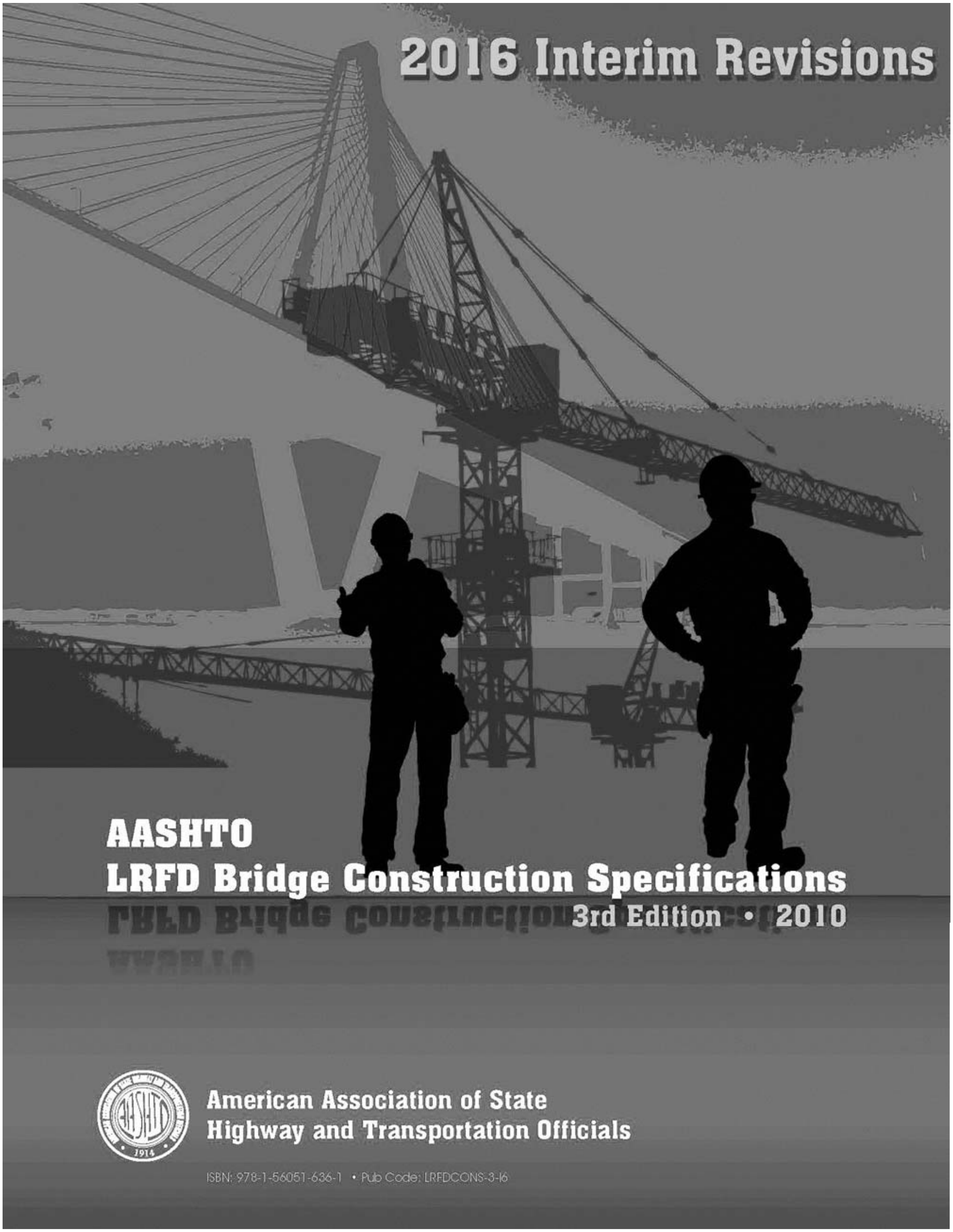


2016 Revision

American Association
of State Highway and
Transportation Officials
AASHTO

444 North Capitol Street, NW, Suite 249
Washington, DC 20001
202-624-5800 phone/202-624-5806 fax
www.transportation.org

ISBN: 978-1-56051-636-1 Pub Code: LRFDCONS-3-I6

2016 Revision

American Association of
State Highway and
Transportation Officials

AASHTO
The Voice of Transportation

444 North Capitol Street, NW, Suite 249
Washington, DC 20001
202-624-5800 phone/202-624-5806 fax
www.transportation.org

ISBN: 978-1-56051-617-0 Pub Code: LRFDCONS-3-I5

American Association of
State Highway and
Transportation Officials

AASHTO
The Voice of Transportation

444 North Capitol Street, NW, Suite 249
Washington, DC 20001
202-624-5800 phone/202-624-5806 fax
www.transportation.org

ISBN: 978-1-56051-595-1 Pub Code: LRFDCONS-3-I4

American Association of
State Highway and
Transportation Officials

AASHTO
The Voice of Transportation

444 North Capitol Street, NW Suite 249
Washington, DC 20001
202-624-5800 phone/202-624-5806 fax
www.transportation.org

ISBN: 978-1-56051-561-6 Pub Code: LRFDCONS-3-I3

American Association of State Highway and Transportation Officials
444 North Capitol Street, NW Suite 249
Washington, DC 20001
202-624-5800 phone/202-624-5806 fax
www.transportation.org

ISBN: 978-1-56051-518-0 Pub Code: LRFDCONS-3-I2

.

American Association of State Highway and Transportation Officials
444 North Capitol Street, NW Suite 249
Washington, DC 20001
202-624-5800 phone/202-624-5806 fax
www.transportation.org

ISBN: 978-1-56051-452-7

Publication Code: LRFDCONS-3

PREFACE

Units

The *AASHTO LRFD Bridge Construction Specifications*, Third Edition, uses U.S. Customary units only. Per a decision by the subcommittee in 2009, SI units will no longer be included in this edition or future interims.

References

If a standard is available as a stand-alone publication—for example, the ACI standards—the title is *italicized* in the text and listed in the references. If a standard is available as part of a larger publication—for example, the AASHTO materials specifications—the standard's title is not italicized and the larger publication—in this case, *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 29th Edition—is listed in the references.

Unit Abbreviations 2010 Revision

Most of the abbreviations commonly used in LRFD Construction are listed below.
Also, please note the following:

- Abbreviations for singular and plural are the same.
- Most units of time have one-letter abbreviations. Unit abbreviations are always set in roman type, while variables and factors are set in *italic* type. Thus, "2 h" is the abbreviation for "two hours."

Table i—Frequently-Used Unit Abbreviations

Unit	Abbreviation
cubic foot	ft^3
cubic inch	$in.^3$
cubic yard	yd^3
degrees Fahrenheit	ºF
foot	ft
foot-kip	ft-kip
foot per hour	ft/h
foot per minute	ft/min
foot per second	ft/s
foot pound	ft · lb
foot pound-force	ft · lbf
foot second	ft · s
gallon	gal
hour	h
Hertz	Hz
inch	in.
joule	J
kilonewton	kN
kilopascal	kPa
kip per foot	kip/ft
kip per square inch	ksi
kip per square foot	kip/ft^2
megapascal	MPa
microinch	µin
micron	µm
mile	mi
minute	min (min. for "minimum")

Table i (continued)—Frequently-Used Unit Abbreviations

Unit	Abbreviation
newton	N
newton meter	N · m
newton per meter	N/m
ounce	oz
pascal	Pa
pascal second	Pa · s
pound	lb
pound-force	lbf
pound-force foot	lbf · ft
pound-force inch	lbf · in.
pound-force per foot	lbf/ft
pound-force per inch	lbf/in.
pound-force per pound	lbf/lb
pound-force per square foot	psf
pound-force per square inch	psi
pound per cubic foot	lb/ft^3
pound per cubic inch	lb/in.^{3}
pound per cubic yard	lb/yd^3
pound per foot	lb/ft
pound per inch	lb/in.
pound per hour	lb/h
pound per square foot	lb/ft^2
pound per yard	lb/yd
radian	rad
radian per second	rad/s
quart	qt
second	s
square inch	in.^{2}
square foot	ft^2
square mile	mi^2
square yard	yd^2
year	yr

Note: There are no abbreviations for *day*, *degree* (angle), *kip*, *mil*, or *ton*.

AASHTO Publications Staff
February 2010

PREFACE

Unit Abbreviations

Table i—Frequently-Used Unit Abbreviations

Revise the Table as follows:

Unit	Abbreviation
cubic foot	ft^3
cubic inch	$in.^3$
cubic yard	yd^3
degrees Fahrenheit	ºF
foot	ft
foot-kip	ft-kip
foot per hour	ft/h
foot per minute	ft/min
foot per second	ft/s
foot pound	ft · lb
foot pound-force	ft · lbf
foot second	ft · s
gallon	gal
hour	h
Hertz	Hz
inch	in.
~~joule~~	~~J~~
~~kilonewton~~	~~kN~~
~~kilopascal~~	~~kPa~~
kip per foot	kip/ft
kip per square inch	ksi
kip per square foot	kip/ft^2
~~megapascal~~	~~MPa~~
microinch	μin
micron	μm
mile	mi
minute	min (min. for “minimum”)
~~newton~~	~~N~~
~~newton meter~~	~~N · m~~
~~newton per meter~~	~~N/m~~
ounce	oz
~~pascal~~	~~Pa~~
~~pascal second~~	~~Pa · s~~
pound	lb
pound-force	lbf

2010 Revision

Table i (continued)—Frequently-Used Unit Abbreviations

Unit	Abbreviation
pound-force foot	lbf · ft
pound-force inch	lbf · in.
pound-force per foot	lbf/ft
pound-force per inch	lbf/in.
pound-force per pound	lbf/lb
pound-force per square foot	psf
pound-force per square inch	psi
pound per cubic foot	lb/ft^3
pound per cubic inch	lb/in.^{3}
pound per cubic yard	lb/yd^3
pound per foot	lb/ft
pound per inch	lb/in.
pound per hour	lb/h
pound per square foot	lb/ft^2
pound per yard	lb/yd
radian	rad
radian per second	rad/s
quart	qt
second	s
square inch	in.^{2}
square foot	ft^2
square mile	mi^2
square yard	yd^2
year	yr

Note: There are no abbreviations for *day*, *degree* (angle), *kip*, *mil*, or *ton*.

FOREWORD

The first broadly recognized national standard to design and construct bridges in the United States was published in 1931 by the American Association of State Highway Officials (AASHO), the predecessor to AASHTO. With the advent of the automobile and the establishment of highway departments in all of the American states dating back to just before the turn of the century, the design, construction, and maintenance of most U.S. bridges was the responsibility of these departments and, more specifically, the chief bridge engineer within each department. It was natural, therefore, that these engineers, acting collectively as the AASHTO Highways Subcommittee on Bridges and Structures, would become the author and guardian of this first bridge standard.

This first publication was entitled *Standard Specifications for Highway Bridges and Incidental Structures*. It quickly became the *de facto* national standard and, as such, was adopted and used by not only the state highway departments but also other bridge-owning authorities and agencies in the United States and abroad. The title was soon revised to *Standard Specifications for Highway Bridges* and new editions were released about every four years. AASHTO released the 17th and final edition in 2002.

The body of knowledge related to the design of highway bridges has grown enormously since 1931 and continues to do so. Theory and practice have evolved greatly, reflecting advances through research in understanding the properties of materials, in improved materials, in more rational and accurate analysis of structural behavior, in the advent of computers and rapidly advancing computer technology, in the study of external events representing particular hazards to bridges such as seismic events and stream scour, and in many other areas. The pace of advances in these areas has accelerated in recent years. To accommodate this growth in bridge engineering knowledge, the Subcommittee on Bridges and Structures has been granted authority under AASHTO's governing documents to approve and issue Bridge Interims each year, not only with respect to the Standard Specifications but also to enhance the twenty-odd additional publications on bridges and structures engineering that are under its stewardship.

In 1986, the Subcommittee submitted a request to the AASHTO Standing Committee on Research to assess U.S. bridge design specifications, to review foreign design specifications and codes, to consider design philosophies alternative to those underlying the Standard Specifications, and to render recommendations based on these investigations. This work was accomplished under the National Cooperative Highway Research Program (NCHRP), an applied research program directed by the AASHTO Standing Committee on Research and administered on behalf of AASHTO by the Transportation Research Board (TRB). The work was completed in 1987, and, as might be expected with continuing research, the Standard Specifications were found to have discernible gaps, inconsistencies, and even some conflicts. Beyond this, the specification did not reflect or incorporate the most recently developing design philosophy, load-and-resistance factor design (LRFD), a philosophy which has been gaining ground in other areas of structural engineering and in other parts of the world such as Canada and Europe.

From its inception until the early 1970s, the sole design philosophy embedded within the Standard Specifications was one known as working stress design (WSD). WSD establishes allowable stresses as a fraction or percentage of a given material's load-carrying capacity, and requires that calculated design stresses not exceed those allowable stresses. Beginning in the early 1970s, WSD was adjusted to reflect the variable predictability of certain load types, such as vehicular loads and wind forces, through adjusting design factors, a design philosophy referred to as load factor design (LFD). Both WSD and LFD are reflected in the current edition of the Standard Specifications.

A further philosophical extension considers the variability in the properties of structural elements, in similar fashion to load variabilities. While considered to a limited extent in LFD, the design philosophy of LRFD takes variability in the behavior of structural elements into account in an explicit manner. LRFD relies on extensive use of statistical methods, but sets forth the results in a manner readily usable by bridge designers and analysts.

With the advent of these specifications, bridge engineers had a choice of two standards to guide their designs, the long-standing AASHTO *Standard Specifications for Highway Bridges*, and the alternative, newly adopted *AASHTO LRFD Bridge Design Specifications*, and its companions, *AASHTO LRFD Bridge Construction Specifications* and *AASHTO LRFD Movable Highway Bridge Design Specifications*. Subsequently, the Federal Highway Administration (FHWA) and the states mandated that LRFD standards be used to design all new and total replacement bridges after 2007. For more information on FHWA's LRFD policy, please visit http://www.fhwa.dot.gov/bridge/lrfd/index.htm.

A new edition of these specifications will be published every two years, followed by an interim edition the immediate year after its release. The Interim Specifications have the same status as AASHTO standards, but are tentative revisions approved by at least two-thirds of the Subcommittee. These revisions are voted on by the AASHTO member departments prior to the publication of each new edition of this book and, if approved by at least two-thirds of the members, they are included in the next new edition as standards of the Association. AASHTO members are the 50 State Highway or Transportation Departments, the District of Columbia, and Puerto Rico. Each member has one vote. The U.S. Department of Transportation is a nonvoting member.

Annual Interim Specifications are generally used by the States after their adoption by the Subcommittee. Orders for these annual Interim Specifications may be placed by visiting our website, bookstore.transportation.org, or by calling 1-800-231-3475 (toll free within the U.S. and Canada). A free copy of the current publication catalog can be downloaded from our website or requested from the Publications Sales Office.

The Subcommittee would also like to thank Mr. John M. Kulicki, Ph.D., and his associates at Modjeski and Masters for their valuable assistance in the preparation of the LRFD Specifications.

AASHTO encourages suggestions to improve these specifications. They should be sent to the Chairman, Subcommittee on Bridges and Structures, AASHTO, 444 North Capitol Street, N.W., Suite 249, Washington, DC 20001. Inquiries as to intent or application of the specifications should be sent to the same address.

AASHTO Highways Subcommittee on Bridges and Structures
February 2010

ABBREVIATED TABLE OF CONTENTS

The *AASHTO LRFD Bridge Construction Specifications*, Third Edition, contains the following 32 sections:

TABLE OF CONTENTS

SECTION 1

STRUCTURE EXCAVATION AND BACKFILL

1.1—GENERAL

Structure excavation shall consist of the removal of all material, of whatever nature, necessary for the construction of foundations for bridges, retaining walls, and other major structures, in accordance with the contract documents or as directed by the Engineer.

If not otherwise provided for in the contract, structure excavation shall include the furnishing of all necessary equipment and the construction and subsequent removal of all cofferdams, shoring, and water control systems which may be necessary for the execution of the work.

If not otherwise specified in the contract documents, it shall also include the placement of all necessary backfill, including any necessary stockpiling of excavated material which is to be used in backfill, and the disposing of excavated material which is not required for backfill, in roadway embankments or as provided for excess and unsuitable material in Subsection 203.02, *AASHTO Guide Specifications for Highway Construction*.

If the contract does not include a separate pay item or items for such work, structure excavation shall include all necessary clearing and grubbing and the removal of existing structures within the area to be excavated.

Classification, if any, of excavation will be indicated in the contract documents and set forth in the proposal.

The removal and disposal of buried natural or man-made objects are included in the class of excavation in which they are located, unless such removal and disposal are included in other items of work. However, in the case of a buried man-made object, the removal and disposal of such object will be paid for as extra work and its volume will not be included in the measured quantity of excavation, if:

- its removal requires the use of methods or equipment not used for other excavation on the project,
- its presence was not indicated in the contract drawings,
- its presence could not have been ascertained by site investigation, including contact with identified utilities within the area, and
- the Contractor so requests in writing prior to its removal.

C1.1

Subsection 203.02 is located in the *AASHTO Guide Specifications for Highway Construction.*

1.2—WORKING DRAWINGS

Whenever specified in the contract drawings, the Contractor shall provide working drawings, accompanied by calculations where appropriate, of excavation procedures, embankment construction, and backfilling operations. This plan shall show the details of shoring, bracing, slope treatment, or other protective system proposed for use and shall be accompanied by design calculations and supporting data in sufficient detail to permit an engineering review of the proposed design.

The working drawings for protection from caving shall be submitted sufficiently in advance of proposed use to allow for their review; revision, if needed; and approval without delay to the work.

Working drawings shall be approved by the Engineer prior to performance of the work involved, and such approval shall not relieve the Contractor of any responsibility under the contract for the successful completion of the work.

1.3—MATERIALS

Material used for backfill shall be free of frozen lumps, wood, or other degradable or hazardous matter and shall be of a grading such that the required compaction can be consistently obtained using the compaction methods selected by the Contractor.

Permeable material for underdrains shall conform to *AASHTO Guide Specifications for Highway Construction*, Subsection 704.01.

C1.3

Subsection 704.01 is located in the *AASHTO Guide Specifications for Highway Construction*.

1.4—CONSTRUCTION

1.4.1—Depth of Footings

The elevation of the bottoms of footings, as shown in the contract documents, shall be considered as approximate only and the Engineer may order, in writing, such changes in dimensions or elevation of footings as may be necessary to secure a satisfactory foundation.

1.4.2—Foundation Preparation and Control of Water

1.4.2.1—General

Where practical, all substructures shall be constructed in open excavation and, where necessary, the excavation shall be shored, braced, or protected by cofferdams constructed in accordance with the requirements contained in Article 3.3, “Cofferdams and Shoring.” When footings can be placed in the dry without the use of cofferdams, backforms may be omitted with the approval of the Engineer and the entire excavation filled with concrete to the required elevation of the top of the footing. The additional concrete required shall be furnished and placed at the expense of the Contractor. Temporary water control systems shall conform to the requirements contained in Article 3.4, “Temporary Water Control Systems.”

1.4.2.2—Excavations within Channels

When excavation encroaches upon a live streambed or channel, unless otherwise permitted, no excavation shall be made outside of caissons, cribs, cofferdams, steel piling, or sheeting, and the natural streambed adjacent to the structure shall not be disturbed without permission from the Engineer. If any excavation or dredging is made at the site of the structure before caissons, cribs, or cofferdams are sunk or are in place, the Contractor shall, without extra charge, after the foundation base is in place, backfill all such excavation to the original ground surface or riverbed with material satisfactory to the Engineer. Material temporarily deposited within the flow area of streams from foundation or other excavation shall be removed and the stream flow area freed from obstruction thereby.

1.4.2.3—Foundations on Rock

When a foundation is to rest on rock, the rock shall be freed from all loose material, cleaned, and cut to a firm surface, either level, stepped, or roughened, as may be directed by the Engineer. All seams shall be cleaned out and filled with concrete, mortar, or grout before the footing is placed.

Where blasting is required to reach footing level, any loose, fractured rock caused by overbreak below bearing level shall be removed and replaced with concrete or grouted at the Contractor's expense.

1.4.2.4—Foundations Not on Rock

When a foundation is to rest on an excavated surface other than rock, special care shall be taken not to disturb the bottom of the excavation, and the final removal of the foundation material to grade shall not be made until just before the footing is to be placed.

Where the material below the bottom of footings not supported by piles has been disturbed, it shall be removed and the entire space filled with concrete or other approved material at the Contractor's expense. Under footings supported on piles, the over-excavation or disturbed volumes shall be replaced and compacted as directed by the Engineer.

1.4.2.5—Approval of Foundation

After each excavation is completed, the Contractor shall notify the Engineer that the excavation is ready for inspection and evaluation and no concrete or other footing material shall be placed until the Engineer has approved the depth of the excavation and the character of the foundation material.

1.4.3—Backfill

Backfill material shall conform to the provisions of Article 1.3, "Materials." If sufficient material of suitable quality is not available from excavation within the project limits, the Contractor shall import such material as directed by the Engineer.

Unless otherwise specified in the contract documents, all spaces excavated and not occupied by abutments, piers, or other permanent work shall be refilled with earth up to the surface of the surrounding ground, with a sufficient allowance for settlement. Except as otherwise provided, all backfill shall be thoroughly compacted to the density of the surrounding ground and its top surface shall be neatly graded. Fill placed around piers shall be deposited on both sides to approximately the same elevation at the same time. Rocks larger than 3.0-in. maximum dimension shall not be placed against the concrete surfaces.

Embankment construction shall conform to the requirements of *AASHTO Guide Specifications for Highway Construction*, Subsection 203.02. The fill at retaining walls, abutments, wingwalls, and all bridge bents in embankment shall be deposited in well-compacted, horizontal layers not to exceed 6.0 in. in thickness and shall be brought up uniformly on all sides of the structure or facility. Backfill within or beneath embankments, within the roadway in excavated areas, or in front of abutments and retaining walls or wingwalls shall be compacted to the same density as required for embankments.

No backfill shall be placed against any concrete structure until permission has been given by the Engineer. The placing of such backfill shall also conform to the requirements of Article 8.15.2, "Earth Loads." The backfill in front of abutments and wingwalls shall be placed first to prevent the possibility of forward movement. Jetting of the fill behind abutments and wingwalls will not be permitted.

Adequate provision shall be made for the thorough drainage of all backfill. French drains, consisting of at least 2.0 ft^3 of permeable material wrapped in filter fabric to prevent clogging and transmission of fines from the backfill, shall be placed at each weep hole.

Backfilling of metal and concrete culverts shall be done in accordance with the requirements of Sections 26, "Metal Culverts," and 27, "Concrete Culverts."

C1.4.3

Subsection 203.02 is located in *AASHTO Guide Specifications for Highway Construction*.

1.5—MEASUREMENT AND PAYMENT

1.5.1—Measurement

The quantity to be paid for as structure excavation shall be measured by the cubic yard. The quantities for payment will be determined from limits shown in the contract documents or ordered by the Engineer. No deduction in structure excavation pay quantities will be made where the Contractor does not excavate material which is outside the limits of the actual structure but within the limits of payment for structure excavation.

Unless otherwise specified in the contract documents, pay limits for structure excavation shall be taken as:

- the horizontal limits shall be vertical planes 18.0 in. outside of the neat lines of footings or structures without footings,
- the top limits shall be the original ground or the top of the required grading cross-section, whichever is lower, and
- the lower limits shall be the bottom of the footing or base of structure, or the lower limit of excavation ordered by the Engineer.

When foundations are located within embankments and the specifications require the embankment to be constructed to a specified elevation that is above the bottom of the footing or base of structure prior to construction of the foundation, then such specified elevation will be considered to be the original ground.

When it is necessary, in the opinion of the Engineer, to carry the foundations below the elevations shown in the contract documents, the excavation for the first 3.0 ft of additional depth will be included in the quantity for which payment will be made under this item. Excavation below this additional depth will be paid for as extra work, unless the Contractor states in writing that payment at contract prices is acceptable.

1.5.2—Payment

Unless otherwise provided, structure excavation, measured as provided in Article 1.5.1, "Measurement," will be paid for by the cubic yard for the kind and class specified in the contract documents.

Payment for structure excavation shall include full compensation for all labor, material, equipment, and other items that may be necessary or convenient to the successful completion of the excavation to the elevation of the bottom of footings or base of structure.

Full compensation for controlling and removing water from excavations and for furnishing and installing or constructing all cofferdams, shoring, and all other facilities necessary to the operations, except concrete seal courses that are shown in the contract documents, and their subsequent removal, shall be considered as included in the contract price for structure excavation, unless the contract document provides for their separate payment.

Unless otherwise specified in the contract documents, the contract price for structure excavation shall include full payment for all handling and storage of excavated materials that are to be used as backfill, including any necessary drying, and the disposal of all surplus or unsuitable excavated materials. Any clearing, grubbing, or structure removal that is required but not paid for under other items of the contract documents will be considered to be included in the price paid for structure excavation.

Unless the contract document provides for its separate payment, the contract price for structure excavation shall include full compensation for the placing and compacting of structure backfill. The furnishing of backfill material from sources other than excavation will be paid for at the contract unit price for the material being used or as extra work if no unit price has been established.

1.6—REFERENCE

AASHTO. 2008. *AASHTO Guide Specifications for Highway Construction*, Ninth Edition, GSH-9, American Association of State Highway and Transportation Officials, Washington, DC.

TABLE OF CONTENTS

SECTION 2

REMOVAL OF EXISTING STRUCTURES

2.1—DESCRIPTION

This work shall consist of the removal, wholly or in part, and satisfactory disposal or salvage of all bridges, retaining walls, and other major structures that are designated to be removed in the contract documents. Unless otherwise specified, the work also includes any necessary excavation and the backfilling of trenches, holes, or pits that result from such removal. It also includes all costs for environmental and health monitoring systems or programs as may be required.

2.2—WORKING DRAWINGS

Working drawings showing methods and sequence of removal shall be prepared:

- when structures or portions of structure are specified to be removed and salvaged,
- when removal operations will be performed over or adjacent to public traffic or railroad property, or
- when specified in the contract documents.

The working drawings shall be submitted to the Engineer for approval at least ten days prior to the proposed start of removal operations. Removal work shall not begin until the drawings have been approved. Such approval shall not relieve the Contractor of any responsibility under the contract documents for the successful completion of the work.

When salvage is required, the drawings shall clearly indicate the markings proposed to designate individual segments of the structure.

2.3—CONSTRUCTION

2.3.1—General

Except for utilities and other items that the Engineer may direct the Contractor to leave intact, the Contractor shall raze, remove, and dispose of each structure or portion of structure designated to be removed. All concrete and other foundations shall be removed to a depth of at least 2.0 ft below ground elevation or 3.0 ft below subgrade elevation, whichever is lower. Unless otherwise specified in the contract documents, the Contractor has the option to either pull piles or cut them off at a point not less than 2.0 ft below groundline. Cavities left from structure removal shall be backfilled to the level of the surrounding ground and, if within the area of roadway construction, shall be compacted to meet the requirements of the contract documents for embankment.

Explosives shall not be used except at locations and under conditions specified in the contract documents. All blasting shall be completed before the placement of new work.

2.3.2—Salvage

Materials which are designated to be salvaged under the contract documents, either for reuse in the project or for future use by the Owner, shall remain the property of the Owner and shall be carefully removed in transportable sections and stockpiled near the site at a location designated by the Engineer. The Contractor shall restore or replace damaged or destroyed material without additional compensation.

Rivets and bolts that must be removed from steel structures to be salvaged shall be removed by cutting the heads with a chisel, after which they shall be punched or drilled from the hole, or by any other method that will not injure the members for reuse and will meet the approval of the Engineer. Prior to dismantling, all members or sections of steel structures shall be match-marked with paint in accordance with the diagram or plan approved by the Engineer.

All bolts and nails shall be removed from lumber deemed salvageable by the Engineer as part of the salvage of timber structures.

2.3.3—Partial Removal of Structures

When structures are to be widened or modified and only portions of the existing structure are to be removed, these portions shall be removed in such a manner as to leave the remaining structure undamaged and in proper condition for the use contemplated. Methods involving the use of blasting or wrecking balls shall not be used within any span or pier unless the entire span or pier is to be removed. Any damage to the portions remaining in service shall be repaired by the Contractor at the Contractor's expense.

Before beginning concrete removal operations involving the removal of a portion of a monolithic concrete element, a saw cut approximately 1.0 in. deep shall be made to a true line along the limits of removal on all faces of the element that will be visible in the completed work.

Old concrete shall be carefully removed to the lines designated by drilling, chipping, or other methods approved by the Engineer. The surfaces presented as a result of this removal shall be reasonably true and even, with sharp, straight corners that will permit a neat joint with the new construction or be satisfactory for the use contemplated. Where existing reinforcing bars are to extend from the existing structure into new construction, the concrete shall be removed so as to leave the projecting bars clean and undamaged. Where projecting bars are not to extend into the new construction, they shall be cut off flush with the surface of the old concrete.

During full-depth removal of deck concrete over steel beams or girders which are to remain in place, the Contractor shall exercise care so as not to notch, gouge, or distort the top flanges. Any damage shall be repaired at the direction of the Engineer and at the expense of the Contractor. Repairs may include grinding, welding, heat-straightening, or member replacement, depending on the location and severity of the damage.

2.3.4—Disposal

Any material not designated for salvage will belong to the Contractor. Except as provided herein, the Contractor shall store or dispose of such material outside of the right-of-way. If the material is disposed of on private property, the Contractor shall secure written permission from the property owner and shall furnish a copy of each agreement to the Engineer. Waste materials may be disposed of in an Owner's site when such sites are described in the contract documents.

Unless otherwise provided in the contract documents, removed concrete may be buried in adjacent embankments, provided it is broken into pieces which can be readily handled and incorporated into embankments and is placed at a depth of not less than 3.0 ft below finished grade and slope lines. The removed concrete shall not be buried in areas where piling is to be placed or within 10.0 ft of trees, pipelines, poles, buildings, or other permanent objects or structures, unless permitted by the Engineer. Removed concrete may also be disposed of outside the right-of-way as provided above.

The contract documents shall indicate all known hazardous material including paint history. Hazardous material shall be properly disposed of and appropriate records maintained.

2.4—MEASUREMENT AND PAYMENT

The work, as prescribed for by this item, shall be measured as each individual structure or portion of a structure to be removed. Payment will be made on the basis of the lump-sum bid price for the removal of each structure or portion of structure as specified in the contract documents.

The above prices and payments shall be full compensation for all work, labor, tools, equipment, excavation, backfilling, materials, proper disposal, and incidentals necessary to complete the work, including salvaging materials not to be reused in the project when such salvaging is specified and not otherwise paid for.

Full compensation for removing and salvaging materials that are to be reused in the project shall be considered as included in the contract document prices paid for reconstructing, relocating, or resetting the items involved, or in such other contract pay items that may be designated in the contract documents; no additional compensation will be allowed therefore.

SECTION 3: TEMPORARY WORKS

TABLE OF CONTENTS

SECTION 3

TEMPORARY WORKS

3.1—GENERAL

3.1.1—Description

This work shall consist of the construction and removal of temporary facilities that are generally designed by the Contractor and employed by the Contractor in the execution of the work, and whose failure to perform properly could adversely affect the character of the contract work or endanger the safety of adjacent facilities, property, or the public. Such facilities include but are not limited to falsework, forms and form travelers, cofferdams, shoring, water control systems, and temporary bridges. Appropriate reductions in allowable stresses and decreases in resistance factors or imposed loads shall be used for design when other than new or undamaged materials are to be used. To the extent possible, calculations shall include adjustments to section properties to account for damage or section loss.

Unless otherwise permitted, the design of the temporary works shall be based on *AASHTO LRFD Bridge Design Specifications* load factors specified in Articles 3.4.1 and 3.4.2, and all applicable load combinations shall be investigated. Such investigation shall include construction loads; member capacity based on field condition which account for section loss, deterioration of capacity, and any alterations of the structure; support conditions during all construction phases; and the appropriate distribution of construction stockpiled materials and construction equipment.

C3.1.1

FHWA issued Technical Advisory T5140.24 on October 29, 1993, concerning *Bridge Temporary Works*. The Secretary of the U.S. Department of Transportation was directed by Congress to develop specifications and guidelines for use in constructing bridge temporary works. The falsework collapse of the Maryland Route 198 Bridge over the Baltimore/Washington Parkway and the fact that no national standard code or specification was available on bridge temporary works precipitated the mandate.

The guide specifications, handbook, and certification program are useful in conjunction with this Section of these Specifications. States are encouraged to review these publications and incorporate appropriate portions into their specifications. The *Construction Handbook* (see below) is a useful instructional tool for field inspection personnel.

The following publications are useful reference documents in the preparation of specifications for the design, review and inspection of temporary works:

- *Synthesis of Falsework, Formwork, and Scaffolding for Highway Bridge Structures,* Nov. 1991 (FHWA-RD-91-062)
- *Guide Standard Specifications for Bridge Temporary Works*, Nov. 1993 (FHWA-RD-93-031)
- *Guide Design Specifications for Bridge Temporary Works*, Nov. 1993 (FHWA-RD-93-032)
- *Certification Program for Bridge Temporary Works,* Nov. 1993 (FHWA-RD-93-033)
- *Construction Handbook for Bridge Temporary Works,* Nov. 1993 (FHWA-RD-93-034)
- *Steel Bridge Erection Guide Specification,* 2007 (NSBA S10.1)

3.1.2—Working Drawings

Whenever specified in the contract documents or requested by the Engineer, the Contractor shall provide working drawings with design calculations and supporting data in sufficient detail to permit a structural review of the proposed design of a temporary work. When concrete is involved, such data shall include the sequence and rate of placement. Sufficient copies shall be furnished to meet the needs of the Engineer and other entities with review authority. The working drawings shall be submitted sufficiently in advance of proposed use to allow for their review; revision, if needed; and approval without delay to the work.

The Contractor shall not start the construction of any temporary work for which working drawings are required until the drawings have been approved by the Engineer. Such approval will not relieve the Contractor of responsibility for results obtained by use of these drawings or any of the Contractor's other responsibilities under the contract.

3.1.3—Design

The design of temporary works shall conform to the *AASHTO LRFD Bridge Design Specifications* or the *Guide Design Specifications for Bridge Temporary Works*, or to other established and generally accepted design code or specification for such work.

When manufactured devices are to be employed, the design shall not result in loads on such devices in excess of the load ratings recommended by their Manufacturer. For equipment where the rated capacity is determined by load testing, the design load shall be as stated in the *Guide Specifications for Bridge Temporary Works.*

The load rating used for special equipment, such as access scaffolding, may be under the jurisdiction of OSHA and/or other State/Local regulations. However, in no case shall the rating exceed 80 percent of the maximum load sustained during load testing of the equipment.

When required by statute or specified in the contract documents, the design shall be prepared and the drawings signed by a Registered Professional Engineer.

C3.1.3

Article 3.1.3 specifies the use of the *AASHTO LRFD Bridge Design Specifications* or the *Guide Design Specifications for Bridge Temporary Works*, unless another recognized specification is accepted by the Engineer.

The *Guide Design Specifications for Bridge Temporary Works* is referenced for design loads.

Access scaffolding is covered under the Occupational Safety and Health Administration (OSHA) but stability trusses used for erection of structural steel are designed as falsework.

3.1.4—Construction

Temporary works shall be constructed in conformance with the approved working drawings. The Contractor shall verify that the quality of the materials and work employed are consistent with that assumed in the design.

3.1.5—Removal

Unless otherwise specified, all temporary works shall be removed and shall remain the property of the Contractor upon completion of their use. The area shall be restored to its original or planned condition and cleaned of all debris.

3.2—FALSEWORK AND FORMS

3.2.1—General

The working drawings for falsework shall be prepared and sealed by a Registered Professional Engineer whenever the height of falsework exceeds 14.0 ft or whenever traffic, other than workers involved in constructing the bridge, will travel under the bridge.

Falsework and forms shall be of sufficient rigidity and strength to safely support all loads imposed and to produce in the finished structure the lines and grades indicated in the contract documents. Forms shall also impart the required surface texture and rustication and shall not detract from the uniformity of color of the formed surfaces.

C3.2.1

Falsework is considered to be any temporary structure which supports structural elements of concrete, steel, masonry, or other materials during their construction or erection.

Forms are considered to be the enclosures or panels which contain the fluid concrete and withstand the forces due to its placement and consolidation. Forms may in turn be supported on falsework.

Form travelers, as used in segmental cantilever construction, are considered to be a combination of falsework and forms.

3.2.2—Falsework Design and Construction

3.2.2.1—Loads

The design load for falsework shall consist of the sum of dead and live vertical loads and any horizontal loads.

As a minimum, dead loads shall include the weight (mass) of the falsework and all construction material to be supported. The combined unit weight (density) of concrete, reinforcing and prestressing steel, and forms shall be assumed to be not less than 0.16 kip/ft^3 of normal-weight concrete or 0.13 kip/ft^3 of lightweight concrete that is supported.

Live loads shall consist of the actual weight (mass) of any equipment to be supported, applied as concentrated loads at the points of contact and a uniform load of not less than 0.02 kip/ft^2 applied over the area supported, plus 0.075 kip/ft applied at the outside edge of deck overhangs.

The horizontal load used for the design of the falsework bracing system shall be the sum of the horizontal loads due to equipment; construction sequence, including unbalanced hydrostatic forces from fluid concrete; stream flow, when applicable; and an allowance for wind. However, in no case shall the horizontal load to be resisted in any direction be less than two percent of the total dead load.

For post-tensioned structures, the falsework shall also be designed to support any increase in or redistribution of loads caused by prestressing of the structure.

Loads imposed by falsework onto existing, new, or partially completed structures shall not exceed those permitted in Article 8.15, "Application of Loads."

C3.2.2.1

In the SI units, normal-weight concrete is known as normal-density concrete and lightweight concrete is known as low-density concrete.

3.2.2.2—Foundations

Falsework shall be founded on a solid footing, safe against undermining, protected from softening, and capable of supporting the loads imposed on it. When requested by the Engineer, the Contractor shall demonstrate by suitable load tests that the soil bearing values assumed for the design of the falsework footings do not exceed the supporting capacity of the soil.

Falsework which cannot be founded on a satisfactory footing shall be supported on piling which shall be spaced, driven, and removed in an approved manner.

3.2.2.3—Deflections

For cast-in-place concrete structures, the calculated deflection of falsework flexural members shall not exceed $^1/_{240}$ of their span irrespective of the fact that the deflection may be compensated for by camber strips.

3.2.2.4—Clearances

Unless otherwise provided, the minimum dimensions of clear openings to be provided through falsework for roadways that are to remain open to traffic during construction shall be at least 5.0 ft greater than the width of the approach traveled way, measured between barriers when used. The minimum vertical clearance over Interstate routes and freeways shall be 14.5 ft, and 14.0 ft over other classes of roadways.

3.2.2.5—Construction

Falsework shall be constructed and set to grades that allow for its anticipated settlement and deflection, and for the vertical alignment and camber indicated in the contract documents or ordered by the Engineer for the permanent structure. When directed by the Engineer, variable-depth camber strips shall be used between falsework beams and soffit forms to accomplish this.

Suitable screw jacks, pairs of wedges, or other devices shall be used at each post to:

- adjust falsework to grade,
- permit minor adjustments during the placement of concrete or structural steel should observed settlements deviate from those anticipated, and
- allow for the gradual release of the falsework.

The Contractor shall provide for accurate measurement of falsework settlement during the placing and curing of the concrete.

Falsework or formwork for deck slabs on girder bridges shall be supported directly on the girders so that there will be no appreciable differential settlement during placing of the concrete. Girders shall either be braced and tied to resist any forces that would cause rotation or torsion in the girders caused by the placing of concrete for diaphragms or decks or be shown to be adequate for those effects. Unless specifically permitted, welding of falsework support brackets or braces to structural steel members or reinforcing steel shall not be allowed.

3.2.3—Formwork Design and Construction

3.2.3.1—General

Forms shall be of wood, steel, or other approved material and shall be mortar tight and of sufficient rigidity to prevent objectional distortion of the formed concrete surface caused by pressure of the concrete and other loads incidental to the construction operations.

Forms for concrete surfaces exposed to view shall produce a smooth surface of uniform texture and color substantially equal to that which would be obtained with the use of plywood conforming to the National Institute of Standards and Technology Product Standard PS 1 for Exterior B-B Class I Plywood. Panels lining such forms shall be arranged so that the joint lines form a symmetrical pattern conforming to the general lines of the structure. The same type of form-lining material shall be used throughout each element of a structure. Such forms shall be sufficiently rigid so that the undulation of the concrete surface shall not exceed 0.125 in. when checked with a 5.0-ft straightedge or template. All sharp corners shall be filleted with approximately 0.75-in. chamfer strips.

C3.2.3.1

Forms for concrete structures using plywood refers to the National Institute of Standards and Technology Product Standards PS 1, *Construction and Industrial Plywood*.

Concrete shall not be deposited in the forms until all work connected with constructing the forms has been completed, all debris has been removed, all materials to be embedded in the concrete have been placed for the unit to be cast, and the Engineer has inspected the forms and materials.

3.2.3.2—Design

The structural design of formwork shall conform to the ACI Standard, *Recommended Practice for Concrete Formwork* (ACI 347), or some other generally accepted and permitted standard. In selecting the hydrostatic pressure to be used in the design of forms, consideration shall be given to the maximum rate of concrete placement to be used, the effects of vibration, the temperature of the concrete, and any expected use of set-retarding admixtures or pozzolanic materials in the concrete mix.

C3.2.3.2

Formwork design refers to ACI 347-78, *Recommended Practice for Concrete Formwork.*

3.2.3.3—Construction

Forms shall be set and held true to the dimensions, lines, and grades of the structure prior to and during the placement of concrete. Forms may be given a bevel or draft at projections, such as copings, to ensure easy removal. Prior to reuse, forms shall be cleaned, inspected for damage, and, if necessary, repaired. When forms appear to be defective in any manner, either before or during the placement of concrete, the Engineer may order the work stopped until defects have been corrected.

Forms shall be treated with form oil or other approved release agent before the reinforcing steel is placed. Material which will adhere to or discolor the concrete shall not be used.

Except as provided herein, metal ties or anchorages within the forms shall be so constructed as to permit their removal to a depth of at least 1.0 in. from the face without injury to the concrete. Ordinary wire ties may be used only when the concrete will not be exposed to view and where the concrete will not come in contact with salts or sulfates. Such wire ties, upon removal of the forms, shall be cut back at least 0.25 in. from the face of the concrete with chisels or nippers; for green concrete, nippers shall be used. Fittings for metal ties shall be of such design that, upon their removal, the cavities that are left will be of the smallest possible size. The cavities shall be filled with cement mortar and the surface left sound, smooth, even, and uniform in color.

When epoxy-coated reinforcing steel is required, all metal ties, anchorages, or spreaders that remain in the concrete shall be of corrosion-resistant material or coated with a dielectric material.

For narrow walls and columns where the bottom of the form is inaccessible, an access opening shall be provided in the forms for cleaning out extraneous material immediately before placing the concrete.

3.2.3.4—Tube Forms

Tubes used as forms to produce voids in concrete slabs shall be properly designed and fabricated or otherwise treated to make the outside surface waterproof. Prior to concrete placement, such tubes shall be protected from the weather and stored and installed by methods that prevent distortion or damage. The ends of tube forms shall be covered with caps that shall be made mortar tight and waterproof. If wood or other material that expands when moist is used for capping tubes, a premolded rubber joint filler 0.25 in. in thickness shall be used around the perimeter of the caps to permit expansion. A polyvinyl chloride (PVC) vent tube shall be provided near each end of each tube. These vents shall be constructed to provide positive venting of the voids. After exterior form removal, the vent tube shall be trimmed to within 0.5 in. of the bottom surface of the finished concrete.

Anchors and ties for tube forms shall be adequate to prevent displacement of the tubes during concrete placement.

3.2.3.5—Stay-in-Place Forms

Stay-in-place deck soffit forms, such as corrugated metal or precast concrete panels, may be used if shown in the contract documents or approved by the Engineer. Prior to the use of such forms, the Contractor shall provide a complete set of details to the Engineer for review and approval. Unless otherwise noted, the contract documents for structures should be dimensioned for the use of removable forms. Any changes necessary to accommodate stay-in-place forms, if approved, shall be at the expense of the Contractor.

3.2.4—Removal of Falsework and Forms

3.2.4.1—General

Falsework or forms shall not be removed without approval of the Engineer. In the determination of the time for the removal of falsework and forms, consideration shall be given to the location and character of the structure, the weather, the materials used in the mix, and other conditions influencing the early strength of the concrete.

Methods of removal likely to cause overstressing of the concrete or damage to its surface shall not be used. Supports shall be removed in such a manner as to permit the structure to uniformly and gradually take the stresses due to its own weight (mass). For structures of two or more spans, the sequence of falsework release shall be as specified or approved in the contract documents.

3.2.4.2—Time of Removal

If field operations are not controlled by beam or cylinder tests, the following minimum periods of time, exclusive of days when the temperature is below 40°F, shall have elapsed after placement of concrete before falsework is released or forms are removed:

Falsework for:

• Spans over 14.0 ft	14 days
• Spans of 14.0 ft or less	10 days
• Bent caps not yet supporting girders	10 days

Forms:

• Not supporting the dead weight (mass) of the concrete	24 h
• For interior cells of box girders and for railings	12 h

If high early strength is obtained with Type III cement or by the use of additional cement, these periods may be reduced as directed.

Where field operations are controlled by cylinder tests:

- Generally, the removal of supporting forms or falsework shall not begin until the concrete is found to have the specified compressive strength.

- In no case shall supports be removed in less than seven days after placing the concrete.

- Forms shall not be removed until the concrete has sufficient strength to prevent damage to the surface.

- Falsework for post-tensioned portions of structures shall not be released until the prestressing steel has been tensioned.

- Falsework supporting any span of a continuous or rigid frame bridge shall not be released until the aforementioned requirements have been satisfied for all of the structural concrete in that span and in the adjacent portions of each adjoining span for a length equal to at least one-half the length of the span where falsework is to be released.

Unless otherwise specified or approved in the contract documents, falsework shall be released before the railings, copings, or barriers are placed for all types of bridges. For arch bridges, the time of falsework release relative to the construction of elements of the bridge above the arch shall be as shown in the contract documents or directed by the Engineer.

3.2.4.3—Extent of Removal

All falsework and forms shall be removed except:

- Portions of driven falsework piles which are more than 1.0 ft below subgrade within roadbeds, or 2.0 ft below the original ground or finished grade outside of roadbeds, or 2.0 ft below the established limits of any navigation channel.

- Footing forms where their removal would endanger the safety of cofferdams or other work.
- Forms from enclosed cells where access is not provided.
- Deck forms in the cells of box girder bridges that do not interfere with the future installation of utilities shown in the contract documents.

3.3—COFFERDAMS AND SHORING

3.3.1—General

Cofferdams shall be constructed to adequate depths to assure stability and to adequate heights to seal off all water. They shall be safely designed and constructed and be made as watertight as is necessary for the proper performance of the work which must be done inside them. In general, the interior dimensions of cofferdams shall be such as to give sufficient clearance for the construction of forms and the inspection of their exteriors, and to permit pumping from outside the forms. Cofferdams that are tilted or moved laterally during the process of sinking shall be righted, reset, or enlarged so as to provide the necessary clearance. This shall be solely at the expense of the Contractor.

The Contractor shall control the ingress of water so that footing concrete can be placed in the dry. The Contractor shall determine if a seal is required, and, if required, shall determine the depth of the seal and the cure time required and shall be fully responsible for the performance of the seal. After the seal has cured, the cofferdam shall be pumped out and the balance of the masonry placed in the dry. When weighted cofferdams are employed and the weight is utilized to partially overcome the hydrostatic pressure acting against the bottom of the foundation seal, special anchorage such as dowels or keys shall be provided to transfer the entire weight of the cofferdam into the foundation seal. During the placing and curing of a foundation seal, the elevation of the water inside the cofferdam shall be controlled to prevent any flow through the seal and, if the cofferdam is to remain in place, it shall be vented or ported at or below low-water level.

Shoring shall be adequate to support all loads imposed and shall comply with any applicable safety regulations.

C3.3.1

Cofferdams and shoring consist of those structures used to temporarily hold the surrounding earth and water out of excavations and to protect adjacent property and facilities during construction of the permanent work.

A concrete seal conforming to the requirements of Section 8, “Concrete Structures,” shall be placed under water below the elevation of the footing.

3.3.2—Protection of Concrete

Cofferdams shall be constructed so as to protect green concrete against damage from sudden fluctuations in water level and to prevent damage to the foundation by erosion. No struts or braces shall be used in cofferdams or shoring systems in such a way as to extend into or through the permanent work, without written permission from the Engineer.

3.3.3—Removal

Unless otherwise provided or approved, cofferdams and shoring with all sheeting and bracing shall be removed after the completion of the substructure, with care being taken not to disturb or otherwise injure the finished work.

3.4—TEMPORARY WATER CONTROL SYSTEMS

3.4.1—General

Temporary water control systems consist of dikes, by-pass channels, flumes and other surface water diversion works, cut-off walls, and pumping systems, including wellpoint and deep well systems, used to prevent water from entering excavations for structures.

3.4.2—Working Drawings

Working drawings for temporary water control systems, when required, shall include details of the design and the equipment, operating procedures to be employed, and location of point or points of discharge. The design and operation shall conform to all applicable water pollution and erosion control requirements.

3.4.3—Operations

Pumping from the interior of any foundation enclosure shall be done in such manner as to preclude the possibility of the movement of water through any fresh concrete. No pumping will be permitted during the placing of concrete or for a period of at least 24 hours thereafter, unless it be done from a suitable sump separated from the concrete work by a watertight wall or other effective means, subject to approval of the Engineer.

Pumping to unwater a sealed cofferdam shall not commence until the seal has set sufficiently to withstand the hydrostatic pressure.

Pumping from wellpoints or deep wells shall be regulated so as to avoid damage by subsidence to adjacent property.

3.5—TEMPORARY BRIDGES

3.5.1—General

Temporary bridges shall be constructed, maintained, and removed in a manner that will not endanger the work or the public.

C3.5.1

Temporary bridges include detour bridges for use by the public, haul road bridges, and other structures, such as conveyor bridges, used by the Contractor.

3.5.2—Detour Bridges

When a design is furnished by the Owner, detour bridges shall be constructed and maintained to conform to either such design or an approved alternative design. When permitted by the Specifications, the Contractor may submit a proposed alternative design. Any alternative design must be equivalent in all respects to the design and details furnished by the Owner and is subject to approval by the Engineer. The working drawings and design calculations for any alternative design shall be signed by a Registered Professional Engineer.

C3.5.2

When a design is not furnished by the Owner, the Contractor shall prepare the design and furnish working drawings to the Engineer for approval. The design shall provide the clearances, alignment, load capacity, and other design parameters specified or approved in the contract documents. The design shall conform to the *AASHTO LRFD Bridge Design Specifications*. If design live loads are not otherwise specified in the contract documents, 75 percent of the HL93 loading may be used. The working drawings and design calculations shall be signed by a Registered Professional Engineer.

The "design of detour bridges" refers to the *AASHTO LRFD Bridge Design Specifications*, 2007.

3.5.3—Haul Bridges

When haul road bridges or other bridges which are not for public use are proposed for construction over any right-of-way that is open to the public or that is over any railroad, working drawings showing complete design and details, including the maximum loads to be carried, shall be submitted to the Engineer for approval. Such drawings shall be signed by a Registered Professional Engineer. The design shall conform to *AASHTO LRFD Bridge Design Specifications* when applicable or to other appropriate standards.

C3.5.3

The design of haul bridges refers to the *AASHTO LRFD Bridge Design Specifications*, 2007.

3.5.4—Maintenance

The maintenance of temporary bridges for which working drawings are required shall include their replacement in case of partial or complete failure. In case of the Contractor's delay or inadequate progress in making repairs and replacement, the Owner reserves the right to furnish such labor, materials, and supervision of the work as may be necessary to restore the structure for proper movement of traffic. The entire expense of such restoration and repairs shall be considered a part of the cost of the temporary structure and where such expenditures are incurred by the Owner, they shall be charged to the Contractor.

3.6—MEASUREMENT AND PAYMENT

Unless otherwise specified in the contract documents, payment for temporary works shall be considered to be included in the payment for the various items of work for which they are used and no separate payment shall be made therefore.

When an item for concrete seals for cofferdams is included in the contract, such concrete will be measured and paid for as provided in Section 8, "Concrete Structures."

When an item or items for temporary bridges, cofferdams, shoring systems, or water control systems is included in the bid schedule, payment will be the lump-sum bid for each such structure or system which is listed on the bid schedule and which is constructed and removed in accordance with the requirements of the contract documents. Such payment includes full compensation for all costs involved with the furnishing of all materials and the construction, maintenance, and removal of such temporary works.

3.7—REFERENCES

AASHTO. 1995. *Construction Handbook for Bridge Temporary Works*, CHBTW-1, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 1995. *Guide Design Specifications for Bridge Temporary Works*, GSBTW-1, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

ACI. 1978. *Recommended Practice for Concrete Framework*, American Concrete Institute, Farmington Hills, MI.

CCEER. 2006. *Recommendations for the Design of Beams and Posts in Bridge Falsework,* CCEER 05/11, Center for Civil Engineering Earthquake Research, University of Nevada, Reno, NV, Jan. 2006.

FHWA. 1991. *Synthesis of Falsework, Formwork, and Scaffolding for Highway Bridge Structures*, FHWA-RD-91-062, Federal Highway Administration, U.S. Department of Transportation, Washington, DC, Nov. 1991.

FHWA. 1993. *Bridge Temporary Works*, TS140.24, Federal Highway Administration, U.S. Department of Transportation, Washington, DC, Oct. 1993.

FHWA. 1993. *Guide Standard Specifications for Bridge Temporary Works*, FHWA-RD-93-031, Federal Highway Administration, U.S. Department of Transportation, Washington, DC, Nov. 1993.

FHWA. 1993. *Guide Design Specifications for Bridge Temporary Works*, FHWA-RD-93-032, Federal Highway Administration, U.S. Department of Transportation, Washington, DC, Nov. 1993. See also AASHTO GSBTW-1, listed above.

FHWA. 1993. *Certification Program for Bridge Temporary Works*, FHWA-RD-93-033, Federal Highway Administration, U.S. Department of Transportation, Washington, DC, Nov. 1993.

FHWA. 1993. *Construction Handbook for Bridge Temporary Works*, FHWA-RD-93-034, Federal Highway Administration, U.S. Department of Transportation, Washington, DC, Nov. 1993. See also AASHTO CHBTW-1, listed above.

NIST. 1995. *Construction and Industrial Plywood*, Voluntary Product Standard PS 1-95, National Institute of Standards and Technology, U.S. Department of Commerce, Gaithersburg, MD.

NSBA. 2007. *Steel Bridge Erection Guide Specification,* S10.1, National Steel Bridge Alliance, Chicago, IL. See also AASHTO NSBASBEGS-1 (2007).

SSRP. 2006. *Full Scale Load Testing of Sand-Jacks,* SSRP-05/06, University of California, San Diego, CA, June 2006.

TABLE OF CONTENTS

SECTION 4

DRIVEN FOUNDATION PILES

4.1—INTRODUCTION

This work shall consist of furnishing and driving foundation piles of the type and dimensions designated in the contract documents, including cutting off or building up foundation piles when required. This Specification also covers providing test piles and performing loading tests. Piling shall conform to and be installed in accordance with these Specifications; at the location; and to the elevation, penetration, and required nominal resistance shown in the contract documents or as directed by the Engineer.

Except when test piles are required, the Contractor shall furnish the piles in accordance with the dimensions shown in the contract documents. When test piles are required, the production pile lengths shown in the contract documents shall be used for estimating purposes only and the actual lengths to be furnished for production piles shall be determined by the Engineer after the test piles have been driven and tested. The lengths given in the order list provided by the Engineer shall include only the lengths anticipated for use in the completed structure. The Contractor shall increase the lengths shown or ordered to provide for fresh heading and for such additional length as may be necessary to suit the method of operation, without added compensation.

C4.1

For the purposes of this document, "nominal resistance" is considered synonymous with "ultimate pile capacity."

Driven pile lengths are estimated for bidding purposes from soil investigation, static analysis, and perhaps local experience. Rarely, however, are these estimated lengths used to control production pile installations. Usually dynamic methods (e.g., dynamic testing, wave equation, or dynamic formula) are used to evaluate nominal resistance of test piles or the early production piles and then develop a "driving criterion" with a specified number of blows per unit penetration ("blow count"). For larger projects, a static load test is sometimes used to confirm the pile nominal resistance and establish a driving criterion. The blow count criterion determined by the test piles is usually applied to production piles to ensure that they will achieve similar nominal resistances as the test piles. The blow count is in effect an additional quality assurance test.

The objective of this Specification is to provide a criterion by which the Owner can ensure that designated piles are properly installed and the Contractor can expect equitable compensation for work performed. The Owner's responsibility is to estimate the pile lengths required to safely support the design load. Pile lengths should be estimated based on subsurface explorations, testing, and analysis, which are completed during the design phase. Pile contractors who enter contractual agreements to install piles for an Owner should not be held accountable or indirectly penalized for inaccuracies in estimated lengths. The Contractor's responsibility is to provide and install designated piles, undamaged, to the requirements specified by the Engineer. This work is usually accomplished within an established framework of restrictions necessary to ensure a "good" pile foundation. The price bid for this item of work will reflect the Contractor's estimate of both actual cost to perform the work and perceived risk.

4.2—MATERIALS

4.2.1—Steel Piles

4.2.1.1—Rolled Structural Steel Piles

4.2.1.1.1—Specifications for Steel Properties

Steel used in rolled structural steel piles shall conform to the following Standard Specifications of the American Society for Testing and Materials (ASTM):

- ASTM A36/A36M: Standard Specification for Carbon Structural Steel
- ASTM A572/A572M: Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel
- ASTM A992/A992M: Standard Specification for Structural Steel Shapes

The above listing does not exclude the use of steel ordered or produced to other than the listed specifications or other published ASTM specifications that establish its properties and suitability.

Steel for cast steel shoes, if used, shall conform to ASTM A148/A148M (Grade 90-60).

C4.2.1.1.1

A 36/A 36M is not readily available from structural mills. Better economy and availability will be realized by specifying high-strength (50 ksi) ASTM A572/A572M or ASTM A992/A992M.

Pile shoes should be considered when structural steel shapes are driven through obstructions or to sloping hard rock. Pile shoes are discussed in Article 4.4.2.2.2.

4.2.1.1.2—Minimum Dimensions

Sections of such piles shall be of “H” or “W” shape and shall comply with the following requirements:

- The flange projection shall not exceed 14 times the minimum thickness of metal in either the flange or the web, and flange widths shall not be less than 80 percent of the depth of the section.
- The nominal depth in the direction of the web shall not be less than 8.0 in.

Flanges and web shall have a minimum nominal thickness of not less than 0.375 in.

4.2.1.2—Steel Pipe Piles

4.2.1.2.1—Specification for Steel

Steel pipe piles shall consist of steel pipe conforming to the project plans.

(a) Minimum Dimensions:

Pipes shall have an outside diameter and a minimum nominal wall thickness as shown in the contract documents.

(b) Ends of closed-end pipe piles shall be closed with a flat plate or a forged or cast steel conical point, or other end closure of approved design. End plates shall have a minimum thickness of 0.75 in. The end plate shall be cut flush with the outer pile wall. The end of the pipe shall be beveled before welding to the end plate using a partial penetration groove weld.

C4.2.1.2.1

Open-end pipe piles sometimes are filled and closed-end steel pipe piles are usually filled with concrete as detailed in Article 4.2.1.2.2.

Typically, ASTM A252 Grade 2 is used. However, consideration should be given to using Grade 3, which provides additional strength with little increase in cost. While ASTM A252 is a commonly used specification and performs well in most applications, structures with seismic or special conditions may require additional qualifications. One example of this is as follows: Pipe shall be ASTM A252, but dimensional tolerance as per API 5L and elongation of 25 percent minimum in 2.0 in. The carbon equivalency shall not exceed 0.05 percent. API 5L could be specified, but it requires hydrostatic testing and 48.0 in. outside diameter is the largest diameter covered by API 5L.

Bearing piles are usually of no less than 8.0-in. diameter. Some special applications may have smaller diameters. Generally, wall thicknesses should not be less than 0.188 in. Larger pile diameters generally require larger wall thickness. In some cases, a larger thickness may be desirable for both open-end and closed-end pipe piles. Very thin-wall pipe piles may be difficult to drive in some cases and a thicker wall may be required. Pipes installed open-ended may require a suitable cutting shoe.

Larger diameter pipes may require thicker end plates and/or reinforcement.

4.2.1.2.2—Concrete for Concrete-Filled Pipe Piles

Before concrete is placed in the pile, the pile shall be inspected by an acceptable method to confirm the full pile length and dry bottom condition. If accumulations of water in pipes are present, the water shall be removed before the concrete is placed.

The concrete for concrete-filled pipe piles shall have a minimum compressive strength of 2.5 ksi and a slump of not less than 6.0 in. and not more than 10.0 in. Concrete shall be placed in each pile in a continuous operation.

No concrete shall be placed until all driving within a radius of 15.0 ft of the pile has been completed, or all driving within the above limits shall be discontinued until the concrete in the last pile cast has set for at least two days.

C4.2.1.2.2

A drop light or mirror system, downhole camera or weighted tape with attached dry cloth, are possible inspection methods.

It is not necessary to use a tremie or centering cone when placing concrete in pipe piles. It is impossible to center the concrete in a batter pile.

Continuous operation may include changing of concrete supply trucks or other brief interruptions.

4.2.2—Timber Piles

4.2.2.1—General

The Contractor shall supply pressure-treated Southern Pine or Douglas fir piles conforming to ASTM D25, i.e., new and clean peeled one piece from butt to tip. Piles not meeting ASTM D25 requirements shall be rejected.

C4.2.2.1

Timber piles are normally ordered only in 5.0-ft incremental lengths. Usually, Douglas fir piles are available on the west coast, and Southern pine in eastern locations. Southern pine is available up to 80.0 ft in length. Douglas fir piles are available up to 120.0 ft. Most timber piles are 7.0- or 8.0-in. in diameter at the tip. Because of the natural taper of the pile of 1.0-in. diameter reduction for each 10.0 ft of pile length, the butt diameter depends on tip diameter and the pile length. Most commonly available pile sizes are provided in the Timber Piling Council's "Timber Pile Design and Construction Manual."

4.2.2.2—Submittals

Certification by treating plant stating type, pressure process used, net amount of preservative retained, and compliance with applicable standards shall be submitted to the Engineer. Any structural connections such as for uplift loads shall be shown in the submittal.

4.2.2.3—Field Fabrication

Where specified, timber piles shall be fitted with metal shoes as specified in Article 4.4.2.2.3 of this Specification. If the pile top is trimmed to the final cut-off elevation, cut surfaces at the pile head shall be treated as specified in Article 4.4.7.2.

4.2.2.4—Pressure Treatment

Pressure treatment shall be in accordance with the American Wood Preserver's Association (APWA) Standard U1-06 Use Category System: User Specification for Treated Wood. Category UC4C shall be taken to apply to land and fresh water piling and foundation piling. Categories UC5A, UC5B, and UC5C should generally be taken to apply to timber piling for salt and brackish water application when continuous marine exposure is expected.

4.2.2.5—Required Retentions

The preservative retentions and penetrations provided in the AWPA Standard U1-06 Commodity Specifications E and G for the Use Category specified shall apply. The minimum preservative retentions shall be as specified in Table 4.2.2.5-1.

Table 4.2.2.5-1—Preservative Retention Requirements

Use Category	Species[a]	Retention Specifications (lb/ft³)						
		Creosote		Pentachlorophenol				
		CR/CR-S[b]	CR-PS[b]	PCP-A/PCP-C[b]	CuN[b]	ACZA[b]	CCA[b]	ACQ-C[b]
UC4C	Southern Pine	12.0	12.0	0.60	0.10	0.80	0.80	0.80
	Coastal Douglas fir	17.0	17.0	0.85	0.14	1.00	1.00[a]	NR[b]
	Interior Douglas fir	17.0	17.0	0.85	NR[b]	1.00	NR[b]	NR[b]
UC5A	Southern Pine	16.0	NR[b]	NR[b]	NR[b]	1.5	1.5	NR[b]
	Coastal Douglas fir	16.0	NR[b]	NR[b]	NR[b]	1.5	NR[b]	NR[b]
UC5B and UC5C	Southern Pine	20.0	NR[b]	NR[b]	NR[b]	2.5	2.5	NR[b]
	Coastal Douglas fir	16.0	NR[b]	NR[b]	NR[b]	2.5	NR[b]	NR[b]

Notes:

[a] The listing of Coastal Douglas fir as an acceptable species for treatment with CCA is not intended to imply that this species can generally be satisfactorily treated. Treatment for this species is usually satisfactory only if the material is chosen from permeable wood, selected by treatment trials. If treatable wood is not available, the treatment of Coastal Douglas fir with CCA is not recommended.

[b] CR/CR-S =Creosote or Creosote Solution; CR-PS = Creosote-Petroleum Solution; PCP-A/PCP-C = Pentachlorophenol, Type A or C Solvent; CuN = Copper Naphthenate; ACZA = Ammoniacal Copper Zinc Arsenate; CCA = Chromated Copper Arsenate; ACQ = Alkali Copper Quat, Type C; NR = not recommended.

4.2.3—Prestressed Concrete Piles

Production of piles shall be in accordance with Prestressed Concrete Institute (PCI) MNL-116, *Manual for Quality Control for Plants and Production of Structural Precast Concrete Products*.

C4.2.3

For additional information, see the reprint of PCI's "Precast Prestressed Concrete Piles," Chapter 20 of the *PCI Bridge Design Manual* (September 2004), Publication Number BM-20-04.

4.2.3.1—Forms

Forms for prestressed concrete piles shall conform to the general requirements for concrete form work as provided in PCI MNL-116, *Manual for Quality Control for Plants and Production of Structural Precast Concrete Products*.

4.2.3.2—Casting

Concrete shall be cast continuously within three days after pretensioning steel; however, concrete shall not be cast in forms until placement of reinforcement and anchorages has been inspected and approved by the pile Manufacturer's quality control representative. Each pile shall have dense concrete, straight smooth surfaces, and reinforcement retained in its proper position during fabrication. Unless self-consolidating concrete is used, the concrete shall be compacted by vibrating with a vibrator head smaller than the minimum distance between the pretensioning steel. Ensure that pile end surfaces are perpendicular to the longitudinal axis of the pile.

C4.2.3.2

Continuous casting operation may include changing concrete supply trucks or other brief interruptions.

4.2.3.3—Finish

Finish of piles shall be in accordance with PCI MNL-116, *Manual for Quality Control for Plants and Production of Structural Precast Concrete Products*. Standard finish shall be that the formed sides are reasonably smooth from casting against approved forms. Standard finish of the top shall be a float finish with edges tooled.

C4.2.3.3

Special finishes, if required by the Engineer, can be listed here.

4.2.3.4—Curing and Protection

Curing of piles shall be in accordance with PCI MNL-116, *Manual for Quality Control for Plants and Production of Structural Precast Concrete Products*. Piles shall be cured using moist curing or accelerated steam curing.

No pile shall be driven until it is sufficiently cured so as to resist handling and driving stresses without damage.

In cold weather, an extended curing period may be required, as specified in the contract documents. Concrete shall be protected from freezing until the compressive strength reaches at least 0.8 f'_c.

C4.2.3.4

Local experience and driving conditions may require longer than seven days' curing time or a minimum concrete strength before driving. Piles driven early may show a higher risk of breakage. If ordered by the Engineer to drive early, the Contractor should not bear the risk of damaged piles.

Air entrainment, water/cement ratio, and type of cement are all important factors in the design of concrete piles for harsh environments. ACI 318-02 Chapter 4, Article 4.3, discusses these issues that contribute to durability in harsh environments such as seawater and sulfate soils.

If exposed to freezing conditions, dowel holes should be protected from water intrusion. For more information on cold weather requirements, refer to PCI MNL-116, *Manual for Quality Control for Plants and Production of Structural Precast Concrete Products.*

4.2.3.5—Prestressing

Prestressing of piles shall be in accordance with PCI MNL-116, *Manual for Quality Control for Plants and Production of Structural Precast Concrete Products*.

4.2.3.6—Shop Drawings

The Contractor shall submit the required number of shop drawings for prestressed concrete piles to the Engineer, indicating pile dimensions, materials, tendon arrangement, and prestressing forces proposed for use, and any addition or rearrangement of reinforcing steel from that shown in the contract documents. Construction of the piles shall not begin until the Engineer has approved the drawings.

4.2.3.7—Storage and Handling

Handling, storing, and transporting prestressed concrete piles shall be done in such a manner as to avoid excessive bending stresses, cracking, spalling, or other injurious result.

C4.2.3.7

Cracks can be repaired, if necessary, by injecting epoxy under pressure into the cracks. Generally recognized guidelines suggest that cracks wider than 0.007 in. can be successfully injected. Smaller cracks often need no repair.

4.3—PROTECTIVE COATINGS

If there is a required protection, the Contractor shall be responsible for restoring or repair any damage to the coating.

4.4—DRIVING PILES

4.4.1—Pile Driving Equipment

All pile driving equipment, including the pile driving hammer, hammer cushion, helmet, pile cushion, and other appurtenances to be furnished by the Contractor shall be approved in advance by the Engineer before any driving can take place. Pursuant to obtaining this approval, the Contractor shall submit a description of pile driving equipment to the Engineer at least two weeks before pile driving is to begin. The description shall contain sufficient detail so that the proposed driving system can be evaluated by wave equation analysis.

If the nominal resistance is to be determined by static load test, dynamic test, quick static load test, or wave equation analysis, the Contractor shall submit to the Engineer results of a wave equation analysis to show that the piles are drivable.

If the nominal resistance is to be determined by dynamic formula, a wave equation analysis is not required. The blow count required by the dynamic formula shall not exceed 10 blows per in.

The following hammer efficiencies shall be used in a wave equation analysis of vertical piles unless better information is available.

Hammer Type	*Efficiency (in Percent)*
Drop	25 to 40
Single-acting air/steam	67
Double-acting air/steam	50
Diesel	80
Hydraulic or diesel with built-in energy measurement	95

Hammer efficiencies shall be adjusted for batter driving.

In addition to the other requirements of these Specifications, the criterion that the Contractor and the Engineer will use to evaluate the driving equipment shall consist of both the required number of hammer blows per in. at the required nominal resistance and the pile driving stresses over the entire driving process. The required number of hammer blows indicated by the wave equation analysis at the required nominal resistance shall be between 2 and 10 blows per in. for the driving equipment to be deemed acceptable.

In addition, for the driving equipment to be deemed acceptable, the pile stresses, which are determined by the wave equation analysis for the entire driving operation, shall not exceed the values below:

C4.4.1

The actual hammer performance is a variable that can be accurately assessed only through dynamic measurements, as in Article 4.4.4.3.

Drop hammer efficiency can be highly variable depending on the drop mechanism. A lower efficiency for drop hammer will produce more conservative estimates of nominal resistance, but a higher efficiency would be more conservative when assessing driving stresses.

Diesel hammers operate at variable ram strokes. Hydraulic hammers are often operated at less than full stroke to prevent overstressing piles.

- For steel piles, compressive driving stress shall not exceed 90 percent of the yield point of the pile material.

- For concrete piles, tensile stresses shall not exceed 0.095 multiplied by the square root of the concrete compressive strength, f'_c, in kips per square in. plus the effective prestress value, i.e., $[0.095(f'_c)^{0.5}$ + prestress] where f'_c is given in ksi, $[7.9(f'_c)^{0.5}$ + prestress] where f'_c is given in kPa and compressive stresses shall not exceed 85 percent of the compressive strength minus the effective prestress value, i.e., $(0.85f'_c$ – prestress).

- For timber piles, the compressive driving stress shall not exceed $\varphi_{da}F_{co}$, where F_{co} is the base resistance of wood in compression parallel to the grain as specified in Article 8.4.1.3, and φ_{da} is the resistance factor for stresses incurred during pile driving specified in Article 8.5.2.2 of the *AASHTO LRFD Bridge Design Specifications*, and φ_{da} is equal to 1.15.

During pile driving operations, the Contractor shall use the approved system. Any change in the driving system shall be considered only after the Contractor has submitted revised pile driving equipment data and wave equation analysis. The Contractor shall be notified of the acceptance or rejection of the driving system changes within two working days of the Engineer's receipt of the requested change. The time required for submission, review, and approval of a revised driving system shall not constitute the basis for a contract time extension to the Contractor.

Approval of pile driving equipment shall not relieve the Contractor of responsibility to drive piles, free of damage, to the required nominal resistance and, if specified, the minimum penetration, shown in the contract documents.

4.4.1.1—Hammers

4.4.1.1.1—General

Piles shall be driven with an impact or vibratory hammer conforming to these Specifications.

Pile driving hammers shall be of the size needed to develop the energy required to drive the piles at a blow count that does not exceed 10 blows per in. at the required nominal resistance.

C4.4.1.1.1

The intent is to select a size of hammer with sufficient reserve capacity at normal operating conditions depending on the anticipated subsoil conditions and local experience. The Contractor may be asked by the Engineer to drive to a higher blow count to penetrate an unforeseen thin dense layer or minor obstruction. Jetting or drilling may be preferred means to penetrate a dense layer, as discussed in Articles 4.4.1.2.6, 4.4.2.1.2, and 4.4.2.1.3. Overdriving will often damage the pile and/or hammer.

4.4.1.1.2—Drop Hammers

Drop hammers shall not be used for concrete piles or for piles whose required nominal resistance exceeds 60.0 tons.

Where drop hammers are permitted, the ram shall have a weight not less than 1.0 ton and the height of drop shall not exceed 12.0 ft. In no case shall the ram weight of drop hammers be less than the combined weight of helmet and pile. All drop hammers shall be equipped with hammer guides and a helmet to ensure concentric impact.

C4.4.1.1.2

Lighter drop weights might be insufficient to spool the crane winch.

4.4.1.1.3—Air Hammers

If a dynamic formula is used to establish the required blow count, the weight of the striking parts of air hammers used shall not be less than one-third the weight of pile and drive cap, and in no case shall the striking part have a weight less than 1.4 tons. If a wave equation analysis is used to establish the required blow count and driving stresses, this limitation on ram weight shall not apply.

The plant and equipment furnished for air-hammers shall have sufficient capacity to maintain, under working conditions, the pressure at the hammer specified by the Manufacturer. The hose connecting the compressor with the hammer shall be at least the minimum size recommended by the Manufacturer.

Hammer performance shall be evaluated at the end of driving by measuring blows per minute and comparing these blows with the Manufacturer's recommendations.

C4.4.1.1.3

Smaller ram weight hammers can be used for special applications.

4.4.1.1.4—Diesel Hammers

If open-end (single-acting) diesel hammers are not equipped with a device to measure impact velocity at all times during pile driving operations, the stroke shall be obtained by measuring the speed of operation either manually or with a device that makes the measurement automatically.

Closed-end (double-acting) diesel hammers shall be equipped with a bounce chamber pressure gauge in good working order, mounted near ground level so as to be easily read by the Engineer. The Contractor shall provide a correlation chart of bounce chamber pressure and potential energy.

C4.4.1.1.4

Either impact velocity or stroke measurement is required and should be recorded. Jump sticks to visually measure stroke should not be used for safety reasons.

It is important to record stroke or bounce chamber pressure with the blow count.

4.4.1.1.5—Hydraulic Hammers

Hydraulic hammers shall be equipped with a system for measuring ram energy. The system shall be in good working order and the results shall be easily and immediately available to the Engineer.

C4.4.1.1.5

The measurement of impact velocity makes it possible to calculate the kinetic energy of the ram at impact. The measurement device may display either impact velocity or energy. This information shall be recorded with the blow count.

4.4.1.1.6—Vibratory Hammers

Vibratory or other pile driving methods may be used only when specified in the contract documents or in writing by the Engineer. Except when pile lengths have been evaluated from static load test piles, the nominal resistance of piles driven with vibratory hammers shall be verified by additional driving of the first pile driven in each group of 10 piles with an impact hammer of suitable energy to measure the nominal resistance before driving the remaining piles in the group. In case of variable soils, additional piles shall be verified by an impact hammer as directed by the Engineer. All piles that rely primarily on point bearing capacity shall be redriven with an impact hammer.

Vibratory hammers shall not be used to drive concrete piles.

4.4.1.1.7—Additional Equipment or Methods

In case the required penetration is not obtained by the use of a hammer complying with the minimum requirements above, the Contractor may be required to provide a hammer of greater energy or, when permitted, resort to supplemental methods such as jetting or predrilling.

4.4.1.2—Driving Appurtenances

4.4.1.2.1—Hammer Cushion

All impact pile driving equipment except drop hammers shall be equipped with a suitable thickness of hammer cushion material to prevent damage to the hammer or pile. Hammers designed such that a hammer cushion is not required shall be excluded from this requirement.

Where applicable, hammer cushions shall be made of durable, manufactured materials that will retain uniform properties during driving. Wood, wire rope, or asbestos hammer cushions shall not be used. A striker plate shall be placed on the hammer cushion to ensure uniform compression of the cushion material. The hammer cushion shall be replaced by the Contractor before driving is permitted to continue whenever there is a reduction of hammer cushion thickness exceeding 25 percent of the original thickness or, for air hammers, when the reduction in thickness exceeds the Manufacturer's recommendations.

C4.4.1.2.1

For hammers requiring cushion material, use of a durable hammer cushion material that will retain uniform properties during driving is mandatory to accurately relate blow count to nominal resistance. Nondurable materials that deteriorate during driving cause erratic estimates of nominal resistance and, if allowed to dissolve, result in damage to the pile or driving system.

4.4.1.2.2—Helmet

Piles driven with impact hammers shall be fitted with a helmet to distribute the hammer blow uniformly and concentrically to the pile head. The surface of the helmet in contact with the pile shall be plane and smooth and shall be aligned parallel with the hammer base and the pile top. It shall be guided by the leads and not be free-swinging. The helmet shall fit the pile head in such a manner as to maintain concentric alignment of hammer and pile.

For special types of piles, appropriate driving heads, mandrels, or other devices shall be provided so that the piles may be driven without damage.

For timber piles, the least inside helmet or hammer base horizontal dimension shall not exceed the pile head diameter by more than 2.0 in. If the timber pile diameter slightly exceeds the least helmet or hammer base dimension, the pile head shall be trimmed to fit the helmet.

C4.4.1.2.2

Pipe piles and timber piles that are approximately round sections are frequently driven using square helmets. If the helmet dimension is much larger than the pipe diameter, then a centering fixture is required.

The timber top greatest diameter can be slabbed with a chain saw to a reduced effective width to fit the helmet dimension and to a length sufficient for the helmet depth, provided that the slabbed length is above the final cut-off elevation.

4.4.1.2.3—Pile Cushion

A pile cushion shall protect the heads of concrete piles. The cushion thickness placed on the pile head before driving shall be selected by wave equation analysis so that the limiting driving stresses are not exceeded. If the required driving blow count is determined by a dynamic formula, the cushion shall have a thickness of at least 4.0 in.

A new pile cushion shall be provided if, during driving, the cushion begins to smoke or excessive compression occurs. The pile cushion dimensions shall be such as to distribute the blow of the hammer uniformly over the entire cross-section of the pile.

Pile cushions shall be protected from the weather and kept dry before use. Pile cushion shall not be soaked in any liquid unless approved by the Engineer. The use of manufactured pile cushion materials in lieu of a wood pile cushion shall be evaluated on a case-by-case basis.

A used pile cushion in good condition shall be used for restrike tests.

C4.4.1.2.3

Wood pile cushions may become overly compressed and hard after about 1,500 hammer blows. If the hammer energy is relatively low, the cushion can last even longer. In easy driving conditions, it is possible to drive more than one pile with a cushion.

A cushion that has been exposed to less than 50 blows is generally not suitable for restrike tests.

In the case of batter piles, a horizontal brace may be required between the crane and the leads.

4.4.1.2.4—Leads

Pile driving leads that align the pile and the hammer in proper positions throughout the driving operation shall be used. Leads shall be constructed in a manner that affords freedom of movement of the hammer while maintaining alignment of the hammer and the pile to ensure concentric impact for each blow.

The leads shall be designed to permit proper alignment of battered piles when applicable.

Leads may be either fixed or swinging type. Swinging leads, when used, shall be fitted with a pile gate at the bottom of the leads. The leads shall be adequately embedded in the ground or the pile constrained in a structural frame such as a template to maintain alignment.

4.4.1.2.5—Followers

Followers shall be used only when approved in writing by the Engineer or when specified in the contract documents.

For concrete piles, a pile cushion shall be used at the pile top, and suitability of the follower shall be checked by wave equation analysis to verify the blow count, driving stresses, and nominal resistance.

For steel or timber piles, if a wave equation analysis is not performed, the follower shall have an impedance between 50 percent and 200 percent of the pile impedance.

The follower and pile shall be maintained in proper alignment during driving. The follower shall be of such material and dimensions to permit the piles to be driven to the blow count determined to be necessary.

C4.4.1.2.5

The pile driven with a follower should be checked with a wave equation and selected piles with either a static test or dynamic test on the pile and/or on the follower. This eliminates the need to drive a longer full-length test pile in each bent or footing. The longer pile will have higher-than-normal tension stresses, probably different blow counts, and adds significant cost to the project because longer leads and bigger equipment is required to drive that pile.

Impedance is the product of elastic modulus times cross-sectional area divided by material wavespeed (wavespeed is typically 16,800 ft/sec for steel or 12,500 ft/sec for concrete).

The final position of the pile can be verified by checking the position and inclination of the follower at the end of driving.

4.4.1.2.6—Jetting

Jetting shall be permitted only if specified in the contract documents or approved in writing by the Engineer.

The Contractor shall determine the number of jets and the volume and pressure of water at the jet nozzles necessary to freely erode the material adjacent to the pile.

The Contractor shall control and dispose of all jet water in a manner satisfactory to the Engineer, or as specified in the contract documents. If jetting is specified or approved by the Engineer and is performed according to the specifications or as approved by the Engineer, the Contractor shall not be held responsible for any damage to the site caused by jetting operations. If jetting is used for the Contractor’s convenience, the Contractor shall be responsible for all damages to the site caused by jetting operations.

Unless otherwise indicated by the Engineer or the contract documents, jet pipes shall be removed before or when the pile tip is 5.0 ft above the minimum or final tip elevation, and the pile shall then be driven without jetting to the final tip elevation or to the required nominal resistance with an impact hammer. If the required nominal resistance is not reached at the final tip elevation, the pile may be allowed to set up and then the required nominal resistance will be determined by restriking the pile.

C4.4.1.2.6

Jetting is the use of water and air to facilitate pile penetration by displacing the soil.

Predrilling can also be used to facilitate the penetration of the pile, as specified in Article 4.4.2.1.2.

This may be a situation of excessive jetting below the design toe elevation of the pile. In a case in which the driving resistance is low during driving, consideration should be given to adjusting the jetting criteria, upon approval by the Engineer. The 5 ft above the pile toe should be considered as a first estimate and not necessarily final a criterion.

4.4.2—Preparation for Driving

4.4.2.1—Site Work

4.4.2.1.1—Excavation

If practical, piles shall not be driven until after the excavation is complete. Any material forced up between the piles shall be removed to the correct elevation before concrete for the foundation is placed.

Unless otherwise approved by the Engineer, piles at bridge ends shall not be driven until roadway embankments are placed.

C4.4.2.1.1

In some cases, such as high water table, it may be necessary to drive the piles before excavating. Also, in a case in which the footings are closely spaced, it may not be possible to move the piling rig around in the site. In these cases, it is common to use a follower to drive the piles to final grade before excavating for the pile cap. Alternatively, a longer pile can be driven and cut off at the proper elevation.

4.4.2.1.2—Predrilling to Facilitate Driving

When required by the contract documents, the Contractor shall predrill holes of a size specified, at pile locations, and to the depths shown in the contract documents or approved in writing by the Engineer. Any void space remaining around the pile after completion of driving shall be filled with sand or other approved material. The use of spuds shall not be permitted in lieu of predrilling, unless specified in the contract documents or approved in writing by the Engineer. Material resulting from drilling holes shall be disposed of as approved by the Engineer.

C4.4.2.1.2

Predrilling is a process where a hole is drilled with a continuous flight auger or a wet rotary bit to remove some soil or loosen the strata. Predrilling is usually used in the case where driving the pile will displace the upper soil enough to push adjoining piles out of the proper position or limit vibration in the upper layers.

Normally, predrilled holes are smaller than the diameter or diagonal of the pile cross-section and sufficient to allow penetration of the pile to the specified elevation. If subsurface obstructions are encountered, the hole diameter may be increased to the least dimension that is adequate for pile installation or to avoid obstructions.

Jetting can also be used to facilitate driving. Jetting is specified in Article 4.4.1.2.6.

4.4.2.1.3—Additional Requirements for Predrilled Holes in Embankments

If required by the contract documents, piles to be driven through compacted fill or embankment of a depth greater than 5.0 ft shall be driven in holes predrilled to natural ground. After driving the pile, the space around the pile shall be filled to the ground surface with sand or other approved material. Material resulting from predrilling holes shall be disposed of as approved by the Engineer.

C4.4.2.1.3

The predrilled hole should have a diameter not more than the greatest dimension of the pile cross-section plus 6.0 in.

4.4.2.2—Preparation of Piling

4.4.2.2.1—Pile Heads

For steel and timber piling, the pile heads shall be cut and maintained square with the longitudinal axis of the pile. Precast concrete pile heads shall be flat, smooth, and perpendicular to the longitudinal axis of the pile to prevent eccentric impacts from the helmet. Prestressing strands shall be cut off below the surface of the end of the pile. For concrete or timber piles, the pile head shall be chamfered on all sides.

C4.4.2.2.1

The goal of a well-prepared pile head is to provide uniform contact and thereby reduce the potential of pile top damage.

Pile top distortions should be removed before assessing blow count acceptance for the driving criterion.

Prestressed concrete piles may also be chamfered along their length.

4.4.2.2.2—Collars

When timber piles are required to be driven to more than 100.0 tons nominal resistance or when driving conditions require it, collars, bands, or other devices shall be provided to protect piles against splitting and brooming.

4.4.2.2.3—Pile Shoes and End Plates

Pile shoes shall be used when specified by the Engineer or in the contract documents to protect all types of piles when hard driving or obstructions are expected.

Steel pile shoes shall be fabricated from cast steel conforming to ASTM A148/A148M (Grade 90-60).

End plates used on closed-end pipe piles shall be made of ASTM A36/A36M steel or better. The diameter and thickness shall be specified by the Engineer.

When shoes are required by soil conditions, the tips of timber piles shall conform to the approved steel shoes to ensure a firm uniform contact and prevent local stress concentrations in the timber.

C4.4.2.2.3

Pile shoes are sometimes called pile tips or points. Shoes are sometimes specified when not needed; to save cost, do not use shoes unless necessary.

A pile driving acceptance criterion should be developed that will prevent damage to the pile toe. Steel piles driven into soft rock may not require toe protection. When hard rock, sloping rock, or obstructions are expected, the pile toe should be protected with cast steel shoes.

Pile shoes used at the option of the Contractor shall be of a type approved by the Engineer.

4.4.3—Driving

Unless approved by the Engineer, piles shall be driven to:

- the required nominal resistance, or
- the required nominal resistance and minimum tip elevation, if specified, or
- the specified tip elevation.

The blow count shall always be measured, either during initial driving or by redriving with a warm hammer after a wait period, as determined by the Engineer.

For diesel hammers, the stroke shall be recorded. For hydraulic hammers, either energy or impact velocity shall be recorded.

If water jets are used in connection with the driving, the nominal resistance shall be determined from the results of driving after the jets have been withdrawn.

The procedure used in driving the piles shall not subject them to excessive and undue abuse producing crushing and spalling of the concrete, injurious splitting, splintering and brooming of the wood, or excessive deformation of the steel.

C4.4.3

A minimum pile penetration should only be specified if needed to ensure that uplift, lateral stability, depth to resist downdrag, depth to resist scour, and depth for structural lateral resistance are met for the strength or extreme event limit state. Minimum pile tip elevations may be required for the extreme event and service limit states. For example, a normally consolidated layer of cohesive soil below the pile tips might settle under the pile loads, causing an undesirable vertical deflection.

For soils that show a large amount of slowly developing setup and for which sufficient time is not available to verify the setup by restriking a pile, the piles may be driven to a specified depth.

The required blow count is determined either by a static load test, dynamic testing, or wave equation analysis.

The penetration per blow or blow count is usually required for quality control. The blow count is the number of hammer blows required to cause 1.0 ft or 1.0 in. of penetration. Sometimes in easy driving, usually at the beginning of driving a pile, the penetration may be so large that it is recorded as feet per blow. There may be a few cases of very easy driving in soft soils with large setup where measuring blow count may not be necessary. However, in almost all cases, the driving record (record of blow count per unit penetration for the entire driving of a pile) is important if questions arise at some time after completion of driving.

The hammer can be warmed up by striking a previously driven pile at least 20 hammer blows.

Jetting is discussed in more detail in Article 4.4.1.2.6.

4.4.3.1—Heaved Piles

If pile heave is observed, level readings referenced to a fixed datum shall be taken by the Engineer on all piles immediately after installation and periodically thereafter as adjacent piles are driven to determine the pile heave range.

During the driving process for adjacent piles, piles shall be redriven:

- if they heave more than 0.5 in. and end bearing is dominant, or
- if they heave more than 1.5 in. and shaft friction is dominant.

If pile heave is detected for pipe or shell piles that have been filled with concrete, the piles shall be redriven to original position after the concrete has obtained sufficient strength, and a proper hammer-pile cushion system, satisfactory to the Engineer, is used. The Contractor shall be paid for all work performed in conjunction with redriving piles because of pile heave provided the initial driving was done in accordance with the specified installation sequence.

4.4.3.2—Obstructions

If piles encounter unforeseeable, isolated obstructions, the Contractor shall be paid for the cost of obstruction removal and for all remedial design or construction measures caused by the obstruction.

C4.4.3.2

Removal would apply only if the obstruction is near ground surface.

4.4.3.3—Installation Sequence

The order of installing piles in pile groups shall be either starting from the center of the group and proceeding outward in both directions or starting at the outside row and proceeding progressively across the group.

4.4.3.4—Practical Refusal

The selection of a practical refusal blow count limit is difficult because it can depend on the site soil profile, the pile type, and possibly hammer Manufacturer limitations to prevent hammer damage. In no case shall driving continue for more than 3.0 in. at practical refusal driving conditions.

C4.4.3.4

In cases in which the driving is easy until near the end of driving, a higher blow count sometimes may be satisfactory, but if a high blow count is required over a large percentage of the depth, even 10 blows per in. may be too large. Blow counts greater than 10 blows per in. should be used with care, particularly with concrete or timber piles.

In the case of hard rock, the driving criterion should be based on a blows-per-in. criterion and should address limiting the blows following an abrupt refusal to prevent damage. Typically, an example limiting driving criterion is 5 blows per 0.5 in. Refer to Article 4.4.2.2.2 for pile shoes.

4.4.3.5—Limiting Driving Stresses

Unless specified otherwise in the contract documents or by the Engineer, the stresses induced during driving shall not exceed the limits set forth in Article 4.4.1.

4.4.3.6—Driving of Probe Piles

Where required in the contract documents, probe piles shall be furnished to the lengths specified and driven at the locations and to the elevations, nominal resistances, or blow counts directed by the Engineer before other piles are ordered. All piles shall be driven with approved impact hammers unless specifically stated otherwise in the contract documents. The same type and size hammer shall be used on the production piles.

The approval of driving equipment shall conform to the requirements of these Specifications. Unless otherwise approved by the Engineer, the Contractor shall excavate the ground at each probe pile to the elevation of the bottom of the footing before the pile is driven (see Article 4.4.2.1.1 and Commentary). Additional probe piles shall be driven at locations selected by the Engineer to explore possible subsurface variations.

When ordered by the Engineer, probe piles driven to plan grade and not having the required nominal resistance shall be spliced and driven until the required bearing is obtained.

C4.4.3.6

In the context used here, probe piles are those driven to determine the required pile length at various locations on the site. In some parts of the country they are known as indicator piles or test piles. The use of probe piles is particularly common when concrete piles are used.

In general, the specified length of probe piles will be greater than the estimated length of production piles in order to explore the variation of soil conditions.

Probe piles that do not attain the hammer blow count, or required dynamic tests that predict nominal resistance at the specified depth may be allowed to “set up” for a period of 12 to 24 hours, as determined by the Engineer, before being redriven. When possible, the hammer should be warmed up before redriving begins by applying at least 20 blows to another pile. If the specified nominal resistance is not attained on redriving, the Engineer may direct the Contractor to drive a portion or all of the remaining probe pile length and repeat the setup–redrive procedure.

4.4.3.7—Accuracy of Driving

Piles shall be driven with a variation of not more than 0.25 in./ft (1:50) from the vertical or not more than 0.5 in./ft (1:25) from the batter shown in the contract documents, except that piles for trestle bents shall be driven so that the cap may be placed in its proper location without adversely affecting the resistance of the piles.

After driving, the pile head shall be within 6.0 in. of plan locations for all piles capped below final grade, and shall be within 3.0 in. of plan locations for bent caps supported by piles,

No pile shall be nearer than 4.0 in. from any edge of the cap. Any increase in pile cap dimensions or reinforcing caused by out-of-position piles shall be at the Contractor’s expense.

C4.4.3.7

The amount that a pile can be out of position may be determined by the structural engineer. Tight tolerances of 3 in. or less are not practical.

While the Contractor should make every effort to install piles at the planned location and at the planned batter, deviations in actual accuracy obtained may occur for many reasons, including obstructions. To avoid otherwise needless increases in costs, tight specifications in plan location should be specified only when absolutely necessary.

4.4.4—Determination of Nominal Resistance

4.4.4.1—General

The nominal resistance of piles will be determined by the Engineer using the method specified in the contract documents. The method used to determine resistance of piles during or after installation shall be consistent with the pile resistance verification methodology assumed during the project design phase in accordance with Article 10.5.5.2.3 of the *AASHTO LRFD Bridge Design Specifications*.

C4.4.4.1

When comparing various capacity determination methods, higher resistance factors for the more reliable methods result in more useable load per pile or fewer piles per project and thus cost savings. Consideration should be given to the potential for change in nominal resistance after the end of driving. The effect of soil relaxation or setup should be considered in the determination of nominal resistance for soils that are likely to be subject to these phenomena. For example, if setup is present, the pile can be driven to a lesser criterion and lesser capacity. In this case, a wait period to allow for gain due to setup, with confirmation of the nominal resistance by a retest (static or dynamic) will be needed.

4.4.4.2—Static Load Tests

If a static load test is used to determine the pile axial resistance, the test shall not be performed less than five days after the test pile was driven unless approved by the Engineer or otherwise specified in the contract documents. The static load test shall follow the procedures specified in ASTM D1143/D1143M, and the loading procedure should follow the Quick Load Test Method, unless detailed longer-term load-settlement data are needed, in which case the standard loading procedure should be used. Testing equipment and measuring systems shall conform to ASTM D1143/D1143M. The equipment to conduct the static load test shall be supplied by the entity specified in the contract documents. The Engineer or the testing laboratory shall perform the test.

The Contractor shall submit detailed contract documents of the proposed loading apparatus, prepared by a licensed professional engineer, to the Engineer for approval. The submittal shall include calibrations for the hydraulic jack, load cell, and pressure gauge conducted within 30 days before mobilization to the job site. When the approved method requires the use of tension (anchor) piles that will later be used as permanent piles in the work, such tension piles shall be of the same type and size as the production piles and shall be driven in the location of permanent piles where feasible.

While performing the static load test, the Contractor shall provide safety equipment and employ adequate safety procedures. Adequate support for the static load test plates, jack, and ancillary devices shall be provided to prevent them from falling in the event of a release of load due to hydraulic failure, test pile failure, or other cause.

The method of defining failure of the static load test shall be as defined in the contract documents or by the Engineer. Based on the static load test results, the Engineer shall provide the driving criteria for production pile acceptance.

C4.4.4.2

The Quick Test Procedure is desirable because it avoids problems that frequently arise when performing a static test that cannot be started and completed within an eight-hour period. Tests that extend over a longer period are difficult to perform because of the limited number of experienced personnel that are usually available. The Quick Test has proven to be easily performed in the field and the results usually are satisfactory. However, if the formation in which the pile is installed may be subject to significant creep settlement, alternative procedures provided in ASTM D1143/D1143M should be considered.

The practice varies widely across the country regarding who supplies the testing equipment, measuring systems, and jack. The requirements should be stated in the contract documents.

Requirements and guidelines for interpretation of static load test results and the development of driving criteria for production pile acceptance are provided in Article 10.7.3.8.2 of the *AASHTO LRFD Bridge Design Specifications*.

Reaction piles, if used and if driven in production pile locations, should be reseated by redrive if the test is a compression test.

The pile's nominal resistance may increase (soil setup) or decrease (relaxation) after the end of driving. Therefore, it is essential that static load testing be performed after equilibrium conditions in the soil are re-established. Static load tests performed before equilibrium conditions have re-established will underestimate the long-term pile nominal resistance in soil setup conditions and overestimate the long-term nominal resistance in relaxation cases. For piles driven in clays, into weathered shale, or in sandy silts and sands, specifications should require a delay period to elapse between driving and load testing of two weeks, seven days, or five to seven days, respectively.

When specified, tension static load tests shall be conducted in accordance with ASTM D3689. When specified, lateral load tests shall be conducted in accordance with ASTM D3966.

Due to jack ram friction, loads indicated by a jack pressure gauge are commonly 10 to 20 percent higher than the actual load imposed on the pile.

When static load tests are used to control production pile driving, the time required to analyze the static load test results and establish driving criteria should be specified so that the delay time to the Contractor is clearly identified.

4.4.4.3—Dynamic Testing

Dynamic testing shall be conducted in accordance with ASTM D4945. The Contractor shall prepare for the required instrument attachment as directed by the Engineer.

The Contractor shall drive the pile as directed by the Engineer. If directed by the Engineer, the Contractor shall reduce the driving energy transmitted to the pile by using additional cushion or reducing the energy output of the hammer to maintain acceptable stresses in the piles. If nonaxial driving is indicated by dynamic measurements, the Contractor shall immediately realign the driving system.

If the required nominal resistance is not achieved at the end of driving, the Contractor shall restrike the dynamic test pile following a waiting period specified in the contract documents or as directed by the Engineer. Once the waiting period is completed, the dynamic testing instruments shall be reattached, the pile redriven, and the dynamic test repeated. The hammer shall be warmed up before restrike begins. The maximum penetration required during restrike shall be 3.0 in., or a maximum of 20 blows shall be obtained, whichever occurs first.

C4.4.4.3

Dynamic Testing is often called “High Strain Dynamic Pile Testing” and requires impacting the pile with the pile driving hammer or a large drop weight and measuring force and velocity in the pile with pile analyzer instruments.

The Contractor should attach the instruments to the pile after the pile is placed in the leads.

Dynamic Testing estimates the nominal resistance at the time of testing and, as a minimum, generally requires a signal matching analysis of the data. However, dynamic testing can also evaluate the reliability of wave equation analyses for driveability by measuring pile stresses during driving and performance of the hammer in transferring energy to the pile.

Because the nominal resistance of a pile may change substantially during and after pile driving, waiting after driving for additional testing may be beneficial for a safe and economical pile foundation. If possible, the dynamic test should be performed as a restrike test if the Engineer anticipates significant time-dependent increases in nominal strength, called setup, or reductions, called relaxation.

When high blow counts are anticipated during restrike, it is important that the largest possible energy be applied for the earliest blows.

It is desirable to adjust the hammer energy so that the blow count is between 2 and 10 blows per in. Nominal resistance may be overpredicted at blow counts below 2 blows per in. Nominal resistance may be underpredicted at blow counts above 10 blows per in.

About 20 blows are usually required to warm up a diesel or hydraulic hammer. If a previously driven pile is not available to strike for warming up the hammer, the Contractor may choose to use something else such as timber pads on the ground.

When dynamic tests are specified on production piles, the first pile driven in each foundation area is often tested.

The restrike time and frequency should be based on the time-dependent strength change characteristics of the soil. The following minimum restrike durations are often used:

Soil Type	*Time Delay Until Restrike*
Clean Sands	1 Day
Silty Sands	2 Days
Sandy Silts	3–5 Days
Silts and Clays	7–14 Days*
Shales	7 Days

* Longer times sometimes required.

Specifying too short of a restrike time for friction piles in fine-grained soils may result in pile length overruns. Testing personnel should have attained an appropriate level of expertise on the Pile Driving Contractors Association (PDCA)-endorsed examination for providers of dynamic testing services.

The time necessary to analyze the dynamic test results and provide the test results to the Contractor once testing is completed should be stated in the specifications. The time required for the Engineer to review the test results and provide driving criteria should be provided in the specifications, but should not exceed three working days.

4.4.4.4—Wave Equation Analysis

When specified in the contract documents, the Engineer using a wave equation analysis shall determine the driving criterion necessary to reach the required nominal resistance of the pile. Soil and pile properties to be used in this analysis shall be as shown in the contract documents or as determined by the Engineer. The Contractor shall supply the Engineer the necessary information on the proposed driving equipment to perform the wave equation analysis.

C4.4.4.4

A wave equation analysis is sometimes used to establish a driving criterion in preparation for performing a static or a dynamic test.

Without dynamic test results with signal matching analysis and/or static load test data, considerable judgment is required to use the wave equation to predict the pile bearing resistance. Key soil input values that affect the predicted resistance include the soil damping and quake values, the skin friction distribution (e.g., such as those that could be obtained from a pile bearing static analysis), and the anticipated amount of soil setup or relaxation. Furthermore, the actual hammer performance is a variable that can only be accurately assessed through dynamic measurements, although "standard" input values are available.

4.4.4.5—Dynamic Formula

When using a dynamic formula, the particular formula shall be specified in the contract documents. A dynamic formula should not be used if the required nominal resistance is more than 600.0 kips.

Formulas shall be considered applicable only where:

- the head of the pile is not broomed, crushed, or otherwise damaged, and
- a follower is not used.

C4.4.4.5

The *Engineering News* formula has been in use for many years by some agencies, in spite of the fact that its accuracy has been questioned (e.g., Peck, et al., 1974), and through comparison to static load test data, its inaccuracy has recently been documented (Paikowsky, et al., 2004; Allen, 2005). The FHWA Gates formula has been demonstrated to provide improved accuracy relative to the *Engineering News* formula (Paikowsky, et al., 2004; Allen, 2005), and hence it is the preferred dynamic formula, if a dynamic formula is used.

If a dynamic formula is used to establish the driving criterion, the FHWA Gates Formula specified herein should be used. The nominal pile resistance as measured during driving using this method shall be taken as follows:

$$R_{ndr} = 1.75\sqrt{E_d}\,\log_{10}(10N_b) - 100 \qquad (4.4.4.5\text{-}1)$$

where:

R_{ndr} = nominal pile resistance measured during pile driving (kips)

E_d = developed hammer energy. This is the kinetic energy in the ram at impact for a given blow. If ram velocity is not measured, it may be assumed equal to the potential energy of the ram at the height of the stroke, taken as the ram weight times the stroke (ft-lbs)

N_b = number of hammer blows for 1.0 in. of pile permanent set (blows/in.)

If a dynamic formula other than those provided herein is used, it shall be calibrated based on measured static load test results to obtain an appropriate resistance factor, consistent with Article C10.5.5.2 of the *AASHTO LRFD Bridge Design Specifications*.

The *Engineering News* formula, modified to predict a nominal bearing resistance, may be used. The nominal pile resistance using this method shall be taken as:

$$R_{ndr} = \frac{12E_d}{(s + 0.1)} \qquad (C4.4.4.5\text{-}1)$$

where:

R_{ndr} = nominal pile resistance measured during driving (kips)

E_d = developed hammer energy. This is the kinetic energy in the ram at impact for a given blow. If ram velocity is not measured, it may be assumed equal to the potential energy of the ram at the height of the stroke, taken as the ram weight times the stroke (ft-kips)

s = pile permanent set (in.)

4.4.5—Splicing of Piles

Where splices are unavoidable for steel or concrete piles, their number, locations, and details shall be subject to approval of the Engineer.

4.4.5.1—Steel Piles

Full-length piles shall be used where practicable. If splicing is permitted, the method of splicing shall be in accordance with ANSI/AWS D1.1 or as approved by the Engineer. Either shielded arc or submerged arc welding should be used when splicing steel piles. Only certified welders shall perform welding. Mechanical splices that are not welded shall be used for compression piles only.

4.4.5.2—Concrete Piles

Full-length piles shall be used where practical. Where splicing is permitted, concrete pile splice details shall conform to the contract documents, or as approved by the Engineer. Mechanical splices including drive–fit splices may also be used.

C4.4.5.2

Drive–fit mechanical splices are for compression piles only. Mechanical splices designed for tension are available

4.4.5.3—Timber Piles

Timber piles shall not be spliced unless specified in the contract documents or in writing by the Engineer.

4.4.6 Defective Piles

- The pile is withdrawn if practicable, and replaced by a new and, if necessary, longer pile.

- One or more replacement piles are driven adjacent to the defective pile.

A pile driven below the specified butt elevation shall be corrected by one of the following methods approved by the Engineer for the pile in question.

- The pile is spliced or built up as otherwise provided herein.

- A sufficient portion of the footing is extended down to properly embed the pile.

A pile driven out of its proper location, specified in the contract documents or by the Engineer, shall be corrected by one of the following methods approved by the Engineer for the pile in question:

- One or more replacement piles are driven next to the out-of-position piles.

- The footing is extended laterally to incorporate the out-of-location pile.

- Additional reinforcement is added.

All such remedial materials and work shall be furnished at the Contractor's expense.

C4.4.6

The Engineer's determination may be influenced by the pile size and material and the soil conditions.

If piles are driven below cut-off elevation, build-ups are generally required. The concrete at the top of the pile should be cut away, leaving the reinforcing steel exposed for a length as specified in Section 5 of the *AASHTO LRFD Bridge Design Specifications*. The final cut of the concrete should be perpendicular to the axis of the pile. Reinforcement similar to that used in the pile should be securely fastened to the projecting steel and the necessary formwork shall be placed, with care being taken to prevent leakage along the pile. The concrete should be equal to or higher than the quality used in the pile. Just before placing concrete, the top of the pile should be thoroughly flushed with water, allowed to dry, and then covered with a thin coating of neat cement, mortar, or other suitable bonding material. The forms should remain in place for at least seven days and should then be carefully removed and the entire exposed surface of the pile finished as previously specified.

4.4.7—Pile Cut-Off

4.4.7.1—General

All piles shall be cut off to a true plane at the elevations required and anchored to the structure as shown in the contract documents. All cut-off lengths of piling shall remain the property of the Contractor and shall be properly disposed of.

4.4.7.2—Special Requirements for Timber Piles

Timber piles shall be cut to the elevations shown on the contract documents. The length of pile above the cut-off elevation shall be sufficient to permit the complete removal of all material damaged by driving.

Immediately after making final cut-off on treated timber foundation piles, the cut area shall be given a liberal application of copper naphthenate until visible evidence of further penetration has ceased. The copper naphthenate solution shall have minimum two percent copper metal.

C4.4.7.2

Disposal in landfills is the normal requirement. Cut-off pile ends may not be burned in open fires, stoves, or fireplaces. Treated wood may be burned in commercial or industrial incinerators or boilers. Burning should be in compliance with local, state, and federal regulations.

Treated marine piling exposed to the weather shall be capped with a permanently fixed coating such as epoxy or with conical or other caps attached to the piles.

Piling supporting timber structures where the piles are cut off, but not concrete capped, shall be treated with a liberal application of copper naphthenate until visible evidence of further penetration has ceased. In addition, a layer of saturated building felt or fiberglass cloth that overlaps the side of the pile at least 2.0 in. shall be securely fastened and completely covered with 20-gauge thick galvanized metal or aluminum sheet.

All cuts, injuries, and holes as would occur from the removal of nails or spikes that would penetrate the treating zone, as well as bolt holes for connections, shall be treated by applying coal-tar roof cement meeting ASTM D5643.

Cut-off pile ends shall be properly disposed of in compliance with local, state, and federal regulations.

4.5—MEASUREMENT AND PAYMENT

4.5.1—Method of Measurement

4.5.1.1—Timber, Steel, and Concrete Piles

4.5.1.1.1—Piles Furnished

The quantities of pile to be paid for shall be the sum of the lengths in feet. The piles shall be of the types and lengths indicated in the contract documents or ordered in writing by the Engineer, furnished in compliance with the material requirements of these Specifications and stockpiled or installed in good condition at the site of the work by the Contractor, and accepted by the Engineer.

When extensions of piles are necessary, the extension length ordered in writing by the Engineer shall be included in the total length of piling furnished.

4.5.1.1.2—Piles Driven

The quantities of driven piles of each type to be paid for shall be the quantities of acceptable piles of each type that were driven.

4.5.1.2—Pile Splices and Pile Shoes

Where pile splices or protective pile tip shoes are shown in the contract documents, the number of pile splices or shoes measured for payment shall be those shown in the contract documents, or ordered in writing by the Engineer, and actually installed on piles used in the work. No payment shall be made for splices or shoes used at the option of the Contractor. When not shown in the contract documents, pile splices or shoes ordered by the Engineer shall be paid for as extra work.

4.5.1.3—Static Load Tests

The quantity of static load tests to be paid for shall be the number of load tests completed.

Test piles for static load tests, whether incorporated into the permanent structure or not, shall be measured as provided for the test piles furnished and test piles driven and shall be paid for under the appropriate pay item.

C4.5.1.3

Not all static load tests yield the predicted results. A properly performed test that fails to yield the predicted results is still a successfully completed test and should be paid for as such.

4.5.2—Basis of Payment

4.5.2.1—Unit Cost Contracts

The quantities, determined as specified, shall be paid for at the contract documents' price per unit of measurement, respectively, for each of the general pay items listed below, for each size and type of pile shown in the contract documents.

Pay Item	*Pay Unit*
Mobilization and Demobilization	Lump Sum
Piles Furnished	LF or Each
Piles Driven	LF or Each
Test Piles, Furnished	LF or Each
Test Piles, Driven	LF or Each
Static Pile Load Test	Each
Dynamic Pile Test (during driving)	Each
Dynamic Pile Test (during restrike)	Each
Splices	Each
Pile Shoes	Each
Predrilling or Preaugering	LF or Each
Jetting	LF or Each
Cut-off (over 5.0 ft lengths only)	Each
Spudding (Punching)	Per Hr.
Delays, Downtime, or Out-of-Sequence Moves	Per Hr.

Note: LF = linear foot

Payment for piles furnished shall be taken to include full compensation for all costs involved in the furnishing and delivery of all piles to the project site.

Payment for piles driven shall be taken to include full compensation for all costs involved in the actual driving and for all costs for which compensation is not provided under other specified pay items involved with the furnishing of labor, equipment, and materials used to install the piles.

Payment for static or dynamic tests shall be taken to include full compensation for providing labor, equipment, and materials needed to perform the load tests as specified. If the dynamic pile test requires substantial repositioning or idle time of the crane, additional compensation for out-of-sequence moves shall be paid at the bid rate for this item.

C4.5.2.1

Mobilization and demobilization is generally considered to be for one each, and grouped as a single priced item (lump sum). For jobs that could have more than one mobilization and demobilization, such as sequenced jobs, it would be appropriate to use the term "each," rather than "lump sum."

It also may be appropriate to separate mobilization and demobilization prices for major subcontractors.

Piles whose price per foot changes with length (such as timber piles) do not lend themselves well to unit price contracts. In the event that piles exceed the bid length by 5.0 ft or more, an adjustment in unit prices is probably appropriate. Longer piles may cause transportation problems.

Dynamic pile tests to evaluate hammer performance and driving stresses during driving require a brief interruption to the driving of the Test Pile to attach the sensors to the pile.

Dynamic pile tests to evaluate capacity often are made during restrike to take advantage of the common setup or guard against relaxation. If the restrike is for a pile nearby the current crane location, the interruption will be brief.

Some pay items (such as pile shoes) can be included in the "furnished pile" pay item, if established before bid.

Spudding is generally driving or dropping a steel member to create a pathway through obstructions.

If cut-off lengths become excessive, additional costs will be incurred.

Delays or downtime caused by the Owner, agent(s), or subcontractor(s), and out-of-sequence moves will be charged at the rate established in the pay item.

Payment for pile splices, shoes, or lugs shall be taken to include full compensation for all costs involved with furnishing all materials and performing the work involved with attaching or installing splices, shoes, or lugs to the piles.

Payment for predrilling, jetting, or spudding shall be taken to include full compensation for providing labor, equipment, and materials needed to perform these pile installation aid procedures.

Payment for cut-off shall be taken to include full compensation for providing labor and equipment needed to adapt the pile top to the specified cut-off elevation and to properly dispose of the removed material.

Payment for delays or downtime shall be taken to include full compensation for unproductive time caused by the owner, his agent, or his subcontractor.

4.5.2.2—Lump Sum Contracts

Payment shall be a lump sum for the piles as specified in the contract documents.

There shall be no change in contract price if the specified pile does not drive to the plan-tip elevation due to refusal caused by soil strata or obstructions.

The bid form shall include the following items to accommodate changes in pile quantities. If the Engineer determines that pile lengths or number of piles are to be changed, the lump sum shall be adjusted as follows:

Pay Item	*Pay Unit*
Increase (Add)	
Longer piles, up to 5.0 ft	LF
Longer piles, 6.0 to 10.0 ft	LF
Decrease (Deduct)	
Shorter piles, up to 5.0 ft	LF
Shorter piles, 6.0 to 10.0 ft	LF
Increase (Add)	
Added piles	Each
Decrease (Deduct)	
Deleted piles	Each

Added or deleted piles apply only up to 10 percent of the original quantity. Changes greater than this shall require a change in the unit prices. Pile length changes of more than 10.0 ft shall require renegotiation of the contract.

If changes occur during driving, unanticipated work shall be paid as an extra.

C4.5.2.2

This method of bidding may be useful in design build or for rapid construction situations. Many private sector projects are bid on a lump sum basis

The specification may call for predrilling or jetting to facilitate penetration.

The unit prices apply to piles before manufacture. Pile lengths are determined by the Engineer. No credit is due for any length of properly installed pile left above cut-off elevation. In the case of piles that are normally supplied in stock increments (normally 5.0 ft), the unit price is to be applied to the entire length of pile ordered (e.g., a 31.0-ft pile may be paid as 35.0 ft due to order lengths).

4.6—REFERENCES

AASHTO. 2002. *Standard Specifications for Highway Bridges*, 17th Edition, HB-17, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2009. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 29th Edition, HM-29. American Association of State Highway and Transportation Officials, Washington, DC. Includes AASHTO M, R, and T standards, which are also available individually in downloadable form.

ACI. 2005. “Durability Requirements,” *Building Code Requirements for Structural Concrete and Commentary*, ACI 318, American Concrete Institute, Farmington Hills, MI, Chapter 4.

Allen, Tony M. 2005. *Development of Geotechnical Resistance Factors and Downdrag Load Factors for LRFD Foundation Strength Limit State Design*, FHWA-NHI-05-052, National Highway Institute, Federal Highway Administration, U.S. Department of Transportation, Washington, DC, February 2005.

API. March 2004. *Specification for Line Pipe*, 43rd Edition, API 5L, American Petroleum Institute, Washington, DC.

AWPA. 2002. *Standard for the Care of Preservative-Treated Wood Products*, AWPA M4-02, American Wood Protection Association, Birmingham, AL.

AWPA. 2004. *Use Category System: User Specification for Treated Wood*. APWA U1-06, American Wood Protection Association, Birmingham, AL.

Collin, James G., Ph.D., P.E. 2002. *Timber Piling Design and Construction Manual*, Timber Piling Council in conjunction with American Wood Preservers Institute.

Engineering News-Record formula. Based on a formula published in 1888 by A. M. Wellington, editor of *Engineering News-Record*, New York, NY.

Paikowsky, S. G., B. Birigisson, M. McVay, T. Nguyen, C. Kuo, G. Baecher, B. Ayyab, K. Stenersen, K. O’Malley, L. Chernauskas, and M. O’Neill. 2004. *Load and Resistance Factor Design (LRFD) for Deep Foundations*, NCHRP Final Report 507, Transportation Research Board, Washington, DC.

PCI. 1999. *Manual for Quality Control for Plants and Production of Structural Precast Concrete Products*, 4th Edition, PCI MNL-116. Precast/Prestressed Concrete Institute, Chicago, IL.

PCI. 2004. “Precast Prestressed Concrete Piles,” *PCI Bridge Design Manual*, BM-20-04, Precast/Prestressed Concrete Institute, Chicago, IL, Chapter 20.

Peck, R. B., W. E. Hansen, and T. H. Thornburn. 1974. *Foundation Engineering*, 2nd Edition, John Wiley and Son, Inc., New York, NY.

TABLE OF CONTENTS

SECTION 5

DRILLED SHAFTS

5.1—DESCRIPTION

This item of work shall consist of furnishing all materials, labor, tools, equipment, services, and incidentals necessary to construct the drilled shafts in accordance with the contract documents and this Specification.

5.2—SUBMITTALS, APPROVALS AND MEETINGS

At least four weeks prior to the start of drilled shaft construction, the Contractor shall submit four copies of a project reference list to the Engineer for approval, verifying the successful completion by the Contractor of at least three separate foundation projects within the last five years with drilled shafts of similar size (diameter and depth) and difficulty to those shown in the Plans, and with similar subsurface geotechnical conditions. A brief description of each project and the owner's contact person's name and current phone number shall be included for each project listed.

C5.2

Electronic versions of all submittals should be encouraged. All submissions should be made concurrently to all on the distribution list.

5.2.1—Experience and Personnel

At least two weeks prior to the start of drilled shaft construction, the Contractor shall submit four copies of a list identifying the on-site supervisors and drill rig operators assigned to the project to the Engineer for approval. The list shall contain a detailed summary of each individual's experience in drilled shaft excavation operations, and placement of assembled reinforcing cages and concrete in drilled shafts.

- On-site supervisors shall have a minimum two years experience in supervising construction of drilled shaft foundations of similar size (diameter and depth) and difficulty to those shown in the Plans, and similar geotechnical conditions to those described in the geotechnical report. The work experience shall be direct supervisory responsibility for the on-site drilled shaft construction operations. Project management level positions indirectly supervising on-site drilled shaft construction operations shall not be considered to be acceptable for this experience requirement.
- Drill rig operators shall have a minimum one year experience in construction of drilled shaft foundations.

The Engineer shall approve or reject the Contractor's qualifications and field personnel within ten working days after receipt of the submission. Work shall not be started on any drilled shaft until the Contractor's qualifications and field personnel are approved by the Engineer. The Engineer may suspend the drilled shaft construction if the Contractor substitutes unapproved field personnel without prior approval by the Engineer. The Contractor shall be fully liable for the additional costs resulting from the suspension of work and no adjustments in contract time resulting from such suspension of work shall be allowed.

5.2.2—Drilled Shaft Installation Plan

At least four weeks prior to the start of drilled shaft construction, the Contractor shall submit four copies of a drilled shaft installation plan narrative for acceptance by the Engineer. In preparing the narrative, the Contractor shall reference the available subsurface geotechnical data provided in the contract boring logs and any geotechnical report(s) prepared for this project. This narrative shall provide at a minimum the following information:

- Description of overall construction operation sequence and the sequence of drilled shaft construction when in groups or lines.
- A list, description and capacities of proposed equipment, including but not limited to cranes, drills, augers, bailing buckets, final cleaning equipment, and drilling unit. As appropriate, the narrative shall describe why the equipment was selected and describe equipment suitability to the anticipated site and subsurface conditions. The narrative shall include a project history of the drilling equipment demonstrating the successful use of the equipment on shafts of equal or greater size in similar subsurface geotechnical conditions.
- Details of drilled shaft excavation methods, including proposed drilling methods, methods for cleanout of the bottom of the excavation hole, and a disposal plan for excavated material, drilling slurry, and regulated/hazardous waste (if applicable). If appropriate, this shall include a review of method suitability to the anticipated site and subsurface geotechnical conditions, including boulders and obstruction removal techniques if such are indicated in the contract subsurface geotechnical information.
- Details of the method(s) to be used to ensure drilled shaft hole stability (i.e., prevention of caving, bottom heave, etc., using temporary casing, slurry, or other means) during excavation and concrete placement. The details shall include a review of method suitability to the anticipated site and subsurface geotechnical conditions.

C5.2.2

The agency should recognize that the depth of the requested narrative should be appropriate to the complexity of the project.

When the Contract requires a minimum penetration into a bearing layer, as opposed to a specified shaft tip elevation, and the bearing layer elevation at each shaft cannot be accurately determined, insert the following: “Variations in the bearing layer elevation from that shown in the Plans are anticipated. The Contractor shall have equipment on-site capable of excavating an additional 20 percent of depth below that shown in the Plans.”

Where the installation of drilled shafts will take place adjacent to existing sensitive installations prone to damage due to the instability of uncased drilled shaft holes or where subsurface soil strata do not lend themselves to an uncased construction technique due to stability concerns, the owner may specify the use and limits of the temporary casing.

In areas of the country that are subject to high seismic forces, the designer may limit the drilled shaft diameter to that used in design calculations. Such limitations may include restrictions on telescoping of casing or limiting the amount of excavation prior to the introduction of casing.

- Detailed procedures for mixing, using, maintaining, and disposing of the slurry shall be provided. A detailed mix design (including all additives and their specific purpose in the slurry mix), and a discussion of its suitability to the anticipated subsurface geotechnical conditions, shall also be provided for the proposed slurry.
- The submittal shall include a detailed plan for quality control of the selected slurry, including tests to be performed, test methods to be used, and minimum and/or maximum property requirements which must be met to ensure that the slurry functions as intended, considering the anticipated subsurface conditions and shaft construction methods, in accordance with the slurry manufacturer's recommendations and these Specifications.

As a minimum, the slurry quality control plan shall include the following tests:

Property	Test Method
Density (lb/ft^3)	Mud Weight (Density), API 13B-1, Section 1
Viscosity (s/qt)	Marsh Funnel and Cup, API 13B-1, Section 2.2
pH	Glass Electrode, pH Meter or pH Paper
Sand Content (%)	Sand, API 13B-1, Section 5

- Reinforcing steel shop drawings, details of reinforcement placement including type and location of all splices, reinforcement cage support and centralization methods.
- Where casings are proposed or required, casing dimensions and detailed procedures for permanent casing installation, temporary casing installation and removal, and methods of advancing the casing along with the means to be utilized for excavating the drilled shaft hole in accordance with Article 5.4.
- Where temporary casings are used, details of the method to extract the temporary casings and maintaining shaft reinforcement in proper alignment and location and maintaining the concrete slump to keep concrete workable during casing extraction.

- Details of concrete placement, including a time schedule, proposed operational procedures for pumping, and a sample uniform yield form to be used by the Contractor for plotting the volume of concrete placed versus the depth of shaft for all shaft concrete placement.
- Procedure of tremie methods used.

Anomalies due to head loss of tremie need to be addressed as to the procedure to avoid, inspect and repair if needed.

- The method to be used to form a horizontal construction joint during concrete placement.

Horizontal construction joints may become necessary due to equipment breakdown or loss of concrete supply during drilled shaft concrete placement.

Where top of drilled shafts cutoff elevations are below the water surface, a sealed cofferdam arrangement is generally required to construct the joint.

- Where applicable, a description of the material to be used to temporarily backfill a drilled shaft excavation hole during a stoppage of the excavation operation, as well as the method used to place and remove the material.
- A description of the method and materials that will be used to fill or eliminate all voids below the top of shaft between the plan shaft diameter and excavated shaft diameter, or between the shaft casing and surrounding soil, if permanent casing is specified.

In seismic design situations, the backfill material and placement method should attempt to replicate the existing ground conditions as closely as possible.

- Methods of the removal and disposal of contaminated concrete.

The Engineer shall evaluate the shaft installation plan for conformance with the Contract Plans and Specifications within ten working days after receipt of the submission. At the option of the Owner, a Shaft Installation Plan Submittal Meeting may be scheduled following review of the Contractor's initial submittal of the plan. Those attending the Shaft Installation Plan Submittal Meeting shall include the following:

- The superintendent, on-site supervisors, and other Contractor personnel involved in the preparation and execution of the drilled shaft installation plan.
- The Project Engineer and Owner's personnel involved with the structural, geotechnical, and construction review of the shaft installation plan together with Owner's personnel who will provide inspection and oversight during the drilled shaft construction phase of project.

5.2.3—Slurry Technical Assistance

If slurry is used to construct the drilled shafts, the Contractor shall provide, or arrange for, technical assistance from the slurry Manufacturer as specified in Article 5.4.3.4.1. The Contractor shall submit four copies of the following to the Engineer:

- The name and current phone number of the slurry manufacturer's technical representative assigned to the project.
- The name(s) of the Contractor's personnel assigned to the project and trained by the slurry manufacturer's technical representative in the proper use of the slurry. The submittal shall include a signed training certification letter from the slurry manufacturer for each individual, including the date of the training.

5.2.4—Approvals

Work shall not begin until all the required submittals have been accepted in writing by the Engineer. All procedural acceptances given by the Engineer shall be subject to trial in the field and shall not relieve the Contractor of the responsibility to satisfactorily complete the work.

5.2.5—Drilled Shaft Preconstruction Conference

A shaft preconstruction conference shall be held at least five working days prior to the Contractor beginning any shaft construction work at the site to discuss investigative boring information, construction procedures, personnel, equipment to be used, and other elements of the accepted shaft installation plan as specified in Article 5.2.2. If slurry is used to construct the shafts, the frequency of scheduled site visits to the project site by the slurry manufacturer's representative shall be discussed. Those attending shall include:

- The superintendent, on-site supervisors, and other key personnel identified by the Contractor as being in charge of excavating the shaft, placing the casing and slurry as applicable, placing the steel reinforcing bars, and placing the concrete. If slurry is used to construct the shafts, the slurry manufacturer's representative and a Contractor's employee trained in the use of the slurry, as identified to the Engineer in accordance with Article 5.4.3.4.1, shall also attend.
- The Project Engineer, key inspection personnel, and appropriate representatives of the Owner.

C5.2.5

Meetings may need to be held in order to obtain agreement on the shaft submittal but the shaft conference should only be held after approval.

Attendees on Owner's behalf should include representatives having experience in construction, materials, structural and geotechnical design.

If the Contractor's key personnel change, or if the Contractor proposes a significant revision of the approved shaft installation plan, an additional conference may be held at the request of the Engineer before any additional shaft construction operations are performed.

5.3—MATERIALS

5.3.1—Concrete

Concrete used in the construction of drilled shafts shall conform to Article 8.2, "Classes of Concrete." The concrete slump shall be as follows:

Dry placement methods	6.0–8.0 in.
Casing removal methods	8.0–10.0 in.
Tremie placement methods	8.0–10.0 in.

Slump loss of more than 4 in.shall not be permitted during the period equal to the anticipated pour period plus 2.0 h. A minimum of 6.0 in. slump shall be required for this time period. Slump life may be extended through the use of retarders and mid-range water reducers if approved by the Engineer.

C5.3.1

To achieve maximum workability the following mix characteristics are recommended:

- A maximum course aggregate of 0.375 in. in wet hole pours or shafts with dense reinforcing configurations.
- Use of rounded in lieu of crushed aggregates.
- Consider using fly ash as a cement replacement and as a fluidfier.

In cases where dense reinforcing configurations close to the minimum opening size limits are specified, it is suggested that slumps of 8.0–10.0 in. be used even in dry placement methods.

5.3.2—Reinforcing Steel

Reinforcing steel used in the construction of shafts shall conform to AASHTO M 31M/M 31.

Reinforcing steel shall be bundled in order to meet the clear spacing requirements between the vertical reinforcement bars. Rolled hoops or bundled spirals shall be used in order to maximize clear space between horizontal reinforcement.

C5.3.2

Current practice regarding minimum clear space between reinforcement elements is to have clear distance between parallel longitudinal and parallel transverse reinforcing bars not be less than five times the maximum aggregate size or 5.0 in., whichever is greater, per Article 5.13.4.5.2 of the *AASHTO LRFD Bridge Design Specifications*. Recent research indicates that clear distance between parallel longitudinal and parallel transverse reinforcing bars of ten times the maximum aggregate size provides for improved flow of concrete through the cage to ensure the integrity of the concrete outside of the reinforcing cage. Prevailing practice varies regarding minimum opening size amongst the various owners. Their experience indicates that the current requirements contained in the *AASHTO LRFD Bridge Design Specifications* produce desired results when the requirements in the construction specifications are fully applied.

Reinforcing steel for shafts poured inside temporary casings should not have hooks to the outside.

5.3.3—Casings

All permanent structural casing shall be of steel conforming to ASTM A36/A36M or ASTM A252 Gr. 2 unless specified otherwise in the Plans. All splicing of permanent structural casing shall be in accordance with Article 6.13.3, "Welded Connections," of the *AASHTO LRFD Bridge Design Specifications*.

C5.3.3

Permanent structural casing is defined as casing designed as part of the shaft structure providing stiffness or load carrying capacity and installed to remain in place after construction is complete.

All permanent casing shall be of ample strength to resist damage and deformation from transportation and handling, installation stresses, and all pressures and forces acting on the casing. For permanent nonstructural casing, corrugated casing may be used.

All temporary casing shall be a smooth wall structural steel, except where corrugated metal pipe is shown in the Plans as an acceptable alternative material. All temporary casing shall be of ample strength to resist damage and deformation from transportation and handling, installation and extraction stresses, and all pressures and forces acting on the casing. The casing shall be capable of being removed without deforming and causing damage to the completed shaft, and without disturbing the surrounding soil.

Temporary casing is defined as casing installed to facilitate shaft construction only, is not designed as part of the shaft structure, and is completely removed after shaft construction is complete, unless otherwise shown in the Plans.

Permanent nonstructural casing is defined as casing designed to remain in place to assist in the construction of the drilled shaft.

The casing shall be watertight and clean prior to placement in the excavation.

The outside diameter of the casing shall not be less than the specified diameter of the shaft.

Where seismic design requires that the shaft be constructed to the diameters indicated in the drawings, the Engineer shall specify that telescoping casing will not be allowed.

In cases where seismic design governs, the inside diameter of the casing shall not be greater than the specified diameter of the shaft plus 6.0 in., unless otherwise specified in the plans.

Where the minimum thickness of the casing is specified in the Plans, it shall be considered to satisfy structural design requirements only. The Contractor shall increase the casing thickness from the minimum specified thickness as necessary to satisfy the construction installation requirements.

5.3.4—Mineral Slurry

Mineral slurry shall be used in conformance with the quality control plan specified in Article 5.2.2.

Mineral slurry shall conform to the following requirements:

Property	Test	Requirement
Density (lb/ft^3)	Mud Weight (Density) API 13B-1, Section 1	64.3 to 72
Viscosity (s/qt)	Marsh Funnel and Cup API 13B-1, Section 2.2	28 to 50
pH	Glass Electrode, pH Meter, or pH Paper	8 to 11
Sand Content (%) (immediately prior to placing concrete)	Sand API 13B-1, Section 5	4.0 max

C5.3.4

Unit weights stated are exclusive of weighting agents that may be proposed by the contractor with the agreement of the slurry manufacturer's representative.

Some slurry systems incorporate a weighting agent when utilizing salt water in slurry. This may add up to 5 lb/ft^3 to the unit weight.

Where it is necessary to use a mineral slurry in salt water applications, it is recommended that attapulgite or sepiolite be used in lieu of bentonite.

When approved by the Engineer, slurry may be used in salt water, and the allowable densities may be increased up to 2 lb/ft^3. Slurry temperature shall be at least 40°F when tested.

5.3.5—Polymer Slurry

Polymer slurries shall be used in conformance with the manufacturer's recommendations and shall conform to the quality control plan specified in Article 5.2.2. Only synthetic slurry systems which have been approved by the Owner may be used. The polymer slurry shall conform to the following requirements:

Property	Test	Requirement
Density (lb/ft^3)	Mud Weight (Density) API 13B-1, Section 1	64 max
Viscosity (s/qt)	Marsh Funnel and Cup API 13B-1, Section 2.2	32 to 135
pH	Glass Electrode, pH Meter or pH Paper	8 to 11.5
Sand Content (%) (immediately prior to placing concrete)	Sand API 13B-1, Section 5	1.0 max

When approved by the Engineer, polymer slurry may be used in salt water, and the allowable densities may be increased up to 2.0 lb/ft^3.

The sand content of polymer slurry prior to final cleaning and immediately prior to placing concrete shall be less than or equal to 1.0 percent, in accordance with American Petroleum Institute API 13B-1, Section 5. Slurry temperature shall be at least 40°F when tested.

5.3.6—Water Slurry

Water may be used as slurry when casing is used for the entire length of the drilled hole.

Water slurry shall conform to the following requirements:

Property	Test	Requirement
Density (lb/ft^3)	Mud Weight (Density) API 13B-1, Section 1	64 max
Sand Content (%)	Sand API 13B-1, Section 5	1.0 max

C5.3.6

A water slurry is water that is maintained as clean as possible during its use. The mixing of water with naturally occurring site materials is not recommended.

When approved by the Engineer, slurry may be used in salt water, and the allowable densities may be increased up to 2 lb/ft^3. Slurry temperature shall be at least 40°F when tested.

5.3.7—Access Tubes for Cross-Hole Sonic Log Testing

Access tubes for cross-hole sonic log testing shall be steel pipe of 0.145 in. minimum wall thickness and at least 1.5 in. inside diameter.

The access tubes shall have a round, regular inside diameter free of defects and obstructions, including all pipe joints, in order to permit the free, unobstructed passage of 1.3 in. maximum diameter source and receiver probes used for the cross-hole sonic log tests. The access tubes shall be watertight, free from corrosion with clean internal and external faces to ensure good bond between the concrete and the access tubes. The access tubes shall be fitted with watertight threaded caps on the bottom and the top.

5.3.8—Grout

Grout for filling the access tubes at the completion of the cross-hole sonic log tests shall be a neat cement grout with a maximum water/cement ratio of 0.45.

5.4—CONSTRUCTION

5.4.1—Drilled Shaft Excavation

Shafts shall be excavated to the required depth as shown in the Plans or as directed by the Engineer. Once the excavation operation has been started, the excavation shall be conducted in a continuous operation until the excavation of the shaft is completed, except for pauses and stops as noted, using approved equipment capable of excavating through the type of material expected. Pauses during this excavation operation, except for casing splicing and removal of obstructions, shall not be allowed. The Contractor shall provide temporary casing at the site in sufficient quantities to meet the needs of the anticipated construction method.

Pauses, defined as interruptions of the excavation operation, may be allowed only for casing splicing and removal of obstructions. Shaft excavation operation interruptions not conforming to this definition shall be considered stops.

If the shaft excavation is not complete at the end of the shift or series of continuous shifts, the shaft excavation operation may be stopped, provided the Contractor, before the end of the work day, protects the shaft as indicated in Article 5.4.2.

If slurry is present in the shaft excavation, the Contractor shall conform to the requirements of Article 5.4.3.4.2 regarding the maintenance of the minimum level of drilling slurry throughout the stoppage of the shaft excavation operation, and shall recondition the slurry to the required slurry properties in accordance with Article 5.3 prior to recommencing shaft excavation operations.

5.4.2—Drilled Shaft Excavation Protection

Shaft excavations shall not be left open overnight unless cased full depth or otherwise protected against sidewall instability. The use of slurry to protect a shaft during a drilling stoppage or overnight shutdown may be approved by the Engineer.

Casing of shafts in stable rock formations during stoppages shall not be required.

C5.4.2

An open excavation is defined as a shaft that has not been filled with concrete, or temporarily backfilled with a material approved by the Engineer in accordance with Article 5.2.2 or protected in accordance with Article 5.4.3.

5.4.3—Drilled Shaft Excavation Protection Methods

The Contractor shall bear full responsibility for selection and execution of the method(s) of stabilizing and maintaining the shaft excavation. The walls and bottom of the shaft excavation shall be protected so that sidewall caving and bottom heave are prevented from occurring, and so that the soil adjacent to the shaft is not disturbed. The Contractor may excavate the shaft without excavation protection provided it can be demonstrated that the soil/rock is stable within or above the zones of seepage.

C5.4.3

Project specific requirements may dictate that specific shaft excavation protection methods should be used. For example, the Contract may require that permanent casing be used if very soft soils are present that will not support the weight of the wet concrete when the casing is extracted, or if the foundations for an immediately adjacent structure are present and must be protected from movement.

Acceptable protection methods include the use of casing, drilling slurry, or both.

5.4.3.1—Temporary Casing Construction Method

In stable soils, the Contractor shall conduct casing installation and removal operations and shaft excavation operations such that the adjacent soil outside the casing and shaft excavation for the full height of the shaft is not disturbed.

If the Contractor is utilizing casing that is adequately sealed into competent soils such that the water cannot enter the excavation, the Contractor may, with the Engineer's approval, continue excavation in soils below the water table provided the water level within the casing does not rise or exhibit flow.

As the temporary casing is withdrawn, a sufficient head of fluid concrete shall be maintained to ensure that water or slurry outside the temporary casing will not breach the column of freshly placed concrete.

Casing extraction shall be at a slow, uniform rate with the pull in line with the axis shaft. Excessive rotation of the casing shall be avoided to limit deformation of the reinforcing steel cage.

The Contractor shall remove all temporary casings from the excavation as concrete placement is completed, unless permission has been received from the Engineer to leave specified temporary casings in place.

C5.4.3.1

Disturbed soil is defined as soil whose geotechnical properties have been changed from those of the original in-situ soil, and whose altered condition adversely affects the performance of the shaft foundation.

Movement of the casing by rotation, exerting downward pressure, and tapping to facilitate extraction or extraction with a vibratory hammer is acceptable. The duration of vibration during casing extraction with a vibratory hammer should be limited in order to minimize potential segregation of the concrete.

5.4.3.2—Permanent Casing Construction Method

Where permanent casing is specified, excavation shall conform to the specified outside diameter of the shaft. After the casing has been filled with concrete, all void space occurring between the casing and shaft excavation shall be filled with a material which approximates the geotechnical properties of the in-situ soils, in accordance with the shaft installation plan specified in Article 5.2.2 and as approved by the Engineer.

Tops of permanent casings for the shafts shall be removed to the top of the shaft or finished ground line, whichever is lower, unless the top of the permanent casing is shown in the Plans at a different elevation. For those shafts constructed within a permanent body of water, tops of permanent casings for shafts shall be removed to the low water elevation, unless directed otherwise by the Engineer.

C5.4.3.2

As outlined in Article C5.2.2 the backfill of accidental over-excavation outside the casing may require the use of materials which closely approximate the lateral response of the native soils.

In other cases the engineer may require that foundation materials be sealed against evaporation or water introduction. In those cases, the engineer may require that any annular space around a permanent casing be filled with structural grout.

5.4.3.3—Alternative Casing Methods

Shaft casing shall be equipped with cutting teeth or a cutting shoe and installed by either rotating or oscillating the casing.

C5.4.3.3

This alternative may be specified if vibratory placement or extraction of casing is not permitted.

Soils consisting of gravel and cobble mixtures, or matrix supported boulders where the matrix is loose and granular, tend to be susceptible to caving and sloughing, and usually require casing to stabilize the shaft side walls. These materials also make vibratory casing installation very difficult and risky for both the Contracting Agency and the Contractor. In such cases, the installation of temporary and/or permanent casing by either a rotating or an oscillating method may be required.

5.4.3.4—Slurry

The Contractor shall use slurry, in accordance with Article 5.3, to maintain a stable excavation during excavation and concrete placement operations once water begins to enter the shaft excavation and remain present.

The Contractor shall use slurry to maintain stability during shaft excavation and concrete placement operations in the event water begins to enter the shaft excavation at a rate greater than 12.0 in./h; or if the Contactor is not able to restrict the amount of water in the shaft to less than 3.0 in. prior to concreting, or to equilibrate water pressure on the sides and base of the shaft excavation when ground water is encountered or anticipated based on the available subsurface data.

C5.4.3.4

Many situations will require the contractor to utilize both slurry and casing techniques in the same hole.

5.4.3.4.1—Slurry Technical Assistance

If slurry is used, the manufacturer's representative, as identified to the Engineer in accordance with Article 5.2.3, shall:

- provide technical assistance for the use of the slurry,
- be at the site prior to introduction of the slurry into a drilled hole, and
- remain at the site during the construction and completion of a minimum of one shaft to adjust the slurry mix to the specific site conditions.

After the manufacturer's representative is no longer present at the site, the Contractor's employee trained in the use of the slurry, as identified to the Engineer in accordance with Article 5.2.3, shall be present at the site throughout the remainder of shaft slurry operations for this project to perform the duties specified above.

5.4.3.4.2—Minimum Level of Slurry in the Excavation

Where slurry is used to maintain a stable excavation, the slurry level in the excavation shall be maintained to obtain hydrostatic equilibrium throughout the construction operation at a height required to provide and maintain a stable hole, but not less than 5.0 ft. above the water table.

The Contractor shall provide casing, or other means, as necessary to meet these requirements.

The slurry level shall be maintained above all unstable zones a sufficient distance to prevent bottom heave, caving or sloughing of those zones.

Throughout all stops in shaft excavation operations, the Contractor shall monitor and maintain the slurry level in the excavation to the greater of the following elevations:

- no lower than the water level elevation outside the shaft, or
- an elevation as required to provide and maintain a stable hole.

C5.4.3.4.2

Recommended slurry levels are as follows:

- not less than 5.0 ft. for mineral slurries,
- not less than 10.0 ft. for water slurries, and
- not less than 10.0 ft. for polymer slurries, except when a lesser dimension is specifically recommended by the slurry manufacturer for the site conditions and construction.

Artesian conditions may require slurry levels even greater than noted for the above slurry types.

5.4.3.4.3—Cleaning Slurry

The Contractor shall clean, re-circulate, de-sand, or replace the slurry as needed in order to maintain the required slurry properties in accordance with Articles 5.3.4 and 5.3.5. Sand content shall be within specified limits as specified in the Contract, prior to concrete placement.

5.4.4—Obstructions

When obstructions are encountered, the Contractor shall notify the Engineer promptly. When efforts to advance past the obstruction to the design shaft tip elevation results in a reduction in the rate of advance and/or change in approved means and methods relative to the approved shaft installation plans, then the Contractor shall remove, bypass or break up the obstruction under the provisions of Article 5.5.1.3.

C5.4.4

An obstruction is defined as a specific object (including, but not limited to, boulders, logs, and man-made objects) encountered during the shaft excavation operation which prevents or hinders the advance of the shaft excavation.

If the agency chooses to limit obstruction removal to "unknown obstructions" it places a heavy burden on the Foundation Report to accurately describe the obstructions that a contractor should anticipate.

5.4.5—Protection of Existing Structures

The Contractor shall control operations to prevent damage to existing structures and utilities. Preventative measures shall include, but are not limited to, selecting construction methods and procedures that will prevent excessive caving of the shaft excavation and monitoring and controlling the vibrations from the driving of casing or sheeting, drilling of the shaft, or from blasting, if permitted.

C5.4.5

This Section will be used for site-specific issues such as shallow foundations adjacent drilled shaft work or adjacent vibration sensitive installations. The Agency may choose to specify casing installation in advance of excavation or may restrict the amount of vibration a contractor may use to install or remove casing or perform drilling operations.

5.4.6—Slurry Sampling and Testing

Mineral slurry and polymer slurry shall be mixed and thoroughly hydrated in slurry tanks, lined ponds, or storage areas. The Contractor shall draw sample sets from the slurry storage facility and test the samples for conformance with the appropriate specified material properties before beginning slurry placement in the drilled hole. Slurry shall conform to the quality control plan included in the shaft installation plan in accordance with Article 5.2.2 and approved by the Engineer. A sample set shall be composed of samples taken at mid-height and within 2.0 ft. of the bottom of the storage area.

The Contractor shall sample and test all slurry in the presence of the Engineer, unless otherwise directed. The date, time, names of the persons sampling and testing the slurry, and the results of the tests shall be recorded. A copy of the recorded slurry test results shall be submitted to the Engineer at the completion of each shaft, and during construction of each shaft when requested by the Engineer.

Sample sets of all slurry, composed of samples taken at mid-height and within 2.0 ft. of the bottom of the shaft, shall be taken and tested during drilling as necessary to verify the control of the properties of the slurry. As a minimum, sample sets of polymer slurry shall be taken and tested at least once every 4.0 h after beginning its use during each shift.

Sample sets of all slurry, as specified, shall be taken and tested immediately prior to placing concrete.

The Contractor shall demonstrate to the satisfaction of the Engineer that stable conditions are being maintained. If the Engineer determines that stable conditions are not being maintained, the Contractor shall immediately take action to stabilize the shaft. The Contractor shall submit a revised shaft installation plan that addresses the problem and prevents future instability. The Contractor shall not continue with shaft construction until the damage that has already occurred is repaired in accordance with the Specifications, and until receiving the Engineer's approval of the revised shaft installation plan.

5.4.7—Drilled Shaft Excavation Inspection

The Contractor shall use appropriate means, such as a cleanout bucket, air lift, or hydraulic pump, to clean the bottom of the excavation of all drilled shafts. For wet drilled shaft excavations in soils, the base of the excavation shall be covered with not more than 3.0 in. of sediment or loose or disturbed material just prior to placing concrete. For dry drilled shaft excavations in soils, the base of the excavation shall be covered with not more than 1.5 in. sediment or loose or disturbed material just prior to placing concrete. For wet or dry drilled shaft excavations in rock, the base of the excavation shall be covered with not more than 0.5 in. for 50 percent of the base area of sediment or loose or disturbed material just prior to placing concrete.

The excavated shaft shall be inspected and approved by the Engineer prior to proceeding with construction. The bottom of the excavated shaft shall be sounded with an airlift pipe, a tape with a heavy weight attached to the end of the tape, or other means acceptable to the Engineer to determine that the shaft bottom meets the requirements in the Contract.

C5.4.7

The amount of sediment left on the base of the shaft can be determined by using a weighted tape and bouncing it on the bottom of the shaft. If the weight strikes the bottom of the excavation with an immediate stop, the shaft has little or no sediment. If the weight slows down and sinks to a stop, then excessive sediment exists.

The cleanliness of the shaft base is a requirement not only for end bearing and settlement considerations but also to obtain an uncontaminated concrete pour.

Mucker buckets, airlifts, and special coring tools that have gates that can be opened and closed are typically used. If cleanliness of less than 2.0 in. of loose material is required, the inspection must utilize camera techniques that enable visual inspection. Flocculents may be required for wet excavation.

5.4.8—Assembly and Placement of Reinforcing Steel

The Contractor shall show bracing and any extra reinforcing steel required for fabrication of the cage on the shop drawings. The Contractor will be responsible for engineering the temporary support and bracing of reinforcing cages to ensure that they maintain their planned configuration during assembly, transportation, and installation. As a minimum:

- At least 4 vertical bars of each cage, equally spaced around the circumference, shall be tied at all reinforcement intersections with double wire ties.
- At least 25 percent of remaining reinforcement intersections in each cage shall be tied with single wire ties. Tied intersections shall be staggered from adjacent ties.
- Bracing shall be provided to prevent collapse of the cage during assembly, transportation, and installation.

Successful completion of these minimum baseline requirements for reinforcement cage will in no way relieve the Contractor of full responsibility for engineering the temporary support and bracing of the cages during construction.

The reinforcement shall be carefully positioned and securely fastened to provide the minimum clearances listed below, and to ensure that no displacement of the reinforcing steel cage occurs during placement of the concrete.

C5.4.8

Allowable tolerance of the reinforcing cage is based on minimum CRSI intersection tie requirements, plus whatever additional ties and bracing necessary to maintain the cage shape.

Recommended concrete cover to reinforcing steel:

Shaft Diameter	**Minimum Concrete Cover**
Less than or equal to 3.0 ft.	3.0 in.
Greater than 3.0 ft. and less than 5.0 ft.	4.0 in.
5.0 ft. or larger	6.0 in.

The steel reinforcing cage shall be securely held in position throughout the concrete placement operation. The reinforcing steel in the shaft shall be tied and supported so that the location of the reinforcing steel will remain within allowable tolerance. Concrete spacers or other approved noncorrosive spacing devices shall be used at sufficient intervals (near the bottom, the top, and at intervals not exceeding 10.0 ft. vertically) to ensure concentric spacing for the entire cage length. The number of spacers required at each level shall be one spacer for each 1.0 ft. of excavation diameter, with a minimum of four spacers at each level. The spacers shall be of adequate dimension to ensure an annular space between the outside of the reinforcing cage and the side of the excavation along the entire length of the shaft as shown in the plans. Acceptable feet made of plastic or concrete (bottom supports) shall be provided to ensure that the bottom of the cage is maintained at the proper distance above the base of the excavation, unless the cage is suspended from a fixed base during the concrete pour.

Bracing steel that constricts the interior of the reinforcing cage shall be removed after lifting the cage if freefall concrete or wet tremie methods of concrete placement are to be used.

5.4.9—Concrete Placement, Curing, and Protection

Concrete placement shall commence as soon as possible after completion of drilled shaft excavation by the Contractor and inspection by the Engineer. Immediately prior to commencing concrete placement, the shaft excavation and the properties of the slurry (if used) shall conform to Article 5.3. Concrete placement shall continue in one operation to the top of the shaft, or as shown in the Plans.

If water is not present (a dry shaft), the concrete shall be deposited through the center of the reinforcement cage by a method that prevents segregation of aggregates on the reinforcement cage. The concrete shall be placed such that the free fall is vertical down the center of the shaft without hitting the sides, the steel reinforcing bars, or the steel reinforcing bar cage bracing.

If water exists in amounts greater than 3.0 in. in depth or enters at a rate of more than 12.0 in./h, then the shaft excavation shall be filled with slurry to at least the level specified in Article 5.4.3.4.2 and concrete placed by tremie methods.

Throughout the underwater concrete placement operation, the discharge end of the tube shall remain submerged in the concrete at least 5.0 ft. and the tube shall always contain enough concrete to prevent water from entering. The concrete placement shall be continuous until the work is completed, resulting in a seamless, uniform shaft. If the concrete placement operation is interrupted, the Engineer may require the Contractor to prove by core drilling or other tests that the

C5.4.9

Free-fall concrete can be guided to the center of the shaft with the use of a centering hopper.

Because of the nature of drilled shaft mix designs, it is unnecessary to vibrate the concrete.

A practical definition of a dry shaft is when the amount of standing water in the base of the shaft prior to concreting is less than or equal to 3.0 in. and water is entering the shaft at a rate of less than 12.0 in./h.

In cases where it is possible to pour tremie placed shafts to ground surface, the contractor should consider placing concrete until a minimum of 18.0 in. of concrete, measured vertically, has been expelled to eliminate contaminates in the top of the shaft pour.

shaft contains no voids or horizontal joints. If testing reveals voids or joints, the Contractor shall repair them or replace the shaft at no expense to the Contracting Agency. Responsibility for coring for testing costs, and calculation of time extension, shall be in accordance with Article 5.4.12.

Before placing any fresh concrete against concrete deposited in water or slurry (construction joint), the Contractor shall remove all scum, laitance, loose gravel, and sediment on the surface of the concrete deposited in water or slurry and chip off any high spots on the surface of the existing concrete that would prevent any steel reinforcing bar cage from being placed in the position required by the Plans.

Contractor shall complete a concrete yield plot for each wet shaft poured by tremie methods. This yield plot shall be submitted to the Agency within 24.0 h of completion of the concrete pour.

The contractor shall not perform shaft excavation operations within three diameters of a newly poured shaft within 24.0 h of the placement of concrete and only when the concrete has reached a minimum compressive strength of 1800 psi.

5.4.10—Tremies

When placing concrete underwater, the Contractor shall use a concrete pump or gravity tremie. A tremie shall have a hopper at the top that empties into a watertight tube at least 8.0 in. in diameter. If a pump is used, a watertight tube shall be used with a minimum diameter of 4.0 in., except as noted herein. The discharge end of the tube on the tremie or concrete pump shall include a device to seal out water while the tube is first filled with concrete. In lieu of a seal at the discharge end of the pipe, the Contractor may opt to place a “pig” or “rabbit” in the hopper prior to concrete placement that moves through the tremie when pushed by the concrete forcing water or slurry from the tremie pipe.

C5.4.10

A pig or rabbit is a flexible device that fills the entire cross-section (at least 110 percent) of the tremie tube and creates an impermeable separation between the concrete in the tremie and the slurry.

A tremie (with pig or end cap seal) or pump extension should be used for all wet placements so that the water does not mix with the concrete as it is being placed in the excavation. Trapped air in the pump line or tremie will cause mixing of the concrete and any available water.

Mark the tremie pipe so that tremie insertion and concrete head may be determined. In addition, it is good practice to know the volume placed per stroke of the concrete pump to validate the concrete head.

Reinsertion of a tremie or pump implies a loss of head. Removal of contaminated concrete is advisable, and coring or other Cross-hole Sonic Logging (CSL) testing should be done.

5.4.11—Drilled Shaft Construction Tolerances

Drilled shafts shall be constructed so that the center of the poured shaft at the top of the shaft or mudline, whichever is lower, is within the following horizontal tolerances:

Shaft Diameter	Tolerance
Less than or equal to 2.0 ft	3.0 in.
Greater than 2.0 ft and less than 5.0 ft.	4.0 in.
5.0 ft. or larger	6.0 in.

C5.4.11

Lateral plan deviation less than specified should be shown on the contract plans.

Drilled shafts in soil shall be within 1.5 percent of plumb. Drilled shafts in rock shall be within 2.0 percent of plumb. Plumbness shall be measured from the top of the poured shaft elevation or mudline, whichever is lower.

During drilling or excavation of the shaft, the Contractor shall make frequent checks on the plumbness, alignment, and dimensions of the shaft. Any deviation exceeding the allowable tolerances shall be corrected with a procedure approved by the Engineer.

Drilled shaft steel reinforcing bars shall be no higher than 6.0 in. above or 3.0 in. below the plan elevation.

5.4.12—Integrity Testing

Cross-hole Sonic Logging (CSL) testing shall be performed on shafts as specified in the Contract. The Contractor shall accommodate the CSL testing by furnishing and installing access tubes in accordance with Article 5.3.7.

The Contractor shall install access tubes for CSL testing in all drilled shafts, except as otherwise noted, to permit access for the CSL test probes. If, in the opinion of the Engineer, the condition of the shaft excavation permits shaft construction in the dry, the Engineer may specify that the testing be omitted.

The Contractor shall securely attach the access tubes to the interior of the reinforcement cage of the shaft. One access tube shall be furnished and installed for each foot of shaft diameter, rounded to the nearest whole number, as shown in the Plans. A minimum of three tubes shall be required. The access tubes shall be placed around the shaft, inside the spiral or hoop reinforcement and 3.0 in. clear of the vertical reinforcement, at a uniform spacing measured along the circle passing through the centers of the access tubes. If these minimums cannot be met due to close spacing of the vertical reinforcement, then the access tubes shall be bundled with the vertical reinforcement.

If trimming the cage is required and access tubes for CSL testing are attached to the cage, the Contractor shall either shift the access tubes up the cage or cut the access tubes, provided that the cut tube ends are adapted to receive the watertight cap, as specified.

The access tubes shall be installed in straight alignment and as parallel to the vertical axis of the reinforcement cage as possible. The access tubes shall extend from the bottom of the shaft to at least 2.0 ft. above the top of the shaft. Splice joints in the access tubes, if required to achieve full length access tubes, shall be watertight. The Contractor shall clear the access tubes of all debris and extraneous materials before installing the access tubes. Care shall be taken to prevent damaging the access tubes during reinforcement cage installation and concrete placement operations in the shaft excavation.

C5.4.12

CSL testing is used as a regular inspection method for wet placement shafts using tremie concrete methods. Other Nondestructive Testing (NDT) methods available include Gamma-Gamma (GG) testing and Pulse Echo Testing. Cave-ins along the outer perimeter of the cage may not be detected by CSL testing which could lessen the shaft capacity.

CSL testing should only be used for shafts placed in the dry where visual inspection indicates that irregularities in concrete placement may have occurred.

Tubes may be placed either with reinforcing steel or midspan between vertical reinforcements. There are concerns that CSL tubes placed midspan will effectively negate the clear spacing requirements specified in Article C5.3.2 for the reinforcing steel. There are competing concerns that CSL tubes bundled with large reinforcing steel bundles will create a large blockage for proper flow of concrete. These issues must be resolved on a case-by-case basis.

If the reinforcing steel does not extend to the bottom of the shaft, the CSL tubes should be extended to the shaft bottom.

The access tubes shall be filled with potable water before concrete placement, and the top watertight threaded caps shall be reinstalled.

Prior to performing any CSL testing operations specified herein, the Contractor shall remove the concrete at the top of the shaft down to sound concrete.

The Owner shall perform CSL testing and analysis on all completed shafts designated for testing by the Engineer. The Contractor shall give at least 48 h notice to the Engineer of the time the concrete in each shaft will be sufficiently cured to allow for cross-hole sonic log testing.

The testing shall be performed after the shaft concrete has cured at least 96.0 h. Additional curing time prior to testing may be required if the shaft concrete contains admixtures, such as set retarding admixture or water reducing admixture. The additional curing time prior to testing required under these circumstances shall not be grounds for additional compensation or extension of time to the Contractor. No subsequent construction shall be performed on the completed shaft until the CSL tests are approved and the shaft accepted. The CSL shall be completed within seven days of placement of the shaft.

The agency may opt to require the Contractor to provide the Integrity Testing and provide the results to the agency for its approval.

After placing the shaft concrete and before beginning the CSL testing of a shaft, the Contractor shall inspect the access tubes. Each access tube that the test probe cannot pass through shall be replaced, at the Contractor's expense, with a 2.0 in. diameter hole cored through the concrete for the entire length of the shaft. Unless directed otherwise by the Engineer, cored holes shall be located approximately 6.0 in. inside the reinforcement and shall not damage the shaft reinforcement. Descriptions of inclusions and voids in cored holes shall be logged and a copy of the log shall be submitted to the Engineer. Findings from cored holes shall be preserved, identified as to location, and made available for inspection by the Engineer.

If a single tube is blocked, the Engineer may perform CSL testing on the remaining tubes. If no anomalies are noted, the Engineer may waive the requirement to provide the cored alternative hole.

The Engineer shall determine final acceptance of each shaft, based on the CSL test results and analysis for the tested shafts and a review of the visual inspection reports for the subject shaft, and will provide a response to the Contractor within three working days after receiving the test results and analysis submittal.

The Engineer may approve the continuation of shaft construction prior to approval and acceptance of the first shaft if the Engineer's observations of the construction of the first shaft are satisfactory, including, but not limited to, conformance to the shaft installation plan, as approved by the Engineer; and the Engineer's review of the Contractor's daily reports and the Inspector's daily logs concerning excavation, steel reinforcing bar placement, and concrete placement are satisfactory.

If the Engineer determines that the concrete placed under slurry for a given shaft is structurally inadequate, that shaft shall be rejected. The placement of concrete under slurry shall be suspended until the Contractor submits to the Engineer written changes to the methods of shaft construction needed to prevent future structurally inadequate shafts and receives the Engineer's written approval of the submittal.

If the Contractor requests, the Engineer may direct that additional testing be performed at a shaft. At the Engineer's request, the Contractor shall drill a core hole in any questionable quality shaft (as determined from CSL testing and analysis or by observation of the Engineer) to explore the shaft condition.

Prior to beginning coring, the Contractor shall submit the method and equipment to be used to drill and remove cores from shaft concrete to the Engineer and shall not begin coring until it has received the Engineer’s written approval. The coring method and equipment shall provide for complete core recovery and shall minimize abrasion and erosion of the core.

The recovery system most often used to ensure undamaged core recovery is the triple barrel system.

If subsequent testing at a shaft indicates the presence of a defect(s) in the shaft, the testing costs and the delay costs resulting from the additional testing shall be borne by the Contractor. If this additional testing indicates that the shaft has no defect, the testing costs and the delay costs resulting from the additional testing shall be paid by the Contracting Agency, and, if the shaft construction is on the critical path of the Contractor’s schedule, a time extension equal to the delay created by the additional testing shall be granted.

A defect is defined as a feature which will result in inadequate or unsafe performance under strength, service, and extreme event loads, or inadequate serviceability over the design life of the drilled shaft, in the opinion of the Engineer.

For all shafts determined to be unacceptable, the Contractor shall submit a plan for further investigation and remedial action to the Engineer for approval. All modifications to the dimensions of the shafts, as shown in the Plans, that are required by the investigation and the remedial action plan shall be supported by calculations and working drawings. The Contractor shall not begin repair operations until receiving the Engineer's approval of the investigation and remedial action plan.

If a defect is discovered, it is recommended that three dimensional tomography be conducted to better define the extent of the defect.

All access tubes and cored holes shall be dewatered and filled with grout after tests are completed and the shaft is accepted. The access tubes and cored holes shall be filled using grout tubes that extend to the bottom of the tube or hole or into the grout already placed.

5.5—MEASUREMENT AND PAYMENT

5.5.1—Measurement

5.5.1.1—Drilled Shafts in Soil

Soil excavation for shaft including haul shall be measured by the lineal feet of shaft excavated for each diameter. The lineal feet shall be computed using the top of shaft soil elevation, as defined below, and the bottom elevation shown in the Plans, unless adjusted by the Engineer, less all rock excavation measured as specified in Article 5.5.1.2.

Except as otherwise specified, the top of shaft soil elevation shall be defined as the highest existing ground point within the shaft diameter. For shafts where the top of shaft is above the existing ground line and where the Plans show embankment fill placed above the existing ground line to the top of shaft and above, the top of shaft soil elevation shall be defined as the top of shaft concrete. Excavation through embankment fill placed above the top of shaft shall not be included in the measurement.

5.5.1.2—Drilled Shafts in Rock

Rock excavation for shaft including haul shall be measured by the lineal feet of shaft excavated for each diameter. The lineal feet shall be computed using the shaft diameter shown in the Plans, the top of the rock line, defined as the highest bedrock point within the shaft diameter, and the bottom elevation shown in the Plans, unless adjusted by the Engineer.

Top of rock elevation for bidding purposes shall be determined by the geologist's determination in the contract documents. Actual top of rock for payment purposes may differ from that shown in the contract documents based on the rock definition contained in Article C5.5.1.2.

C5.5.1.2

Rock is defined as that consolidated mass of mineral material having an Unconfined Compressive Strength (UCS) in an intact sample of at least one sample of 1000 psi minimum. This definition falls between class 1 and 2 of the relative rating system for rock classification outlined in Table 10.4.6.4-1 of the *AASHTO LRFD Bridge Design Specifications*.

The geologic determination for measurement purposes may be different from top of rock for design purposes to account for decomposed, weathered, or shattered rock.

In some formations, such as pinnacle limestone, top of rock elevations may vary widely across the shaft diameter, precluding the use of a single boring to accurately determine top of rock.

Some regional practices, such as the use of rig penetration rates to determine the top of rock, may need to be considered when developing rock pay quantities.

5.5.1.3—Obstruction Removal

Obstructions identified under Article 5.4.4 will be measured per hour of time spent working on obstructions.

C5.5.1.3

Alternatively, obstruction removal can be paid based on a force account basis.

The use of an hourly rate eliminates the necessity to maintain records of equipment on site and determine whether equipment was being used, on standby or available for use elsewhere.

The hourly rate method does leave the process open to abuse through unbalanced bidding.

The alternative method of measuring and paying for obstruction removal includes payment on Force Account. While this eliminates the abuses of bid unbalancing, it does create a tremendous amount of administration to determine rates for equipment not commonly rated and record all equipment used or on standby. In addition, careful tracking of the equipment used and the effect of the obstruction removal on the equipment on site not used directly for obstruction removal, but subsequently idled by the obstruction event, will be needed.

Obstruction measurement and payment can be limited to unanticipated obstructions only. This method limits the incidence of obstructions and their payment. However, it places a heavy burden on the foundation report to accurately describe all known obstructions and also encourages the contractor to carry costly contingencies in its bid, thereby potentially increasing bid prices unnecessarily.

5.5.1.4—Trial Drilled Shafts

Drilled shafts that are installed prior to installation of contract drilled shafts for the purpose of demonstrating to the engineer the adequacy of the methods proposed shall be measured per each for shafts installed successfully.

5.5.1.5—Exploration Holes

Exploration holes specified in the contract by the Engineer for purposes of confirming geotechnical properties of soil and rock and to determine the founding elevation of the proposed shafts will be measured per lineal foot for exploration holes installed. Exploration holes may be drilled prior to shaft excavation or from the base of the excavation shaft. The top elevation shall be defined as ground surface at time of exploration hole drilling. The bottom of elevation shall be defined as the bottom of the exploration hole.

5.5.1.6—Permanent Casing—Furnishing and Placing

Furnishing and placing permanent casing shall be measured by the number of linear feet of each diameter of required permanent casing installed, as specified in Article 5.3.3. Upper limit of casing payment shall be defined as the lower of :

- original ground or
- base of footing,

if excavated prior to shaft installation. Lower limit shall be the elevation indicated in the Contract Plans.

5.5.1.7—Load Tests

Load tests shall be measured per each for tests carried successfully to the capacity specified or shaft failure.

5.5.1.8—Cross-Hole Sonic Logging Casing

CSL access tube shall be measured by the linear feet of tube furnished and installed.

C5.5.1.8

When the Contract requires a minimum penetration into a bearing layer, as opposed to a specified shaft tip elevation, and the bearing layer elevation at each shaft cannot be accurately determined, replace Article 5.5.1.8 with: CSL access tube will be measured by the linear foot of tube required based on the design depth shown in the Plans plus the length required to extend the shaft reinforcement by set percentage of the length.

5.5.1.9—Drilled Shaft Construction

Concrete for shaft shall be measured by the cubic yard of concrete in place. The cubic yards shall be computed using the shaft diameter shown in the Plans and the top and bottom elevations shown in the Plans, unless adjusted by the Engineer.

C5.5.1.9

In cases where concrete is poured to limits of excavation (i.e., to ground surface), serious consideration should given to combining bid items such as excavation, concrete placement, and reinforcing steel placement (where rebar cages are constant in section throughout the entire shaft).

5.5.1.10—Reinforcing Steel

Steel reinforcing bar for shaft shall be measured by the computed weight in pounds of all reinforcing steel in place, as shown in the Plans. Bracing for steel reinforcing bar cages shall be considered incidental to this item of work.

5.5.2—Payment

5.5.2.1—Drilled Shafts in Soil

Payment for the item "Soil Excavation For Shaft Including Haul," per lineal foot for each diameter, includes all costs in connection with furnishing, mixing, placing, maintaining, containing, collecting, and disposing of all mineral, synthetic, and water slurry and disposal of all excavated materials. Temporary casing required to complete shaft excavation shall be included in this bid item.

5.5.2.2—Drilled Shafts in Rock

Payment for the item "Rock Excavation For Shaft Including Haul." per lineal foot for each diameter, includes all costs in connection with disposal of spoil and associated water. Temporary casing, if necessary, is included in this bid item.

5.5.2.3—Obstruction Removal

Payment for removing shaft obstructions shall be made for the changes in shaft construction methods necessary to remove the obstruction based on hours spent at contract bid rates.

C5.5.2.3

See commentary in Article 5.4.4 and 5.5.1.3 for additional guidance.

5.5.2.4—Trial Drilled Shafts

Trial drilled shafts shall be paid on the basis of number of drilled shafts directed by the Engineer and installed successfully. Payment for trial drilled shafts shall include mobilization, excavation and disposal of drill spoil, concrete, and rebar, if necessary.

5.5.2.5—Exploration Holes

Exploration holes installed at the direction of the Engineer shall be paid per lineal foot of exploration hole installed.

5.5.2.6—Permanent Casing—Furnishing and Placing

Payment for the item "Furnishing and Placing Permanent Casing For ____ Diameter. Shaft" shall be paid per linear foot.

5.5.2.7—Load Tests

Load tests shall be paid per test installed successfully to failure or to the specified load.

5.5.2.8—Cross-Hole Sonic Logging Casing

Payment for the item "CSL Access Tube" shall be paid per linear foot installed.

C5.5.2.8

If CSL testing is to be provided by the Contractor, then add the following Measurement and Payment specification.

Mobilization for CSL Test Paid per each mobilization to test shafts.

"CSL Test," per each shaft tested.
CSL test will be measured once per shaft tested.

5.5.2.9—Drilled Shaft Construction

Payment for the item "Concrete For Shaft" shall be paid per cubic yard.

5.5.2.10—Reinforcing Steel

Payment for the item "Steel Reinforcing. Bar For Shaft" shall be paid per pound, including all costs in connection with furnishing and installing steel reinforcing bar spacers and centralizers.

5.6—REFERENCES

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2009. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 29th Edition, HM-29. American Association of State Highway and Transportation Officials, Washington, DC.

API. 2003. *Recommended Practice for Field Testing Water-Based Drilling Fluids*, 3rd Edition, ANSI/API RP 13B-1. American Petroleum Institute, Washington, DC.

TABLE OF CONTENTS

SECTION 6

GROUND ANCHORS

6.1—DESCRIPTION

This work shall consist of designing, furnishing, installing, testing, and stressing permanent cement-grouted ground anchors in accordance with these Specifications and the contract documents.

C6.1

If there is any doubt as to the feasibility of using ground anchors for a particular project, special test anchors should be called for in the contract documents. Production anchors are often nonredundant structural members, difficult to inspect, and located in critical support areas. Assurance of success may be worth the added expense.

6.2—WORKING DRAWINGS

At least four weeks before work is to begin, the Contractor shall submit to the Engineer for review and approval complete working drawings and design calculations describing the ground anchor system or systems intended for use. The submittal shall include the following:

1) A ground anchor schedule giving:

- Ground anchor number,
- Ground anchor design load,
- Type and size of tendon,
- Minimum total anchor length,
- Minimum bond length,
- Minimum tendon bond length, and
- Minimum unbonded length.

2) A drawing of the ground anchor tendon and the corrosion protection system, including details for the following:

- Spacers separating elements of tendon and their location,
- Centralizers and their location,
- Unbonded length corrosion protection system,
- Bond length corrosion protection system,
- Anchorage and trumpet,
- Anchorage corrosion protection system,
- Drilled or formed hole size,
- Level of each stage of grouting, and

C6.2

The contract documents generally give the Contractor considerable latitude in the selection of materials and method of installation that may be used; therefore, complete working drawings are required to control the work.

- Any revisions to structure details necessary to accommodate the ground anchor system intended for use.

3) The grout mix design and procedures for placing the grout.

The Engineer shall approve or reject the Contractor's working drawings within four weeks of receipt of a complete submittal. No work on ground anchors shall begin until working drawings have been approved in writing by the Engineer. Such approval shall not relieve the Contractor of any responsibility under the contract documents for the successful completion of the work.

6.3—MATERIALS

6.3.1—Prestressing Steel

Ground anchor tendons shall consist of single or multiple elements of prestressing steel, anchorage devices, and, if required, couplers conforming to the requirements described in Section 10, "Prestressing." The following materials are acceptable for use as ground anchor tendons:

- AASHTO M 203M/M 203 (ASTM A416/A416M—uncoated seven-wire strand)
- ASTM A886/A866M (indented, seven-wire strand)
- ASTM A882/A882M (epoxy-coated, seven-wire strand)

C6.3.1

A positive Hoyer effect is present in all pretensioned bonded members. The strand is pulled to and held at a high tension while concrete is cast around it and cured. In the tensioned state, the diameter of the strand is reduced compared to the diameter in an untensioned state. When the tension in the strand is transferred from the external anchors to the concrete, the stress in the strand at the end of the concrete member goes from a high stress to zero stress. At the points of reduced stress and zero stress, the diameter of the wires increases and the wires press tightly against the surrounding concrete, creating a high friction which is an important factor in transferring the total force. This is called the Hoyer effect, identified by Jack R. Janney in 1954, "Nature of Bond in Prestressed Concrete," *Journal of American Concrete Institute*, Volume 25, May 1954.

When a pull-out load is applied to an untensioned smooth wire strand as it is in a ground anchor, the reduction in the strand diameter due to a negative Hoyer effect significantly decreases the capacity of the strand to transfer its tension to the concrete surrounding it.

As its tension is increased and its diameter decreased, the adhesive bond of the indented strand decreases in the same manner as that of smooth wire strand, but the mechanical bond provided by concrete in the indentations remains effective, giving the indented strand much higher capacity to transfer its tension under a pullout loading.

The tension in an epoxy-coated strand is transferred to the concrete by the embedment of the grit on its surface into the concrete around it. The reduction in strand diameter due to the negative Hoyer effect is not large enough to have any significant effect on the strand's capacity to transfer tension.

6.3.2—Grout

Cement shall be Type I, II, or III portland cement conforming to AASHTO M 85 (ASTM C150). Cement used for grouting shall be fresh and shall not contain any lumps or other indications of hydration or "pack set."

C6.3.2

Although sand is not generally used in grouting small diameter holes, it may have advantages with larger diameter holes. Fly ash and pozzolans are also occasionally used as filler material. Accelerators are not permitted

Aggregate shall conform to the requirements for fine aggregate described in Section 8, "Concrete Structures."

Admixtures may be used in the grout subject to the approval of the Engineer. Expansive admixtures may only be added to the grout used for filling sealed encapsulations, trumpets, and anchorage covers. Accelerators shall not be used.

Water for mixing grout shall be potable, clean, and free of injurious quantities of substances known to be harmful to portland cement or prestressing steel.

because of concern that some may cause corrosion of the prestressing steel.

6.3.3—Steel Elements

Bearing plates shall be fabricated from steel conforming to AASHTO M 270M/M 270 (ASTM A709/A709M), Grade 36 (Grade 250) minimum, or be a ductile iron casting conforming to ASTM A536.

Trumpets used to provide a transition from the anchorage to the unbonded length corrosion protection shall be fabricated from a steel pipe or tube conforming to the requirements of ASTM A53/A53M for pipe or ASTM A500 for tubing. Minimum wall thickness shall be 0.20 in.

Anchorage covers used to enclose exposed anchorages shall be fabricated from steel, steel pipe, steel tube, or ductile cast iron conforming to the requirements of AASHTO M 270M/M 270 (ASTM A709/A709M), Grade 36 (Grade 250) for steel, ASTM A53/A53M for pipe, ASTM A500 for tubing, or ASTM A536 for ductile cast iron. Minimum thickness shall be 0.10 in.

6.3.4—Corrosion Protection Elements

Corrosion-inhibiting grease shall conform to the requirements of *Specification for Unbonded Single Strand Tendons*, Section 3.2.5, published by the Post-Tensioning Institute.

Sheath for the unbonded length of a tendon shall consist of one of the following:

- Seamless polyethylene (PE) tube having a minimum wall thickness of 60 mils ± 10 mils. Polyethylene shall be classified by ASTM D3350.

- Seamless polypropylene tube having a minimum wall thickness of 60 mils ± 10 mils. Polypropylene shall be classified by ASTM D4101.

- Heat-shrinkable tube consisting of a radiation cross-linked polyolefin tube internally coated with an adhesive sealant. The minimum tube wall thickness before shrinking shall be 24 mils. The minimum adhesive sealant thickness shall be 20 mils.

- Corrugated polyvinyl chloride (PVC) tube having a minimum wall thickness of 30 mils.

Encapsulation for the tendon bond length shall consist of one of the following:

- Corrugated high-density polyethylene (HDPE) tube having a minimum wall thickness of 30 mils and conforming to AASHTO M 252 requirements.

- Deformed steel tube or pipe having a minimum wall thickness of 25 mils.

- Corrugated PVC tube having a minimum wall thickness of 30 mils.

- Fusion-bonded epoxy conforming to the requirements of AASHTO M 317M/M 317 (ASTM D3963/D3963M), except that it shall have a film thickness of 15 mils.

6.3.5—Miscellaneous Elements

Bondbreaker for a tendon shall consist of smooth plastic tube or pipe that is resistant to aging by ultraviolet light and that is capable of withstanding abrasion, impact, and bending during handling and installation.

Spacers for separation of elements of a multi-element tendon shall permit the free flow of grout. They shall be fabricated from plastic, steel, or material which is not detrimental to the prestressing steel. Wood shall not be used.

Centralizers shall be fabricated from plastic, steel, or material that is not detrimental to either the prestressing steel or any element of the tendon corrosion protection. Wood shall not be used. The centralizer shall be able to maintain the position of the tendon so that a minimum of 0.5 in. of grout cover is obtained on the tendons, or over the encapsulation.

6.4—FABRICATION

Tendons for ground anchors may be either shop- or field-fabricated from materials conforming to the requirements of Article 6.3, “Materials.” Tendons shall be fabricated as shown on the approved working drawings. The tendon shall be sized so that the maximum test load does not exceed 80 percent of the minimum guaranteed ultimate strength of the tendon.

6.4.1—Bond Length and Tendon Bond Length

The Contractor shall determine the bond length necessary to satisfy the load test requirements. The minimum bond length shall be 10.0 ft in rock, 15.0 ft in soil, or the minimum length shown in the contract documents. The minimum tendon bond length shall be 10.0 ft.

6.4.1.1—Grout-Protected Ground Anchor Tendon

Spacers shall be placed along the tendon bond length of multi-element tendons so that the prestressing steel will bond to the grout. They shall be located at 10.0-ft maximum centers with the upper one located a maximum of 5.0 ft from the top of the tendon bond length and the lower one located a maximum of 5.0 ft from the bottom of the tendon bond length.

Centralizers shall be placed along the bond length. They shall be located at 10.0-ft maximum centers with the upper one located a maximum of 5.0 ft from the top of the bond length and the lower one located 1.0 ft from the bottom of the bond length. Centralizers are not required on tendons installed utilizing a hollow-stem auger if it is grouted through the auger and the drill hole is maintained full of a stiff grout 9.0-in. slump or less during extraction of the auger. A combination centralizer-spacer may be used.

Centralizers are not required on tendons installed utilizing a pressure injection system in coarse-grained soils using grouting pressures greater then 0.150 ksi.

C6.4.1.1

Experience has shown that sufficient grout cover is maintained around pressure-grouted anchors installed in coarse-grained soils without the use of centralizers.

6.4.1.2—Encapsulation-Protected Ground Anchor Tendon

The tendon bond length shall be encapsulated by a grout-filled corrugated plastic or deformed steel tube, or by a fusion-bonded epoxy coating. The tendon shall be grouted inside the encapsulation either prior to inserting the tendon in the drill hole or after the tendon has been placed in the drill hole. Punching holes in the encapsulation and allowing the grout to flow from the encapsulation to the drill hole, or vice versa, shall not be permitted. The tendon shall be centralized within the encapsulation and the tube sized to provide an average of 0.20 in. of grout cover for the prestressing steel. For grout-protected ground anchor tendons, spacers and centralizers shall be used to satisfy the same requirements specified in Article 6.4.1.1, “Grout-Protected Ground Anchor Tendons.” The anchorage device of tendons protected with fusion-bonded epoxy shall be electrically isolated from the structure.

C6.4.1.2

Fusion-bonded epoxy encapsulations may have holidays present in the coating. Electrical isolation of the tendon from the structure will prevent the development of a long-line galvanic corrosion cell between the structure and the tendon bond length portion.

6.4.2—Unbonded Length

The unbonded length of the tendon shall be a minimum of 15.0 ft or as indicated in the contract documents or approved working drawings.

Corrosion protection shall be provided by a sheath completely filled with corrosion-inhibiting grease or grout, or a heat-shrinkable tube. If grease is used to fill the sheath, provisions shall be made to prevent it from escaping at the ends. The grease shall completely coat the tendon and fill the interstices between the wires of seven-wire strands. Continuity of corrosion protection shall be provided at the transition from the bonded length to unbonded length of the tendon.

If the sheath provided is not a smooth tube, then a separate bondbreaker must be provided to prevent the tendon from bonding to the anchor grout surrounding the unbonded length.

6.4.3—Anchorage and Trumpet

Nonrestressable anchorages may be used unless restressable anchorages are specified in the contract documents.

Bearing plates shall be sized so that the bending stresses in the plate and average bearing stress on the concrete, if applicable, do not exceed the nominal resistances described in *AASHTO LRFD Bridge Design Specifications*, Article 5.10.9.7.2. The size of bearing plates shall not be less than that shown in the contract documents or on the approved working drawings.

The trumpet shall be welded to the bearing plate. The trumpet shall have an inside diameter at least 0.25 in. greater than the diameter of the tendon at the anchorage. The trumpet shall be long enough to accommodate movements of the structure during testing and stressing. For strand tendons with encapsulation over the unbonded length, the trumpet shall be long enough to enable the tendons to make a transition from the diameter of the tendon in the unbonded length to the diameter of the tendon at the anchorhead without damaging the encapsulation. Trumpets filled with corrosion-inhibiting grease shall have a permanent Buna-N rubber or approved equal seal provided between the trumpet and the unbonded length corrosion protection. Trumpets filled with grout shall have a temporary seal provided between the trumpet and the unbonded length corrosion protection.

C6.4.3

The nominal resistance of bearing plates refers to Article 5.10.9.7.2, "Bearing Resistances," of the *AASHTO LRFD Bridge Design Specifications*, 2004.

6.4.4—Tendon Storage and Handling

Tendons shall be stored and handled in such a manner as to avoid damage or corrosion. Damage to tendon prestressing steel as a result of abrasions, cuts, nicks, welds, and weld splatter will be cause for rejection by the Engineer. Grounding of welding leads to the prestressing steel is not permitted. A slight rusting, provided it is not sufficient to cause pits visible to the unaided eye, shall not be cause for rejection. Prior to inserting a tendon into the drilled hole, its corrosion protection elements shall be examined for damage. Any damage found shall be repaired in a manner approved by the Engineer.

Repairs to encapsulation shall be in accordance with the tendon Supplier's recommendations.

C6.4.4

Smooth sheathing may be repaired with ultra high molecular weight polyethylene (PE) tape, spiral wound around the tendon so as to completely seal the damaged area. The pitch of the spiral is to be such that a double thickness of tape is ensured at all points.

6.5—INSTALLATION

The Contractor shall select the drilling method, the grouting procedure, and grouting pressure to be used for the installation of the ground anchor as necessary to satisfy the load test requirements.

6.5.1—Drilling

The drilling method used may be core drilling, rotary drilling, percussion drilling, auger drilling, or driven casing. The method of drilling used shall prevent loss of ground above the drilled hole that may be detrimental to the structure or existing structures. Casing for anchor holes, if used, shall be removed, unless permitted by the Engineer to be left in place. The location, inclination, and alignment of the drilled hole shall be as shown in the contract documents. Inclination and alignment shall be within ±3 degrees of the planned angle at the bearing plate, and within ±1.0 ft of the planned location at the ground surface (point of entry).

C6.5.1

The longitudinal axis of the drilled hole and that of the tendon must be parallel. The tendon must not be bent to accommodate connecting the bearing plate to the structure.

6.5.2—Tendon Insertion

The tendon shall be inserted into the drilled hole to the desired depth without difficulty. When the tendon cannot be completely inserted, it shall be removed and the drill hole cleaned or redrilled to permit insertion. Partially inserted tendons shall not be driven or forced into the hole.

6.5.3—Grouting

A neat cement grout or sand cement grout conforming to Article 6.3.2, "Grout," shall be used. Admixtures, if used, shall be mixed in quantities not to exceed the Manufacturer's recommendations.

The grouting equipment shall produce a grout free of lumps and undispersed cement. A positive-displacement grout pump shall be used. The pump shall be equipped with a pressure gage to monitor grout pressures. The pressure gage shall be capable of measuring pressures of at least 0.150 ksi or twice the actual grout pressures used, whichever is greater. The grouting equipment shall be sized to enable the grout to be pumped in one continuous operation. The mixer shall be capable of continuously agitating the grout.

The grout shall be injected from the lowest point of the drill hole. The grout may be pumped through grout tubes, casing, hollow-stem augers, or drill rods. The grout may be placed before or after insertion of the tendon. The quantity of the grout and the grout pressures shall be recorded. The grout pressures and grout takes shall be controlled to prevent excessive heave of the ground or fracturing of rock formations.

Except where indicated below, the grout above the top of the bond length may be placed at the same time as the bond length grout but it shall not be placed under pressure. The grout at the top of the drill hole shall stop 6.0 in. from the back of the structure or from the bottom of the trumpet, whichever is lowest.

If the ground anchor is installed in a fine-grained soil using a drilled hole larger than 6.0 in. in diameter, then the grout above the top of the bond length shall be placed after the ground anchor has been load-tested. The entire drill hole may be grouted at the same time if it can be demonstrated that the ground anchor system does not derive a significant portion of its load resistance from the soil above the bond length portion of the ground anchor.

Pressure grouting techniques shall be utilized if grout-protected tendons are used for ground anchors in rock. Pressure grouting requires that the drill hole be sealed and that the grout be injected until a 0.05 ksi grout pressure can be maintained on the grout within the bond length for a period of 5 min.

Upon completion of grouting, the grout tube may remain in the drill hole provided it is filled with grout.

After grouting, the tendon shall not be loaded for a minimum of three days.

6.5.4—Trumpet and Anchorage

The corrosion protection surrounding the unbonded length of the tendon shall extend into the trumpet a minimum of 6.0 in. beyond the bottom seal in the trumpet.

The corrosion protection surrounding the unbonded length of the tendon shall not contact the bearing plate or the anchorhead during load testing or stressing.

The bearing plate and anchorhead shall be placed perpendicular to the axis of the tendon.

The trumpet shall be completely filled with corrosion-inhibiting grease or grout. The grease may be placed any time during construction. The grout shall be placed after the ground anchor has been load-tested. The Contractor shall demonstrate that the procedures selected for placement of either grease or grout will produce a completely filled trumpet.

C6.5.4

The most critical area to protect from corrosion is in the vicinity of the anchorage. Below the bearing plate, the corrosion protection over the unbonded length is usually terminated to expose the bare tendon. Above the bearing plate, the bare tendon is gripped by either wedges, nuts, or deformations in the case of wires. Regardless of the type of tendon, the gripping mechanism creates stress concentrations at the connection. In addition, an aggressive corrosive environment may exist at the anchorhead since oxygen is readily available. The vulnerability of this area is demonstrated by the fact that most tendon failures occur within a short distance of the anchorhead. Extreme care is required in order to ensure that the prestressing steel is well protected in this area.

Anchorages not encased in concrete shall be covered with a corrosion-inhibiting grease-filled or grout-filled steel enclosure.

6.5.5—Testing and Stressing

Each ground anchor shall be load-tested by the Contractor using either the performance test or the proof test procedures specified herein. No load greater than ten percent of the design load may be applied to the ground anchor prior to load testing. The test load shall be simultaneously applied to the entire tendon.

6.5.5.1—Testing Equipment

A dial gage or vernier scale capable of measuring displacements to 0.001 in. shall be used to measure ground anchor movement. It shall have adequate travel so total ground anchor movement can be measured without resetting the device.

A hydraulic jack and pump shall be used to apply the test load. The jack and a calibrated pressure gage shall be used to measure the applied load. The pressure gage shall be graduated in 0.100 ksi increments or less. When the theoretical elastic elongation of the total anchor length at the maximum test load exceeds the ram travel of the jack, the procedure for recycling the jack ram shall be included on the working drawings. Each increment of test load shall be applied as rapidly as possible.

A calibrated reference pressure gage shall be available at the site. The reference gage shall be calibrated with the test jack and pressure gage.

An electrical resistance load cell and readout shall be provided when performing a creep test.

The stressing equipment shall be placed over the ground anchor tendon in such a manner that the jack, bearing plates, load cell, and stressing anchorage are axially aligned with the tendon and the tendon is centered within the equipment.

C6.5.5.1

Experience has shown that electrical resistance load cells frequently do not perform satisfactorily under field conditions. Hence, they are not recommended for measurement of load. Load cells are, however, very sensitive to small changes in load and are used to monitor changes in load during a creep test.

6.5.5.2—Performance Test

Five percent of the ground anchors or a minimum of three ground anchors, whichever is greater, shall be performance tested in accordance with the following procedures. The Engineer shall select the ground anchors to be performance tested. The remaining anchors shall be tested in accordance with the proof-test procedures.

The performance test shall be made by incrementally loading and unloading the ground anchor in accordance with the following schedule unless a different maximum test load and schedule are indicated in the contract documents:

C6.5.5.2

If a different maximum test load is to be required, a schedule similar to this one should be described in the contract documents.

- The load shall be raised from one increment to another immediately after recording the ground anchor movement.

- The ground anchor movement shall be measured and recorded to the nearest 0.001 in. with respect to an independent fixed reference point at the alignment load and at each increment of load.

- The load shall be monitored with a pressure gage.

- The reference pressure gage shall be placed in series with the pressure gage during each performance test.

If the load determined by the reference pressure gage and the load determined by the pressure gage differ by more than ten percent, the jack, pressure gage, and reference pressure gage shall be recalibrated. At load increments other than the maximum test load, the load shall be held just long enough to obtain the movement reading.

Table 6.5.5.2-1—Performance Test Schedule

Load	Load
AL	*AL*
0.25*DL**	0.25*DL*
AL	0.50*DL*
0.25*DL*	0.75*DL*
0.50*DL**	1.00*DL*
AL	1.20*DL**
0.25*DL*	*AL*
0.50*DL*	0.25*DL*
0.75*DL**	0.50*DL*
AL	0.75*DL*
0.25*DL*	1.00*DL*
0.50*DL*	1.20*DL*
0.75*DL*	1.33*DL** (Max. test load)
1.00*DL**	Reduce to lock-off load (Article 6.5.5.6)

where:

AL = Alignment load

DL = Design load for ground anchor

* = Graph required, as specified herein

The alignment load is a small load, normally less than ten percent of the design load, applied to the ground anchor in order to keep the testing equipment in position during testing.

The maximum test load in a performance test shall be held for 10 min. The jack shall be repumped as necessary in order to maintain a constant load. The loadhold period shall start as soon as the maximum test load is applied, and the ground anchor movement shall be measured and recorded at 1 min, 2, 3, 4, 5, 6, and 10 min. If the ground anchor movements between 1 min and 10 min exceeds 0.04 in., the maximum test load shall be held for an additional 50 min. If the load-hold is extended, the ground anchor movement shall be recorded at 15 min, 20, 25, 30, 45, and 60 min.

A graph shall be constructed showing a plot of ground anchor movement versus load for each load increment marked with an asterisk (*) in Table 6.5.5.2-1 and a plot of the residual ground anchor movement of the tendon at each alignment load versus the highest previously applied load. Graph format shall be approved by the Engineer prior to use.

6.5.5.3—Proof Test

Those anchors not subjected to a performance test shall be tested as specified herein.

The proof test shall be performed by incrementally loading the ground anchor in accordance with the following schedule unless a different maximum test load and schedule are indicated in the contract documents. The load shall be raised from one increment to another immediately after recording the ground anchor movement. The ground anchor movement shall be measured and recorded to the nearest 0.001 in. with respect to an independent fixed reference point at the alignment load and at each increment of load. The load shall be monitored with a pressure gage. At load increments other than the maximum test load, the load shall be held just long enough to obtain the movement reading.

Table 6.5.5.3-1—Proof Test Schedule

Load	Load
AL	1.00*DL*
0.25*DL*	1.20*DL*
0.50*DL*	1.33*DL* (max. test load)
0.75*DL*	Reduce to lock-off load

where:

AL = Alignment load

DL = Design load for ground anchor

C6.5.5.3

If a different maximum test load is to be required, a schedule similar to the one given in this Article should be described in the contract documents.

The maximum test load in a proof test shall be held for 10 min. The jack shall be repumped as necessary in order to maintain a constant load. The load-hold period shall start as soon as the maximum test load is applied, and the ground anchor movement shall be measured and recorded at 1 min, 2, 3, 4, 5, 6, and 10 min. If the ground anchor movement between 1 min and 10 min exceeds 0.04 in., the maximum test load shall be held for an additional 50 min. If the load-hold is extended, the ground anchor movement shall be recorded at 15 min, 20, 30, 45, and 60 min. A graph shall be constructed showing a plot of ground anchor movement versus load for each load increment in the proof test. Graph format shall be approved by the Engineer prior to use.

6.5.5.4—Creep Test

Creep tests shall be performed if specified in the contract documents. The Engineer shall select the ground anchors to be creep tested.

The creep test shall be made by incrementally loading and unloading the ground anchor in accordance with the performance test schedule used. At the end of each loading cycle, the load shall be held constant for the observation period indicated in the creep test schedule below unless a different maximum test load is indicated in the contract documents. The times for reading and recording the ground anchor movement during each observation period shall be 1 min, 2, 3, 4, 5, 6, 10, 15, 20, 25, 30, 45, 60, 75, 90, 100, 120, 150, 180, 210, 240, 270, and 300 min as appropriate. Each load-hold period shall start as soon as the test load is applied. In a creep test, the pressure gage and reference pressure gage shall be used to measure the applied load, and the load cell shall be used to monitor small changes of load during a constant load-hold period. The jack shall be repumped as necessary in order to maintain a constant load.

Table 6.5.5.4-1—Creep Test Schedule

AL	Observation Period, min
0.25*DL*	10
0.50*DL*	30
0.75*DL*	30
1.00*DL*	45
1.20*DL*	60
1.33*DL*	300

A graph shall be constructed showing a plot of the ground anchor movement and the residual movement measured in a creep test as described for the performance test. Also, a graph shall be constructed showing a plot of the ground anchor creep movement for each load-hold as a function of the logarithm of time. Graph formats shall be approved by the Engineer prior to use.

C6.5.5.4

If creep tests are required, at least two ground anchors should be creep-tested. If a different maximum test load is to be required, a schedule similar to this one should be described in the contract documents.

6.5.5.5—Ground Anchor Load Test Acceptance Criteria

A performance-tested or proof-tested ground anchor with a 10-min load-hold shall be deemed to be acceptable if:

- The ground anchor resists the maximum test load with less than 0.04 in. of movement between 1 min and 10 min; and
- The total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length, or
- For a performance-tested ground anchor in competent rock, the total movement at the maximum test load may not exceed the theoretical elastic elongation of the unbonded length plus 50 percent of the theoretical elastic elongation of the bonded length.

A performance-tested or proof-tested ground anchor with a 60-min load-hold shall be deemed to be acceptable if the:

- Ground anchor resists the maximum test load with a creep rate that does not exceed 0.08 in. in the last log cycle of time; and
- Total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length.

For a performance-tested ground anchor in competent rock, the total movement at the maximum test load may not exceed the theoretical elastic elongation of the unbonded length plus 50 percent of the theoretical elastic elongation of the bonded length.

A creep-tested ground anchor shall be deemed to be acceptable if the:

- Ground anchor carries the maximum test load with a creep rate that does not exceed 0.08 in. in the last log cycle of time and
- Total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length.
- For a performance-tested ground anchor in competent rock, the total movement at the maximum test load may not exceed the theoretical elastic elongation of the unbonded length plus 50 percent of the theoretical elastic elongation of the bonded length.

If the total movement of the ground anchor at the maximum test load does not exceed 80 percent of the theoretical elastic elongation of the unbonded length, the ground anchor shall be replaced at the Contractor's expense.

A ground anchor which has a creep rate greater than 0.08 in. per log cycle of time can be incorporated into the structure, but its design nominal resistance shall be equal to one-half of its failure load. The failure load is the load resisted by the ground anchor after the load has been allowed to stabilize for 10 min.

When a ground anchor fails, the Contractor shall modify the design and/or the installation procedures. These modifications may include, but are not limited to, installing a replacement ground anchor, reducing the design load by increasing the number of ground anchors, modifying the installation methods, increasing the bond length, or changing the ground anchor type. Any modification which requires changes to the structure shall be approved by the Engineer. Any modifications of design or construction procedures shall be without additional cost to the Owner and without extension of the contract-documents time.

Retesting of a ground anchor will not be permitted, except that regrouted ground anchors may be retested.

6.5.5.6—Lock-Off

Upon successful completion of the load testing, the ground anchor load shall be reduced to the lock-off load indicated in the contract documents and transferred to the anchorage device. The ground anchor may be completely unloaded prior to lock-off. After transferring the load and prior to removing the jack, a lift-off load reading shall be made. The lift-off load shall be within ten percent of the specified lock-off load. If the load is not within ten percent of the specified lock-off load, the anchorage shall be reset and another lift-off load reading shall be made. This process shall be repeated until the desired lock-off load is obtained.

6.6—MEASUREMENT AND PAYMENT

Ground anchors will be measured and paid for by the number of units installed and accepted as shown in the contract documents or ordered by the Engineer. No change in the number of ground anchors to be paid for will be made because of the use by the Contractor of an alternative number of ground anchors.

The contract unit price paid for ground anchors shall include full compensation for furnishing all labor, materials, tools, equipment, and incidentals, and for doing all the work involved in installing the ground anchors (including testing), complete in place, as specified in these Specifications, the contract documents, and as directed by the Engineer.

C6.6

Some agencies prefer to pay for performance tests and creep tests separately to avoid the uncertainty of testing costs. Local experience will determine the desirability of such separate pay clauses.

6.7—REFERENCES

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

Janney, J. R. 1954. "Nature of Bond in Prestressed Concrete," *Journal of American Concrete Institute*, American Concrete Institute, Vol. 25, May 1954.

PTI. 2000. *Specification for Unbonded Single Strand Tendons*, 2nd Edition, Post-Tensioning Institute, Phoenix, AZ, Section 3.2.5.

SECTION 7: EARTH-RETAINING SYSTEMS

TABLE OF CONTENTS

SECTION 7

EARTH-RETAINING SYSTEMS

7.1—DESCRIPTION

This work shall consist of furnishing and installing earth-retaining systems in accordance with the contract documents and these specifications.

C7.1

Earth-retaining systems include concrete and masonry gravity walls, reinforced concrete retaining walls, sheet pile and soldier pile walls (with and without ground anchors or other anchorage systems), crib and cellular walls, and mechanically stabilized earth walls.

7.2—WORKING DRAWINGS

Working drawings and design calculations shall be submitted to the Engineer for review and approval at least four weeks before work is to begin. Such submittals shall be required:

- for each alternative proprietary or nonproprietary earth-retaining system proposed, as permitted or specified in the contract documents,
- complete details for the system to be constructed are not included in the contract documents, and
- when otherwise required by the contract documents or these specifications. Working drawings and design calculations shall include the following:
    - existing ground elevations that have been verified by the Contractor for each location involving construction wholly or partially in original ground,
    - layout of wall that will effectively retain the earth, but not less in height or length than that shown for the wall system in the contract documents,
    - complete design calculations substantiating that the proposed design satisfies the design parameters in the contract documents,
    - complete details of all elements required for the proper construction of the system, including complete material specifications,
    - earthwork requirements including specifications for material and compaction of backfill,
    - details of revisions or additions to drainage systems or other facilities required to accommodate the system, and
    - other information required in the contract documents or requested by the Engineer.

The Contractor shall not start work on any earth-retaining system for which working drawings are required until such drawings have been approved by the Engineer. Approval of the Contractor's working drawings shall not relieve the Contractor of any responsibility under the contract documents for the successful completion of the work.

7.3—MATERIALS

7.3.1—Concrete

7.3.1.1—Cast-in-Place

Cast-in-place concrete shall conform to the requirements of Section 8, “Concrete Structures.” The concrete shall be Class A unless otherwise indicated in the contract documents.

7.3.1.2—Pneumatically Applied Mortar

Pneumatically applied mortar shall conform to the requirements of Section 24, “Pneumatically Applied Mortar.”

7.3.1.3—Precast Elements

The materials, manufacturing, storage, handling, and erection of precast concrete elements shall conform to the requirements in Article 8.13, “Precast Concrete Members.” Unless otherwise shown in the contract documents or on the approved working drawings, portland cement concrete used in precast elements shall conform to Class A (AE) with a minimum compressive strength at 28 days of 4.0 ksi.

7.3.1.4—Segmental Concrete Facing Blocks

Masonry concrete blocks used as wall-facing elements shall have a minimum compressive strength of 4 ksi and a water absorption limit of five percent. In areas of repeated freeze-thaw cycles, the facing blocks shall be tested in accordance with ASTM C1262 to demonstrate durability. The facing blocks shall meet the requirements of ASTM C1372, except that acceptance regarding durability under this testing method shall be achieved if the weight loss of each of four of the five specimens at the conclusion of 150 cycles does not exceed one percent of its initial weight. Blocks shall also meet the additional requirements of ASTM C140. Facing blocks directly exposed to spray from deiced pavements shall be sealed after erection with a water-resistant coating or be manufactured with a coating or additive to increase freeze-thaw resistance.

7.3.2—Reinforcing Steel

Reinforcing steel shall conform to the requirements of Section 9, “Reinforcing Steel.”

7.3.3—Structural Steel

Structural steel shall conform to AASHTO M 270M/M 270 (ASTM A709/A709M), Grade 36 (Grade 250), unless otherwise specified in the contract documents.

7.3.4—Timber

Timber shall conform to the requirements of Section 16, "Timber Structures," and Article 4.2.2, "Timber Piles."

7.3.5—Drainage Elements

7.3.5.1—Pipe and Perforated Pipe

Pipe and perforated pipe shall conform to Subsection 708, "Concrete, Clay, and Plastic Pipe," and Section 709, "Metal Pipe," of the *AASHTO Guide Specifications for Highway Construction*.

7.3.5.2—Filter Fabric

Filter fabric shall conform to Subsection 620, "Filter Fabric," of the *AASHTO Guide Specifications for Highway Construction*,

7.3.5.3—Permeable Material

Permeable material shall conform to Subsection 704, "Aggregate for Drainage," of the *AASHTO Guide Specifications for Highway Construction*, unless otherwise specified in the contract documents or on the approved working drawings.

7.3.5.4—Geocomposite Drainage Systems

Geocomposite drainage systems shall conform to the requirements specified in the contract documents or the approved working drawings.

7.3.6—Structure Backfill Material

7.3.6.1—General

All structure backfill material shall consist of material free from organic material or other unsuitable material as determined by the Engineer. Gradation will be determined by AASHTO T 27 (ASTM C136). Grading shall be as follows, unless otherwise specified.

Sieve Size	*Percent Passing*
3.0 in. (75 mm)	100
No. 4 (4.75 mm)	35–100
No. 30 (600 µm)	20–100
No. 200 (75 µm)	0–15

7.3.6.2—Crib and Cellular Walls

Structure backfill material for crib and cellular walls shall be of such character that it will not sift or flow through openings in the wall. For wall heights over 20.0 ft, the following grading shall be required:

Sieve Size	*Percent Passing*
3.0 in. (75 mm)	100
No. 4 (4.75 mm)	25–70
No. 30 (600 µm)	5–20
No. 200 (75 µm)	0–5

7.3.6.3—Mechanically Stabilized Earth Walls

Structure backfill material for mechanically stabilized earth walls shall conform to the following grading, internal friction angle and soundness requirements:

Sieve Size	*Percent Passing*
4.0 in. (100 mm)	100
No. 40 (425 µm)	0–60
No. 200 (75 µm)	0–15

*Plasticity Index (PI), as determined by AASHTO T 90, shall not exceed 6.

The material shall exhibit an angle of internal friction of not less than 34 degrees, as determined by the standard Direct Shear Test, AASHTO T 236 (ASTM D3080), on the portion finer than the No. 10 (2.00-mm) sieve, utilizing a sample of the material compacted to 95 percent of AASHTO T 99, Methods C or D (with oversized correction as outlined in Note 7) at optimum moisture content. No testing is required for backfills where 80 percent of sizes are greater than 0.75 in.

The materials shall be substantially free of shale or other soft, poor durability particles. The material shall have a magnesium sulfate soundness loss of less than 30 percent after four cycles.

Additionally, the backfill material shall meet the following electrochemical requirements when steel soil reinforcement is to be used:

- pH of 5 to 10,
- resistivity not less than 30 Ω · m,
- chlorides not greater than 100 ppm, and
- Sulfates not greater than 200 ppm.

7.4—EARTHWORK

7.4.1—Structure Excavation

Structure excavation for earth-retaining systems shall conform to the requirements of Section 1, "Structure Excavation and Backfill," and as provided below.

7.4.2—Foundation Treatment

Foundation treatment shall conform to the requirements of Article 1.4.2, "Foundation Preparation and Control of Water," unless otherwise specified in the contract documents or included in the approved working drawings. If subexcavation of foundation material is indicated, the Contractor shall perform the excavation to the limits shown. Material excavated shall be replaced with structure backfill material meeting the requirements for the particular earth-retaining system to be constructed unless a different material is specified in the contract documents. The material shall be compacted to a density not less than 95 percent of the maximum density as determined by AASHTO T 99, Methods C or D (with oversize correction as outlined in Note 7).

7.4.3—Structure Backfill

Placement of structure backfill material shall conform to the requirements of Articles 1.4.3, "Backfill," and 7.6, "Construction." Material used shall conform to the requirements of Article 7.3.6.

7.5—DRAINAGE

Drainage facilities shall be constructed in accordance with the details shown on the approved working drawings or in the contract documents and these specifications.

7.5.1—Concrete Gutters

Concrete gutters shall be constructed to the profile indicated in the contract documents or on the approved working drawings. Pneumatically applied mortar shall conform to the requirements of Section 24, "Pneumatically Applied Mortar." Outlet works shall be provided at sags in the profile, at the low ends of the gutter, and at other indicated locations.

7.5.2—Weep Holes

Weep holes shall be constructed at the locations shown in the contract documents or on the approved working drawings. A minimum of 2.0 ft^3 of permeable material encapsulated with filter fabric shall be placed at each weep hole.

Joints between precast concrete retaining-wall face panels which function as weep holes shall be covered with filter fabric. The filter fabric shall be bonded to the face panels with adhesive conforming to Federal Specification MMM-A-121. The face panels which are to receive the filter fabric shall be dry and thoroughly cleaned of dust and loose materials.

7.5.3—Drainage Blankets

Drainage blankets consisting of permeable material encapsulated in filter fabric, collector pipes, outlet pipes, and cleanout pipes shall be constructed as specified in the contract documents or on the approved working drawings.

The subgrade to receive the filter fabric shall conform to the compaction and elevation tolerance specified and shall be free of loose or extraneous material and sharp objects that may damage the filter fabric during installation. The fabric shall be stretched, aligned, and placed in a wrinkle-free manner. Adjacent borders of the fabric shall be overlapped from 12.0 in. to 18.0 in. Should the fabric be damaged, the torn or punctured section shall be repaired by placing a piece of fabric that is large enough to cover the damaged area and to meet the overlap requirement.

The permeable material shall be placed in horizontal layers and thoroughly consolidated along with and by the same methods specified for structure backfill. Ponding and jetting of permeable material or structure backfill material adjacent to permeable material shall not be permitted. During spreading and compaction of the permeable material and structure backfill or embankment material, a minimum of 6.0 in. of such material shall be maintained between the fabric and the Contractor's equipment.

The perforated collector pipe shall be placed within the permeable material to the flow line elevations shown.

Outlet pipes shall be placed at sags in the flow line, at the low end of the collector pipe, and at other locations shown or specified in the contract documents. Rock slope protection, when required at the end of outlet pipes, shall conform to the details in the contract documents or approved working drawings and the requirements in Section 22, “Slope Protection.”

Cleanout pipes shall be placed at the high ends of collector pipes and at other locations as specified in the contract documents.

7.5.4—Geocomposite Drainage Systems

Geocomposite drainage systems shall be installed at the locations shown in the contract documents or on the approved working drawings. The geocomposite drainage material shall be placed and secured tightly against the excavated face, lagging, or back of wall as specified in the contract documents. When concrete is to be placed against geocomposite drainage material, the drainage material shall be protected against physical damage and grout leakage.

7.6—CONSTRUCTION

The construction of earth-retaining systems shall conform to the lines and grades indicated in the contract documents, on the working drawings, or as directed by the Engineer.

7.6.1—Concrete and Masonry Gravity Walls, Reinforced Concrete Retaining Walls

Stone masonry construction shall conform to the requirements of Section 14, "Stone Masonry." Concrete construction shall conform to the requirements of Section 8, "Concrete Structures." Reinforced concrete block masonry shall conform to the requirements of Section 15, "Concrete Block and Brick Masonry."

Vertical precast concrete wall elements with cast-in-place concrete footing support shall be adequately supported and braced to prevent settlement or lateral displacement until the footing concrete has been placed and has achieved sufficient strength to support the wall elements.

The exposed face of concrete walls shall receive a Class 1 finish as specified in Section 8, "Concrete Structures," unless a special architectural treatment is specified in the contract documents, or on the approved working drawings.

7.6.2—Sheet Pile and Soldier Pile Walls

This work shall consist of constructing continuous walls of timber, steel, or concrete sheet piles, and the constructing of soldier pile walls with horizontal facing elements of timber, steel, or concrete.

7.6.2.1—Sheet Pile Walls

Steel sheet piles shall be of the type and weight (mass) specified in the contract documents. Steel sheet piles shall conform to the requirements of AASHTO M 202M/M 202 (ASTM A328/A328M), AASHTO M 270M/M 270 (ASTM A709/A709M) Grade 50 (Grade 345), or to the specifications for "Piling for Use in Marine Environments" in ASTM A690/A690M. Painting of steel sheet piles, when required, shall conform to Article 13.2, "Painting Metal Structures."

Timber sheet piles, unless otherwise specified or permitted in the contract documents, shall be treated in accordance with Section 17, "Preservative Treatment of Wood." The piles shall be of the dimensions, species, and grade of timber specified in the contract documents. The piles may be either cut from solid material or made by building up with three planks securely fastened together. The piles shall be drift sharpened at their lower ends so as to wedge adjacent piles tightly together during driving.

Concrete sheet piles shall conform to the details specified in the contract documents or the approved working drawings. The manufacture and installation shall conform, in general, to the requirements for precast concrete bearing piles in Section 4, "Driven Foundation Piles." Concrete sheet piles detailed to have a tongue and groove joint on the portion below ground and a double-grooved joint on the exposed portion shall, after installation, have the upper grooves cleaned of all sand, mud, or debris and grouted full. Unless otherwise provided in the contract documents or approved in writing by the Engineer, grout shall be composed of one part cement and two parts sand. The grout shall be deposited through a grout pipe placed within a watertight plastic sheath extending the full depth of the grout slot formed by the grooves in two adjacent pilings and which, when filled, completely fills the slot.

Sheet piles shall be driven to the specified penetration or bearing capacity in accordance with the requirements of Section 4, "Driven Foundation Piles."

After driving, the tops of sheet piles shall be neatly cut off to a straight line at the elevation specified in the contract documents or as directed by the Engineer.

Sheet pile walls shall be braced by wales or other bracing system, as shown in the contract documents or directed by the Engineer.

Timber waling strips shall be properly lapped and joined at all splices and corners. The wales shall preferably be in one length between corners and shall be bolted near the tops of the piles.

When specified in the contract documents or on the approved working drawings, reinforced concrete caps shall be constructed in accordance with Section 8, "Concrete Structures."

7.6.2.2—Soldier Pile Walls

Soldier piles shall be either driven piles or piles constructed in a drilled shaft excavation to the specified penetration or bearing capacity indicated in the contract documents.

Driven piles shall be furnished and installed in accordance with the requirements of Section 4, "Driven Foundation Piles." The piles shall be of the type indicated in the contract documents.

Piles constructed in a drilled shaft excavation shall conform to the details shown in the contract documents. Construction of the shaft excavation and placement of concrete or lean concrete backfill shall be in accordance with Section 5, "Drilled Piles and Shafts." The structural component of the soldier pile placed in the shaft excavation shall be as specified in the contract documents. Reinforced concrete, either cast-in-place or precast, shall conform to the requirements of Section 8, "Concrete Structures." Timber members shall conform to the requirements of Section 16, "Timber Structures," and Section 17, "Preservative Treatment of Wood." Steel members shall conform to the requirements of Section 11, "Steel Structures." Painting of steel members, if required, shall conform to Section 13, "Painting."

Concrete backfill placed around precast concrete, timber or steel pile members in the drilled shaft excavation shall be commercially available portland cement concrete with a cement content not less than five sacks/yd^3. Lean concrete backfill shall consist of commercial quality concrete sand, water, and not greater than one sack/yd^3 of portland cement. The limits for placement of concrete and lean concrete shall be specified in the contract documents.

The facing spanning horizontally between soldier piles shall conform to the materials and details in the contract documents or on the approved working drawings. Timber lagging shall conform to the requirements in Section 16, "Timber Structures" and Section 17, "Preservative Treatment of Wood." Precast concrete lagging or facing panels and cast-in-place concrete facing shall conform to the requirements in Section 8, "Concrete Structures." Concrete anchors, welded connections, and bolted connections for securing facing elements to the soldier piles shall conform to the details in the contract documents.

The exposed surface of concrete wall facing shall receive a Class 1 finish as specified in Section 8, "Concrete Structures," unless a special architectural treatment is specified in the contract documents or on the approved working drawings.

7.6.2.3—Anchored Sheet Pile and Soldier Pile Walls

7.6.2.3.1—General

The construction of anchored walls shall consist of constructing sheet pile and soldier pile walls anchored with a tie-rod and concrete anchor system or with ground anchors.

Sheet pile and soldier pile wall construction shall conform to the requirements of Articles 7.6.2.1 and 7.6.2.2, respectively.

7.6.2.3.2—Wales

Wales consisting of either timber, steel, or concrete shall conform to the details in the contract documents or on the approved working drawings. The alignment of wales shall be such that tie-rods or ground anchors can be installed without bending. Timber wales shall conform to the requirements of Section 16, "Timber Structures," and Section 17, "Preservative Treatments of Wood." Steel wales shall conform to the requirements of Section 11, "Steel Structures." Concrete wales shall conform to the requirements of Section 8, "Concrete Structures."

7.6.2.3.3—Concrete Anchor System

Concrete anchor systems, consisting of either drilled shafts or reinforced concrete shapes placed within the limits of soil or rock excavation, with or without pile support, shall conform to the details in the contract documents or on the approved working drawings.

Battered anchor piles shall be driven to the proper batter shown. The tension anchor piles shall be furnished with adequate means of anchorage to the concrete anchor block.

Drilled shaft concrete anchors shall conform to the details in the contract documents or on the approved working drawings and be constructed in conformance with Section 5, "Drilled Piles and Shafts."

7.6.2.3.4—Tie-Rods

Tie-rods shall be round steel bars conforming to AASHTO M 270M/M 270 (ASTM A709/A709M), Grade 36 (Grade 250), unless otherwise specified in the contract documents. Corrosion protection shall be provided as specified in the contract documents. Care shall be taken in the handling and backfilling operations to prevent damage to the corrosion protection or bending of the tie-rod itself.

The connection of the tie-rods to the soldier piles, wales, wall face, and concrete anchors shall conform to the details specified in the contract documents.

7.6.2.3.5—Ground Anchors

Ground anchors shall be constructed in conformance with the requirements of Section 6, "Ground Anchors."

The connection of ground anchors to soldier piles, wales, or wall face shall conform to the details in the contract documents or on the approved working drawings.

7.6.2.3.6—Earthwork

Earthwork shall conform to the requirements in Article 7.4.

Unless otherwise specified in the contract documents, excavation in front of the wall shall not proceed more than 3.0 ft below a level of tie-rods or ground anchors until such tie-rods and anchors or ground anchors are complete and accepted by the Engineer.

Placement of lagging shall closely follow excavation in front of the wall such that loss of ground is minimized.

7.6.3—Crib Walls and Cellular Walls

This work shall consist of constructing timber, concrete, or steel crib walls and concrete monolithic cell walls complete with backfill material within the cells formed by the members.

7.6.3.1—Foundation

In addition to the requirements of Article 7.4.2, the foundation or bed course material shall be finished to exact grade and cross slope so that the vertical or battered face alignment will be achieved.

When required, timber mud sills, concrete leveling pads, or concrete footings shall conform to the details specified in the contract documents. Timber mud sills shall be firmly and evenly bedded in the foundation material. Concrete for leveling pads or footings shall be placed against the sides of excavation in the foundation material.

7.6.3.2—Crib Members

Unless otherwise specified in the contract documents, timber header and stretcher members shall conform to the requirements of Section 16, "Timber Structures," and shall be the same as for caps, posts, and sills. Preservative treatment shall conform to the requirements of Section 17, "Preservative Treatment of Wood." The size of the members shall be as shown in the contract documents.

Concrete header and stretcher members shall conform to the requirements of Section 8, "Concrete Structures," for precast concrete members. The dimensions of the members and minimum concrete strength shall be as specified in the contract documents or on the approved working drawings.

Steel crib members consisting of base plates, columns, stretchers, and spacers shall be fabricated from sheet steel conforming to AASHTO M 218. Thickness of members shall be as specified. Crib members shall be so fabricated that members of the same nominal size and thickness shall be fully interchangeable. No drilling, punching, or drifting to correct defects in manufacture shall be permitted. Any members having holes improperly punched shall be replaced. Bolts, nuts, and miscellaneous hardware shall be galvanized in accordance with AASHTO M 232M/M 232 (ASTM A153/A153M).

7.6.3.3—Concrete Monolithic Cell Members

Concrete monolithic cell members consisting of four-sided cells of uniform height and various depths shall be cast in conformance with the requirements set forth for precast members in Section 8, "Concrete Structures." The minimum concrete compressive strength shall be 4.0 ksi. The exposed cell face shall have a Class 1 finish; faces not exposed to view shall have a uniform surface finish free of open pockets of aggregate or surface distortions in excess of 0.25 in. The protruding keys and recesses for keys on the tops and bottoms of the side walls of the cells shall be accurately located.

7.6.3.4—Member Placement

Timber and concrete crib members shall be placed in successive tiers at spacings conforming to the specified details for the particular height of wall being constructed. Drift bolts at the intersection of timber header and stretcher members shall be accurately installed so that minimum edge distances are maintained. At the intersection of concrete header and stretcher members, asphalt felt shims or other approved material shall be used to obtain uniform bearing between the members.

Steel column sections, stretchers, and spacers shall conform to the proper length and weight (mass) as specified. These members shall be accurately aligned to permit completing the bolted connections without distorting the members. Bolts at the connections shall be torqued to not less than 25.0 ft · lb.

Concrete monolithic cell members of the proper sizes shall be successively stacked in conformance with the layout specified in the contract documents or on the approved working drawings. Care shall be exercised in placing the members to prevent damage to the protruding keys. Damaged or ill-fitting keys shall be repaired using a method approved by the Engineer.

7.6.3.5—Backfilling

The cells formed by the wall members shall be backfilled with structure backfill material conforming to the requirements in Article 7.3.6. Backfilling shall progress simultaneously with the erection of the members forming the cells. Backfill material shall be so placed and compacted as to not disturb or damage the members. Placement of backfill shall be in uniform layers not exceeding 1.0 ft in thickness unless otherwise proposed by the Contractor and approved by the Engineer. Compaction shall be to a density of at least 95 percent of the maximum density as determined by AASHTO T 99, Method C. Backfilling behind the wall to the limits of excavation shall conform to the same requirements unless otherwise indicated or approved.

7.6.4—Mechanically Stabilized Earth Walls

The construction of mechanically stabilized earthwalls shall consist of constructing a facing system to which steel or polymeric soil reinforcement is connected and the placing of structure backfill material surrounding the soil reinforcement.

7.6.4.1—Facing

Facing consisting of either precast concrete panels, cast-in-place concrete, or welded wire fabric shall conform to the details and materials specified in the contract documents or on the approved working drawings.

Precast concrete panels shall be cast in conformance with the requirements set forth for precast members in Section 8, "Concrete Structures." The concrete compressive strength shall be that specified in the concrete documents or 4.0 ksi, whichever is greater. The exposed face shall have a Class 1 finish, or the architectural treatment specified in the contract documents or on the approved working drawings. The face not exposed to view shall have a uniform surface finish free of open pockets of aggregate or surface distortions in excess of 0.25 in. Soil reinforcement connection hardware shall be accurately located and secured during concrete placement and shall not contact the panel reinforcing steel. Joint filler, bearing pads, and joint cover material shall be as specified in the contract documents.

Cast-in-place concrete facing shall be constructed in conformance with the requirements in Section 8, "Concrete Structures." Soil reinforcement extending beyond the temporary facing shall be embedded in the facing concrete the minimum dimensions specified in the contract documents or on the approved working drawings.

Welded wire facing, either temporary or permanent, shall be formed by a 90–degree bend of the horizontal soil reinforcement. The vertical portion of the soil reinforcement forming the face shall be connected to the succeeding upper level of soil reinforcement. A separate backing mat and hardware cloth shall be placed immediately behind the vertical portion of soil reinforcement. Its wire size and spacing shall be as specified in the contract documents.

7.6.4.2—Soil Reinforcement

All steel soil reinforcement and any steel connection hardware shall be galvanized in accordance with AASHTO M 111M/M 111 (ASTM A123/A123M).

Steel strip reinforcement shall be hot-rolled to the required shape and dimensions. The steel shall conform to ASTM A572/A572M, Grade 65 (Grade 450), unless otherwise specified in the contract documents.

Welded wire fabric reinforcement shall be shop fabricated from cold-drawn wire of the sizes and spacings specified in the contract documents or on the approved working drawings. The wire shall conform to the requirements of AASHTO M 32M/M 32 (ASTM A82); fabricated fabric shall conform to the requirements of AASHTO M 55M/M 55 (ASTM A185).

Polymeric reinforcement shall be of the type and size specified in the contract documents or on the approved working drawings and shall conform to the specified material and manufacturing requirements.

Connection hardware shall conform to the contract documents or the approved working drawings.

The installation of instrumentation for monitoring corrosion shall conform to the requirements specified.

7.6.4.3—Construction

When required, a precast reinforced or a cast-in-place concrete leveling pad shall be provided at each panel foundation level. Prior to placing the leveling pads, the foundation material shall conform to the requirements of Article 7.4.2.

Precast concrete panels and welded wire fabric facing shall be placed and supported as necessary so that their final position is vertical or battered as shown in the contract documents or on the approved working drawings within a tolerance acceptable to the Engineer.

Joint filler, bearing pads, and joint covering material shall be installed concurrent with face panel placement.

Backfill material conforming to the requirement in Article 7.3.6 shall be placed and compacted simultaneously with the placement of facing and soil reinforcement. Placement and compaction shall be accomplished without distortion or displacement of the facing or soil reinforcement. Sheeps foot or grid-type rollers shall not be used for compacting backfill within the limits of the soil reinforcement. At each level of soil reinforcement the backfill material shall be roughly leveled to an elevation approximately 0.1 ft above the level of connection at the facing before placing the soil reinforcement. All soil reinforcement shall be uniformly tensioned to remove any slack in the connection or material.

7.7—MEASUREMENT AND PAYMENT

Unless otherwise designated in the contract documents, earth-retaining systems shall be measured and paid for by the square foot. The square foot area for payment shall be based on the vertical height and length of each section built, except in the case when alternative earth-retaining systems are permitted in the contract documents. When alternative earth-retaining systems are permitted, the square foot area for payment will be based on the vertical height and length of each section of the system type designated as the basis of payment whether or not it is actually constructed. The vertical height of each section shall be taken as the difference in elevation on the outer face, from the bottom of the lowermost face element for systems without footings, and from the top of footing for systems with footings, to the top of the wall, excluding any barrier.

The contract price paid per square foot for earth-retaining systems shall include full compensation for furnishing all labor, materials, tools, equipment, and incidentals, and for doing all the work involved in constructing the earth-retaining systems including, but not limited to, earthwork, piles, footings, and drainage systems, complete in place, as specified in the contract documents, in these Specifications and as directed by the Engineer.

Full compensation for revisions to drainage system or other facilities made necessary by the use of an alternative earth-retaining system shall be considered as included in the contract price paid per square foot for earth-retaining system and no adjustment in compensation will be made therefore.

7.8—REFERENCES

AASHTO. 2008. *AASHTO Guide Specifications for Highway Construction*, Ninth Edition, GSH-9, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2009. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 29th Edition, HM-29, American Association of State Highway and Transportation Officials, Washington, DC.

GSA. 1966. *Adhesive, Bonding Vulcanized Rubber to Steel*, Federal Specification MMM-A-121, U.S. General Services Administration.

SECTION 8: CONCRETE STRUCTURES

TABLE OF CONTENTS

SECTION 8

CONCRETE STRUCTURES

8.1—GENERAL

8.1.1—Description

This work shall consist of furnishing, placing, finishing, and curing concrete in bridges, culverts, and miscellaneous structures in accordance with these specifications and conforming to the lines, grades, and dimensions specified in the contract documents. The work may include elements of structures constructed by cast-in-place and precast methods using either plain (unreinforced), reinforced, or prestressed concrete or any combination thereof.

8.1.2—Related Work

Other work involved in the construction of concrete structures shall be as specified in the applicable sections of this Specification, especially Section 3, "Temporary Works," Section 9, "Reinforcing Steel," and Section 10, "Prestressing."

8.1.3—Construction Methods

Whenever the contract documents permit the Contractor to select the method or equipment to be used for any operation, it shall be the Contractor's responsibility to employ methods and equipment which will produce satisfactory work under the conditions encountered and which will not damage any partially completed portions of the work.

Falsework and forms shall conform to the requirements of Section 3, "Temporary Works."

Generally, all concrete shall be fully supported until the required strength and age has been reached. However, the slip form method may be permitted for the construction of pier shafts and railings providing the Contractor's plan assures that:

- the results will be equal in all respect to those obtained by the use of fixed forms, and
- adequate arrangements will be provided for curing, finishing, and protecting the concrete.

8.2—CLASSES OF CONCRETE

8.2.1—General

The class of concrete to be used in each part of the structure shall be as specified in the contract documents. If not specified, the Engineer shall designate the class of concrete to be used.

8.2.2—Normal-Weight (-Density) Concrete

Ten classes of normal-weight (-density) concrete are provided for in these specifications as listed in Table 8.2.2-1, except that for concrete on or over saltwater or exposed to deicing chemicals, the maximum water/cement ratio shall be 0.45.

Coarse aggregate for Class B and Class B(AE) shall be furnished into separate sizes as shown in Table 8.2.2-1.

C8.2.2

With high performance concrete, it is desirable that the specifications be performance-based. Class P(HPC) is intended for use in prestressed concrete members with a specified concrete compressive strength greater than 6.0 ksi and should always be used for specified concrete strengths greater than 10.0 ksi. Class A(HPC) is intended for use in cast-in-place construction where performance criteria in addition to concrete compressive strengths are specified. Other criteria might include shrinkage, chloride permeability, freeze-thaw resistance, deicer scaling resistance, abrasion resistance, or heat of hydration.

For both classes of concrete, a minimum cement content is not included since this should be selected by the producer based on the specified performance criteria. Maximum water-cementitious materials ratios have been included. The value of 0.40 for Class P(HPC) is less than the value of 0.49 for Class P, whereas the value of 0.45 for Class A(HPC) is the same as that for Class A(AE). For Class P(HPC) concrete, a maximum size of coarse aggregate is specified since it is difficult to achieve the higher concrete compressive strengths with aggregates larger than 0.75 in. For Class A(HPC) concrete, the maximum aggregate size should be selected by the producer based on the specified performance criteria.

Air content for Class A(HPC) and P(HPC) should be set with trial tests but a minimum of two percent is recommended.

The 28-day specified compression strength may not be appropriate for strengths greater than 6.0 ksi.

Table 8.2.2-1—Classification of Normal-Weight Concrete

Class of Concrete	Minimum Cement Content	Maximum Water/ Cementitious Material Ratio	Air Content Range	Size of Coarse Aggregate Per AASHTO M 43 (ASTM D448)	Size Number [a]	Specified Compressive Strength
	lb/yd^3	lb per lb	%	Nominal Size		ksi at days
A	611	0.49	—	1.0 in. to No. 4	57	4.0 at 28
A(AE)	611	0.45	6 ± 1.5	1.0 in. to No. 4	57	4.0 at 28
B	517	0.58	—	2.0 in. to 1.0 in. and 1.0 in. to No. 4	3 57	2.4 at 28
B(AE)	517	0.55	5 ± 1.5	2.0 in. to 1.0 in. and 1.0 in. to No. 4	3 57	2.4 at 2p8
C	658	0.49	—	0.5 in. to No. 4	7	4.0 at 28
C(AE)	658	0.45	7 ± 1.5	0.5 in. to No. 4	7	4.0 at 28
P	564	0.49	—[b]	1.0 in. to No. 4 or 0.75 in. to No. 4	7 67	≤ 6.0 at [b]
S	658	0.58	—	1.0 in. to No. 4	7	—
P(HPC)	—[c]	0.40	—[b]	≤0.75 in.	67	> 6.0 at [b]
A(HPC)	—[c]	0.45	—[b]	—[c]	—[c]	≤ 6.0 at [b]

Notes:
[a] As noted in AASHTO M 43 (ASTM D448), Table 1–Standard Sizes of Processed Aggregate.
[b] As specified in the contract documents.
[c] Minimum cementitious materials content and coarse aggregate size to be selected to meet other performance criteria specified in the contract.

8.2.3—Lightweight (Low-Density) Concrete

2016 Revision

Lightweight (low-density) concrete shall conform to the requirements specified in the contract documents. When the contract documents require the use of natural sand for a portion or all of the fine aggregate, the natural sand shall conform to AASHTO M 6.

8.3—MATERIALS

8.3.1—Cements

Portland cements shall conform to the requirements of AASHTO M 85 (ASTM C150) and blended hydraulic cements shall conform to the requirements of AASHTO M 240 (ASTM C595) or ASTM C1157. For Type 1P portland-pozzolan cement, the pozzolan constituent shall not exceed 20 percent of the weight (mass) of the blend and the loss on ignition of the pozzolan shall not exceed five percent.

Except for Class P(HPC) and Class A(HPC) or when otherwise specified in the contract documents, only Type I, II, or III portland cement; Types IA, IIA, or IIIA

C8.3.1

ASTM C1157 is a performance specification that does not require restrictions on the composition of the cement or its constituents. It can be used to accept cements not conforming to AASHTO M 85 (ASTM C150) and AASHTO M 240 (ASTM C595).

The low alkali requirement of AASHTO M 85 (ASTM C150) does not provide protection against alkali-silica reactivity in all cases. A better approach is provided in AASHTO M 6 and M 80.

air entrained portland cement; or Types IP or IS blended hydraulic cements shall be used. Types IA, IIA, and IIIA cements may be used only in concrete where air entrainment is required.

Low-alkali cements conforming to the requirements of AASHTO M 85 (ASTM C150) for low-alkali cement shall be used when specified in the contract documents or when ordered by the Engineer as a condition of use for aggregates of limited alkali-silica reactivity.

Unless otherwise permitted, the product of only one mill of any one brand and type of cement shall be used for like elements of a structure that are exposed to view, except when cements must be blended for reduction of any excessive air entrainment where air-entraining cement is used.

For Class P(HPC) and Class A(HPC), trial batches using all intended constituent materials shall be made prior to concrete placement to ensure that cement and admixtures are compatible. Changes in the mill, brand, or type of cement shall not be permitted without additional trial batches.

8.3.2—Water

Water used in mixing and curing of concrete shall be subject to approval and shall be reasonably clean and free of oil, salt, acid, alkali, sugar, vegetable, or other injurious substances. Water shall be tested in accordance with, and shall meet the suggested requirements of AASHTO T 26. Water known to be of potable quality may be used without test. Where the source of water is relatively shallow, the intake shall be so enclosed as to exclude silt, mud, grass, or other foreign materials.

Mixing water for concrete in which steel is embedded shall not contain a chloride ion concentration in excess of 1,000 ppm or sulfates as SO_4 in excess of 1,300 ppm.

8.3.3—Fine Aggregate

Fine aggregate for concrete shall conform to the requirements of AASHTO M 6.

8.3.4—Coarse Aggregate 2010 Revision

Coarse aggregate for concrete shall conform to the requirements of AASHTO M 80.

8.3.5—Combined Aggregates

Blends of fine and coarse aggregates shall conform to the requirements of AASHTO M XXI.

C8.3.5

The use of a combined aggregate grading can result in the use of less water, cementitious materials, and paste and lead to improved fresh and hardened concrete properties.

See attached Appendix A8 for proposed AASHTO M XXI.

8.3.6—Lightweight (Low-Density) Aggregate

Lightweight (low-density) aggregate for concrete shall conform to the requirements of AASHTO M 195 (ASTM C330).

8.3.7—Air-Entraining and Chemical Admixtures

Air-entraining admixtures shall conform to the requirements of AASHTO M 154 (ASTM C260).

Chemical admixtures shall conform to the requirements of AASHTO M 194 (ASTM C494/C494M). Unless otherwise specified in the contract documents, only Type A, Type B, Type D, Type F, or Type G shall be used.

Admixtures containing chloride ion (CL) in excess of one percent by weight (mass) of the admixture shall not be used in reinforced concrete. Admixtures in excess of 0.1 percent shall not be used in prestressed concrete.

A Certificate of Compliance signed by the Manufacturer of the admixture shall be furnished to the Engineer for each shipment of admixture used in the work. Said Certificate shall be based upon laboratory test results from an approved testing facility and shall certify that the admixture meets the above specifications.

If more than one admixture is used, documentation demonstrating the compatibility of each admixture with all other proposed admixtures, and the sequence of application to obtain the desired effects, shall be submitted by the Contractor.

Air-entraining and chemical admixtures shall be incorporated into the concrete mix in a water solution. The water so included shall be considered to be a portion of the allowed mixing water.

C8.3.7

The types of chemical admixtures are as follows:

- Type A—Water-reducing
- Type B—Retarding
- Type D—Water-reducing and retarding
- Type F—Water-reducing and high-range
- Type G—Water-reducing, high-range, and retarding

8.3.8—Mineral Admixtures

Mineral admixtures in concrete shall conform to the following requirements:

- Fly ash pozzolans and calcined natural pozzolans—AASHTO M 295 (ASTM C618)
- Ground granulated blast-furnace slag—AASHTO M 302 (ASTM C989)
- Silica fume—AASHTO M 307 (ASTM C1240)

Fly ash as produced by plants that utilize the limestone injection process or use compounds of sodium, ammonium, or sulfur, such as soda ash, to control stack emissions shall not be used in concrete.

A Certificate of Compliance, based on test results and signed by the producer of the mineral admixture certifying that the material conforms to the above specifications, shall be furnished for each shipment used in the work.

C8.3.8

Pozzolans (fly ash, silica fume) and slag are used in the production of Class P(HPC) and Class A(HPC) concretes to extend the service life.

Where special materials other than those identified above are included in a concrete mix design, the properties of those materials shall be determined by methods specified in the contract documents.

Occasionally, it may be appropriate to use other materials; for example, when concretes are modified to obtain very high strengths through the introduction of special materials, such as:

- Silica fume,
- Cements other than portland or blended hydraulic cements,
- Proprietary high early strength cements,
- Ground granulated blast-furnace slag, and
- Other types of cementitious and/or pozzolanic materials.

8.3.9—Steel

Materials and installation of reinforcing and prestressing steel shall conform to the requirements of Sections 9, "Reinforcing Steel," and 10, "Prestressing," respectively.

8.4—PROPORTIONING OF CONCRETE

8.4.1—Mix Design

8.4.1.1—Responsibility and Criteria 2010 Revision

The Contractor shall design and be responsible for the performance of all concrete mixes used in structures. The mix proportions selected shall produce concrete that is sufficiently workable and finishable for all uses intended and shall conform to the requirements in Table 8.2.2-1 and all other requirements of this Section.

For normal-weight (-density) concrete, the absolute volume method, such as described in American Concrete Institute Publication 211.1, shall be used in selecting mix proportions. For Class P(HPC) with fly ash, the method given in American Concrete Institute Publication 211.4 shall be permitted. For lightweight (low-density) concrete, the mix proportions shall be selected on the basis of trial mixes, with the cement factor rather than the water/cement ratio being determined by the specified strength, using methods such as those described in American Concrete Institute Publication 211.2.

The mix design shall be based on the specified properties. When strength is specified, select an average concrete strength sufficiently above the specified strength so that, considering the expected variability of the concrete and test procedures, no more than one in ten strength tests will be expected to fall below the specified strength. Mix designs shall be modified during the course of the work when necessary to ensure compliance with the specified fresh and hardened concrete properties. For Class P(HPC) and Class A(HPC), such modifications

C8.4.1.1

Normal-weight (-density) mix design refers to the American Concrete Institute (ACI), Publication 211.1, 1991. Lightweight (low-density) mix design refers to the ACI Publication 211.2, 1998.

For Class P(HPC) with fly ash, the method given in ACI Publication 211.4, 1993, is permitted.

In Class P(HPC) and Class A(HPC) concretes, properties other than compressive strength are also important, and the mix design should be based on specified properties rather than only compressive strength.

shall only be permitted after trial batches to demonstrate that the modified mix design will result in concrete that complies with the specified concrete properties.

8.4.1.2—Trial Batch Tests

For classes A, A(AE), P, P(HPC), and A(HPC) concrete; for lightweight (low-density) concrete; and for other classes of concrete when specified in the contract documents or ordered by the Engineer, satisfactory performance of the proposed mix design shall be verified by laboratory tests on trial batches. The results of such tests shall be furnished to the Engineer by the Contractor or the Manufacturer of precast elements at the time the proposed mix design is submitted.

If materials and a mix design identical to those proposed for use have been used on other work within the previous year, certified copies of concrete test results from this work that indicate full compliance with these specifications may be substituted for such laboratory tests.

The average values obtained from trial batches for the specified properties, such as strength, shall exceed design values by a certain amount based on variability. For compressive strength, the required average strength used as a basis for selection of concrete proportions shall be determined in accordance with AASHTO M 241 (ASTM C685/C685M).

C8.4.1.2

In Class P(HPC) and Class A(HPC) concretes, properties other than compressive strength are also important. However, if only compressive strength is specified, AASHTO M 241 (ASTM C685/C685M) provides the method to determine the required average strength.

8.4.1.3—Approval

All mix designs and any modifications thereto shall be approved by the Engineer prior to use. Mix design data provided to the Engineer for each class of concrete required shall include the name, source, type, and brand of each of the materials proposed for use and the quantity to be used per cubic meter of concrete.

8.4.2—Water Content

For calculating the water/cement ratio of the mix, the weight (mass) of the water shall be that of the total free water in the mix which includes the mixing water, the water in any admixture solutions, and any water in the aggregates in excess of that needed to reach a saturated-surface-dry condition.

The amount of water used shall not exceed the limits listed in Table 8.2.2-1 and shall be further reduced as necessary to produce concrete of the consistencies listed in Table 8.4.2-1 at the time of placement.

Table 8.4.2-1—Normal-Weight Concrete Slump Test Limits

Type of Work	Nominal Slump, in.	Maximum Slump, in.
Formed Elements: Sections over 12.0 in. Thick Sections 12.0 in. Thick or Less	 1–3 1–4	 5 5
Cast-in-Place Piles and Drilled Shafts Not Vibrated	5–8	9
Concrete Placed under Water	5–8	9
Filling for Riprap	3–7	8

When Type F or G high-range, water-reducing admixtures are used, Table 8.4.2-1 slump limits may be exceeded as permitted by the Engineer.

When the consistency of the concrete is found to exceed the nominal slump, the mixture of subsequent batches shall be adjusted to reduce the slump to a value within the nominal range. Batches of concrete with a slump exceeding the maximum specified shall not be used in the work.

If concrete of adequate workability cannot be obtained by the use of the minimum cement content allowed, the cement and water content shall be increased without exceeding the specified water/cement ratio, or an approved admixture shall be used.

8.4.3—Cement Content

The minimum cement content shall be as listed in Table 8.2.2-1 or otherwise specified in the contract documents. For Class P(HPC), the total cementitious materials content shall be specified not to exceed 1000 lb/yd^3 of concrete. For other classes of concrete, maximum cement or cement plus mineral admixture content shall not exceed 800 lb/yd^3 of concrete. The actual cement content used shall be within these limits and shall be sufficient to produce concrete of the required strength, consistency, and performance.

C8.4.3

Many high-strength concretes require a cementitious materials content greater than the traditional AASHTO limit of 800 lb/yd^3. However, when cementitious materials contents in excess of 1000.0 lb/yd^3 are required in high-strength concrete, optimization of other constituent materials or alternative constituent materials should be considered.

8.4.4—Mineral Admixtures

Mineral admixtures shall be used in the amounts specified in the contract documents. For all classes of concrete except Classes P(HPC) and A(HPC), when Types I, II, IV, or V AASHTO M 85 (ASTM C150) cements are used and mineral admixtures are neither specified in the contract documents nor prohibited, the Contractor will be permitted to replace:

- up to 25 percent of the required portland cement with fly ash or other pozzolan conforming to AASHTO M 295 (ASTM C618),
- up to 50 percent of the required portland cement with slag conforming to AASHTO M 302 (ASTM C989), or
- up to ten percent of the required portland cement with silica fume conforming to AASHTO M 307 (ASTM C1240).

When any combination of fly ash, slag, and silica fume are used, the Contractor will be permitted to replace up to 50 percent of the required portland cement. However, no more than 25 percent shall be fly ash and no more than ten percent shall be silica fume. The weight (mass) of the mineral admixture used shall be equal to or greater than the weight (mass) of the portland cement replaced.

C8.4.4

Mineral admixtures are widely used in concrete in the percentages given. For Class P(HPC) and Class A(HPC) concretes, different percentages may be used if trial batches substantiate that such amounts provide the specified properties.

A 25-percent maximum of portland cement replacement is permitted for all classes, except for Classes P(HPC) and A(HPC), which have a 50-percent maximum portland cement replacement.

In calculating the water-cementitious materials ratio of the mix, the weight (mass) of the cementitious materials shall be considered to be the sum of the weight (mass) of the portland cement and the mineral admixtures.

For Class P(HPC) and Class A(HPC) concrete, mineral admixtures (pozzolans or slag) shall be permitted to be used as cementitious materials with portland cement in blended cements or as a separate addition at the mixer. The amount of mineral admixture shall be determined by trial batches. The water-cementitious materials ratio shall be the ratio of the weight (mass) of water to the total cementitious materials, including the mineral admixtures. The properties of the freshly mixed and hardened concrete shall comply with specified values.

8.4.5—Air-Entraining and Chemical Admixtures

Air-entraining and chemical admixtures shall be used as specified in the contract documents. Otherwise, such admixtures may be used, at the option and expense of the Contractor when permitted by the Engineer, to increase the workability or alter the time of set of the concrete.

8.5—MANUFACTURE OF CONCRETE

The production of ready-mixed concrete and concrete produced by stationary mixers shall conform to the requirements of AASHTO M 157 and the requirements of this Article.

8.5.1—Storage of Aggregates

The handling and storage of concrete aggregates shall be such as to prevent segregation or contamination with foreign materials. The methods used shall provide for adequate drainage so that the moisture content of the aggregates is uniform at the time of batching. Different sizes of aggregate shall be stored in separate stock piles sufficiently removed from each other to prevent the material at the edges of the piles from becoming intermixed.

When specified in Table 8.2.2-1 or in the contract documents, the coarse aggregate shall be separated into two or more sizes in order to secure greater uniformity of the concrete mixture.

8.5.2—Storage of Cement

The Contractor shall provide suitable means for storing and protecting cement against dampness. Cement which for any reason has become partially set or which contains lumps of caked cement shall be rejected. Cement held in storage for a period of over three months if bagged or six months if bulk, or cement which for any reason the Engineer may suspect is damaged, shall be subject to a retest before being used in the work.

Copies of cement records shall be furnished to the Engineer showing, in such detail as the Engineer may reasonably require, the quantity used during the day or run at each part of the work.

8.5.3—Measurement of Materials

Materials shall be measured by weighing, except as otherwise specified in the contract documents or where other methods are specifically authorized. The apparatus provided for weighing the aggregates and cement shall be suitably designed and constructed for this purpose. Each size of aggregate and the cement shall be weighed separately. The accuracy of all weighing devices shall be such that successive quantities can be measured to within one percent of the desired amount. Cement in standard packages need not be weighed but bulk cement shall be weighed. The mixing water shall be measured by volume or by weight (mass). The accuracy of measuring the water shall be within a range of error of not over one percent. All measuring devices shall be subject to approval and shall be tested, at the Contractor's expense, when deemed necessary by the Engineer.

When volumetric measurements are authorized for projects, the weight (mass) proportions shall be converted to equivalent volumetric proportions. In such cases, suitable allowance shall be made for variations in the moisture condition of the aggregates, including the bulking effect in the fine aggregate.

When sacked cement is used, the quantities of aggregates for each batch shall be exactly sufficient for one or more full sacks of cement and no batch requiring fractional sacks of cement will be permitted.

C8.5.3

The conventional sack of cement has a volume of 1.0 ft^3 and a weight of 94.0 lb.

8.5.4—Batching and Mixing Concrete

8.5.4.1—Batching

The size of the batch shall not exceed the capacity of the mixer as guaranteed by the Manufacturer or as determined by the Standard Requirements of the Associated General Contractors of America.

The measured materials shall be batched and charged into the mixer by means that will prevent loss of any materials due to effects of wind or other causes.

8.5.4.2—Mixing

The concrete shall be mixed only in the quantity required for immediate use. Mixing shall be sufficient to thoroughly intermingle all mix ingredients into a uniform mixture. Concrete that has developed an initial set shall not be used. Retempering concrete shall not be permitted.

For other than transit-mixed concrete, the first batch of concrete materials placed in the mixer shall contain a sufficient excess of cement, sand, and water to coat the inside of the drum without reducing the required mortar content of the mix.

When mixer performance tests as described in AASHTO M 157 are not made, the required mixing time for stationary mixers shall be not less than 90 s nor more than 5 min. The minimum drum revolutions for transit mixers at the mixing speed recommended by the Manufacturer shall not be less than 70 and not less than that recommended by the Manufacturer.

The timing device on stationary mixers shall be equipped with a bell or another suitable warning device adjusted to give a clearly audible signal each time the lock is released. In case of failure of the timing device, the Contractor shall be permitted to operate the mixer while the timing device is being repaired, provided he furnishes an approved timepiece equipped with minute and second hands. If the timing device is not placed in good working order within 24 h, further use of the mixer shall be prohibited until repairs are made.

For small quantities of concrete needed in emergencies or for small noncritical elements of the work, concrete may be hand-mixed using methods approved by the Engineer.

Between uses, any mortar coating inside of mixing equipment which sets or dries shall be cleaned from the mixer before use is resumed.

8.5.5—Delivery

The organization supplying concrete shall have sufficient plant capacity and transporting apparatus to ensure continuous delivery at the rate required. The rate of delivery of concrete during concreting operations shall be such as to provide for the proper handling, placing, and finishing of the concrete. The rate shall be such that the interval between batches shall not exceed 20 min and shall be sufficient to prevent joints within a monolithic pour caused by placing fresh concrete against concrete in which initial set has occurred. The methods of delivering and handling the concrete shall be such as will facilitate placing with the minimum of rehandling and without damage to the structure or the concrete.

8.5.6—Sampling and Testing

Compliance with the requirements indicated in this Section shall be determined in accordance with the following standard methods of AASHTO or ASTM:

- Sampling Fresh Concrete, AASHTO T 141 (ASTM C172)
- Weight per Cubic Foot, Yield, and Air Content (Gravimetric) of Concrete, AASHTO T 121 (ASTM C138/C138M)
- Sieve Analysis of Fine and Coarse Aggregate, AASHTO T 27 (ASTM C136)
- Slump of Portland Cement Concrete, AASHTO T 119 (ASTM C143/C143M)

- Air Content of Freshly Mixed Concrete by the Pressure Method, AASHTO T 152 (ASTM C231)
- Specific Gravity and Absorption of Fine Aggregate, AASHTO T 84 (ASTM C128)
- Specific Gravity and Absorption of Coarse Aggregate, AASHTO T 85 (ASTM C127)
- Determining Density of Structural Lightweight (Low-Density) Concrete, ASTM C567
- Making and Curing Concrete Test Specimens in the Laboratory, ASTM C192/C192M)
- Making and Curing Concrete Test Specimens in the Field., AASHTO T 23 (ASTM C31/C31M)
- Compressive Strength of Cylindrical Concrete Specimens, AASHTO T 22 (ASTM C39/C39M)

8.5.7—Evaluation of Concrete Strength

8.5.7.1—Tests

A strength test shall consist of the average strength of at least two 6.0 × 12.0-in. or at least three 4.0 × 8.0-in. compressive strength test cylinders fabricated from material taken from a single randomly selected batch of concrete, except that, if any cylinder should show evidence of improper sampling, molding, or testing, said cylinder shall be discarded and the strength test shall consist of the strength of the remaining cylinder(s). A minimum of three cylinders shall be fabricated for each strength test when the specified strength exceeds 5.0 ksi.

C8.5.7.1

The use of 4.0 × 8.0-in. cylinders for measuring concrete compressive strengths is increasing. Test results using the smaller size cylinder have a higher variability compared to 6.0 × 12.0-in. cylinders. This can be offset by requiring three cylinders of the smaller size compared to two for the larger size. Since measurement of compressive strength is more critical for high-strength concrete, three cylinders are required for both cylinder sizes.

8.5.7.2—For Controlling Construction Operations

For determining adequacy of cure and protection, and for determining when loads or stresses can be applied to concrete structures, test cylinders shall be cured at the structure site under conditions that are not more favorable than the most unfavorable conditions for the portions of the structure which they represent as described in AASHTO T 23 (ASTM C31/C31M), Article 9.4. Sufficient test cylinders shall be made and tested at the appropriate ages to determine when operations such as release of falsework, application of prestressing forces, or placing the structure in service can occur.

8.5.7.3—For Acceptance of Concrete

For determining compliance of concrete with a specified strength, test cylinders shall be cured under controlled conditions as described in Article 9.3 of AASHTO T 23 (ASTM C31/C31M) and tested at the specified age. Samples for acceptance tests for each class of concrete shall be taken not less than once a day nor less than once for each 150.0 yd^3 of concrete, or once for each major placement.

Except for Class P(HPC) and Class A(HPC) concrete, any concrete represented by a test that indicates a strength that is less than the specified compressive strength at the specified age by more than 0.500 ksi will be rejected and shall be removed and replaced with acceptable concrete. Such rejection shall prevail unless either:

- The Contractor, at the Contractor's expense, obtains and submits evidence of a type acceptable to the Engineer that the strength and quality of the rejected concrete is acceptable. If such evidence consists of cores taken from the work, the cores shall be obtained and tested in accordance with the standard methods of AASHTO T 24M/T 24 (ASTM C42/C42M), or

- The Engineer determines that said concrete is located where it will not create an intolerable detrimental effect on the structure and the Contractor agrees to a reduced payment to compensate the Owner for loss of durability and other lost benefits.

For Class P(HPC) and Class A(HPC) concrete, any concrete represented by a test that indicates a strength that is less than the specified compressive strength at the specified age will be rejected and shall be removed and replaced with acceptable concrete.

C8.5.7.3

The concrete age when the specified strength is to be achieved must be shown in the contract documents.

The 28-day strength has historically been used for strengths under 6.0 ksi. With the longer time frame involved with HPC, 56 days or later may be more appropriate. Setting the 56-day strength as f'_c with a 28-day goal may also be appropriate.

8.5.7.4—For Control of Mix Design

Whenever the average of three consecutive tests, which were made to determine acceptability of concrete, falls to less than 0.150 ksi above the specified strength, or any single test falls more than 0.200 ksi below the specified strength, the Contractor shall, at the Contractor's expense, make corrective changes in the materials, mix proportions, or concrete manufacturing procedures before placing additional concrete of that class. Such changes shall be approved by the Engineer prior to use.

8.5.7.5—Precast Concrete Cured by the Waterproof Cover Method, Steam, or Radiant Heat

When a precast concrete member is cured by the waterproof cover method, steam, or radiant heat, the compressive strength test cylinders made for any of the above purposes shall be cured under conditions similar to the member. Such concrete shall be considered to be acceptable whenever a test indicates that the concrete has reached the specified compressive strength provided such strength is reached no later than the specified age for the compressive strength.

Test cylinders shall be cured by only one of the following methods:

- For concrete with specified design compressive strengths less than or equal to 6.0 ksi, test cylinders shall be stored next to the member and under the same covers such that the cylinders are exposed to the same temperature conditions as the member.

- For all specified concrete strengths, test cylinders shall be match-cured in chambers in which the temperature of the chamber is correlated with the temperature in the member prior to release of the prestressing strands. Temperatures of the chamber and member shall be verified by use of temperature sensors in the chamber and member. Unless specified otherwise, temperature sensors in I-beams shall be located at the center of gravity of the bottom flange. For other members, the temperature sensors shall be located at the center of the thickest section. The location shall be specified in the contract documents. After release of the prestressing strands, cylinders shall be stored in a similar temperature and humidity environment as the member.

8.6—PROTECTION OF CONCRETE FROM ENVIRONMENTAL CONDITIONS

8.6.1—General

Precautions shall be taken as needed to protect concrete from damage due to weather or other environmental conditions during placing and curing operations. Concrete that has been frozen or otherwise damaged by weather conditions shall be either repaired to an acceptable condition or removed and replaced.

The temperature of the concrete mixture immediately before placement shall be between 50°F and 90°F, except as otherwise provided herein.

8.6.2—Rain Protection

Under conditions of rain, the placing of concrete shall not commence or shall be stopped unless adequate protection is provided to prevent damage to the surface mortar or damaging flow or wash of the concrete surface.

8.6.3—Hot-Weather Protection

When the ambient temperature is above 90°F, the forms, reinforcing steel, steel beam flanges, and other surfaces which will come in contact with the mix shall be cooled to below 90°F by means of a water spray or other approved methods.

The temperature of the concrete at time of placement shall be maintained within the specified temperature range by any combination of the following:

- Shading the materials storage areas or the production equipment.
- Cooling the aggregates by sprinkling with water which conforms to the requirements of Article 8.3.2, "Water."
- Cooling the aggregates or water by refrigeration or replacing a portion or all of the mix water with ice that is flaked or crushed to the extent that the ice will completely melt during mixing of the concrete.
- Injecting liquid nitrogen.

8.6.4—Cold-Weather Protection

8.6.4.1—Protection during Cure

When there is a probability of air temperatures below 35°F during the cure period, the Contractor shall submit for approval by the Engineer prior to concrete placement a cold-weather concreting and curing plan detailing the methods and equipment that will be used to ensure that the required concrete temperatures are maintained. The concrete shall be maintained at a temperature of not less than 45°F for the first six days after placement, except that when pozzolan or slag are used, this period shall be as shown in Table 8.6.4.1-1.

Table 8.6.4.1-1—Pozzolan Cement and Temperature Control Period

Percentage of Cement Replaced, by Weight (Mass), with Pozzolans	Slag	Required Period of Controlled Temperature
10%	25%	8 Days
11–15%	26–35%	9 Days
16–25%	36–50%	10 Days

C8.6.4.1

Addition of pozzolans or slag may result in slower development of properties. Therefore, longer curing periods may be needed. Thermal heating and cooling rates are limited to minimize the thermal strains. Dosing for further retardation using admixtures should be done by trial batch.

The requirement in Table 8.6.4.1-1 for an extended period of controlled temperature may be waived if a compressive strength of 65 percent of the specified design strength is achieved in six days using site-cured cylinders, the match-curing system, or the maturity method.

When combinations of materials are used as cement replacement, the required period of controlled temperature shall be at least six days and shall continue until a compressive strength of 65 percent of the specified design strength is achieved using site-cured cylinders, the match-curing system, or the maturity method.

If external heating is employed, the heat shall be applied and withdrawn gradually and uniformly so that no part of the concrete surface is heated to more than 90°F or caused to change temperature by more than 20°F in 8 h.

When requested by the Engineer, the Contractor shall provide and install two maximum-minimum type thermometers at each structure site. Such thermometers shall be installed as directed by the Engineer so as to monitor the temperature of the concrete and the surrounding air during the cure period.

8.6.4.2—Mixing and Placing

When the air temperature is below 35°F, the temperature of the concrete at the time of placement in sections less than 12.0 in. thick shall be not less than 60°F. Regardless of air temperature, aggregates shall be free of ice, frost, and frozen lumps when batched and concrete shall not be placed against any material whose temperature is 32°F or less.

8.6.4.3—Heating of Mix

When necessary in order to produce concrete of the specified temperature, either the mix water, the aggregates, or both shall be heated prior to batching. Heating shall be done in a manner which is not detrimental to the mix and does not prevent the entrainment of the required amount of air. The methods used shall heat the materials uniformly. Aggregates shall not be heated directly by gas or oil flame or on sheet metal over fire. Neither aggregates nor water shall be heated to over 150°F. If either are heated to over 100°F, they shall be mixed together prior to the addition of the cement so that the cement does not come into contact with materials which are in excess of 100°F.

8.6.5—Special Requirements for Bridge Decks

During periods of low humidity, wind, or high temperatures and prior to the application of curing materials, concrete being placed and finished for bridge decks shall be protected from damage due to rapid evaporation. Such protection shall be adequate to prevent premature crusting of the surface or an increase in drying cracking. Such protection shall be provided by raising the humidity of the surrounding air with fog sprayers operated upwind of the deck, by employing wind-breaks or sun-shades, additionally reducing the temperature of the concrete, scheduling placement during the cooler times of days or nights, or any combination thereof.

For bridge decks that are located over or adjacent to salt water or when specified in the contract documents, the maximum temperature of the concrete at time of placement shall be 80°F.

8.6.6—Concrete Exposed to Salt Water

Unless otherwise specified in the contract documents, concrete for structures exposed to salt or brackish water shall comply with the requirements of Class A(HPC) concrete. Such concrete shall be mixed for a period of not less than 2 min and the water content of the mixture shall be carefully controlled and regulated so as to produce concrete of maximum impermeability. The concrete shall be thoroughly consolidated as necessary to produce maximum density and a complete lack of rock pockets. Unless otherwise indicated in the contract documents, the clear distance from the face of the concrete to the reinforcing steel shall be not less than 4.0 in. No construction joints shall be formed between levels of extreme low water and extreme high water or the upper limit of wave action as determined by the Engineer. Between these levels, the forms shall not be removed, or other means provided, to prevent salt water from coming in direct contact with the concrete for a period of not less than 30 days after placement. Except for the repair of any rock pockets and the plugging of form tie holes, the original surface as the concrete comes from the forms shall be left undisturbed. Special handling shall be provided for precast members to avoid even slight deformation cracks.

C8.6.6

Penetration of harmful solutions accelerates the deterioration of concrete. The most widely experienced environmental distress is the corrosion of the reinforcing steel. Chloride solutions destroy the protective coating around the reinforcing steel, initiating and accelerating the corrosion of the steel. Concrete should be prepared using the proper ingredients and proportions and cured for a period of time before exposure to the severe environment such that the penetration of the harmful solutions is minimized.

8.6.7—Concrete Exposed to Sulfate Soils or Sulfate Water

When the contract documents identify the area as containing sulfate soils or sulfate water, the concrete that will be in contact with such soil or water shall be Class A(HPC) and shall be mixed, placed, and protected from contact with soil or water as required for concrete exposed to salt water, except that the protection period shall be not less than 72 h.

C8.6.7

Sulfate soils or water may contain high levels of sulfates of sodium, potassium, calcium, or magnesia. Penetration of sulfate solutions into concrete may result in chemical reactions that cause disintegration of concrete. Therefore, special precautions may be needed to minimize the intrusion of harmful sulfate solutions. Avoidance of construction joints that may facilitate the intrusion of sulfate solutions, proper material selection and proportioning, production of low permeability concrete, and avoidance of cracking through proper curing are needed.

8.7—HANDLING AND PLACING CONCRETE

8.7.1—General

Concrete shall be handled, placed, and consolidated by methods that will not cause segregation of the mix and will result in a dense homogeneous concrete that is free of voids and rock pockets. The methods used shall not cause displacement of reinforcing steel or other materials to be embedded in the concrete. Concrete shall be placed and consolidated prior to initial set and in no case more than 1.5 h after the cement was added to the mix. Concrete shall not be retempered.

Concrete shall not be placed until the forms, all materials to be embedded, and, for spread footings, the adequacy of the foundation material, have been inspected and approved by the Engineer. All mortar from previous placements, debris, and foreign material shall be removed from the forms and steel prior to commencing placement. The forms and subgrade shall be thoroughly moistened with water immediately before concrete is placed against them. Temporary form spreader devices may be left in place until concrete placement precludes their need, after which they shall be removed.

Placement of concrete for each section of the structure shall be done continuously without interruption between planned construction or expansion joints. The delivery rate, placing sequence, and methods shall be such that fresh concrete is always placed and consolidated against previously placed concrete before initial set has occurred in the previously placed concrete.

During and after placement of concrete, care shall be taken not to injure the concrete or break the bond with reinforcing steel. Workers shall not walk in fresh concrete. Platforms for workers and equipment shall not be supported directly on any reinforcing steel. Once the concrete is set, forces shall not be applied to the forms or to reinforcing bars which project from the concrete until the concrete is of sufficient strength to resist damage.

8.7.2—Sequence of Placement

Whenever a concrete placement plan or schedule is specified or approved, the sequence of placement shall conform to the plan. Unless otherwise specifically permitted by the contract documents, the requirements of the following paragraphs shall apply.

8.7.2.1—Vertical Members

Concrete for columns, substructure and culvert walls, and other similar vertical members shall be placed and allowed to set and settle for a period of time before concrete for integral horizontal members, such as caps, slabs, or footings, is placed. Such period shall be adequate to allow completion of settlement due to loss of bleed water and shall be not less than 12 h for vertical members over 15.0 ft in height and not less than 30 min for members over 5.0 ft, but not over 15.0 ft in height.

When friction collars or falsework brackets are mounted on such vertical members and unless otherwise approved, the vertical member shall have been in place at least seven days and shall have attained its specified strength before loads from horizontal members are applied.

8.7.2.2—Superstructures

Unless otherwise permitted, no concrete shall be placed in the superstructure until substructure forms have been stripped sufficiently to determine the character of the supporting substructure concrete.

Concrete for T-beam or deck girder spans whose depth is less than 4.0 ft may be placed in one continuous operation or may be placed in two separate operations; first, to the top of the girder stems, and second, to completion. For T-beam or deck girder spans whose depth is 4.0 ft or more, and unless the falsework is nonyielding, such concrete shall be placed in two operations, and at least five days shall elapse after placement of stems before the top deck slab is placed.

Concrete for box girders may be placed in two or three separate operations consisting of bottom slab, girder stems, and top slab. In either case, the bottom slab shall be placed first and, unless otherwise permitted by the Engineer, the top slab shall not be placed until the girder stems have been in place for at least five days.

8.7.2.3—Arches

The concrete in arch rings shall be placed in such a manner as to load the centering uniformly and symmetrically. Arch rings shall be cast in transverse sections of such size that each section can be cast in a continuous operation. The arrangement of the sections and the sequence of placing shall be as approved and shall be such as to avoid the creation of initial stress in the reinforcement. The sections shall be bonded together by suitable keys or dowels. Unless prohibited by the contract documents, arch barrels for culverts and other arches may be cast in a single continuous operation.

8.7.2.4—Box Culverts

In general, the base slab or footings of box culverts shall be placed and allowed to set before the remainder of the culvert is constructed. For culverts whose wall height is 5.0 ft or less, the sidewalls and top slab may be placed in one continuous operation. For higher culvert walls, the requirements for vertical members shall apply.

8.7.2.5—Precast Elements

The sequence of placement for concrete in precast elements shall be such that sound, well-consolidated concrete that is free of settlement or shrinkage cracks is produced throughout the member.

8.7.3—Placing Methods

8.7.3.1—General

Concrete shall be placed as nearly as possible in its final position, and the use of vibrators for extensive shifting of the weight (mass) of fresh concrete will not be permitted.

Concrete shall be placed in horizontal layers of a thickness not exceeding the capacity of the vibrator to consolidate the concrete and merge it with the previous lift. In no case shall the depth of a lift exceed 2.0 ft. The rate of concrete placement shall not exceed that assumed for the design of the forms as corrected for the actual temperature of the concrete being placed.

When placing operations would involve dropping the concrete more than 5.0 ft, the concrete shall be dropped through a tube fitted with a hopper head or through other approved devices, as necessary to prevent segregation of the mix and spattering of mortar on steel and forms above the elevation of the lift being placed. This requirement shall not apply to cast-in-place piling when concrete placement is completed before initial set occurs in the first placed concrete.

8.7.3.2—Equipment

All equipment used to place concrete shall be of adequate capacity and designed and operated so as to prevent segregation of the mix or loss of mortar. Such equipment shall not cause vibrations that might damage the freshly placed concrete. No equipment shall have aluminum parts which come in contact with the concrete. Between uses, the mortar coating inside of placing equipment which sets or dries out shall be cleaned from the equipment before use is resumed.

Chutes shall be lined with smooth watertight material and, when steep slopes are involved, shall be equipped with baffles or reverses.

Concrete pumps shall be operated such that a continuous stream of concrete without air pockets is produced. When pumping is completed, the concrete remaining in the pipeline, if it is to be used, shall be ejected in such a manner that there will be no contamination of the concrete or separation of the ingredients.

Conveyor belt systems shall not exceed a total length of 550.0 linear ft, measured from end to end of the total assembly. The belt assembly shall be so arranged that each section discharges into a vertical hopper arrangement to the next section. To keep segregation to a minimum, scrapers shall be situated over the hopper of each section so as to remove mortar adhering to the belt and to deposit it into the hopper. The discharge end of the conveyor belt system shall be equipped with a hopper and a chute or suitable deflectors to cause the concrete to drop vertically to the deposit area.

8.7.4—Consolidation

All concrete, except concrete placed under water and concrete otherwise exempt, shall be consolidated by mechanical vibration immediately after placement.

Except as noted herein, vibration shall be internal. External form vibrators may be used for thin sections when the forms have been designed for external vibration.

Vibrators shall be of approved type and design and of a size appropriate for the work. They shall be capable of transmitting vibration to the concrete at frequencies of not less than 75 Hz.

The Contractor shall provide a sufficient number of vibrators to properly compact each batch of concrete immediately after it is placed in the forms. The Contractor shall also have at least one spare vibrator immediately available in case of breakdown.

Vibrators shall be manipulated so as to thoroughly work the concrete around the reinforcement and embedded fixtures and into the corners and angles of the forms. Vibration shall be applied at the point of deposit and in the area of freshly deposited concrete. The vibrators shall be inserted and withdrawn out of the concrete slowly. The vibration shall be of sufficient duration and intensity to thoroughly consolidate the concrete but shall not be continued so as to cause segregation. Vibration shall not be continued at any one point to the extent that localized areas of grout are formed. Application of vibrators shall be at points uniformly spaced and not farther apart than 1.5 times the radius over which the vibration is visibly effective.

Vibration shall not be applied either directly to, or through the reinforcement to, sections or layers of concrete which have hardened to the degree that the concrete ceases to be plastic under vibration. Vibrators shall not be used to transport concrete in the forms.

Where immersion-type vibrators are used to consolidate concrete around epoxy-coated reinforcing steel, the vibrators shall be equipped with rubber or other nonmetallic coating.

Vibration shall be supplemented by such spading as is necessary to ensure smooth surfaces and dense concrete along form surfaces and in corners and locations impossible to reach with the vibrators.

When approved by the Engineer, concrete for small noncritical elements may be consolidated by the use of suitable rods and spades.

8.7.5—Underwater Placement

8.7.5.1—General

Only concrete used in cofferdams to seal out water may be placed under water, unless otherwise specified in the contract documents or specifically approved by the Engineer. If other than Class S concrete is to be placed under water, the minimum cement content of the mix shall be increased by ten percent to compensate for loss due to wash.

To prevent segregation, concrete placed under water shall be carefully placed in a compact mass, in its final position, by means of a tremie, concrete pump, or other approved method and shall not be disturbed after being deposited. Still water shall be maintained at the point of deposit and the forms under water shall be watertight. Cofferdams shall be vented during the placement and cure of concrete to equalize the hydrostatic pressure and thus prevent flow of water through the concrete.

Concrete placed under water shall be placed continuously from start to finish. The surface of the concrete shall be kept as nearly horizontal as practicable. To ensure thorough bonding, each succeeding layer of seal shall be placed before the preceding layer has taken initial set. For large pours, more than one tremie or pump shall be used to ensure compliance with this requirement.

8.7.5.2—Equipment

A tremie shall consist of a watertight tube having a diameter of not less than 10.0 in. and fitted with a hopper at the top. The tremies shall be supported so as to permit free movement of the discharge end over the entire top surface of the work and so as to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end shall be sealed closed at the start of work so as to prevent water from entering the tube before the tube is filled with concrete. After placement has started, the tremie tube shall be kept full of concrete to the bottom of the hopper. If water enters the tube after placement is started, the tremie shall be withdrawn, the discharge end resealed, and the placement restarted. When a batch is dumped into the hopper, the flow of concrete shall be induced by slightly raising the discharge end, always keeping it in the deposited concrete. The flow shall be continuous until the work is completed. When cofferdam struts prevent lateral movement of tremies, one tremie shall be used in each bay.

Concrete pumps used to place concrete under water shall include a device at the end of the discharge tube to seal out water while the tube is first being filled with concrete. Once the flow of concrete is started, the end of the discharge tube shall be kept full of concrete and below the surface of the deposited concrete until placement is completed.

8.7.5.3—Clean-Up

Dewatering may proceed after test specimens cured under similar conditions indicate that the concrete has sufficient strength to resist the expected loads. All laitance or other unsatisfactory materials shall be removed from the exposed surface by scraping, chipping, or other means which will not injure the surface of the concrete before placing foundation concrete.

8.8—CONSTRUCTION JOINTS

8.8.1—General

Construction joints shall be made only where specified in the contract documents, or shown in the pouring schedule, unless otherwise approved. All planned reinforcing steel shall extend uninterrupted through joints. In the case of emergency, construction joints shall be placed as directed by the Engineer and, if directed, additional reinforcing steel dowels shall be placed across the joint. Such additional steel shall be furnished and placed at the Contractor's expense.

8.8.2—Bonding

Unless otherwise specified in the contract documents, horizontal joints may be made without keys, and vertical joints shall be constructed with shear keys. Surfaces of fresh concrete at horizontal construction joints shall be rough floated sufficiently to thoroughly consolidate the surface and intentionally left in a roughened condition. Shear keys shall consist of formed depressions in the surface covering approximately one-third of the contact surface. The forms for keys shall be beveled so that removal will not damage the concrete.

All construction joints shall be cleaned of surface laitance, curing compound, and other foreign materials before fresh concrete is placed against the surface of the joint. Abrasive blast or other approved methods shall be used to clean horizontal construction joints to the extent that clean aggregate is exposed. All construction joints shall be flushed with water and allowed to dry to a surface dry condition immediately prior to placing concrete.

8.8.3—Bonding and Doweling to Existing Structures

When the contract documents specify that new concrete be bonded to existing concrete structures, the existing concrete shall be cleaned and flushed as specified in Article 8.8.2, "Bonding." When the contract documents show reinforcing dowels grouted into holes drilled in the existing concrete at such construction joints, the holes shall be drilled by methods that will not shatter or damage the concrete adjacent to the holes. The diameters of the drilled holes shall be 0.25 in. larger than the nominal diameter of the dowels unless shown otherwise in the contract documents. The grout shall be a neat cement paste of portland cement and water.

The water content shall be not more than 4 gal/94 lb of cement. Retempering of grout will not be permitted. Immediately prior to placing the dowels, the holes shall be cleaned of dust and other deleterious materials, shall be thoroughly saturated with water, shall have all free water removed, and the holes shall be dried to a saturated surface-dry condition. Sufficient grout shall be placed in the holes so that no voids remain after the dowels are inserted. Grout shall be cured for a period of at least three days or until dowels are encased in concrete.

When specified in the contract documents or approved by the Engineer, epoxy may be used in lieu of portland cement grout for the bonding of dowels in existing concrete. When used, epoxy shall be mixed and placed in accordance with the Manufacturer's recommendations.

8.8.4—Forms at Construction Joints

When forms at construction joints overlap previously placed concrete, they shall be retightened before depositing new concrete. The face edges of all joints that are exposed to view shall be neatly formed with straight bulkheads or grade strips, or otherwise carefully finished true to line and elevation.

8.9—EXPANSION AND CONTRACTION JOINTS

8.9.1—General

Expansion and contraction joints shall be constructed at the locations and in accordance with the details specified in the contract documents. Such joints include open joints, filled joints, joints sealed with sealants or waterstops, joints reinforced with steel armor plates or shapes, and joints with combinations of these features.

When preformed elastomeric compression joint seals or bridge deck joint seal assemblies are required, they shall conform to the requirements of Section 19, “Bridge Deck Joint Seals.”

8.9.2—Materials

8.9.2.1—Premolded Expansion Joint Fillers

Premolded fillers shall conform to one of the following specifications:

- Specification for Preformed Expansion Joint Fillers for Concrete Paving and Structural Construction, AASHTO M 213 (ASTM D1751).

- Specification for Preformed Sponge Rubber and Cork Expansion Joint Fillers for Concrete Paving and Structural Construction, AASHTO M 153 (ASTM D1752). Type II (cork) shall not be used when resiliency is required.

- Specification for Preformed Expansion Joint Filler for Concrete, AASHTO M 33 (ASTM D994).

8.9.2.2—Polystyrene Board Fillers

Board fillers shall be expanded polystyrene with a minimum flexural strength of 0.035 ksi, as determined by ASTM C203, and a compressive yield strength of between 0.015 and 0.04 ksi at five percent compression. When specified in the contract documents, or required to prevent damage during concrete placement, the surface of polystyrene board shall be faced with 0.125-in. thick hardboard conforming to ANSI A135.4.

8.9.2.3—Contraction Joint Material

Material placed in contraction joints shall consist of asphalt saturated felt paper or other approved bond-breaking material.

8.9.2.4—Pourable Joint Sealants

Pourable sealants for placement along the top edges of contraction or filled expansion joints shall conform to one of the following:

- Hot-poured sealants shall conform to AASHTO M 282 (ASTM D3406), except that when the sealant will be in contact with asphaltic material, it shall conform to AASHTO M 301 [*discontinued in 2005*].

- Cold-poured sealant shall be silicone type, conforming to Federal Specification TT-S-1543, Class A. The sealant shall be a one-part, low-modulus silicone rubber type with an ultimate elongation of 1,200 percent.

- Polyethylene foam strip, for use when shown in the contract documents, shall be of commercial quality with a continuous, impervious, glazed top surface, suitable for retaining the liquid sealant at the proper elevation in the joint while hardening.

8.9.2.5—Metal Armor

Expansion joint armor assemblies shall be fabricated from steel in conformance with the requirements of Section 23, "Miscellaneous Metal." Assemblies shall be accurately fabricated and straightened at the shop after fabrication and galvanizing as necessary to conform to the concrete section.

8.9.2.6—Waterstops

Waterstops shall be of the type, size, and shape specified in the contract documents. They shall be dense, homogeneous, and without holes or other defects.

8.9.2.6.1—Rubber Waterstops

Rubber waterstops shall be formed from synthetic rubber made exclusively from neoprene, reinforcing carbon black, zinc oxide, polymerization agents, and softeners. This compound shall contain not less than 70 percent by volume of neoprene. The tensile strength shall not be less than 2.75 ksi with an elongation at breaking of 600 percent. The Shore Durometer indication (hardness) shall be between 50 and 60. After seven days in air at temperature of 158 ± 2°F or after four days in oxygen at 158 ± 2°F and 0.300-ksi pressure, the tensile strength shall not be less than 65 percent of the original.

Rubber waterstops shall be formed with an integral cross-section in suitable molds, so as to produce a uniform section with a permissible variation in dimension of ±0.03125 in. No splices will be permitted in straight strips. Strips and special connection pieces shall be well cured in a manner such that any cross-section shall be dense, homogeneous, and free from all porosity. Junctions in the special connection pieces shall be full molded. During the vulcanizing period, the joints shall be securely held by suitable clamps. The material at the splices shall be dense and homogeneous throughout the cross-section.

8.9.2.6.2—Polyvinyl Chloride Waterstops

Polyvinyl chloride (PVC) waterstops shall be manufactured by the extrusion process from an elastomeric plastic compound, the basic resin of which shall be PVC. The compound shall contain any additional resins, plasticizers, stabilizers, or other materials needed to ensure that, when the material is compounded, it will meet the performance requirements given in this specification. No reclaimed PVC or other material shall be used.

The material shall comply with the following physical requirements when tested under the indicated ASTM test method.

Table 8.9.2.6.2-1—ASTM Test Method Requirements

Specific Gravity	ASTM D792	1.35 Max.
Durometer Hardness	ASTM D2240	75 ± 5
Tensile Strength	ASTM D412	1.8 ksi
Elongation	ASTM D412	350%
Cold Brittleness	ASTM D746	–35°F
Stiffness in Flexure	ASTM D747	0.350 ksi

8.9.2.6.3—Copper Waterstops

Sheet copper shall conform to the Specifications for Copper Sheet, Strip, Plate, and Rolled Bar, AASHTO M 138M/M 138 (ASTM B152/B152M), and shall meet the Embrittlement Test of Section 10 of AASHTO M 138M/M 138 (ASTM B152/B152M).

8.9.2.6.4—Testing of Waterstop Material

The Manufacturer shall be responsible for the testing of all waterstop materials, either in company-owned or affiliated laboratory or in a recognized commercial laboratory, and shall submit three certified copies of test results to the Engineer.

8.9.3—Installation

8.9.3.1—Open Joints

Open joints shall be constructed by the insertion and subsequent removal of a wood strip, metal plate, or other approved material. The insertion and removal of the template shall be accomplished without chipping or breaking the corners of the concrete. When not protected by metal armor, open joints in decks and sidewalks shall be finished with an edging tool. Upon completion of concrete finishing work, all mortar and other debris shall be removed from open joints.

8.9.3.2—Filled Joints

When filled joints are specified in the contract documents, premolded-type fillers shall be used unless polystyrene board is specifically called for. Filler for each joint shall consist of as few pieces of material as possible. Abutting edges of filler material shall be accurately held in alignment with each other and tightly fit or taped as necessary to prevent the intrusion of grout. Joint filler material shall be anchored to one side of the joint by waterproof adhesive or other methods so as to prevent it from working out of the joint, but not interfere with the compression of the material.

8.9.3.3—Sealed Joints

Prior to installation of pourable joint sealants, all foreign material shall be removed from the joint, the filler material shall be cut back to the depth shown or approved, and the surface of the concrete that will be in contact with the sealant shall be cleaned by light sandblasting. When required, a polyethylene foam strip shall be placed in the joint to retain the sealant and isolate it from the filler material. The sealant materials shall then be mixed and installed in accordance with the Manufacturer's directions. Any material that fails to bond to the sides of the joint within 24 h after placement shall be removed and replaced.

8.9.3.4—Waterstops

Adequate waterstops of metal, rubber, or plastic shall be placed as specified in the contract documents. Where movement at the joint is provided for, the waterstops shall be of a type permitting such movement without injury. They shall be spliced, welded, or soldered to form continuous watertight joints.

Precautions shall be taken so that the waterstops shall be neither displaced nor damaged by construction operations or other means. All surfaces of the waterstops shall be kept free from oil, grease, dried mortar, or any other foreign matter while the waterstop is being embedded in concrete. Means shall be used to insure that all portions of the waterstop designed for embedment shall be tightly enclosed by dense concrete.

8.9.3.5—Expansion Joint Armor Assemblies

Armor assemblies shall be installed so the top surface matches the plane of the adjacent finished concrete surface throughout the length of the assembly. Positive methods shall be employed in placing the assemblies to keep them in correct position during the placing of the concrete. The opening at expansion joints shall be that designated in the contract documents at normal temperature or as directed by the Engineer for other temperatures, and care shall be taken to avoid impairment of the clearance in any manner.

8.10—FINISHING PLASTIC CONCRETE

8.10.1—General

Unless otherwise specified in the contract documents, after concrete has been consolidated and prior to the application of cure, all surfaces of concrete that are not placed against forms shall be struck-off to the planned elevation or slope and the surface finished by floating with a wooden float sufficiently to seal the surface. While the concrete is still in a workable state, all construction and expansion joints shall be carefully tooled with an edger. Joint filler shall be left exposed.

8.10.2—Roadway Surface Finish

All bridge decks, approach slabs, and other concrete surfaces for use by traffic shall be finished to a smooth skid-resistant surface in accordance with this Article. During finishing operations, the Contractor shall provide suitable and adequate work bridges for proper performance of the work, including the application of fog sprays and curing compound, and for inspecting the work.

8.10.2.1—Striking Off and Floating

After the concrete is placed and consolidated according to Article 8.7, "Handling and Placing Concrete," bridge decks or top slabs of structures serving as finished pavements shall be finished using approved power-driven finishing machines. Hand-finishing methods may be used if approved by the Engineer for short bridges 50.0 ft or less in length or for irregular areas where the use of a machine would be impractical.

All surfaces shall be struck-off by equipment supported by and traveling on rails or headers. The rails, headers, and strike-off equipment shall be of sufficient strength and be adjusted so that the concrete surface after strike-off will conform to the planned profile and cross-section.

The rails or headers shall be set on nonyielding supports and shall be completely in place and firmly secured for the scheduled length for concrete placement before placing of concrete will be permitted. Rails for finishing machines shall extend beyond both ends of the scheduled length for concrete placement a sufficient distance to permit the float of the finishing machine to fully clear the concrete to be placed. Rails or headers shall be adjustable for elevation and shall be set to allow for anticipated settlement, camber, and deflection of falsework as necessary to obtain a finished surface true to the required grade and cross-section. Rails or headers shall be of a type and shall be so installed that no springing or deflection will occur under the weight (mass) of the finishing equipment and shall be so located that finishing equipment may operate without interruption over the entire surface being finished. Rails or headers shall be adjusted as necessary to correct for unanticipated settlement or deflection that may occur during finishing operations. If rail supports are located within the area where concrete is being placed, as soon as they are no longer needed, they shall be removed to at least 2.0 in. below the finished surface and the void filled with fresh concrete.

Before the delivery of concrete is begun, the finishing machine or, if used, the hand-operated strike-off tool shall be operated over the entire area to be finished to check for excessive rail deflections and for proper deck thickness and cover on reinforcing steel, and to verify operation of all equipment. Any necessary corrections shall be made before concrete placement is begun.

The finishing machine shall go over each area of the surface as many times as it is required to obtain the required profile and cross-section. A slight excess of concrete shall be kept in front of the cutting edge of the screed at all times. This excess of concrete shall be carried all the way to the edge of the pour or form and shall not be worked into the slab, but shall be wasted.

After strike-off, the surface shall be finished with a float, roller, or other approved device as necessary to remove any local irregularities and to leave sufficient mortar at the surface of the concrete for later texturing.

During finishing operations, excess water, laitance, or foreign materials brought to the surface during the course of the finishing operations shall not be reworked into the slab, but shall be removed immediately upon appearance by means of a squeegee or straightedge drawn from the center of the slab towards either edge.

The addition of water to the surface of the concrete to assist in finishing operations will not be permitted.

8.10.2.2—Straightedging

After finishing as described above, the entire surface shall be checked by the Contractor with a 10.0-ft metal straightedge operated parallel to the centerline of the bridge and shall show no deviation in excess of 0.125 in. from the testing edge of the straightedge. For deck surfaces that are to be overlaid with 1.0 in. or more of another material, such deviation shall not exceed 0.375 in. in 10.0 ft. Deviations in excess of these requirements shall be corrected before the concrete sets. The checking operation shall progress by overlapping the straightedge at least one-half the length of the preceding pass.

8.10.2.3—Texturing

The surface shall be given a skid-resistant texture by either burlap or carpet dragging, brooming, tining, or by a combination of these methods. The method employed shall be as specified in the contract documents or as approved by the Engineer. Surfaces that are to be covered with a waterproofing membrane deck seal shall not be coarse textured. They shall be finished to a smooth surface, free of mortar ridges and other projections.

This operation shall be done after floating and at such time and in such manner that the desired texture will be achieved while minimizing displacement of the larger aggregate particles.

8.10.2.3.1—Dragged

If the surface texture is to be a drag finish, the surface shall be finished by dragging a seamless strip of damp burlap over the full width of the surface. The burlap drag shall consist of sufficient layers of burlap and have sufficient length in contact with the concrete to slightly groove the surface and shall be moved forward with a minimum bow of the lead edge. The drag shall be kept damp, clean, and free of particles of hardened concrete. As an alternative to burlap, the Engineer may approve or direct that carpet or artificial turf of approved type and size be substituted.

8.10.2.3.2—Broomed

If the surface texture is to be a broom finish, the surface shall be broomed when the concrete has hardened sufficiently. The broom shall be of an approved type. The strokes shall be square across the slab, from edge to edge, with adjacent strokes slightly overlapped, and shall be made by drawing the broom without tearing the concrete but so as to produce regular corrugations not over 0.125 in. in depth. The surface as thus finished shall be free from porous spots, irregularities, depressions, and small pockets or rough spots such as may be caused by the accidental disturbing of particles of coarse aggregate embedded near the surface during the final brooming operation.

8.10.2.3.3—Tined

If the surface is to be tined, the tining shall be in a transverse direction using a wire broom, comb, or finned float having a single row of tines or fins. The tining grooves shall be between 0.0625 in. and 0.1875 in. wide and between 0.125 in. and 0.1875 in. deep, spaced 0.5 in. to 0.75 in. on centers. Tining shall be discontinued 12.0 in. from the curb line on bridge decks. The area adjacent to the curbs shall be given a light broom finish longitudinally. As an alternative, tining may be achieved using an approved machine designed specifically for tining or grooving concrete pavements.

8.10.2.4—Surface Testing and Correction

After the concrete has hardened, the Engineer will inspect finished deck roadway surfaces that will not be overlain with a wearing surface. Any variations in the surface which exceed 0.125 in. from a 10.0-ft straightedge will be marked. The Contractor shall correct such irregularities by the use of concrete planing or grooving equipment which produces a textured surface equal in roughness to the surrounding unground concrete without shattering or otherwise damaging the remaining concrete.

8.10.3—Pedestrian Walkway Surface Finish

After the concrete for sidewalks and decks of pedestrian structures has been deposited in place, it shall be consolidated and the surface shall be struck off by means of a strike board and floated with wooden or cork float. If directed, the surface shall then be lightly broomed in a transverse direction. An edging tool shall be used on edges and expansion joints. The surface shall not vary more than 0.125 in. under a 5.0-ft straightedge. The surface shall have a granular or matte texture that will not be slippery when wet.

Sidewalk surfaces shall be laid out in blocks with an approved grooving tool as specified in the contract documents or as directed by the Engineer.

8.10.4—Troweled and Brushed Finish

Surfaces that are specified in the contract documents to be troweled shall first be finished as specified under Article 8.10.1, "General." Then, after the concrete is partially set, the surface shall be finished to a smooth surface by troweling with a steel trowel until a slick surface free of bleed water is produced. The surface shall then be brushed with a fine brush using parallel strokes.

8.10.5—Surface under Bearings

When metallic masonry plates are to be placed directly on the concrete or on filler material less than 0.125 in. thick, the surface shall first be finished with a float finish. After the concrete has set, the area which will be in contact with the masonry plate shall be ground as necessary to provide full and even bearing. When such plates are to be set on filler material between 0.125 in. and 0.5 in. thick, the concrete surface shall be steel trowel finished without brushing and the flatness of the finished surface shall not vary from a straightedge laid on the surface in any direction within the limits of the masonry plate by more than 0.0625 in. Surfaces which fail to conform to the required flatness shall be ground until acceptable.

Surfaces under elastomeric bearings and under metallic masonry plates which are supported on mortar or filler pads 0.5 in. or greater in thickness shall be finished by wood floating to a flat and even surface free of ridges.

8.11—CURING CONCRETE

8.11.1—General

All newly placed concrete shall be cured so as to prevent loss of water by use of one or more of the methods specified herein. Except for Class A(HPC) concrete, curing shall commence immediately after the free water has left the surface and finishing operations are completed. For Class A(HPC) concrete, water curing shall commence immediately after finishing operations are complete. If the surface of the concrete begins to dry before the selected cure method can be applied, the surface of the concrete shall be kept moist by a fog spray applied so as not to damage the surface.

Curing by other than waterproof cover, steam, or radiant-heat methods with precast concrete shall continue uninterrupted for seven days except for when pozzolans in excess of ten percent, by weight (mass), of the portland cement are used in the mix. When such pozzolans are used, the curing period shall be ten days. For other than top slabs of structures serving as finished pavements and Class A(HPC) concrete, the above curing periods may be reduced and curing terminated when test cylinders cured under the same conditions as the structure indicate that concrete strengths of at least 70 percent of that specified have been reached.

When deemed necessary by the Engineer during periods of hot weather, water shall be applied to concrete surfaces being cured by the liquid membrane method or by the forms-in-place method, until the Engineer determines that a cooling effect is no longer required. Such application of water will be paid for as extra work.

8.11.2—Materials

8.11.2.1—Water

Water shall conform to the requirements of Article 8.3.2, "Water."

8.11.2.2—Liquid Membranes

Liquid membrane-forming compounds for curing concrete shall conform to the requirements of AASHTO M 148 (ASTM C309).

8.11.2.3—Waterproof Sheet Materials

Waterproof paper, polyethylene film, and white burlap polyethylene sheet shall conform to the requirements of AASHTO M 171 (ASTM C171).

8.11.3—Methods

8.11.3.1—Forms-in-Place Method

Formed surfaces of concrete may be cured by retaining the forms in place without loosening for the required time.

8.11.3.2—Water Method

Concrete surface shall be kept continuously wet by ponding, spraying, or covering with materials that are kept continuously and thoroughly wet. Such materials may consist of cotton mats, multiple layers of burlap, or other approved materials that do not discolor or otherwise damage the concrete.

8.11.3.3—Liquid Membrane Curing Compound Method

The liquid membrane method shall not be used on surfaces where a rubbed finish is required or on surfaces of construction joints unless it is removed by sand blasting prior to placement of concrete against the joint. Type 2, white pigmented, liquid membranes may be used only on the surfaces of bridge decks, on surfaces that will not be exposed to view in the completed work, or on surfaces where their use has been approved by the Engineer.

When membrane curing is used, the exposed concrete shall be thoroughly sealed immediately after the free water has left the surface. Formed surfaces shall be sealed immediately after the forms are removed and necessary finishing has been done. The solution shall be applied by power-operated atomizing spray equipment in one or two separate applications. Hand-operated sprayers may be used for coating small areas. Membrane solutions containing pigments shall be thoroughly mixed prior to use and agitated during application. If the solution is applied in two increments, the second application shall follow the first application within 30 min. Satisfactory equipment shall be provided, together with means to properly control and assure the direct application of the curing solution on the concrete surface so as to result in a uniform coverage at the rate of 1 gal/150.0 ft^2.

If rain falls on the newly coated concrete before the film has dried sufficiently to resist damage, or if the film is damaged in any other manner during the curing period, a new coat of the solution shall be applied to the affected portions equal in curing value to that specified above.

8.11.3.4—Waterproof Cover Method

This method shall consist of covering the surface with a waterproof sheet material so as to prevent moisture loss from the concrete. This method may be used only when the covering can be secured adequately to prevent moisture loss.

The concrete shall be wet at the time the cover is installed. The sheets shall be of the widest practicable width and adjacent sheets shall overlap a minimum of 6.0 in. and shall be tightly sealed with pressure sensitive tape, mastic, glue, or other approved methods to form a complete waterproof cover of the entire concrete surface. The paper shall be secured so that wind will not displace it. Should any portion of the sheets be broken or damaged before expiration of the curing period, the broken or damaged portions shall be immediately repaired. Sections that have lost their waterproof qualities shall not be used.

8.11.3.5—Steam or Radiant-Heat Curing Method

This method may be used only for precast concrete members manufactured in established plants.

Steam curing or radiant-heat curing shall be done under a suitable enclosure to contain the live steam or the heat. Steam shall be low-pressure and saturated. Temperature recording devices shall be employed as necessary to verify that temperatures are uniform throughout the enclosure and within the limits specified in the contract documents.

The initial application of the steam or of the heat shall not occur prior to initial set of the concrete except to maintain the temperature within the curing chamber above the specified minimum temperature. The time of initial set may be determined by the Standard Method of Test for Time of Setting of Concrete Mixtures by Penetration Resistance, AASHTO T 197 (ASTM C403/C403M).

During the waiting period, the temperature within the curing chamber shall not be less than 50°F and live steam or radiant heat may be used to maintain the curing chamber at the proper minimum temperature. During this period the concrete shall be kept wet.

C8.11.3.5

Since high-strength concrete generates more heat of hydration than conventional strength concretes, it is important that concrete temperatures be monitored rather than enclosure temperatures. It is also important that transfer of prestressing force to the concrete occur before the temperature of the concrete decreases. Otherwise, vertical cracking in the girders may result.

For Class P(HPC) concrete, temperature-sensing devices should be placed within the concrete to verify that temperatures are uniform throughout the concrete and within the limits specified.

Application of live steam shall not be directed on the concrete or on the forms so as to cause localized high temperatures. During the initial application of live steam or of radiant heat, the temperature within the concrete shall increase at an average rate not exceeding 40°F per hour until the curing temperature is reached. The maximum curing temperature within the concrete shall not exceed 160°F. The maximum temperature shall be held until the concrete has reached the desired strength. In discontinuing the steam application, the concrete temperature shall not decrease at a rate to exceed 40°F per hour until a temperature 20°F above the temperature of the air to which the concrete will be exposed has been reached.

Radiant heat may be applied by means of pipes circulating steam, hot oil, or hot water, or by electric heating elements. Radiant-heat curing shall be done under a suitable enclosure to contain the heat, and moisture loss shall be minimized by covering all exposed concrete surfaces with a plastic sheeting or by applying an approved liquid membrane curing compound to all exposed concrete surfaces. Top surfaces of concrete members to be used in composite construction shall be clear of residue of the membrane curing compound so as not to reduce bond below design limits. Surfaces of concrete members to which other materials will be bonded in the finished structure shall be clear of residue of the membrane curing compound so as not to reduce bond below design limits.

For prestressed members, the transfer of the stressing force to the concrete shall be accomplished immediately after the steam curing or heat-curing has been discontinued.

8.11.4—Bridge Decks

The top surfaces of bridge decks shall be cured by a combination of the liquid membrane curing compound method and the water method. The liquid membrane shall be Type 2, white pigmented, and shall be applied from finishing bridges progressively and immediately after finishing operations are complete on each portion of the deck. The water cure shall be applied not later than 4 h after completion of deck finishing or, for portions of the decks on which finishing is completed.

When Class A(HPC) concrete is used in bridge decks, water cure shall be applied immediately after the finishing of any portion of the deck is complete and shall remain in place for a minimum period of seven days irrespective of concrete strength. If conditions prevent immediate application of the water cure, an evaporation retardant shall be applied immediately after completion of finishing or fogging shall be used to maintain a high relative humidity above the concrete to prevent drying of the concrete surface. Following the water cure period, liquid membrane curing compound may be applied to extend the curing period.

C8.11.4

High performance concrete tends to have very little bleed water, especially when a low water-cementitious materials ratio is used with mineral admixtures. As a result, the evaporation protection of the bleed water on the fresh concrete is lost. The most effective way to protect the concrete is by application of water curing as soon as screeding or tining of the concrete is complete, but no later than 15 min after the concrete is placed in any portion of the deck. If this is not possible, the next best alternative is to prevent or reduce moisture loss from the concrete until the water cure can be applied.

In the water cure method, the concrete surface is kept continuously wet. The most appropriate method is to cover the deck with materials such as cotton mats, multiple layers of burlap, or other materials that do not discolor or damage the concrete surface and to keep these materials continuously and thoroughly wet. The water cure needs to continue for a minimum of seven days irrespective of concrete strength. The use of a curing compound after the water cure extends the curing period while allowing the Contractor to have access to the bridge deck.

The liquid membrane, Type 2, white pigmented is as defined in AASHTO M 148 (ASTM C309).

8.12—FINISHING FORMED CONCRETE SURFACES

8.12.1—General

Surface finishes for formed concrete surfaces shall be classified as follows:

Class 1—Ordinary Surface Finish
Class 2—Rubbed Finish
Class 3—Tooled Finish
Class 4—Sandblast Finish
Class 5—Wire Brush or Scrubbed Finish

All concrete shall be given a Class 1, Ordinary Surface Finish and, in addition, if further finishing is required, such other type of finish as is specified.

If not otherwise specified in the contract documents, exposed surfaces except the soffits of superstructures and the interior faces and bottoms of concrete girders shall also be given a Class 2, Rubbed Finish.

Class 3, 4, or 5 type surface finishes shall be applied only where specified in the contract documents.

8.12.2—Class 1—Ordinary Surface Finish

Immediately following the removal of forms, fins and irregular projections shall be removed from all surfaces that are to be exposed or waterproofed. Bulges and offsets in such surfaces shall be removed with carborundum stones or discs.

Localized, poorly bonded rock pockets or honeycombed concrete shall be removed and replaced with sound concrete or packed mortar as specified in Article 8.14, "Mortar and Grout." If, in the opinion of the Engineer, rock pockets are of such an extent or character as to affect the strength of the structure materially or to endanger the life of the steel reinforcement, the Engineer may declare the concrete defective and require the removal and replacement of the portions of the structure affected.

On all surfaces, the cavities produced by form ties and all other holes, broken corners or edges, and other defects shall be thoroughly cleaned and, after having been thoroughly saturated with water, shall be carefully pointed and trued with a mortar conforming to Article 8.14. For exposed surfaces, white cement shall be added to the mortar in an amount sufficient to result in a patch that, when dry, matches the surrounding concrete. Mortar used in pointing shall be not more than 1 h old. The concrete shall then be rubbed if required or the cure continued as specified in Article 8.10, "Finishing Plastic Concrete." Construction and expansion joints in the completed work shall be left carefully tooled and free of mortar and concrete. The joint filler shall be left exposed for its full length with clean and true edges.

The resulting surfaces shall be true and uniform. Repaired surfaces, the appearance of which is not satisfactory, shall be "rubbed" as specified in Article 8.12.3, "Class 2—Rubbed Finish."

8.12.3—Class 2—Rubbed Finish

After removal of forms, the rubbing of concrete shall be started as soon as its condition will permit. Immediately before starting this work, the concrete shall be thoroughly saturated with water. Sufficient time shall have elapsed before the wetting down to allow the mortar used in the pointing of rod holes and defects to thoroughly set. Surfaces to be finished shall be rubbed with a medium-coarse carborundum stone, using a small amount of mortar on its face. The mortar shall be composed of cement and fine sand, mixed in proportions used in the concrete being finished. Rubbing shall be continued until form marks, projections, and irregularities have been removed; voids have been filled; and a uniform surface has been obtained. The paste produced by this rubbing shall be left in place.

After other work that could affect the surface has been completed, the final finish shall be obtained by rubbing with a fine carborundum stone and water. This rubbing shall be continued until the entire surface is of a smooth texture and uniform color.

After the final rubbing is completed and the surface has dried, it shall be rubbed with burlap to remove loose powder and shall be left free from all unsound patches, paste, powder, and objectionable marks.

When metal forms, fiber forms, lined forms, or plywood forms in good condition are used, the requirement for a Class 2, Rubbed Finish may be waived by the Engineer when the uniformity of color and texture obtained with Class 1 finishing are essentially equal to that which could be attained with the application of a Class 2, Rubbed Finish. In such cases, grinding with powered disc grinders or light sandblasting with fine sand or other means approved by the Engineer may be utilized in conjunction with Class 1 finishing.

8.12.4—Class 3—Tooled Finish

Finish of this character for panels and other like work may be secured by the use of a bushhammer, pick, crandall, or other approved tool. Air tools, preferably, shall be employed. No tooling shall be done until the concrete has set for at least 14 days and as much longer as may be necessary to prevent the aggregate particles from being "picked" out of the surface.

C8.12.4

A tooled finish surface shows a grouping of broken aggregate particles in a matrix of mortar, each aggregate particle being in slight relief.

8.12.5—Class 4—Sandblasted Finish

The thoroughly cured concrete surface shall be sandblasted with hard, sharp sand to produce an even, fine-grained surface in which the mortar has been cut away, leaving the aggregate exposed.

8.12.6—Class 5—Wire-Brushed or Scrubbed Finish

As soon as the forms are removed and while the concrete is yet comparatively green, the surface shall be thoroughly and evenly scrubbed with stiff wire or fiber brushes, using a solution of muriatic acid in the proportion of one part acid to four parts water by volume until the cement film or surface is completely removed and the aggregate particles are exposed, leaving an even pebbled texture presenting an appearance grading from that of fine granite to coarse conglomerate, depending upon the size and grading of aggregate used. When the scrubbing has progressed sufficiently to produce the texture desired, the entire surface shall be thoroughly washed with water to which a small amount of ammonia has been added, to remove all traces of acid.

8.13—PRECAST CONCRETE MEMBERS

8.13.1—General

Precast concrete members shall be constructed and placed in the work in conformance with the details specified in the contract documents.

If approved by the Engineer, precasting methods may be used for elements of the work which are otherwise indicated to be constructed by the cast-in-place method. When such precasting is proposed, the Contractor shall submit working drawings showing construction joint details and any other information required by the Engineer.

8.13.2—Working Drawings

Whenever specified in the contract documents or requested by the Engineer, the Contractor shall provide working drawings for precast members. Such drawings shall include all details not provided in the contract documents for the construction and the erection of the members and shall be approved before any members are cast. Such approval shall not relieve the Contractor of any responsibility under the contract documents for the successful completion of the work.

8.13.3—Materials and Manufacture

The materials and manufacturing processes used for precast concrete members shall conform to the requirements of the other articles in this Section except as those requirements are modified or supplemented by the provisions that follow.

When precast members are manufactured in established casting yards, the Manufacturer shall be responsible for the continuous monitoring of the quality of all materials and concrete strengths. Tests shall be performed in accordance with appropriate AASHTO or ASTM methods. The Engineer shall be allowed to observe all sampling and testing and the results of all tests shall be made available to the Engineer.

Established Precast Concrete Manufacturing Plants shall be certified under the Precast/Prestressed Concrete Institute (PCI) Certification Program or an alternative equivalent program for the category of work being manufactured.

Plant Quality-Control personnel shall be certified in the PCI Quality Control Personnel Certification Program, Level II. Plant Quality-Control Managers shall be certified PCI Level III. These requirements may be met by alternative experience and certification considered to be equivalent.

Precast members shall be cast on rigid beds or pallets. Special care shall be used in casting the bearing surfaces so that they will join properly with other elements of the structure.

For prestressed precast units, several units may be cast in one continuous line and stressed at one time. Sufficient space shall be left between ends of units to permit access for cutting of tendons after the concrete has attained the required strength.

The side forms may be removed as soon as their removal will not cause distortion of the concrete surface, providing that curing is not interrupted. Members shall not be lifted from casting beds until their strength is sufficient to prevent damage.

When cast-in-place concrete will later be cast against the top surfaces of precast beams or girders, these surfaces shall be finished to a coarse texture by brooming with a stiff, coarse broom. Prior to shipment, such surfaces shall be cleaned of laitance or other foreign material by sandblasting or other approved methods.

When precast members are designed to be abutted together in the finished work, each member shall be match-cast with its adjacent segments to ensure proper fit during erection. As the segments are match-cast, they shall be precisely aligned to achieve the final structure geometry. During the alignment, adjustments to compensate for deflections shall be made.

8.13.4—Curing

Unless otherwise permitted, precast members shall be cured by the water method, waterproof cover method, or the steam or radiant-heat method. The use of insulated blankets is permitted with the waterproof cover method. When the waterproof cover method is used, the air temperature beneath the cover shall not be less than 50°F and live steam or radiant heat may be used to maintain the temperature above the minimum value. The maximum concrete temperature during the curing cycle shall not exceed 160°F. The waterproof cover shall remain in place until such time as the compressive strength of the concrete reaches the strength specified for detensioning or stripping.

C8.13.4

All curing methods require concrete temperatures to be within the 50°F to 160°F range. The use of the waterproof cover method allows high-strength concretes to self-cure without the addition of steam or radiant heat. The use of insulated blankets will depend on the external weather conditions.

8.13.5—Storage and Handling

Extreme care shall be exercised in handling and moving precast prestressed concrete members. Precast girders shall be transported in an upright position and the directions of the reactions with respect to the member shall be as in the final position. Support points during transportation and storage should be located within 2.5 ft of their final position; otherwise, their location shall be shown on the shop drawings.

Prestressed concrete members shall not be shipped until tests on concrete cylinders, manufactured of the same concrete and cured under the same conditions as the girders, indicate that the concrete of the particular member has attained a compressive strength equal to the specified design compressive strength of the concrete in the member.

Care shall be taken during storage, hoisting, and handling of the precast units to prevent cracking or damage. Units damaged by improper storage or handling shall be replaced at the Contractor's expense.

8.13.6—Erection

The Contractor shall be responsible for the safety of precast members during all stages of construction. Lifting devices shall be used in a manner that does not cause damaging, bending, or torsional forces. After a member has been erected and until it is secured to the structure, temporary braces shall be provided as necessary to resist wind or other loads.

Precast deck form panels shall be erected and placed so that the fit of mating surfaces shall be such that excessive grout leakage will not occur. If such fit is not provided, joints shall be dry-packed or sealed with an acceptable caulking compound prior to placing the cast-in-place concrete. End panels for skewed structures may be sawed to fit the skew.

8.13.7—Epoxy-Bonding Agents for Precast Segmental Box Girders

8.13.7.1—Materials

Epoxy-bonding agents for match-cast joints shall be thermosetting 100 percent solid compositions that do not contain solvent or any nonreactive organic ingredient except for pigments required for coloring. Epoxy- bonding agents shall be of two components, a resin and a hardener. The two components shall be distinctly pigmented, so that mixing produces a third color similar to the concrete in the segments to be joined, and shall be packaged in proportioned, labeled, ready-to-use containers.

Epoxy-bonding agents shall be formulated to provide application temperature ranges that will permit erection of match-cast segments at substrate temperatures from 40°F to 115°F. If two surfaces to be bonded have different substrate temperatures, the adhesive applicable at the lower temperature shall be used.

Epoxy-bonding agents shall be insensitive to damp conditions during application and, after curing, shall exhibit high bonding strength to cured concrete, good water resistivity, low creep characteristics, and tensile strength greater than the concrete. In addition, the epoxy-bonding agents shall function as a lubricant during the joining of the match-cast segments, as a filler to accurately match the surface of the segments being joined, and as a durable, watertight bond at the joint.

Epoxy-bonding agents shall be tested to determine their workability, gel time, open time, bond and compression strength, shear, and working temperature range. The frequency of the tests shall be as stated in the contract documents.

The Contractor shall furnish the Engineer with samples of the material for quality assurance testing and a certification from a reputable independent laboratory indicating that the material has passed the required tests.

Specific properties of epoxy and the test procedures to be used to measure these properties shall be as described in the following Subarticles.

8.13.7.1.1—Test 1—Sag Flow of Mixed Epoxy-Bonding Agent

Testing Method: No testing is required.

Specification: Mixed epoxy-bonding agent must be an AASHTO M 235M/M 235 Type VI, Grade 3 (non-sagging) consistency at the designated application temperature class for the bonding agents used.

C8.13.7.1.1

The sag flow test of mixed epoxy-bonding agent is not directly measured. Table 1 of AASHTO M 235M/M 235 specifies the properties for the various types, grades, and classes of bonding agents.

8.13.7.1.2—Test 2—Gel Time of Mixed Epoxy-Bonding Agent

Testing Method: ASTM D2471 (except that 1.0-qt and 1.0-gal quantities shall be tested).

Specification: 30 min minimum on 1.0-qt and 1.0-gal quantities at the maximum temperature of the designated application temperature range. (Note: Gel time is not to be confused with open time specified in Test 3).

C8.13.7.1.2

Gel time is determined on samples mixed as specified in the testing method. It provides a guide for the period of time the mixing bonding agent remains workable in the mixing container during which it must be applied to the match-cast joint surfaces.

8.13.7.1.3—Test 3—Open Time of Bonding Agent

Testing Method: Open time is determined using test specimens as detailed in Article 8.13.7.1.4, "Test 4—Three-Point Tensile Bending Test." The epoxy-bonding agent, at the highest specified application temperature, is mixed together and applied as instructed in Test 4 to the concrete prisms, which shall also be at the highest specified application temperature. The adhesive coated prisms shall be maintained for 60 min at the highest specified application temperature with the adhesive coated surface or surfaces exposed and uncovered before joining together. The assembled prisms are then curved and tested as instructed in Test 4.

The epoxy-bonding agent shall be deemed acceptable for the specified application temperature only when essentially total fracturing of concrete paste and aggregate occurs with no evidence of adhesive failure.

C8.13.7.1.3

Open time of bonding agent test measures workability of the epoxy-bonding agent for the erection and post-tensioning operations.

As tested here, open time is defined as the minimum allowable period of elapsed time from the application of the mixed epoxy-bonding agent to the precast segments until the two segments have been assembled together and temporarily post-tensioned.

Construction situations may sometimes require application of the epoxy-bonding agent to the precast section prior to erecting, positioning, and assembling. This operation may require epoxy-bonding agents having prolonged open time. In general, where the erection conditions are such that the sections to be bonded are prepositioned prior to epoxy application, the epoxy-bonding agent shall have a minimum open time of 60 min within the temperature range specified for its application.

8.13.7.1.4—Test 4—Three-Point Tensile Bending Test

Testing Method: ASTM C192 6.0 × 6.0 × 9.0-in. concrete prisms of 6.0-ksi compressive strength at 28 days shall be sand-blasted on one 6.0 × 6.0-in. side to remove mold release agent, laitance, etc., and shall be submerged in clean water at the lower temperature of the specified application temperature range for 72 h. Immediately on removing the concrete prisms from the water, the sandblasted surfaces shall be air-dried for 1 h at the same temperature and 50-percent relative humidity and each shall be coated with approximately a 0.0625-in layer of the mixed bonding agent. The adhesive-coated faces of two prisms shall then be placed together and held with a clamping force normal to the bonded interface of 0.05 ksi. The assembly shall then be wrapped in a damp cloth that is kept wet during the curing period of 24 h at the lower temperature of the specified application temperature range.

After 24 h curing at the lower temperature of the application temperature range specified for the epoxy-bonding agent, the bonded specimen shall be unwrapped, removed from the clamping assembly, and immediately tested. The test shall be conducted using the standard AASHTO T 97 (ASTM C78) test for flexural strength with third-point loading and the standard MR unit. At the same time the two prisms are prepared and cured, a companion test beam shall be prepared of the same concrete, cured for the same period, and tested following AASHTO T 97 (ASTM C78).

Specification: The epoxy-bonding agent is acceptable if the load on the prisms at failure is greater than 90 percent of the load on the reference test beam at failure.

C8.13.7.1.4

The three-point tensile bending test performed on a pair of concrete prisms bonded together with epoxy-bonding agent, determines the bonding strength between the bonding agent and concrete. The bonded concrete prisms are compared to a reference test beam of concrete 6.0 × 6.0 × 18.0 in.

8.13.7.1.5—Test 5—Compression Strength of Cured Epoxy-Bonding Agent

Testing Method: ASTM D695.

Specification: Compressive strength at 77°F shall be a minimum of 2.0 ksi after 24 h of curing at the minimum temperature of the designated application temperature range and 6.0 ksi at 48 h.

C8.13.7.1.5

Compression strength test of cured epoxy-bonding agent measures the compressive strength of the epoxy-bonding agent.

8.13.7.1.6—Test 6—Temperature Deflection of Epoxy-Bonding Agent

Testing Method: ASTM D648.

Specification: A minimum deflection temperature of 122°F at fiber stress loading of 0.260 ksi is required on test specimens cured seven days at 77°F.

C8.13.7.1.6

Temperature deflection test of epoxy-bonding agent determines the temperature at which an arbitrary deflection occurs under arbitrary testing conditions in the cured epoxy-bonding agent. It is a screening test to establish performance of the bonding agent throughout the erection temperature range.

8.13.7.1.7—Test 7—Compression and Shear Strength of Cured Epoxy-Bonding Agent

Testing Method: A test specimen of concrete is prepared in a standard 6.0 × 12.0-in. cylinder mold to have a height at midpoint of 6.0 in. and an upper surface with a 30-degree slope from the vertical. The upper and lower portions of the specimen with the slant surfaces may be formed through the use of an elliptical insert or by sawing a full-sized 6.0 × 12.0-in. cylinder. If desired, 3.0 × 6.0-in. or 4.0 × 8.0-in. specimens may be used. After the specimens have been moist cured for 14 days, the slant surfaces shall be prepared by light sandblasting, stoning, or acid etching, then by washing and drying the surfaces, and finally by coating one of the surfaces with a 10-mil thickness of the epoxy-bonding agent under test. The specimens shall then be pressed together and held in position for 24 h. The assembly shall then be wrapped in a damp cloth that shall be kept wet during an additional curing period of 24 h at the minimum temperature of the designated application temperature range. The specimen shall then be tested at 77°F following AASHTO T 22 (ASTM C39/C39M) procedures. At the same time as the slant cylinder specimens are made and cured, a companion standard test cylinder of the same concrete shall be made, cured for the same period, and tested following AASHTO T 22 (ASTM C39/C39M).

Specification: The epoxy-bonding agent is acceptable for the designated application temperature range if the load on the slant cylinder specimen is greater than 90 percent of the load on the companion cylinder.

C8.13.7.1.7

Compression and shear strength test of cured epoxy-bonding agent is a measure of the compressive strength and shear strength of the epoxy-bonding agent compared to the concrete to which it bonds. The "slant cylinder" specimen with the epoxy-bonding agent is compared to a reference test cylinder of concrete only.

8.13.7.2—Mixing and Installation of Epoxy

As general instructions contained herein cannot cover all situations, specific recommendations and instructions shall be obtained in each case from the Engineer in charge.

Instructions furnished by the supplier for the safe storage, mixing, and handling of the epoxy-bonding agent shall be followed. The epoxy shall be thoroughly mixed until it is of uniform color. Use of a proper-sized mechanical mixer operating at no more than 600 revolutions per minute shall be required. Contents of damaged or previously opened containers shall not be used.

Surfaces to which the epoxy material is to be applied shall be at least 40°F and shall be free from oil, laitance, form release agent, or any other material that would prevent the epoxy from bonding to the concrete surface. All laitance and other contaminants shall be removed by light sandblasting or by high-pressure water blasting with a minimum pressure of 5.0 ksi. Wet surfaces shall be dried before applying epoxy-bonding agents. The surface shall be at least the equivalent of saturated surface dry (no visible water).

Mixing shall not start until the segment is prepared for installation. Application of the mixed epoxy-bonding agent shall be according to the Manufacturer's instructions using trowel, rubber glove, or brush on one or both surfaces to be joined. The coating shall be smooth and uniform in thickness and shall cover the entire surface with a minimum thickness of 0.0625 in. applied on both surfaces. A discernible bead line shall be observed on all exposed contact areas after temporary post-tensioning. Erection operations shall be coordinated and conducted so as to complete the operations of applying the epoxy-bonding agent to the segments, erection, assembling, and temporary post-tensioning of the newly joined segment within 70 percent of the open time period of the bonding agent.

The epoxy material shall be applied to all surfaces to be joined within the first half of the gel time, as shown on the containers. The segments shall be joined within 45 min after application of the first epoxy material placed and a minimum average temporary prestress of 0.04 ksi over the cross-section should be applied within 70 percent of the open time of the epoxy material. At no point of the cross-section shall the temporary prestress be less than 0.03 ksi.

The joint shall be checked immediately after erection to verify uniform joint width and proper fit. Excess epoxy from the joint shall be removed where accessible. All tendon ducts shall be swabbed immediately after stressing, while the epoxy is still in the nongelled condition, to remove or smooth out any epoxy in the conduit and to seal any pockets or air bubble holes that have formed at the joint.

If the jointing is not completed within 70 percent of the open time, the operation shall be terminated and the epoxy-bonding agent shall be completely removed from the surfaces. The surfaces must be prepared again and fresh epoxy shall be applied to the surface before resuming jointing operations.

8.14—MORTAR AND GROUT

8.14.1—General

This work consists of the making and placing of mortar and grout for use in concrete structures other than in prestressing ducts. Such uses include mortar for filling under masonry plates and for filling keyways between precast members where shown in the contract documents, mortar used to fill voids and repair surface defects, grout used to fill sleeves for anchor bolts, and mortar and grout for other such uses where required or approved.

8.14.2—Materials and Mixing

Materials for mortar and grout shall conform to the requirements of Article 8.3, "Materials." The grading of sand for use in grout or for use in mortar when the width or depth of the void to be filled is less than 0.75 in. shall be modified so that all material passes the No. 8 sieve.

Type 1A, air-entraining, portland cement shall be used when air entrainment is required for the concrete against which the grout or mortar is to be placed.

Unless otherwise specified in the contract documents or ordered by the Engineer, the proportion of cement to sand for mortar shall be one to two and for grout shall be one to one. Proportioning shall be by loose volume.

When nonshrink mortar or grout is specified, either a nonshrink admixture or an expansive hydraulic cement conforming to ASTM C845 of a type approved by the Engineer, shall be used.

Only sufficient water shall be used to permit placing and packing. For mortar, only enough water shall be used so that the mortar will form a ball when squeezed gently in the hand.

Mixing shall be done by either hand methods or with rotating paddle-type mixing machines and shall be continued until all ingredients are thoroughly mixed. Once mixed, mortar or grout shall not be retempered by the addition of water and shall be placed within 1 h.

8.14.3—Placing and Curing

Concrete areas to be in contact with the mortar or grout shall be cleaned of all loose or foreign material that would in any way prevent bond, and the concrete surfaces shall be flushed with water and allowed to dry to a surface dry condition immediately prior to placing the mortar or grout.

The mortar or grout shall completely fill and shall be tightly packed into recesses and holes, on surfaces, under structural members, and at other locations specified. After placing, all surfaces of mortar or grout shall be cured by the water method as provided in Article 8.11, "Curing Concrete," for a period of not less than three days.

Keyways, spaces between structural members, holes, spaces under structural members, and other locations where mortar could escape shall be mortar-tight before placing mortar.

No load shall be allowed on mortar that has been in place less than 72 h unless otherwise permitted by the Engineer.

All improperly cured or otherwise defective mortar or grout shall be removed and replaced by the Contractor at the Contractor's expense.

8.15—APPLICATION OF LOADS

8.15.1—General

Loads shall not be applied to concrete structures until the concrete has attained sufficient strength and, when applicable, sufficient prestressing has been completed, so that damage will not occur.

8.15.2—Earth Loads

Whenever possible, the sequence of placing backfill around structures shall be such that overturning or sliding forces are minimized. When the placement of backfill will cause flexural stresses in the concrete, and unless otherwise permitted by the Engineer, the placement shall not begin until the concrete has reached not less than 80 percent of its specified strength.

8.15.3—Construction Loads

Light materials and equipment may be carried on bridge decks only after the concrete has been in place at least 24 h, providing curing is not interfered with and the surface texture is not damaged. Vehicles needed for construction activities and weighing (having a mass) between 1.0 kip and 4.0 kips, and comparable materials and equipment loads, shall be allowed on any span only after the last placed deck concrete has attained a compressive strength of at least 2.4 ksi. Loads in excess of the above shall not be carried on bridge decks until the deck concrete has reached its specified strength. In addition, for post-tensioned structures, vehicles weighing (having a mass) over 4.5 kips, and comparable materials and equipment loads, shall not be allowed on any span until the prestressing steel for that span has been tensioned.

Precast concrete or steel girders shall not be placed on substructure elements until the substructure concrete has attained 70 percent of its specified strength.

Otherwise, loads imposed on existing, new, or partially completed portions of structures due to construction operations shall not exceed the load-carrying capacity of the structure, or portion of structure, as determined by the Strength II Load Combination in Table 3.4.1-1 of the *AASHTO LRFD Bridge Design Specifications*. The compressive strength of concrete (f'_c) to be used in computing the load-carrying capacity shall be the smaller of the actual compressive strength at the time of loading or the specified compressive strength of the concrete.

C8.15.3

Table 3.4.1-1 refers to Article 3.4.1, "Load Factors and Load Combinations," of the *AASHTO LRFD Bridge Design Specifications*, 2007.

8.15.4—Traffic Loads

Unless otherwise provided in the contract documents, traffic shall not be permitted on concrete decks until at least 14 days after the last placement of deck concrete and until such concrete has attained its specified strength.

8.16—SPECIAL REQUIREMENTS FOR SEGMENTAL BRIDGES

8.16.1—Geometry Control

8.16.1.1—Deflection and Camber Data

The Contractor shall submit deflection and/or camber data for each stage of construction as required to construct the structure to its final grade. The procedure used shall account for the effect of the time-dependent prestress losses and creep which will occur during the construction phase. The data for the entire bridge, based on the Contractor's proposed erection sequence, method, and schedule, shall be submitted to the Engineer for review prior to commencing construction of the pier shafts.

The camber of the structure will be monitored by the Contractor at each stage and corrective actions as approved by the Engineer shall be performed by the Contractor to assure proper erection of the structure to its final grade.

8.16.1.2—Geometry Control

The Contractor shall submit to the Engineer for approval a geometric control plan which shall indicate in detail how the survey is to be performed and the Contractor's proposed actions to assure proper erection of the structure to the final grade shown on the design plans. The geometric control plan shall provide for regular monitoring of the superstructure deflections beginning with the addition of the first cantilever segments and concluding with the last cantilever segment. The plan shall include the adjusting procedure to be utilized, should the cantilever, as erected, deviate from the predicted alignment by more than 1.0 in.

The Contractor shall check the elevations and alignment of the structure at every stage of construction and must maintain a record of all these checks and of all adjustments and corrections made. All surveying shall be performed at a time that will minimize the influence of temperature. Corrections by shimming shall be done only when approved by the Engineer.

For precast segmental construction using short line forming techniques, precision surveying systems shall be provided so that levels and horizontal alignment during precasting are measured to an accuracy of ±0.01 in. For all other types of segmental construction and for erection of segmental bridges, surveying shall be provided to an accuracy of ±0.125 in.

C8.16.1.2

It is recommended that the Engineer make independent checks of the elevation and alignment of the structure.

Independent checks of measurements and computations of geometry by the Engineer are recommended before segments are moved from their casting position.

For precast segmental construction using match-cast segments, careful checks of both measurements and computations of geometry shall be made by the Contractor before moving segments from their casting position. Computed coordinates of all sections cast shall be completed before casting a new segment. In addition to the computed as-built casting curves for vertical and horizontal deflections, a cumulative twist curve shall be computed using the measured cross slopes of the individual units as a check on the extrapolated deflections. In computing set-up elevations in the match-cast process, priority shall be given to correcting twist errors by proper counter-rotation. The segment in the match-cast position shall not be subjected to a stress-inducing twist.

Twist errors are calculated as the actual twist minus the design superelevation, if any.

8.16.2—Tolerances

Unless otherwise specified, reinforcement shall be fabricated and placed within the tolerances specified herein (CRSI 1990).

- For specified effective depth, *d*, and minimum clear concrete protection in flexural members, walls, and compression members where:

	Tolerance on *d*	Tolerance on Minimum Cover
8.0 in. or less	±0.375 in.	–0.375 in.
More than 8.0 in.	±0.5 in.	–0.5 in.

  but the tolerance on the clear distance to formed soffits shall be –0.25 in., and in no case shall the tolerance on cover exceed minimum one-third of the minimum cover stipulated on the structural drawings or in the specifications.

- For longitudinal location of bends and ends of bars: ±2.0 in., except at discontinuous ends of members where tolerance shall be ±0. 5 in.

- As long as the total number of bars specified is maintained, a reasonable tolerance in spacing individual bars is ±1.0 in., except where openings, inserts, embedded items, etc. might require some additional shifting of bars.

Embedded ducts for all types of segmental bridge construction shall be positioned to tolerances as specified in Article 10.4.1, “Placement of Ducts.”

Tolerances for completed segments shall be taken as shown in Table 8.16.2-1 and Figure 8.16.2-1. For bridges without an overlay, the flatness of the top slab shall be 0.125 in. in 10.0 ft in the direction of traffic.

Dimensions from segment-to-segment shall be adjusted to compensate for any deviations within a single segment so that the overall alignment of the completed structure will conform to the dimensions shown on the plans.

When cantilever construction is used, the tolerances for the alignment of the opposing cantilevers in a span shall be stated in the contract documents. The forces up and down which can be placed on the end of the cantilever shall also be stated on the design drawings and shall consider the allowable tensile stresses for construction load combinations in Table 5.14.2.3.3-1 of the *AASHTO LRFD Bridge Design Specifications*.

After erection, final post-tensioning, final corrections, and adjustments are complete and the structure has been placed on its permanent bearings, the superstructure shall conform to the grade and alignment shown on the plans, with due consideration of creep and superimposed dead-load deflections within tolerances as specified on the contract drawings.

Finished segment tolerances should not exceed those listed in Table 8.16.2-1.

Table 8.16.2-1—Completed Segment Tolerance for Segmental Box Girder Bridge Construction

Length of Match-Cast Segment (Not Cumulative) (0.125 in./ft +1.0 in. max.)	±0.125 in./ft
Length of Cast-in-Place Segment	±0.5 in., but not greater than +2.0 in. per span
Web Thickness	±0.375 in.
Depth of Bottom Slab	±0.375 in.
Depth of Top Slab	±0.375 in.
Overall Top Slab Width	±0.0625 in./ft, ±1.0 in. max.
Diaphragm Thickness	±0.5 in.
Grade of Form Edge and Soffit	±0.125 in. in 10.0 ft
Tendon Hole Location	±0.125 in.
Position of Shear Keys	±0.25 in.

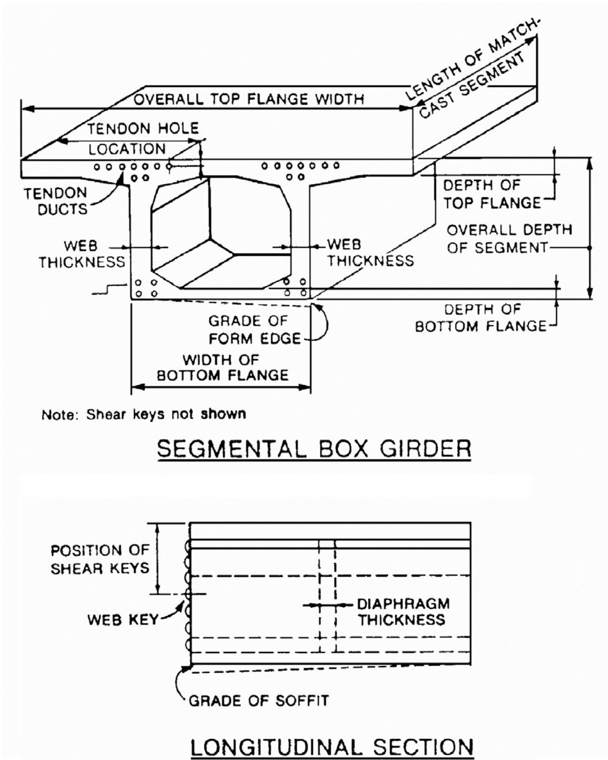


Figure 8.16.2-1—Key Diagram for Segment Tolerances

8.16.3—Shop Drawings and Design Calculations for Construction Procedures

8.16.3.1—General

Sufficiently in advance of the start of superstructure field construction operations, so as to allow the Engineer not less than a 30-calendar-day review period, the Contractor shall submit according to a schedule complete details and information concerning the method, materials, equipment, and procedures the Contractor proposes to use in constructing that portion of the superstructure for which the information is furnished. This submittal shall include a step-by-step erection procedure.

More than one method or technique of erection may be permitted in the overall scope of work. Any subsequent deviation from the approved materials and/or details will not be permitted unless details are submitted by the Contractor and approved by the Engineer in advance of use.

The Contractor's submittals for approval shall include calculations, drawings and information outlined in Article 8.16.3.2, “Design Calculations for Construction Procedures,” and Article 8.16.3.3, “Shop Drawings.” Two sets of all required drawings and calculations shall be submitted and resubmitted if and as necessary until approved by the Engineer. The specified number of distribution copies shall be furnished after approval.

8.16.3.2—Design Calculations for Construction Procedures

Design calculations shall be submitted for falsework, erection devices, formwork, or other temporary construction which may be required and which will be subject to calculated stresses.

Design of the falsework or erection devices for all superstructure concrete shall be done under the direction of and sealed by a registered professional engineer. Calculations shall also be submitted to substantiate the system and method of stressing proposed by the Contractor. Such calculations shall include the required jacking force and elongation of tendons at time of tensioning, tendon stress level at stressing ends after seating, stresses in anchorage zones and distribution plates, stress-strain curves typical of the prestressing steel to be furnished, seating losses, temporary overstresses, and reinforcement required to resist anchor block stresses.

In addition to the above, computations shall be submitted for approval for the following:

- Computations of deflections and required camber due to dead loads, post-tensioning forces, creep, and shrinkage. A tabulation of deflections and camber dimensions shall be included on the shop drawings.
- Computations of jacking forces required at joints during temporary post-tensioning.

C8.16.3.2

Load testing of special erection equipment is recommended.

Note that reinforcement for anchor blocks, as required under these specifications, is not the reinforcement required for global bursting and directional forces due to post-tensioning.

8.16.3.3—Shop Drawings

The Contractor shall submit detailed shop drawings for approval in accordance with the contract documents. The shop drawings shall include, but not necessarily be limited to, the following information:

- Fully and accurately dimensioned views showing the geometry of the segments including all projections, recesses, notches, openings, blockouts, and other pertinent details.
- Details of nonprestressed steel reinforcing shall be clearly shown as to size, spacing, and location, including any special reinforcing required but not shown on the plans.
- Size and type of ducts for all post-tensioning tendons and their horizontal and vertical profiles shall be clearly detailed. Duct supports, grout tubes, vents, and drains shall be shown, including size, type, and locations.
- Details and locations of all other items to be embedded in the segments, such as inserts, lifting devices, and post-tensioning hardware, shall be shown.

- Prestressing details including sizes and properties of tendons, anchorages, plates, assemblies, and stressing equipment, as well as details of the stressing procedure and stressing sequence, details and locations of all couplers, and additional reinforcement necessary to resist anchor block stresses.

- A table giving jacking sequence, jacking forces, and initial elongation of each tendon at each stage of erection for all post-tensioning.

- A table showing elevations and geometry to be used in positioning the forms for the next segment to be cast.

- Graphs, charts, or tables showing the theoretical location of each segment, as erected or placed, shall be furnished to the Engineer for use in checking the erection of the superstructure. Detailed procedures for making geometry corrections shall be described.

- Details of tie-down tendons and temporary and permanent bearing assemblies as required.

- Details of grouting equipment, grout mix design and method of mixing, and placing grout shall be provided.

8.16.4—Forms

8.16.4.1—General

Shop drawings shall be submitted for forms and form travelers as required by the contract documents.

In addition to the requirements of the contract documents, the forms used to cast the concrete segments shall be capable of:

- Match-casting (for precast segmental construction).

- Producing the segments within the tolerances permitted.

- Accommodating blockouts, openings, and protrusions.

- Adjusting to changes in segment geometry as shown in the plans, or for correcting previous minor casting errors to prevent accumulations.

- Stripping without damage to the concrete.

- Providing a tight, leakproof joining to the previous segment.

The bulkhead must be capable of connecting the ducts in a manner to hold their position and prevent intrusion of grout.

Where sections of forms are to be joined on the exterior face of the segment, an offset in excess of 0.0625 in. for flat surfaces and 0.125 in. for corners and bends will not be permitted. Offsets between adjacent matching faces of cast-in-place segments shall not exceed 0.25 in.

Forms shall not be removed until the concrete has attained the release strength specified on the contract plans as evidenced by test cylinders made and cured in the same manner as the segment. Alternatively, maturity meters or instrument control cylinders may be used to evaluate the strength of the concrete in the segment. Care shall be exercised in removing the forms to prevent spalling and chipping of the concrete.

8.16.4.2—Forms for Precast Segmental Construction

All side, bottom, inside, and header forms for precast segmental construction shall be constructed of steel unless use of other materials is approved by the Engineer.

Forms shall be of sufficient thickness, with adequate external bracing and stiffeners, and shall be sufficiently anchored to withstand the forces due to placement and vibration of concrete. Internal bracing and holding devices in forms shall be limited to stay bolts in webs which can be removed from the concrete surface to permit patching following form removal. Joints shall be designed and maintained for mortar tightness.

The grade and alignment of forms shall be checked each time they are set and shall be maintained during the casting of concrete. Slab finish grade will be checked after the concrete is in place.

Metal forms shall be reasonably free from rust, grease, or other foreign materials. All forms shall be cleaned thoroughly prior to each casting operation. End headers shall be maintained to provide a smooth casting surface.

Wood forms may be used on the cast-in-place longitudinal and transverse closure strips.

All form surfaces for casting members shall be constructed and maintained to provide segment tolerances in accordance with Article 8.16.2, "Tolerances."

The faces of all forms, other than end headers, shall be properly cleaned and treated with form oil or other bond breaking coating prior to placing concrete. Bond breaking material between segments and between segments and headers shall be provided in accordance with Article 8.16.7.3, "Separation of Match-Cast Segments." The oil or other materials used for this purpose shall be of a consistency and composition to facilitate form removal. Materials which appreciably stain or react with the concrete will not be constructed to facilitate segment removal without damage to the concrete.

8.16.5—Permanent Bearings

This work shall consist of the furnishing of all materials, fabrication, and installation of the permanent bearings as shown on the plans, as herein specified, and as directed by the Engineer.

The bearings shall be fabricated in conformance with the plans, except as otherwise specifically approved by the Engineer.

Prior to approval of the bearings to be used, the Contractor shall submit a certification by the Manufacturer stating that it and the accessory items meet the requirements set forth. This shall not constitute a waiver on the part of the Owner of any requirements with respect to samples and sampling, and the right is retained to perform any of the tests specified or such test deemed by the Engineer as necessary to qualify the material.

The Contractor shall obtain installation instructions from the supplier of the bearing assemblies and comply with the procedures specified in the installation of the bearing. Shop drawings shall be submitted to the Engineer for approval in accordance with the provisions of the section of these Specifications applicable to the bearing being provided. The adequacy of the design and installation details shall meet with the approval of the Engineer, whose decision shall be final.

8.16.6—Special Provisions for Cast-in-Place Segmental Construction

8.16.6.1—General

The design plans shall be prepared on the assumption that the superstructure will be constructed by a selected method of cast-in-place reinforced concrete segmental construction. Alternate construction methods may be permitted in accordance with the contract documents.

8.16.6.2—Forming System

In addition to the submittals required in Article 8.16.3, the following computations and working drawings shall be submitted for review by the Engineer for compliance with the contract documents:

- Complete details and computations for the forms and form support system, including maximum loadings and stresses created in the completed segments due to equipment forms and concrete placement. Design of form support system shall include adequate allowances for impact loadings which may occur during concrete placement and advancement of forming system.

- Computations of deflection of the forming system during concrete placement.

- Details for temporary supports and tie-downs as needed to stabilize the cantilevers during construction.

- Detailed step-by-step procedure for concrete placement, stressing and advancing the form support system, and adjusting the system for calculated deflection.

- Detailed procedure for fixing the cantilever ends against changes in position or rotation of one cantilever relative to the other during and following placement of concrete for the closure between the cantilevers.

8.16.6.3—Superstructure Construction

This work shall be taken to consist of setting temporary bearings if applicable, casting segments in place, and setting the superstructure on permanent bearings.

The Contractor shall submit complete details and descriptions of the methods, arrangements, and equipment to the Engineer for approval before superstructure construction is started.

The construction method shall include casting of the segments, methods of the tie-down of superstructures during cantilever erection, and method of application of all temporary forces to be used for adjusting horizontal and vertical alignments and to place the structure on permanent bearings. This shall also include control methods to ensure the accuracy of alignment of the completed superstructure.

Work equipment shall be taken to include all machinery, devices, labor, and material which are to be used for erection but will not become a permanent part of the completed superstructure. Equipment must not be operated from or placed upon any part of erected superstructure at any stage of construction other than which specifically meets the requirement of total working load per segment, as allowed by the contract documents, and/or as approved by the Engineer. This includes the post-tensioning hardware; jointing, jacking, grouting equipment, and any other equipment whatsoever; and personnel and materials of any kind.

In addition to segment unbalanced loads which are permitted for the construction method, a 0.010-kip/ft^2 load is permissible. The load shall be taken to include personnel, miscellaneous equipment, and stored material. It shall be the Contractor's responsibility to ensure that this allowable load is not exceeded.

Stressing many be performed in accordance with the following schedule:

- 50 percent of the post-tensioning force may be applied when field cured compression cylinders indicate the compressive strength of the segment concrete is 2.5 ksi, and 18 h have elapsed after completion of concrete placement.

- Form support system may be released and advanced when 50 percent of post-tensioning stress has been applied.

- Transverse stressing shall be staged as necessary to avoid exceeding the allowable stresses in the top slab.

- Tendons shall be fully tensioned prior to placement of concrete for the next segment, except the transverse tendon nearest the segment to be constructed. That tendon shall be stressed to 50 percent, then stressed the remainder when tendons in the new segment are stressed. The form support system shall be designed to avoid overstressing the top slab in the area of the partially stressed tendon.

Construction joints shall be limited to locations shown in the contract documents or as approved, in advance, by the Engineer. All construction joints shall be thoroughly cleaned of laitance and foreign material prior to placing concrete for the abutting section.

Surface of the segment joints shall be prepared in accordance with the contract documents immediately prior to placement of concrete for the next segment.

For placement of closure concrete between cantilevers, the cantilevers shall be fixed to prevent rotation or movement of one cantilever relative to the other. The system for locking the cantilevers and forming for the closure and the procedure for placing the concrete for the closure shall be such that the concrete after the initial set shall not be subjected to tension which could cause cracking.

The Contractor shall submit a construction schedule or check list showing chronological order of every phase and stage of erection and construction of the superstructure.

The Contractor shall prepare a table of elevations and alignments required at each stage of erection, as required by the contract documents, at the check points listed below, or an alternative at Contractor's option, and submit the same to the Engineer.

- One of the lowest corners at the top surface of any temporary bearing pads to be used as datum during erection and to establish a reference point with the actual elevations and alignment required of the permanently positioned superstructure.

- All four corners and centerline (at segment faces) of top slab of pier segments to establish grade and crown.

- Two points on the longitudinal centerline of each pier segment, one on each edge, to establish alignment.

- One point on the longitudinal centerline and at least one corner of each segment along every joint between cast-in-place segments to establish elevations and alignment at every stage of erection.

The temporary bearing pads, if applicable, at the piers shall be very carefully placed. The top surfaces of these pads shall have the correct elevations, alignments, and slopes as required by the plans and so established by the provisions above for temporary bearing pads. Shims may be used underneath the pads to accomplish accuracy. The Contractor shall also devise and provide measures to hold temporary bearing pads in position while the pier segment is being cast.

The Contractor shall check the elevations and alignment of the structure at every stage of construction in accordance with the geometry control plan submitted in accordance with the provisions of Article 8.16.1, "Geometry Control," and shall maintain a record of all these checks and of all adjustments and corrections made.

8.16.7—Special Provisions for Precast Concrete Segmental Construction

8.16.7.1—General

The superstructure shall be erected by the method designed and detailed in the contract documents or by an alternate method submitted by the Contractor. Alternate erection methods may be permitted when specified in the contract documents.

When required by the contract documents, the stressing system and all reinforcement and lifting details shall be successfully demonstrated on a segment designated in the contract documents prior to casting any permanent segments. The segment shall conform to the size and configuration required by the contract documents, including post-tensioning anchorage pockets, reinforcing steel, concrete, and conduits, including curvature and spacing. The tendons designated in the contract documents for this test shall be stressed to the forces shown. No additional payment shall be made for this test.

Casting bed and forms shall be structurally adequate to support the segments without settlement or distortion. The casting bed shall be designed for a method and the hardware needed to adjust and maintain grade and alignment. Details for hardware and adjustment procedures shall be included in the plans and specifications for the casting bed.

C8.16.7.1

Grading of the soffit form and the top portion of each segment shall take into consideration the relative position of the member in the structure.

After the first segment of each unit is cast, all succeeding segments shall be cast against previously cast segments to ensure complete bearing and proper alignment on all mating surfaces.

The anchorage system shall permit tendons to be inserted in the member after erection of segments.

Tendon couplers shall be used only at locations specifically shown in the contract documents or approved by the Engineer. Not more than 50 percent of the tendons shall be coupled at any one section. When temporary external tendons are required by the contract documents, the tendons and anchors shall be in a protective enclosure capable of protecting the tendons from damage by erection equipment and capable of confining a strand or bar tendon that breaks or otherwise releases tension rapidly during or after tensioning and anchorage. Protective enclosure proposals shall be submitted to the Engineer for approval.

Care shall be taken to ensure that deformations of match-cast segments due to thermal gradients caused by the heat of hydration of the new cast concrete do not exceed, at the time of initial set of the new concrete, 0.03125 in. for a single segment or 0.75 in. cumulative for an entire span. These deformations shall be prevented by properly protecting both the match-cast and new cast segments in an isothermal enclosure or with curing blankets and plastic sheeting.

Problems have been reported in the past when segments do not match properly at joints because of the thermal deformations induced during match-casting. The design gradient in the *AASHTO LRFD Bridge Design Specifications* can assist Designers and Constructors in determining to what extent this type of deformation will occur and if additional protection of the match-cast segment is necessary.

8.16.7.2—Fabrication

Reinforcing steel shall be fabricated and placed according to the contract documents. Any conflict or interference with the proper location of ducts and/or reinforcing, or blockouts shall be promptly resolved and corrections made as directed by the Engineer. No reinforcing steel shall be cut and removed to permit proper alignment of stressing conduits. Any bar that cannot be fabricated to clear the post-tensioning duct shall be replaced by additional bars with adequate lap lengths and shall be submitted to the Engineer for approval.

All segments shall be marked on the inside with a unique identification at the time of form removal. This identification shall be used to identify each segment on shop plans, post-tensioning details and calculations, and any other document pertaining to the fabrication and erection of precast concrete segments.

Positive means of holding the conduit in its correct position shall be provided in all cases and shall be indicated on the working drawings submitted for approval. The conduit shall be supported at intervals as specified in Article 10.4.1, "Placement of Ducts," or as shown in the contract documents, and shall be securely fastened to prevent movement during placement of concrete.

C8.16.7.2

Use of integrated shop drawings is recommended to locate conflicts in reinforcement placement.

8.16.7.3—Separation of Match-Cast Segments

The Contractor shall provide equipment to be used for uniform separation of match-cast segments without damage. The method, as well as details of the equipment to be used for separating match-cast segments, shall be included in the shop plans. A bond breaking material shall be used on the web and flanges of the previously cast segment to facilitate separation of segments. The bond breaking material shall be used to break the bond of concrete between the face of previously cast segments and a newly cast segment, as well as the end headers, when required. The bond breaker shall consist of flax soap and talc, or other material approved by the Engineer. A demonstration shall be performed on a 2.0×4.0-ft specimen, prior to the casting of segments, to prove the adequacy of the material. The material shall not be injurious to the concrete and shall permit removal of a segment without pullouts caused by adhesion of the concrete.

8.16.7.4—Handling and Erection of Segments

The Contractor (Fabricator) shall be responsible for proper handling, lifting, storing, transporting, and erection of all segments so that they may be placed in the structure without damage.

Segments shall be maintained in an upright position at all times and shall be stored, lifted, and/or moved in a manner to prevent torsion and other undue stress. Members shall be lifted, hoisted, or stored with lifting devices approved on the shop plans or by other methods approved by the Engineer in writing.

Segments shall not be moved from the casting yard until all curing and strength requirements have been attained and shall be supported in a manner that will minimize warping.

A full-scale test of the lifting and temporary holding hardware shall be performed to demonstrate the adequacy of this equipment prior to beginning any erection of the segments.

8.16.8—Special Provisions for Incremental Launching

8.16.8.1—General

Structures built by the incremental launching method shall comply with the provisions of Article 8.16.3, “Shop Drawings and Design Calculations for Construction Procedures,” and the additional provisions of the following Article.

8.16.8.2—Casting of Segments

Construction of incrementally launched bridges should be based on a weekly cycle for the construction of each segment. When the bottom flange and webs, or portions of webs, of segments are cast first and the top flange afterward, the time between concrete placements shall not exceed three days.

8.16.8.3—Geometric Tolerances

The following tolerances shall not be exceeded in the region of the sliding surfaces:

- In the Forms:

  Vertical deviations in longitudinal and transverse direction: ±0.03125 in.

  Horizontal deviation at the outside of webs: ±0.0625 in.

- On the Launching Bearings:

  Vertical: Longitudinally between piers
  ±0.0625 in.
  Transversely between bearings
  ±0.03125 in.

  Horizontal: Deviation of lateral guides ±0.0625 in.

8.16.8.4—Launching Force

The launching force shall be monitored continuously and checked against the theoretical value. A friction value between zero and four percent shall be maintained. The friction value of zero shall be considered in calculation of the force required to hold back a structure launched on a negative gradient.

C8.16.8.4

Grease may be used on the sliding plates as required to maintain friction values.

8.16.8.5—Pier Monitoring

The deflection of the pier tops shall be continuously monitored. Monitoring devices which automatically switch off the launching equipment in case the permissible pier deflections are exceeded are recommended. Communication between each sliding bearing and the launching equipment shall be provided.

The correct level of all sliding bearings shall be checked at regular intervals. Shim plates shall be kept in stock for all sliding bearings in order to compensate for pier settlements, if any.

8.16.9—Defects and Breakage

Failure of individual wires in a seven-wire strand is acceptable with the approval of the Engineer, provided the total area of wire failure is not more than two percent of the total cross-sectional area of the tendon.

Minor or nonstructural cracks or checks on the surface of the member which, as determined by the Engineer, do not extend to the plane of the nearest reinforcement will be acceptable unless they are numerous and extensive. Diagonal cracks which indicate damage from torsion, longitudinal cracks that follow stressing tendons, or any cracks which extend into the plane of the reinforcing steel and/or prestressed tendons shall be subject to a structural review prior to acceptance. If found acceptable, the cracks shall be repaired by "veeing" out 0.25 in. deep and wide and sealing with epoxy or shall be repaired by epoxy injection.

Minor breakage, spalling, or honeycombing not over 1.0 in. deep shall be repaired in accordance with an established repair procedure submitted to and approved by the Engineer prior to the start of segment fabrication. Major breakage or honeycomb in excess of that specified herein shall be subject to structural review. If found to be satisfactory, these areas shall be repaired as directed by the Engineer. Breakage, spalling, or honeycomb on any mating surface found to be acceptable shall be repaired and the concrete cured prior to casting the mating segment if such segment has not yet been cast.

8.17—MEASUREMENT AND PAYMENT

8.17.1—Measurement

Except for concrete in components of the work for which payment is made under other bid items, all concrete for structures shall be measured by either the cubic yard for each class of concrete included in the contract documents or by the unit for each type of precast concrete member listed in the contract documents.

When measured by the cubic yard, the quantity of concrete shall be computed from the dimensions shown in the contract documents or authorized in writing by the Engineer with the following exceptions:

- The quantity of concrete involved in fillets, scorings, and chamfers 1.0 in.^{2} or less in cross-sectional area shall not be included or deducted.

- When there is a bid item for concrete to be used as a seal course in cofferdams, the quantity of such concrete to be paid for shall include the actual volume of concrete seal course in place, but in no case shall the total volume measured exceed the product of the area between vertical surfaces 1.0 ft outside the neat lines of the seal course as shown in the contract documents and the thickness of the seal course. The thickness of seal course to be paid for shall be the thickness specified in the contract documents or ordered in writing by the Engineer.

The number of precast concrete members of each type listed in the contract documents will be the number of acceptable members of each type furnished and installed in the work.

Expansion joint armor assemblies will be measured and paid for as provided for in Section 23, "Miscellaneous Metal."

Whenever an alternative or option is shown in the contract documents, the quantities of concrete shall be computed on the basis of the dimensions specified in the contract documents and no change in quantities measured for payment shall be made because of the use by the Contractor of such alternatives or options.

8.17.2—Payment

The cubic yards of concrete and the number of precast concrete members, as measured above for each type or class listed in the contract documents, shall be paid for at the contract prices per cubic yard or the contract prices per each member.

Payment for concrete of the various classes and for precast concrete members of the various types shall be considered to be full compensation for the cost of furnishing all labor, materials, equipment, and incidentals; and for doing all the work involved in constructing the concrete work complete in place, as specified in the contract documents. Such payment shall be taken as to include full compensation for furnishing and placing expansion joint fillers, sealed joints, waterstops, drains, vents, miscellaneous metal devices, and the drilling of holes for dowels and the grouting of dowels in drilled holes, unless payment for such work is specified in the contract documents to be included in another bid item.

In addition, payment for precast concrete members shall be considered to be full compensation for the cost of all reinforcing steel, prestressing materials, and other items embedded in the member and for the erection of the members.

8.18—REFERENCES

2010 Revision

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2009. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 29th Edition, HM-29, American Association of State and Highway Transportation Officials, Washington, DC.

ACI Committee 209. 1982. *Prediction of Creep, Shrinkage and Temperature Effects in Concrete Structures*, ACI 209R-82, American Concrete Institute, Farmington Hills, MI.

ACI Committee 211. 1991. *Selecting Proportions for Normal, Heavyweight, and Mass Concrete*, ACI 211.1-91, American Concrete Institute

ACI Committee 211. 1993. *Guide for Selecting Proportions for High-Strength Concrete with Portland Cement and Fly Ash*, ACI 211.4-93, American Concrete Institute, Farmington Hills, MI. Reapproved 2002.

ACI Committee 211. 1998. *Selecting Proportions for Structural Lightweight Concrete*, ACI 211.2-98, American Concrete Institute, Farmington Hills, MI. Reapproved 2004.

ACI Committee 222. 1996. *Corrosion of Metals in Concrete*, ACI 222R-96, American Concrete Institute, Farmington Hills, MI.

ACI Committee 318. 1995. *Building Code Requirements for Reinforced Concrete*, ACI 318-95, American Concrete Institute, Farmington Hills, MI.

ACI Committee 318. 2002. *Building Code Requirements for Structural Concrete*, ACI 318-02 and Commentary, ACI 318R-02, American Concrete Institute, Farmington Hills, MI.

ACI Committee 363. 1998. *Guide to Quality Control and Testing of High-Strength Concrete*, ACI 363.2R-98, American Concrete Institute, Farmington Hills, MI.

ACI Committee 363. 1992. *State-of-the-Art Report on High-Strength Concrete*, ACI 363R-92, American Concrete Institute, Farmington Hills, MI.

ANSI and AHA. 2004. *Basic Hardboard*, ANSI/AHA A135.4, American National Standards Institute, New York, NY.

Cagley, J. R. 2001. "Changing from ACI 318-99 to ACI 318-02," *Concrete International*, June 2001.

Comite Euro-International de Beton (CEB). 1978. *CEB-FIP Model Code for Concrete Structures*. Available from Lewis Brooks, 2 Blagdon Road, New Malden, Surrey, KT3 4AD, England.

CRSI. 2001. *Manual of Standard Practice*, 27th Edition, MSP-1, Concrete Reinforcing Steel Institute, Chicago, IL.

Federation Internationale de la Precontrainte. 1981. "Recommendations for Acceptance and Application of Post-Tensioning Systems," Federation Internationale de la Precontrainte, Paris, France.

FHWA. 2003. *High Performance Concrete,* Federal Highway Administration, HPC Internet Conference, Baltimore, March 2003. Compact Disc.

FHWA and NCBC. 2001. *HPC Bridge Views*, Issue No. 1, Federal Highway Administration and the National Concrete Bridge Council, Skokie, IL, May/June 2001.

Goodspeed, C. H., S. Vanikar, and R. Cook. 1996. "High Performance Concrete Defined for Highway Structures," *Concrete International*, Vol. 18, No. 2, February 1996, pp. 62–67.

GSA. 1996. *Sealing Compound: Silicone Rubber Bases (for Caulking, Sealing, and Glazing in Buildings and Other Structures)*, Federal Specification TT-S-1543B, U.S. General Services Administration, Washington, DC.

Meyers, J. J. and R. L. Carrasquillo. 2000. *Production and Quality Control of High Performance Concrete in Texas Bridge Structures*, Research Report 580/589-1, Center for Transportation Research, The University of Texas at Austin.

Ozyildirim, C., 1984. *4 × 8 Inch Concrete Cylinders versus 6 × 12 Inch Cylinders*, VHTRC 84-R44, Virginia Transportation Research Council, Charlottesville, VA, May 1984.

PCI. 1999. *Manual for Quality Control for Plants and Production of Structural Precast Concrete Products*, MNL-116-99, Precast/Prestressed Concrete Institute, Chicago, IL.

Preston, H. K. 1985. "Testing 7-Wire Strand for Prestressed Concrete: The State of the Art," *Journal of the Prestressed Concrete Institute*, Vol. 30, No. 3, May/June 1985.

PTI. 1990. Guide Specification for Post-Tensioning Materials. In *Post-Tensioning Manual*, 5th Edition, Post-Tensioning Institute, Phoenix, AZ.

PTI. 1990. Recommended Practice for Grouting of Post-Tensioned Prestressed Concrete. In *Post-Tensioning Manual*, 5th Edition, Post-Tensioning Institute, Phoenix, AZ.

Zia, P. and A. Caner. 1993. *Cracking in Large-Sized Long Span Prestressed Concrete AASHTO Girders*, Report No. FHWA/NC/94-003, Center for Transportation Engineering Studies, North Carolina State University, Raleigh, NC.

APPENDIX A8—PROPOSED STANDARD SPECIFICATION FOR COMBINED AGGREGATES FOR HYDRAULIC CEMENT CONCRETE

The following is a proposed standard specification for combined aggregates for hydraulic cement concrete to be included in the *AASHTO Standard Specifications for Transportation Materials and Methods of Sampling, Part 1: Specifications.*

It is suggested that the proposed standard for combined aggregates for hydraulic cement concrete be grouped with M 6-02, Fine Aggregates for Portland Cement Concrete, and M 80-87 (1999), Coarse Aggregate for Portland Cement Concrete, in the table of contents under Aggregates with a heading of "Combined Aggregates for Hydraulic Cement Concrete."

Present specifications are AASHTO M 6 and AASHTO M 80.

[Referenced in Article 8.3.5, "Combined Aggregates."]

APPENDIX A8

AASHTO Designation: M XXI
Standard Specification for Combined Aggregates for Hydraulic Cement Concrete

1. SCOPE

1.1 This specification covers the requirements for combined aggregates for hydraulic cement concrete having a nominal maximum aggregate size of 50 mm (2.0 in.) or less. Fine and coarse aggregate shall be blended to achieve the desired properties. Two approaches are given. One is based on performance and the other on method type.

1.2 The values stated in SI units are to be regarded as standard.

2. REFERENCED DOCUMENTS

2.1 AASHTO Standards:

M 6 Fine Aggregate for Portland Cement Concrete
M 43 Sizes of Aggregate for Road and Bridge Construction
M 80 Coarse Aggregate for Portland Cement Concrete
M 195 Lightweight Aggregates for Structural Concrete
T 22 Compressive Strength of Cylindrical Concrete Specimens
T 23 Making and Curing Concrete Test Specimens in the Field
T 97 Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)
T 119 Slump of Hydraulic Cement Concrete
T 141 Sampling Freshly Mixed Concrete
T 160 Length Change of Hardened Hydraulic Cement Mortar and Concrete
T 198 Splitting Tensile Strength of Cylindrical Concrete Specimens
T 259 Resistance of Concrete to Chloride Ion Penetration
T 277 Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration
PP 34 Estimating the Cracking Tendency of Concrete

2.2 American Concrete Institute Standards:

211.1 Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete
221.2 Standard Practice for Selecting Proportions for Structural Lightweight Concrete

3. SIGNIFICANCE AND USE

3.1 The goal of a combined aggregate grading is to improve the workability of concrete at given water and paste contents or to improve the hardened properties by minimizing the amount of the water and paste for a given workability or to improve both the workability and hardened properties of concrete.

3.2 The shape and texture of the aggregate also have a large influence on the water and paste demand. Trial batches shall be made to ensure that desired concrete properties are achieved.

4. GENERAL REQUIREMENTS

4.1 Fine and coarse aggregates used shall comply with the relevant provisions of AASHTO M 6, M 43, M 80, and M 195 for ordering information, grading (M 43 shall be used unless otherwise permitted) based on a nominal maximum size, uniformity of grading, deleterious substances, and, if specified, reactive aggregates. Tests for performance characteristics of the concrete shall comply with the relevant specifications and specified AASHTO Test Methods for sampling (T 141), making test specimens (T 23), slump (T 119), crack tendency (PP 34), or other test methods as specified.

Proportions of fine and coarse aggregate shall be selected using the performance-based approach of Section 5 or the method-type approach of Section 6.

5. PERFORMANCE-BASED APPROACH

5.1 Contractor shall select the combined aggregate grading and demonstrate with trial batches that the specified properties are achieved (Note 1). It shall be the Contractor's responsibility to ensure that the combined grading provides the specified properties for the project.

Note 1—For proportioning hydraulic cement concrete, the *AASHTO LRFD Bridge Construction Specifications*, Article 8.4.1.1 specify the use of the absolute volume method for normal-density (normal-weight) concrete such as described in ACI publication 211.1 and the use of trial mixes for structural low-density (lightweight) concrete using methods such as described in ACI publication 211.2.

6. METHOD-TYPE APPROACH

6.1 One of the following procedures shall be used to determine combined aggregate grading:

Combined fineness modulus
Coarse factor chart
Power chart
Percent retained on each sieve

Note 2—Details of the procedures are given in the Appendix.

6.2 The specific combined grading to which the aggregate is to be blended, along with the tolerances for quality control, shall be submitted for approval. Concrete characteristics shall be verified by trial batches to ensure that the specified properties are achieved.

APPENDIX

Nonmandatory Information

A.1 METHOD-TYPE APROACH

A 1.1 Fineness Modulus

A1.1.1 The combined fineness modulus (FM) is obtained by adding the total percentages of material in the sample that are coarser than each of a set of sieves (cumulative percentages retained) and dividing the sum by 100 (see ACI 116). [1] FM is an index of the fineness of the material. A higher FM means that the aggregate is coarser. Trial batches are necessary to establish target values.

A1.2 Coarseness Factor

A 1.2.1 Workability factor and coarseness factor are determined for the combined aggregate. The factors are plotted on the chart shown in Figure 1. Zone II is the desired location. [2]

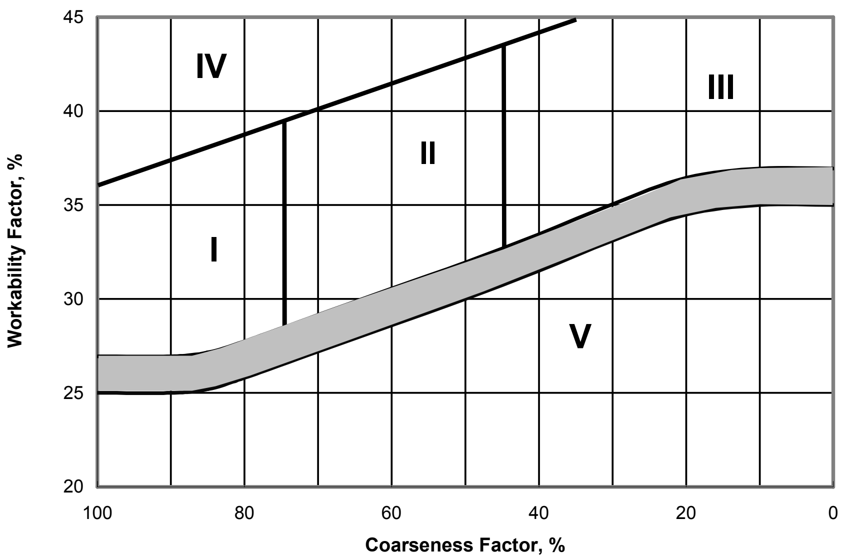


Figure 1—Coarseness Factor Chart

A1.2.2. The workability factor is the percent passing the 2.36-mm (No. 8) sieve adjusted for cementitious materials content of the proposed concrete mix. The measured percent passing the 2.36-mm (No. 8) sieve is increased or decreased by one percentage point for each 22 kg/m^3 (37 lb/cu yd) that the cementitious materials content is above or below 334 kg/m^3 (563 lb/cu yd), respectively.

A1.2.3. The coarseness factor is the cumulative percent retained on the 9.5-mm ($^3/_8$-in.) sieve divided by the cumulative percent retained on the 2.36-mm (No. 8) sieve.

A1.2.4. The five zones in the chart represent the following types of concretes:

I “Gap-graded” and tends to segregate
II Well graded 37.5 to 12.5 mm ($1^1/_2$ to $^1/_2$ in.)
III 12.5 mm ($^1/_2$ in.) and finer
IV Sticky
V Rocky

A1.2.5. Example

Cement Content		362	kg/m^3
Aggregate Data: As shown in Table A1.2.1			
Aggregate ID	Size 57		Sand
SSD Weight, kg	856		503
Specific Gravity	2.77		2.61
Aggregate, % by weight	63		37

Cement Content		611	lb/yd^3
Aggregate Data: As shown in Table A1.2.1			
Aggregate ID	Size 57		Sand
SSD Weight, lb	1887		1108
Specific Gravity	2.77		2.61
Aggregate, % by weight	63		37

Table A1.2.1—Sieve Analysis and Combined Grading

Sieve Size		Size 57 % Passing	Size 57 % Mix	Sand % Passing	Sand % Mix	Combined % Passing	Combined Cum. % Retained	Combined % Retained
50 mm	(2.0 in.)	100.0	63.0	100.0	37.0	100.0	0.0	0.0
37.5 mm	(1.5 in.)	100.0	63.0	100.0	37.0	100.0	0.0	0.0
25.0 mm	(1.0 in.)	100.0	63.0	100.0	37.0	100.0	0.0	0.0
19.0 mm	(0.75 in.)	85.0	53.6	100.0	37.0	90.6	9.4	9.4
12.5 mm	(0.5 in.)	60.0	37.8	100.0	37.0	74.8	25.2	15.8
9.5 mm	(0.375 in.)	35.0	22.1	100.0	37.0	59.1	40.9	15.7
4.75 mm	(No. 4)	10.0	6.3	99.7	36.9	43.2	56.8	15.9
2.36 mm	(No. 8)	0.0	0.0	87.8	32.5	32.5	67.5	10.7
1.18 mm	(No. 16)	0.0	0.0	65.7	24.3	24.3	75.7	8.2
600 μm	(No. 30)	0.0	0.0	33.6	12.4	12.4	87.6	11.9
300 μm	(No. 50)	0.0	0.0	12.7	4.7	4.7	95.3	7.7
150 μm	(No. 100)	0.0	0.0	3.4	1.3	1.3	98.7	3.4
75 μm	(No. 200)	0.0	0.0	0.0	0.0	0.0	100.0	1.3

Q = Cumulative percent retained on 9.5 mm (0.375 in.) sieve = 40.9
R = Cumulative percent retained on 2.36 mm (No. 8) sieve = 67.5
W = Percent passing 2.36 mm (No. 8) sieve = 32.5
C = Cementitious materials content = 362 kg/m^3 (611 lb/yd^3)

Workability Factor (WF) = SI $W + (C - 335)/22 = 32.5 + (362 - 335)/22 = 33.7\%$
= (USC) $W + (C - 564)/38 = 32.5 + (611 - 564)/38 = 33.7\%$

Coarseness Factor (CF) = $100\,(Q/R) = 100\,(40.9/67.5) = 60.6\%$

When the values of CF and WF are plotted on the Coarseness Factor Chart of Figure 1, they fall in Zone II.

A1.3 Power Chart

A1.3.1 In this method, the percent passing each sieve size is plotted against the sieve size in microns raised to power of 0.45 on semilog paper. A best fit straight line is then drawn through data points as shown by the broken line in Figure 2. The combined grading should follow the straight broken line within plus or minus seven percentage points deviation for percent passing. The dash lines in Figure 2 are seven percentage points below and above the broken line indicating the acceptable variability. For aggregates passing the 600 μm (No. 30) sieve, the grading curve may fall below the power chart line to compensate for the presence of fine cementitious materials. [3]

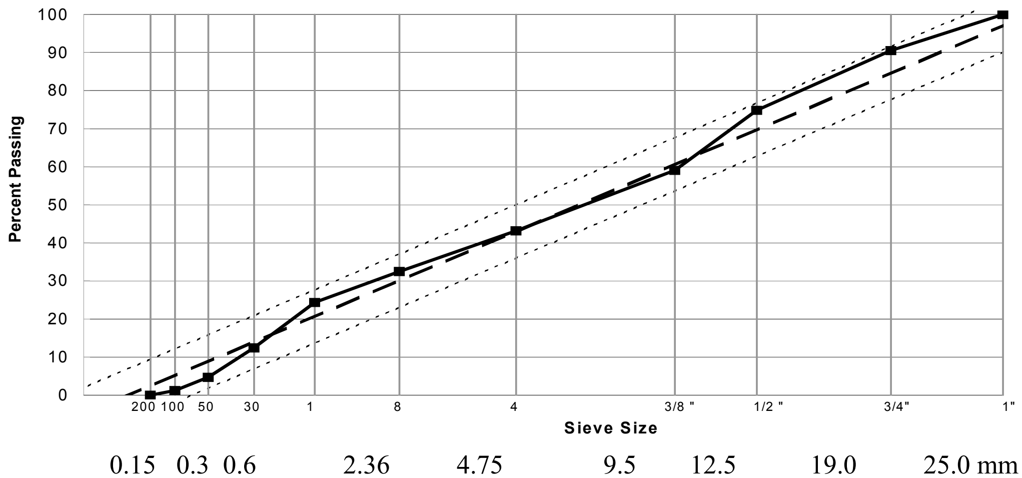


Figure 2—Power Chart

A1.4 Percent Retained on Each Sieve

A 1.4.1 In this method , the percent retained on each sieve size is kept to a limited range and the difference between percent retained on consecutive sieve sizes should be less than ten percentage points.

A1.4.2 ACI 302 suggests limits for the material retained on each sieve for satisfactory reduction in water demand while providing good workability. [4] If the largest size aggregate is 37.5 mm (1.5 in.), the percentage of material retained on each sieve size below the top size and above the 500 µm (No. 100) sieve shall be between 8 and 18. If the largest size aggregate is 25 mm or 19 mm (1.0 in. or 0.75 in.), the range shall be 8 to 22 percent. The ideal range for 600 µm (No. 30) and 300 µm (No. 50) sieves is eight percent to 15 percent retained on each. These ranges are illustrated in Figure 3 together with the data from Table A1.2.1.

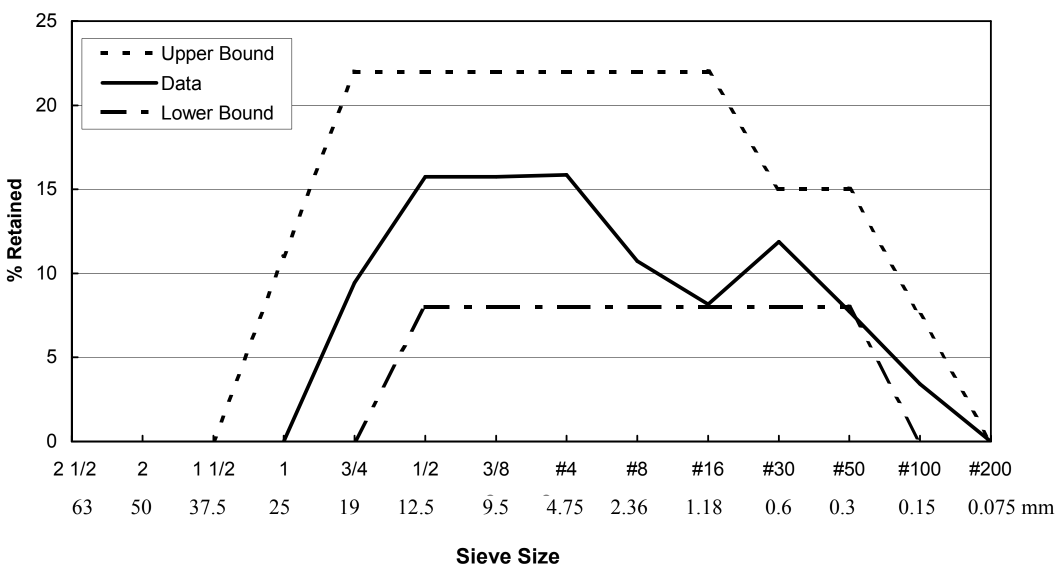


Figure 3—Percent Retained on Each Sieve

---

[1] *Cement and Concrete Technology*, ACI 116-R, ACI Committee 116. 2000.

[2] "Concrete Mixture Optimization," *Concrete International*, Vol. 12, No. 6, June 1990, pp. 33–39.

[3] *Properties of Concrete*, 4th Edition, A. M. Neville, 1996.

[4] *Guide for Concrete Floor and Slab Construction*, ACI 302.1R, ACI Committee 302. 1996.

Back to 2010 Edition

SECTION 8: CONCRETE STRUCTURES

8.3.4—Course Aggregate

Add the following paragraph to the end of this Article:

Reactive or potentially alkali-silica reactive aggregate may be used in concrete having low-alkali cements or a combination of portland or blended cement and pozzolanic materials and/or slag cement.

Insert new commentary to this Article:

C8.3.4

Aggregate reactivity is generally determined by past field performance or by tests (ASTM C295, AASHTO T 303, ASTM C1293, some combination thereof) made on aggregates prior to their use. Refer to Table 56X-l-B of AASHTO's *ASR Guide Specification* (2001) for quantities of reactive constituents (ASTM C295) and expansion limits (AASHTO T 303 and ASTM C1293) that are considered potentially reactive. Reference is also made to FHWA-HIF-09-001.

If aggregates of limited reactivity are used, then the effectiveness of a particular cementitious materials-aggregate combination to mitigate ASR is generally evaluated by ASTM C1567. Data from past field performance, ASTM C1293, or both can also be used to demonstrate satisfactory performance.

Page left intentionally blank.

8.4.1.1—Responsibility and Criteria

C8.4.1.1

Add the following to paragraph 3 of the commentary:

Where required, the specified properties should include resistance to aggregate reactivity. More guidance on this is contained in ACI 201.2R and Caldarone et al. (2005).

Page left intentionally blank.

8.18—REFERENCES

Add the following references:

AASHTO. 2001. *Guide Specification for Highway Construction SECTION 56X, Portland Cement Concrete Resistant to Excessive Expansion Caused by Alkali-Silica Reaction.* American Association of State Highway and Transportation Officials, Washington, DC. http://leadstates.transportation.org/asr/library/gspec.stm

ACI Committee 201. 2001. *Guide to Durable Concrete*, ACI 201.2R-0l. American Concrete Institute, Farmington Hills, MI.

Caldarone, Michael A., Peter C. Taylor, Rachel J. Detwiler, and Shrinivas B. Bhide. 2005. *Guide Specification for High-Performance Concrete for Bridges*, EB233, First Edition. Portland Cement Association, Skokie, IL.

FHWA. 2009. *Report on Determining the Reactivity of Concrete Aggregates and Selecting Appropriate Measures for Preventing Deleterious Expansion in New Concrete Construction*, FHWA-HIF-09-001. Federal Highway Administration, U.S. Department of Transportation, Washingotn, DC.

Back to 2010 Edition

SECTION 8: CONCRETE STRUCTURES

Add a new paragraph to the end of Article 8.2.3 as follows:

8.2.3—Lightweight (Low-Density) Concrete

Lightweight (low-density) concrete shall conform to the requirements specified in the contract documents. When the contract documents require the use of natural sand for a portion or all of the fine aggregate, the natural sand shall conform to AASHTO M 6.

<u>The equilibrium density of lightweight concrete shall be determined by ASTM C567.</u>

2016 Revision

TABLE OF CONTENTS

Section 9

REINFORCING STEEL

9.1—DESCRIPTION

This work shall consist of furnishing and placing reinforcing steel in accordance with these Specifications and in conformity with the contract documents.

9.2—MATERIAL

All reinforcing bars shall be deformed except that plain bars may be used for spirals and ties.

Reinforcing steel shall conform to the requirements herein.

9.2.1—Uncoated Reinforcing

Uncoated reinforcing steel shall conform to one of the following specifications:

- Deformed and Plain Billet-Steel Bars for Concrete Reinforcement—AASHTO M 31M/M 31 (ASTM A615/A615M). Grade 60 (Grade 420) shall be used unless otherwise specified in the contract documents.
- Rail-Steel and Axle-Steel Plain Bars for Concrete Reinforcement—AASHTO M 322M/M 322 (ASTM A996/A996M). Grade 60 (Grade 420) steel shall be used unless otherwise specified in the contract documents.
- Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement—ASTM A706/A706M.
- Deformed Steel Wire for Concrete Reinforcement—AASHTO M 225M/M 225 (ASTM A496).
- Welded Plain Steel Wire Fabric for Concrete Reinforcement—AASHTO M 55M/M 55 (ASTM A185).
- Plain Steel Wire for Concrete Reinforcement—AASHTO M 32M/M 32 (ASTM A82).
- Steel Welded Wire Reinforcement, Deformed, for Concrete—AASHTO M 221M/M 221 (ASTM A497).

C9.2.1

Grade 60 (Grade 420) in ASTM A615/A615M.

Grade 60 (Grade 420) in ASTM A615/A615M.

9.2.2—Epoxy-Coated Reinforcing

The reinforcing steel to be epoxy-coated shall conform to Article 9.2.1.

2010 Edition

When epoxy coating of reinforcing bars is required, the coating materials and process, the fabrication, handling, identification of the bars, and the repair of damaged coating that occurs during fabrication and handling, to the point of shipment to the job site, shall conform to the requirements of AASHTO M 284M/M 284 (ASTM D3963/D3963M) or to ASTM A934/A934M, as specified in the contract documents.

Epoxy-coated reinforcing bars shall be coated in a certified epoxy coating applicator plant in accordance with the Concrete Reinforcing Steel Institute Voluntary Certification Program for Fusion-Bonded Epoxy Coating Applicator Plants, or equivalent.

Epoxy-coated wire and welded wire fabric shall conform to the requirements of ASTM A884/A884M, Class A.

Each shipment of epoxy-coated reinforcing steel shall be accompanied with a Certificate of Compliance signed by the applicator of the coating certifying that the epoxy-coated reinforcing bars conform to the requirements of AASHTO M 317M/M 317 (ASTM D3963/D3963M) and AASHTO M 284M/M 284 (ASTM A775/A775M) or ASTM A934/A934M, or that epoxy-coated wire or welded wire fabric conforms to ASTM A884/A884M, Class A.

9.2.3—Stainless Steel Reinforcing Bars

When specified in the contract document, deformed or plain stainless steel reinforcing bars shall conform to the requirements of ASTM A955/A955M.

9.2.4—Low Carbon, Chromium, Steel Reinforcing Bars

Where specified in the contract documents, deformed, low carbon, chromium, steel reinforcing bars shall conform to the requirements of ASTM A1035/A1035M.

9.2.5—Mill Test Reports

When steel reinforcing bars, other than bars conforming to ASTM A706/A706M, are to be spliced by welding or when otherwise requested, a certified copy of the mill test report showing physical and chemical analysis for each heat or lot of reinforcing bars delivered shall be provided to the Engineer.

9.3—BAR LISTS AND BENDING DIAGRAMS

When the contract documents do not include detailed bar lists and bending diagrams, the Contractor shall provide such lists and diagrams to the Engineer for review and approval. Fabrication of material shall not begin until such lists have been approved. The approval of bar lists and bending diagrams shall in no way relieve the Contractor of responsibility for the correctness of such lists and diagrams. Any expense incident to the revision of material furnished in accordance with such lists and diagrams to make it comply with the design drawings shall be borne by the Contractor.

9.4—FABRICATION

9.4.1—Bending

Bar reinforcement shall be cut and bent to the shapes shown in the contract documents. Fabrication tolerances shall be in accordance with ACI 315-92, "Detailing Manual." All bars shall be bent cold, unless otherwise permitted. Bars partially embedded in concrete shall not be field-bent except as specified in the contract documents.

9.4.2—Hooks and Bend Dimensions

The dimensions of hooks and the diameters of bends measured on the inside of the bar shall be as shown in the contract documents. When the dimensions of hooks or the diameter of bends are not shown, they shall be in accordance with Article 5.10.2 of the *AASHTO LRFD Bridge Design Specifications* or ACI 318/318R-95, Building Code Requirements for Structural Concrete.

C9.4.2

Hook and bend dimensions are specified in Article 5.10.2, "Hooks and Bends," of the *AASHTO LRFD Bridge Design Specifications*.

9.4.3—Identification

Bar reinforcement shall be shipped in standard bundles, tagged and marked in accordance with the *Manual of Standard Practice* of the Concrete Reinforcing Steel Institute.

C9.4.3

The standard for bar reinforcement identification refers to the CRSI *Manual of Standard Practice*, 1996.

9.5—HANDLING, STORING, AND SURFACE CONDITION OF REINFORCEMENT

Steel reinforcement shall be stored above the surface of the ground on platforms, skids, or other supports and shall be protected from mechanical injury and surface deterioration caused by exposure to conditions producing rust. When placed in the work, reinforcement shall be free from dirt, loose rust or scale, mortar, paint, grease, oil, or other nonmetallic coatings that would reduce bond. Epoxy coatings of reinforcing steel in accord with standards in these Specifications shall be permitted. Reinforcement shall be free from injurious defects such as cracks and laminations. Bonded rust, surface seams, surface irregularities, or mill scale shall not be cause for rejection, provided the minimum dimensions, cross-sectional area, and tensile properties of a hand wire-brushed specimen meet the physical requirements for the size and grade of steel specified.

Epoxy-coated reinforcing steel shall be handled and stored by methods that will not damage the epoxy coating. All systems for handling epoxy-coated reinforcement shall have adequately padded contact areas. All bundling bands shall be padded and all bundles shall be lifted with a strong back, multiple supports, or platform bridge so as to prevent bar-to-bar or wire-to-wire abrasion from sags in the bundles. Bars or bundles shall not be dropped or dragged. Epoxy-coated reinforcing steel shall be transported and stored on wooden or padded supports.

Epoxy-coated reinforcing steel shall be immediately protected from sunlight, salt spray, and weather exposure. Provisions shall be made for air circulation around the protected reinforcement to minimize condensation under the protective covering.

9.6—PLACING AND FASTENING

9.6.1—General

Steel reinforcement shall be accurately placed as shown in the contract documents and firmly held in position during the placing and consolidation of concrete. Bars shall be tied at all intersections around the perimeter of each mat and at not less than 2.0-ft centers or at every intersection, whichever is greater, elsewhere. Bundled bars shall be tied together at not more than 6.0-ft centers. For fastening epoxy-coated reinforcement, tie wire and metal clips shall be plastic-coated or epoxy-coated. If uncoated welded wire fabric is shipped in rolls, it shall be straightened into flat sheets before being placed. Welding of crossing bars (tack welding) shall not be permitted for assembly of reinforcement unless authorized in writing by the Engineer.

9.6.2—Support Systems

Reinforcing steel shall be supported in its proper position by use of precast concrete blocks, wire bar supports, supplementary bars, or other approved devices. Such reinforcing supports or devices shall be of such height and placed at sufficiently frequent intervals so as to maintain the distance between the reinforcing steel and the formed surface or the top surface of deck slabs within 0.25 in. of that indicated in the contract documents.

Platforms for the support of workers and equipment during concrete placement shall be supported directly on the forms and not on the reinforcing steel.

9.6.3—Precast Concrete Blocks

Precast concrete blocks shall have a compressive strength not less than that of the concrete in which they are to be embedded. The face of blocks in contact with forms for exposed surfaces shall not exceed 2.0 in. × 2.0 in. in size and shall have a color and texture that will match the concrete surface. When used on vertical or sloping surfaces, such blocks shall have an embedded wire for securing the block to the reinforcing steel. When used in slabs, either such a tie wire or, when the weight (mass) of the reinforcing is sufficient to firmly hold the blocks in place, a groove in the top of the block may be used. For epoxy-coated bars, such tie wires shall be plastic-coated or epoxy-coated.

9.6.4—Wire Bar Supports

Wire bar supports, such as ferrous metal chairs and bolsters, shall conform to industry practice as described in the *Manual of Standard Practice* of the Concrete Reinforcing Steel Institute. Such chairs or bolsters which bear against the forms for exposed surfaces shall be either Class 1—Maximum Protection (Plastic Protected) or Class 2, Type B—Moderate Protection (Stainless Steel Tipped) for which the stainless steel conforms to ASTM A493, Type 430. For epoxy-coated reinforcement, all wire bar supports and bar clips shall be plastic-coated or epoxy-coated.

C9.6.4

The standard for wire bar supports refers to the CRSI *Manual of Standard Practice*, 1996.

9.6.5—Adjustments

Nonprestressed reinforcement used in post-tensioned concrete shall be adjusted or relocated during the installation of prestressing ducts or tendons, as required to provide planned clearances to the prestressing tendons, anchorages, and stressing equipment, as approved by the Engineer.

9.6.6—Repair of Damaged Epoxy Coating

In addition to the requirements of Article 9.2.2, “Epoxy-Coated Reinforcing,” all damaged coating on epoxy-coated reinforcing steel that occurs during shipment, handling, and placing of the reinforcing steel shall be repaired. The maximum amount of repaired damaged areas shall not exceed two percent of the surface area in each linear foot bar length. Should the amount of damaged coating incurred during shipment, handling and placing exceed two percent of the surface area in each linear foot bar length, that bar shall be removed and replaced with an acceptable epoxy-coated bar. The sum of the areas covered with patching material applied during repairs at all stages of the work shall not exceed five percent of the total surface area of any bar. The patching material shall be prequalified as required for the coating material and shall be either identified on the container as meeting the requirements of Annex A1 of AASHTO M 317M/M 317 (ASTM D3963/D3963M) and AASHTO M 284M/M 284 (ASTM A775/A775M) or Annex A1 of ASTM A934/A934M, or shall be accompanied by a Certificate of Compliance certifying that the material meets the requirements of said Annex A1. Patching of damaged areas shall be performed in accordance with the patching material Manufacturer's recommendations. Patches shall be allowed to cure before placing concrete over the coated bars.

9.7—SPLICING OF BARS

9.7.1—General

All reinforcement shall be furnished in the full lengths specified in the contract documents, unless otherwise permitted. Except for splices specified in the contract documents and splices for No. 5 (No. 16) or smaller bars, splicing of bars shall not be permitted without written approval by the Engineer. Splices shall be staggered as far as possible.

9.7.2—Lap Splices

Lap splices shall be of the lengths specified in the contract documents. If not specified in the contract documents, the length of lap splices shall be in accordance with Article 5.11.5.3.1 or Article 5.11.5.5.1 of the *AASHTO LRFD Bridge Design Specifications*, or as approved by the Engineer.

In lap splices, the bars shall be placed and tied in such a manner as to maintain the minimum distance to the surface of the concrete shown in the contract documents. Lap splices shall not be used for No. 14 and No. 18 (No. 43 and No. 57) bars, except as provided in either Article 5.11.5.2.1 or Article 5.11.5.5.1 of the *AASHTO LRFD Bridge Design Specifications*.

C9.7.2

Lap splice requirements refers to Article 5.11.5.3.1, “Lap Splices in Tension,” and Article 5.11.5.5.1, “Lap Splices in Compression,” of the *AASHTO LRFD Bridge Design Specifications*.

Lap splice requirements refer to Article 5.11.5.2.1, “Lap Splices,” of the *AASHTO LRFD Bridge Design Specifications*.

9.7.3—Welded Splices

Welded splices of reinforcing bars shall be used only if detailed in the contract documents or if authorization is made by the Engineer in writing. Welding shall conform to the ANSI/AWS D1.4 *Structural Welding Code—Reinforcing Steel*, and the contract documents.

Welded splices shall not be used on epoxy-coated reinforcing bars. To avoid heating of the coating, no welding shall be performed in close proximity to epoxy-coated bars.

Welded splices shall not be used with reinforcement conforming to ASTM A1035/A1035M.

9.7.4—Mechanical Splices

Mechanical splices shall be used only if preapproved or detailed in the contract documents or authorized in writing by the Engineer. Such mechanical splices shall develop in tension or compression, as required, at least 125 percent of the specified yield strength of the bar being spliced.

When requested by the Engineer, up to two field splices out of each 100, or portion thereof, placed in the work and chosen at random by the Engineer, shall be removed by the Contractor and tested by the Engineer for compliance to the required 125 percent of the specified yield strength of the bars being spliced.

9.8—SPLICING OF WELDED WIRE FABRIC

Sheets of welded wire fabric shall be spliced by overlapping each other sufficiently to maintain a uniform strength and shall be securely fastened at the ends and edges. The edge lap shall not be less than one mesh in width plus 2.0 in.

9.9—SUBSTITUTIONS

Substitution of different size reinforcing bars shall be permitted only when authorized by the Engineer. The substituted bars shall have an area equivalent to the design area or larger, and shall be in accordance with Article 5.7.3.4 of the *AASHTO LRFD Bridge Design Specifications*.

C9.9

Substitution of different size reinforcing bars refers to Article 5.7.3.4, "Control of Cracking by Distribution of Reinforcement," of the *AASHTO LRFD Bridge Design Specifications*.

9.10—MEASUREMENT

Steel reinforcement incorporated in the concrete will be measured in pounds based on the total computed weight (mass) for the sizes and lengths of bars, wire, or welded wire fabric shown in the contract documents or authorized by the Engineer for use in the work.

The weight (mass) of reinforcing bars will be computed using the weights (masses) in Table 9.10-1.

Table 9.10-1—Reinforcing Bars—Nominal Weights and Areas

Bar Size No.	Weight, lb/ft	Cross-Sectional Area, in.^{2}
No. 3	0.376	0.11
No. 4	0.668	0.20
No. 5	1.043	0.31
No. 6	1.502	0.44
No. 7	2.044	0.60
No. 8	2.670	0.79
No. 9	3.400	1.00
No. 10	4.303	1.27
No. 11	5.313	1.56
No. 14	7.65	2.25
No. 18	13.60	4.00

The weight (mass) of wire, welded wire fabric, and plain bars of sizes other than those listed in Table 9.10-1 shall be computed from tables of weights (mass) published by CRSI in MSP-1 or computed using nominal dimensions and an assumed unit weight (density) of 0.2833 lb/in.^{3} The cross-sectional area of wire in hundredths of square inches is equal to its W- or D- (MW- or MD-) Size Number. If the weight per square foot of welded wire fabric is given in the contract documents, that weight (mass) shall be used.

The weight (mass) of reinforcement used in items such as railings and precast members, where payment for the reinforcement is included in the contract price for the item, shall not be included. Threaded bars or dowels placed after the installation of precast members in the work and used to attach such members to cast-in-place concrete shall be included.

No allowance shall be made for clips, wire, separators, wire chairs, and other material used in fastening the reinforcement in place. If bars are substituted upon the Contractor's request and as a result more reinforcing steel is used than specified in the contract documents, only the amount specified in the contract documents shall be included.

The additional reinforcing steel required for splices that are not shown in the contract documents, but are authorized as provided herein, shall not be included.

No allowance shall be made for the weight (mass) of epoxy coating in computing the weight (mass) of epoxy-coated reinforcing steel.

9.11—PAYMENT

Payment for the quantity of reinforcement determined under measurement for each class of reinforcing steel shown in the contract documents shall be made at the contract price per pound. Payment shall be considered to be full compensation for furnishing, fabricating, splicing, and placing of the reinforcing steel, including all incidental work and materials required.

9.12—REFERENCES

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2009. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 29th Edition, HM-29, American Association of State Highway and Transportation Officials, Washington, DC.

AWS. 1998. ANSI/AWS D1.4 *Structural Welding Code—Reinforcing Steel*, American Welding Society, Miami, FL.

CRSI. 1996. *Manual of Standard Practice*, MSP-1, Concrete Reinforcing Steel Institute, Chicago, IL.

SECTION 10: PRESTRESSING

TABLE OF CONTENTS

SECTION 10

PRESTRESSING

10.1—GENERAL

10.1.1—Description

This work shall consist of prestressing precast or cast-in-place concrete by furnishing, placing, and tensioning of prestressing steel in accordance with details shown in the contract documents and as specified in these Specifications. It includes prestressing by either the pretensioning or post-tensioning methods or by a combination of these methods.

This work shall include the furnishing and installation of any appurtenant items necessary for the particular prestressing system to be used, including but not limited to ducts, anchorage assemblies, and grout used for pressure grouting ducts.

When members are to be constructed with part of the reinforcement pretensioned and part post-tensioned, the applicable requirement of this Specification shall apply to each method.

C10.1.1

For cast-in-place prestressed concrete, the term “member” as used in this Section is considered to mean the concrete which is to be prestressed.

10.1.2—Details of Design

Where the design for the prestressing work is not fully detailed in the contract documents, the Contractor shall determine the details or type of prestressing system for use and select materials and details conforming to these Specifications as needed to satisfy the prestressing requirements specified. The system selected shall provide the magnitude and distribution of prestressing force and ultimate strength required by the contract documents without exceeding allowable temporary stresses. Unless otherwise shown in the contract documents, all design procedures, coefficients and allowable stresses, friction, and prestress losses, as well as tendon spacing and clearances, shall be in accordance with either or both the *AASHTO LRFD Bridge Design Specifications* and the *Guide Specifications for Design and Construction of Segmental Concrete Bridges*, as applicable.

The prestressing may be performed by either pretensioning or post-tensioning methods unless the contract documents only specify pretensioning details. If the contract documents only specify pretensioning details, the use of a post-tensioning system shall be allowed only if complete details of any necessary modifications are approved by the Engineer.

Where the effective or working force or stress is specified in the contract documents, it shall be considered to be the force or stress remaining in the prestressing steel after all losses, including creep and shrinkage of concrete, elastic shortening of concrete, relaxation of steel, friction and take up or seating of anchorages, and all other losses peculiar to the method or system of prestressing have taken

C10.1.2

The “details of prestressing design” refers to the *AASHTO LRFD Bridge Design Specifications*, 2007, and *Guide Specifications for Design and Construction of Segmental Concrete Bridges*, 1999.

Structural designers of post-tensioned bridges should review the Post-Tensioning Institute's guide specifications: *Acceptable Standards for Post-Tensioning Systems* and *Guide Specification for Grouting of Post-Tension Structures*.

Contract documents should include a description of one construction method upon which the design is based. Where the design for the prestressing work is fully detailed in the contract documents, contract drawings should normally be developed in sufficient detail to ensure that the design provides for interference-free placement of structural items embedded in the concrete. If the contract contemplates additional engineering in order to complete design details for post-tensioning, the plans should clearly state both the requirements and specific acceptance standards for the additional engineering. Acceptance standards for design and detailing, if different from those established in either or both the *AASHTO LRFD Bridge Design Specifications* and the *Guide Specifications for Design and Construction of Segmental Concrete Bridges*, 2nd Edition, should be specified in such detail that requirements can be met without additional input from the Engineer. For the purposes of this provision, fully detailed prestressing systems are indicated by the presentation of tendon sizes, duct sizes, and anchorage hardware details on the plans.

place or have been provided for. Where the jacking force is specified in the contract documents, it shall be considered to be the force applied to the tendon prior to anchorage and the occurrence of any losses, including the anchor set loss.

10.2—SUPPLEMENTARY DRAWINGS

10.2.1—Working Drawings and Shop Drawings

Where the contract documents do not include complete details for a prestressing system and its method of installation, or where complete details are provided in the contract documents and the Contractor wishes to propose any change, the Contractor shall prepare and submit to the Engineer working drawings of the prestressing system proposed for use. Fabrication or installation of prestressing material shall not begin until the Engineer has approved the drawings.

The working drawings of the prestressing system shall show complete details and substantiating calculations of the method, materials, and equipment the Contractor proposes to use in the prestressing operations, including any additions or rearrangement of reinforcing steel and any revision in concrete dimensions from that specified in the contract

Where the Contractor proposes changes to prestressing details shown on the contract drawings, the contract documents shall clearly state that it is the Contractor's responsibility to complete the details of the prestressing system in accordance with Article 10.1.2 and to prepare supplementary working drawings of the prestressing system proposed for use in accordance with Article 10.2.1.

The provisions of *AASHTO LRFD Bridge Design Specifications* Article 5.14.2.3.9 "Plan Presentation" were developed in 1987 for Section 28, "Specifications, Contract Drawings, and Alternate Construction Methods," of the first edition of the *AASHTO Guide Specifications for Design and Construction of Segmental Concrete Bridges* published by AASHTO in 1989. The provisions of Section 28 were republished in 1999 without change in the second edition of the *AASHTO Guide Specifications for Design and Construction of Segmental Concrete Bridges*.

Under Design-Bid-Build provisions for development of contract drawings it is normally considered to be within the Engineer's design responsibility to prepare complete contract drawings, including drawings in sufficient detail to ensure interference-free placement of all embedded items in the concrete. Where this requirement is not included in the scope of the Engineer's responsibility for development of contract drawings, or where the Contractor proposes substantive changes to the contract drawings, the contract documents should clearly state that it is the Contractor's responsibility to determine details of the post-tensioning system in accordance with Article 10.1.2 of these Specifications and to prepare working drawings of the post-tensioning system proposed for use in accordance with Article 10.2.1 of these Specifications. However, this requirement should not alter the Engineer's responsibilities for assurance that the balance of the design detailing is complete and constructible. Where required by the contract documents, in addition to all working drawings, the Contractor shall prepare integrated placing drawings for all items to be embedded in the concrete in accordance with Article 10.2.2 of these Specifications.

documents. Such details shall outline the method and sequence of stressing and shall include complete specifications and details of the prestressing steel and anchoring devices, working stresses, anchoring stresses, tendon elongations, type of ducts, and all other data pertaining to the prestressing operation, including the proposed arrangement of the prestressing steel in the members.

Working drawings shall be submitted sufficiently in advance of the start of the affected work to allow time for review by the Engineer and correction by the Contractor of the drawings without delaying the work.

Where required on the contract drawings or in the contract documents, the Contractor shall prepare integrated drawings for all items embedded in the concrete. If this provision includes a requirement for additional engineering or design detailing, that requirement shall be clearly stated, as noted in Article 10.1.2.

Shop drawings for post-tensioning and other embedments, such as expansion joints, bearings, and anchor bolts submitted by suppliers shall be reviewed and approved by the Engineer for conformance with the design concept and compliance with the design drawings and Specifications. Where contract-document information on post-tensioning systems is modified by the Contractor, or where contract drawings do not provide detailed dimensional information on the post-tensioning system, it is the Contractor's responsibility to coordinate the placement of the post-tensioning system with other embedments, and to correct any interferences created by the Contractor-supplied PT system or other substitutions. The post-tensioning layout shall govern the layout of the secondary nonprestressed reinforcement. Where necessary, location of nonprestressed reinforcement should be adjusted to clear tendons, subject to the approval of the Engineer.

10.2.2—Integrated Drawings

Where required by the contract documents, in addition to all required working drawings, the Contractor shall prepare composite placing drawings to scale and in sufficient detail to show the relative positions of all items that are to be embedded in the concrete, and their embedment depth, for the portions of the structure that are to be prestressed. Such embedded items shall include the prestressing ducts, vents, anchorage reinforcement and hardware, reinforcing steel, anchor bolts, earthquake restrainers, deck joint seal assemblies, drainage systems, utility conduits, and other such items. Such drawings shall be in sufficient detail to confirm that there will be no conflict between the planned positions of any embedded items and that concrete cover will be adequate. Where the contract calls for the Contractor to develop drawings for post-tensioning systems, or where the Contractor makes modifications to the post-tensioning system(s) shown on the plans, the Contractor shall prepare working drawings for embedded items or propose changes in the dimensions of the work as necessary to eliminate conflicts and provide proper cover wherever conflicts arise with or due to the post-tensioning system. Resolution of conflicts should follow the provisions of Article 10.1.2. Any such revisions shall be approved by the Engineer before work on any affected item is started. All costs involved with the preparation of such drawings and with making the necessary modifications to the work resulting there from shall be borne by the Contractor. This will require integrated drawings to be detailed to a scale and quality sufficient to show double-line reinforcement and post-tensioning tendon details in either two-dimensional or in complete three-dimensional drawings.

10.3—MATERIALS

10.3.1—Prestressing Steel and Anchorages

Prestressing reinforcement shall be high-strength seven-wire strand, high-strength steel wire, or high-strength alloy bars of the grade and type called for in the contract documents and shall conform to the requirements of the following Specifications.

10.3.1.1—Strand

Uncoated seven-wire strand shall conform to the requirements of AASHTO M 203M/M 203 (ASTM A416/A416M). Supplement S1 (Low-Relaxation) shall apply when specified.

10.3.1.2—Wire

2016 Revision

Uncoated stress-relieved steel wire shall conform to the requirements of AASHTO M 204M/M 204 (ASTM A421/A421M).

10.3.1.3—Bars

Uncoated high-strength bars shall conform to the requirements of AASHTO M 275M/M 275 (ASTM A722/A722M). Bars with greater minimum ultimate strength, but otherwise produced and tested in accordance with AASHTO M 275M/M 275 (ASTM A722/A722M), may be used provided they have no properties that make them less satisfactory than the specified material.

10.3.2—Post-Tensioning Anchorages and Couplers

All anchorages and couplers shall develop at least 96 percent of the actual ultimate strength of the prestressing steel, when tested in an unbonded state, without exceeding anticipated set. The coupling of tendons shall not reduce the elongation at rupture below the requirements of the tendon itself. Couplers and/or coupler components shall be enclosed in housings long enough to permit the necessary movements. Couplers for tendons shall be used only at locations specifically indicated in the contract documents and/or approved by the Engineer. Couplers shall not be used at points of sharp tendon curvature.

C10.3.2

The anchorage efficiency test requirement that devices develop 96 percent of the ultimate strength of the prestressing steel has been expressed as actual ultimate strength rather than guaranteed ultimate strength. The reason for this is that the test requirement is to make sure that effects from the hardware used for gripping do not reduce the capacity of the tendons more than four percent. This can only be measured in reference to the actual strength of the particular prestressing steel used in the test.

10.3.2.1—Bonded Systems

Bond transfer lengths between anchorages and the zone where full prestressing force is required under service and ultimate loads shall normally be sufficient to develop the minimum specified ultimate strength of the prestressing steel. When anchorages or couplers are located at critical sections under ultimate load, the ultimate strength required of the bonded tendons shall not exceed the ultimate capacity of the tendon assembly, including the anchorage or coupler, tested in an unbonded state.

Housings shall be designed so that complete grouting of all of the coupler components will be accomplished during grouting of tendons.

10.3.2.2—Unbonded Systems

For unbonded tendons, two dynamic tests shall be performed on a representative anchorage and coupler specimen and the tendon shall withstand, without failure, 500,000 cycles from 60 percent to 66 percent of its minimum specified ultimate strength, and also 50 cycles from 40 percent to 80 percent of its minimum specified ultimate strength. Each cycle shall be taken as the change from the lower stress level to the upper stress level and back to the lower. Different specimens may be used for each of the two tests. Systems utilizing multiple strands, wires, or bars may be tested utilizing a test tendon of smaller capacity than the full-size tendon. The test tendon shall duplicate the behavior of the full-size tendon and generally shall not have less than ten percent of the capacity of the full-size tendon. Dynamic tests shall be required on bonded tendons where the anchorage is located or used in such manner that repeated load applications can be expected on the anchorage; otherwise dynamic tests shall be required only if specified in the contract documents.

Anchorages for unbonded tendons shall not cause a reduction in the total elongation under ultimate load of the tendon to less than two percent measured in a minimum gage length of 10.0 ft.

All the coupling components shall be completely protected with a coating material prior to final encasement in concrete.

10.3.2.3—Special Anchorage Device Acceptance Test

10.3.2.3.1—Test Block Requirements

The test block shall be a rectangular prism. It shall contain those anchorage components which will also be embedded in the structure's concrete. Their arrangement shall comply with the practical application and the suppliers' specifications. The test block shall contain an empty duct of size appropriate for the maximum tendon size which can be accommodated by the anchorage device.

C10.3.2.3.1

Figure C10.3.2.3.1-1 shows a local zone specimen with the local zone confining reinforcement in the upper portion of the specimen and the optional supplementary reinforcement of Article 10.3.2.3.4, "Skin Reinforcement," over the full-length of the specimen. However, an anchorage device supplier could also choose to eliminate such reinforcement in either or both portions of the block.

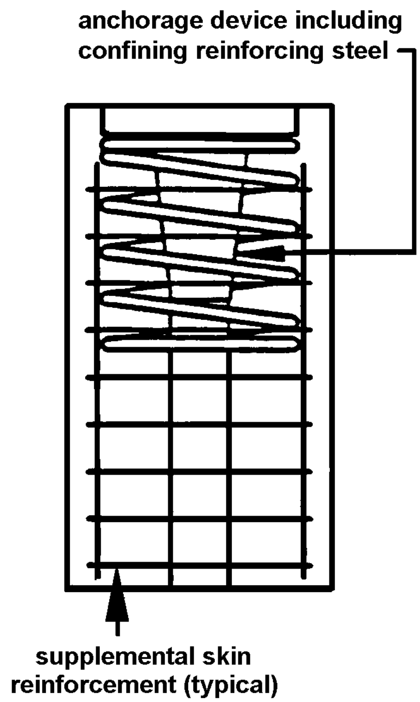


Figure C10.3.2.3.1-1—Special Anchorage Device Acceptance Test Specimen

10.3.2.3.2—Test Block Dimensions

The dimensions of the test block perpendicular to the tendon in each direction shall be the smaller of the minimum edge distance or the minimum spacing specified by the anchorage device supplier, with the stipulation that the cover over any confining reinforcing steel or supplementary skin reinforcement be appropriate for the particular application and environment. The length of the block along the axis of the tendon shall be at least two times the larger of the cross-section dimensions.

10.3.2.3.3—Local Zone Reinforcement

The confining reinforcing steel in the local zone shall be the same as that specified by the anchorage device supplier for the particular system.

10.3.2.3.4—Skin Reinforcement

In addition to the anchorage device and its specified confining reinforcement steel, supplementary skin reinforcement may be provided throughout the specimen. This supplementary skin reinforcement shall be specified by the anchorage device supplier but shall not exceed a volumetric ratio of 0.01.

C10.3.2.3.4

The supplementary reinforcement in the specimen is specified by the anchorage device supplier within the limits of Article 10.3.2.3.4, "Skin Reinforcement." The same amount of reinforcement is also required in the actual structure, as stipulated in Article 9.2.1, "Uncoated Reinforcing." However, other reinforcement in the corresponding portion of the structure (such as minimum reinforcement for creep and shrinkage or bursting reinforcement) may be counted towards this requirement. Since the confinement and supplementary reinforcement in the test specimens will generally be provided in orthogonal directions, similar reinforcement in the actual structure must be furnished to achieve an equivalent orthogonal action.

10.3.2.3.5—Concrete Strength

The concrete strength at the time of stressing shall be greater than the concrete strength of the test specimen at time of testing.

10.3.2.3.6—Test Procedures

Any of the following three test procedures may be deemed to be acceptable:

- Cyclic loading described in Article 10.3.2.3.7,
- Sustained loading described in Article 10.3.2.3.8, or
- Monotonic loading described in Article 10.3.2.3.9.

The loads specified for the tests are given in fractions of the ultimate load F_{pu} of the largest tendon that the anchorage device is designed to accommodate. The specimen shall be loaded in accordance with normal usage of the device in post-tensioning applications, except that load can be applied directly to the wedge plate or equivalent area.

C10.3.2.3.6

Long-term loading has been found to be more critical for the behavior of the local zone than short-term loading. A cyclic loading test gives comparable results to sustained loading tests, but is less time consuming than the sustained loading test (FIB 2000). A monotonic short-term loading test procedure is also included in the provisions. Stricter acceptance criteria are necessary to make the short-term loading test comparable to the other test methods.

Loading in accordance with normal usage of the anchorage device in post-tensioning applications means loading through the wedge plate if available, or over an area formed by the perimeter of the wedge openings pattern. It is not required to load the specimen through the tendon.

10.3.2.3.7—Cyclic Loading Test

10.3.2.3.7a—General

In a cyclic loading test, the load shall be increased to 0.8 F_{pu}. The load shall then be cycled between 0.1 F_{pu} and 0.8 F_{pu} until crack widths stabilize, but for not less than 10 cycles. Crack widths are considered stabilized if they do not change by more than 0.001 in. over the last three readings. Upon completion of the cyclic loading the specimen shall be preferably loaded to failure or, if limited by the capacity of the loading equipment, to at least 1.1 F_{pu}.

C10.3.2.3.7a

The required minimum failure load of 1.1 F_{pu} for cyclic and sustained loading tests reflects the incorporation of the maximum allowable stressing level of 0.8 F_{pu} with a load factor of 1.2 and a ϕ-factor of 0.80. Alternatively, if limited by test equipment capacity, a minimum failure load of 1.0 F_{pu} can be specified, provided the actual concrete strength of the specimen is reduced proportionately.

10.3.2.3.7b—Crack Widths and Patterns

Crack widths and crack patterns shall be recorded at the initial load of 0.8 F_{pu}, at least at the last three consecutive peak loadings before termination of the cyclic loading, and at 0.9 F_{pu}. The maximum load shall also be reported.

10.3.2.3.8—Sustained Loading Test

10.3.2.3.8a—General

In a sustained loading test, the load shall be increased to 0.8 F_{pu} and held constant until crack widths stabilize, but for not less than 48 h. Crack widths shall be considered to be stabilized if they do not change by more than 0.001 in. over the last three readings. After sustained loading is completed, the specimen shall be preferably loaded to failure or, if limited by the capacity of the loading equipment, to at least 1.1 F_{pu}.

10.3.2.3.8b—Crack Widths and Patterns

Crack widths and crack patterns shall be recorded at the initial load of 0.8 F_{pu}, at least three times at intervals of not less than 4 h during the last 12 h before termination of the sustained loading, and during loading to failure at 0.9 F_{pu}. The maximum load shall also be reported.

10.3.2.3.9—Monotonic Loading Test

10.3.2.3.9a—General

In a monotonic loading test, the load shall be increased to 0.9 F_{pu} and held constant for 1 h. The specimen shall then be preferably loaded to failure or, if limited by the capacity of the loading equipment, to at least 1.2 F_{pu}.

C10.3.2.3.9a

In the monotonic loading test the required minimum failure load is increased to 1.2 F_{pu}, reflecting comparative test experience with monotonic, sustained and cyclic loading procedures. Alternatively, if limited by test equipment capacity, a minimum failure load of 1.0 F_{pu} can be specified, provided the actual concrete strength of the specimen is reduced proportionately.

If representative samples out of a series of similar anchorage devices pass the acceptance test, the anchorage device supplier may elect not to test the other anchorage devices in the series. However, the responsibility for the proper performance of such untested anchorage devices remains with the supplier.

10.3.2.3.9b—Crack Widths and Patterns

Crack widths and crack patterns shall be recorded at 0.9 F_{pu} after the 1-h period, and at 1.0 F_{pu}. The maximum load shall also be reported.

10.3.2.3.10—Anchorage Zone Requirements

The strength of the anchorage zone shall exceed:

- Specimens tested under cyclic or sustained loading…………………………………1.1 F_{pu}
- Specimens tested under monotonic loading…………………………………1.2 F_{pu}

The maximum crack width criteria specified below shall be met for moderately aggressive environments:

- No cracks greater than 0.010 in. at 0.8 F_{pu} after completion of the cyclic or sustained loading, or at 0.9 F_{pu} after the 1-h period for monotonic loading.
- No cracks greater than 0.016 in. at 0.9 F_{pu} for cyclic or sustained loading, or at 1.0 F_{pu} for monotonic loading.

For higher aggressivity environments, the crack width criteria shall be reduced by at least 50 percent.

C10.3.2.3.10

The crack width requirements of Article 10.3.2.3.10 are based on recommendations in Article 5.9.1.5, "Crack Control" of the *AASHTO LRFD Bridge Design Specifications*, 2007. A moderately aggressive environment is characterized by moist environments where deicing or sea salts may be present in mists, but where direct exposure to corrosive agents is prevented. This should include most bridge applications.

Records of the anchorage device acceptance test have to be provided by the anchorage device supplier to the Engineer-of-Record and to the constructor. These records must include all the necessary information for proper installation of the anchorage device including all confining and supplementary reinforcement.

10.3.2.3.11—Test Series Requirements

A test series shall consist of three test specimens. Each one of the tested specimens shall meet the acceptance criteria. If one of the three specimens fails to pass the test, a supplementary test of three additional specimens is allowed. The three additional test specimen results shall meet all acceptance criteria of Article 10.3.2.3.10, "Anchorage Zone Requirements."

For a series of similar special anchorage devices, tests shall only be required for representative samples unless tests for each capacity of the anchorages in the series are required by the Engineer-of-Record.

10.3.2.3.12—Records of the Anchorage Device

Records of the anchorage device acceptance test shall include:

- Dimensions of the test specimen.
- Drawings and dimensions of the anchorage device, including all confining reinforcing steel.
- Amount and arrangement of supplementary skin reinforcement.
- Type and yield strength of reinforcing steel.
- Type and compressive strength at time of testing of concrete.

- Type of testing procedure and all measurements required in Articles 10.3.2.3.7 through 10.3.2.3.10 for each specimen.

10.4—PLACEMENT OF DUCTS, STEEL, AND ANCHORAGE HARDWARE

10.4.1—Placement of Ducts

10.4.1.1—General

Ducts shall be rigidly supported at the proper locations in the forms by ties to reinforcing steel which are adequate to prevent displacement during concrete placement. Supplementary support bars shall be used where needed to maintain proper alignment of the duct. Hold-down ties to the forms shall be used when the buoyancy of the ducts in the fluid concrete would lift the reinforcing steel.

Polyethylene duct and metal duct for longitudinal or transverse post-tensioning in the flanges shall be supported at intervals not to exceed 2.0 ft. Polyethylene duct in webs for longitudinal post-tensioning shall be tied to stirrups at intervals not to exceed 2.0 ft, and metal duct for longitudinal post-tensioning in webs shall be tied to stirrups at intervals not to exceed 4.0 ft.

Joints between sections of duct shall be coupled with positive connections which do not result in angle changes at the joints and will prevent the intrusion of cement paste.

After placing of ducts, and reinforcement and forming is complete, an inspection shall be made to locate possible duct damage.

All unintentional holes or openings in the duct shall be repaired prior to concrete placing.

Grout openings and vents shall be securely anchored to the duct and to either the forms or to reinforcing steel to prevent displacement during concrete-placing operations.

After installation in the forms, the ends of ducts shall at all times be sealed as necessary to prevent the entry of water or debris.

During concrete placement for precast segments, mandrels shall be used as stiffeners in each duct and shall extend throughout the length of the segment being cast and at least 2.0 ft into the corresponding duct of the previously cast segment. The mandrels shall be of sufficient rigidity to maintain the duct geometry within the tolerances shown in Table 10.4.1.1-1.

Table 10.4.1.1-1—Duct Position Tolerances

Tolerances	Vertical position in.	Lateral position in.
Horizontal tendons in slabs or in slab regions of larger members	±1/4	±1/2
Longitudinal draped superstructure tendons in webs Tendon over supports or in middle third of span	±1/4	±1/4
Tendon in middle half of web depth	±1/2	±1/4
Longitudinal, generally horizontal, superstructure tendons usually in top or bottom of member	±1/4	±1/4
Horizontal tendons in substructures and foundations	±1/2	±1/2
	Longitudinal position in.	Transverse position in.
Vertical tendons in webs	±1/4	±1/4
Vertical tendons in pier shafts	±1/4	±1/4

Additionally, the following shall apply:

- In all other cases, locate tendons within ±1/4 in. in any direction.
- Entrance and exit angles of tendon paths at anchorages and/or at faces of concrete shall be within ± 3 degrees of desired angle measured in any direction and any deviations in the alignment are accomplished with smooth transitions without any kinks.
- Angle changes at duct joints shall not be greater than ±3 degrees in any direction and must be accomplished with smooth transitions without any kinks.
- Locate anchorages within ±1/4 in. of desired position laterally and ±1 in. along the tendon except that minimum cover requirements shall be maintained.
- Position anchorage confinement reinforcement in the form of spirals, multiple U-shaped bars or links, to be properly centered around the duct and to start within 1/2 in. of the back of the main anchor plate.
- If conflicts exist between the reinforcement and post-tensioning duct, position the post-tensioning duct and adjust the reinforcement locally with the Engineer’s approval.

10.4.1.2—Duct Inlets and Outlets

All ducts for continuous structures shall be supplied with outlets at the high and low points of the duct profile, except where the profile changes are small, as in continuous slabs, and at additional locations as specified in the contract documents. Low-point outlets shall remain open until grouting is started.

C10.4.1.2

See Section 2.8, "Inlets and Outlets," of PTI's *Specification for Grouting of Post-Tension Structures*, 2001.

10.4.1.3—Proving of Post-Tensioning Ducts

Upon completion of concrete placement, the Contractor shall prove that the post-tensioning ducts are free and clear of any obstructions or damage and are able to accept the intended post-tensioning tendons by passing a torpedo through the ducts. The torpedo shall have the same cross-sectional shape as the duct, and 0.25 in. smaller all around than the clear nominal inside dimensions of the duct. No deductions shall be made to the torpedo section dimensions for tolerances allowed in the manufacture or fixing of the ducts. For straight ducts, a torpedo at least 2.0 ft long shall be used. For curved ducts, the length shall be determined so that when both ends touch the outermost wall of the duct, the torpedo is 0.25 in. clear of the innermost wall. If the torpedo will not travel completely through the duct, the Engineer shall reject the member, unless a workable repair can be made to clear the duct, to the satisfaction of the Engineer. Upon completion of the repairs, the torpedo shall pass through the duct easily, by hand, without resorting to excessive effort or mechanical assistance.

C10.4.1.3

Proving the ducts by use of a manually inserted torpedo is intended for segmental construction only. For cast-in-place construction, the Contractor may propose an alternate method acceptable to the Engineer.

10.4.1.4—Duct Pressure Field Test

Before stressing and grouting internal or external tendons, install all grout caps, inlets and outlets and test the tendon with compressed air to determine if duct connections require repair. In the presence of the Engineer, pressurize the tendon to 50 psi and lock-off the outside air source. Record pressure loss for 1 min. A pressure loss of 25 psi is acceptable for tendons having a length of equal to or less than 150 ft and a pressure loss of 15 psi is acceptable for tendons longer than 150 ft. If the pressure loss exceeds the allowable, repair leaking connections using methods approved by the Engineer and retest.

C10.4.1.4

For an external tendon, pressure testing after stressing and before grouting may be specified to verify that the duct was not damaged during stressing.

Use a filtered air supply unless oil and moisture free air can be demonstrated. The air may be tested prior to use by passing air through a white cotton rag for 2 min. If the rag is contaminated, use filtered air.

10.4.2—Placement of Prestressing Steel

10.4.2.1—Placement for Pretensioning

Prestressing steel shall be accurately installed in the forms and held in place by the stressing jack or temporary anchors and, when tendons are to be draped, by hold-down devices. The hold-down devices used at all points of change in slope of tendon trajectory shall be of an approved low-friction type.

Prestressing steel shall not be removed from its protective packaging until immediately prior to installation in the forms and placement of concrete. Openings in the packaging shall be resealed as necessary to protect the unused steel. While exposed, the steel shall be protected as needed to prevent corrosion.

10.4.2.2—Placement for Post-Tensioning

All prestressing steel preassembled in ducts and installed prior to the placement of concrete shall be accurately placed and held in position during concrete placement.

When the prestressing steel is installed after the concrete has been placed, the Contractor shall demonstrate to the satisfaction of the Engineer that the ducts are free of water and debris immediately prior to installation of the steel. The total number of strands in an individual tendon may be pulled into the duct as a unit, or the individual strand may be pulled or pushed through the duct.

Anchorage devices or block-out templates for anchorages shall be set and held so that their axis coincides with the axis of the tendon and anchor plates are normal in all directions to the tendon.

The prestressing steel shall be distributed so that the force in each girder stem is equal or as required by the contract documents, except as provided herein. For box girders with more than two girder stems, at the Contractor's option, the prestressing force may vary up to five percent from the theoretical required force per girder stem provided the required total force in the superstructure is obtained and the force is distributed symmetrically about the centerline of the typical section.

10.4.2.2.1—Protection of Steel After Installation

Prestressing steel used in post tensioned concrete members that is not grouted within the time limit specified below, shall be continuously protected against rust or other corrosion by means of a corrosion inhibitor placed in the ducts or directly applied to the steel. The prestressing steel shall be so protected until grouted or encased in concrete. Prestressing steel installed and tensioned in members after placing and curing of the concrete and grouted within the time limit specified below will not require the use of a corrosion inhibitor described herein and rust which may form during the interval between tendon installation and grouting will not be cause for rejection of the steel.

The permissible interval between tendon installation and grouting without use of a corrosion inhibitor for various exposure conditions shall be taken as follows:

- Very Damp Atmosphere or over Saltwater (Humidity > 70 percent) — 7 days

C10.4.2.2.1

Prestressing steel installed in members prior to placing and curing of the concrete, is not recommended. If installed in the duct, but not grouted within the time limit specified, a corrosion inhibitor placed in the ducts may only be used when directed by the engineer. Most inhibitors are detrimental to bond and removal and reinstallation of strand may be required at the direction of the Engineer.

- Moderate Atmosphere 15 days
  (Humidity from 40 percent to 70 percent)

- Very Dry Atmosphere 20 days
  (Humidity < 40 percent)

After tendons are placed in ducts, the openings at the ends of the ducts shall be sealed to prevent entry of moisture.

When steam curing is used, unless anchorage systems mandate its installation, steel for post-tensioning shall not be installed until the steam curing is completed.

Such tendons shall be protected against corrosion by means of a corrosion inhibitor placed in the ducts or on the steel, or shall be stressed and grouted within seven days after steam curing.

Whenever electric welding is performed on or near members containing prestressing steel, the welding ground shall be attached directly to the steel being welded. All prestressing steel and hardware shall be protected from weld spatter or other damage.

10.4.3—Placement of Anchorage Hardware

The Contractor is responsible for the proper placement of all materials according to the design documents of the Engineer-of-Record and the requirements stipulated by the anchorage device supplier. The Contractor shall exercise all due care and attention in the placement of anchorage hardware, reinforcement, concrete and consolidation of concrete in anchorage zones. Modifications to the local zone details verified under provisions of Article 5.10.9.7.3 of the *AASHTO LRFD Bridge Design Specifications* and Article 10.3.2.3, "Special Anchorage Device Acceptance Test," herein shall be approved by both the Engineer-of-Record and the anchorage device supplier.

C10.4.3

Anchorage zones are very critical regions of a structure. Therefore, construction should follow exactly the specifications by the Engineer-of-Record and the anchorage device supplier. Change of anchorage zone details have to be approved by the Engineer-of-Record and the anchorage device supplier.

Modifications to the local zone details refer to Article 5.10.9.7.3, "Special Anchorage Devices," of the *LRFD Bridge Design Specifications*, 2007.

10.5—IDENTIFICATION AND TESTING

All wire, strand, or bars to be shipped to the site shall be assigned a lot number and tagged for identification purposes. Anchorage assemblies to be shipped shall be likewise identified.

Each lot of wire or bars and each reel of strand reinforcement shall be accompanied by a Manufacturer's Certificate of Compliance, a mill certificate, and a test report. The mill certificate and test report shall include:

- Chemical composition (not required for strand),
- Cross-sectional area,
- Yield and ultimate strengths,
- Elongation at rupture,
- Modulus of elasticity, and

- Stress strain curve for the actual prestressing steel intended for use.

All values certified shall be based on test values and nominal sectional areas of the material being certified.

The Contractor shall furnish to the Engineer for verification testing the samples described in the following subarticles selected from each lot. If ordered by the Engineer, the selection of samples shall be made at the Manufacturer's plant by the Inspector.

All samples submitted shall be representative of the lot to be furnished and, in the case of wire or strand, shall be taken from the same master roll.

The actual strength of the prestressing steel shall not be less than specified by the applicable ASTM Standard, and shall be determined by tests of representative samples of the tendon material in conformance with ASTM Standards.

All of the materials specified for testing shall be furnished free of cost and shall be delivered in time for tests to be made well in advance of anticipated time of use.

10.5.1—Pretensioning Tendons

For pretensioned strands, one sample at least 5.0 ft long shall be furnished in accordance with the requirements of Paragraph 9.1 of AASHTO M 203M/M 203 (ASTM A416/A416M).

10.5.2—Post-Tensioning Tendons

The following lengths shall be furnished for each 20.0 tons, or portion thereof, lot of material used in the work:

- For wires not requiring heading—sufficient length to make up one parallel-lay cable 5.0 ft long consisting of the same number of wires as the cable to be furnished.

- For strand to be furnished with fittings—5.0 ft between near ends of fittings.

- For bars to be furnished with threaded ends and nuts—5.0 ft between threads at ends.

10.5.3—Anchorage Assemblies and Couplers

The Contractor shall furnish for testing one specimen of each size of prestressing tendon, including couplings, of the selected type, with end fittings and anchorage assembly attached, for strength tests only. These specimens shall be 5.0 ft in clear length, measured between ends of fittings. If the results of the test indicate the necessity of check tests, additional specimens shall be furnished without cost.

When dynamic testing is required, the Contractor shall perform the testing and shall furnish certified copies of test results which indicate conformance with the specified requirements prior to installation of anchorages or couplers. The cost of such testing shall be borne by the Contractor.

For prestressing systems previously tested and approved on projects having the same tendon configuration, the Engineer may not require complete tendon samples provided there is no change in the material, design, or details previously approved. Shop drawings or prestressing details shall identify the project on which approval was obtained; otherwise testing shall be conducted.

10.6—PROTECTION OF PRESTRESSING STEEL

All prestressing steel shall be protected against physical damage and rust or other results of corrosion at all times from manufacture to grouting. Prestressing steel shall also be free of deleterious material such as grease, oil, wax or paint. Prestressing steel that has sustained physical damage at any time shall be rejected. The development of pitting or other results of corrosion, other than rust stain, shall be cause for rejection.

Prestressing steel shall be packaged in containers or shipping forms for the protection of the strand against physical damage and corrosion during shipping and storage. A corrosion inhibitor which prevents rust or other results of corrosion shall be placed in the package or form, or shall be incorporated in a corrosion inhibitor carrier-type packaging material, or when permitted by the Engineer, may be applied directly to the steel. The corrosion inhibitor shall have no deleterious effect on the steel or concrete or bond strength of steel to concrete or grout. Packaging or forms damaged from any cause shall be immediately replaced or restored to original condition.

The shipping package or form shall be clearly marked with a statement that the package contains high-strength prestressing steel, and the type of corrosion inhibitor used, including the date packaged.

All anchorages, end fittings, couplers, and exposed tendons, which will not be encased in concrete or grout in the completed work, shall be permanently protected against corrosion.

10.7—CORROSION INHIBITOR

Corrosion inhibitor shall consist of a vapor phase inhibitor (VPI) powder conforming to the provisions of Federal Specification MIL-P-3420F-87 or as otherwise approved by the Engineer. When approved, water soluble oil may be used on tendons as a corrosion inhibitor.

C10.7

Most inhibitors are detrimental to bond and should only be used at the direction of the Engineer.

10.8—DUCTS

10.8.1—General

Ducts used to provide holes or voids in the concrete for the placement of post-tensioned bonded tendons may be either formed with removable cores or may consist of rigid or semi-rigid ducts which are cast into the concrete.

Ducts formed with removable cores shall be formed with no constrictions which would tend to block the passage of grout. All coring materials shall be removed.

Ducts formed by sheath left in place shall be a type that will not permit the intrusion of cement paste. They shall transfer bond stresses as required and shall retain shape under the weight (mass) of the concrete and shall have sufficient strength to maintain their correct alignment without visible wobble during placement of concrete.

10.8.2—Metal Ducts

Sheathing for ducts shall be metal, except as provided herein. Such ducts shall be galvanized ferrous metal and shall be fabricated with either welded or interlocked seams. Galvanizing of welded seams will not be required. Rigid ducts shall have smooth inner walls and shall be capable of being curved to the proper configuration without crimping or flattening. Semi-rigid ducts shall be corrugated and, when tendons are to be inserted after the concrete has been placed, their minimum wall thickness shall be as follows: 26 gage for ducts less than or equal to 2.625-in. diameter, 24 gage for ducts greater than 2.625-in. diameter. When bar tendons are preassembled with such ducts, the duct thickness shall not be less than 31 gage.

10.8.3—Plastic Ducts

For locations in saltwater environment or exposure to deicing chemicals, plastic duct material shall be considered and is recommended.

Corrugated plastic duct to be completely embedded in concrete shall be constructed from either polyethylene or polypropylene. The minimum acceptable radius of curvature shall be established by the duct supplier according to standard test methods. The duct shall have a thickness as shown in Table 10.8.3-1. Ducts shall have a white coating on the outside or shall be of white material with ultraviolet stabilizers added. Polyethylene duct shall be fabricated from resins meeting or exceeding the requirements of ASTM D3350 with a cell classification of 345464A. Polypropylene duct shall be fabricated from resins meeting or exceeding the requirements of ASTM D4101 with a cell classification range of PP0340B14541 to PP0340B67884. Use resin containing antioxidant(s) with a minimum Oxidation Induction Time (OIT) according to ASTM D3895 of not less than 20 min. The OIT test shall be performed on samples taken from the finished product.

C10.8.3

The cell class includes carbon black for sunlight and ultraviolet light protection.

Table 10.8.3-1—Duct Thickness

Duct Shape	Duct Diameter in.	Duct Thickness in.
Flat	any size	0.08
Round	0.9	0.08
Round	2.375	0.08
Round	3.0	0.10
Round	3.35	0.10
Round	4.0	0.12
Round	4.5	0.14
Round	5.125	0.16
Round	5.71	0.16

Rigid smooth black polyethylene ducts for use where the tendon is not embedded in concrete shall be rigid pipe manufactured from 100 percent virgin polyethylene resin meeting the requirements of ASTM D3350 with a minimum cell class of 344464C. A resin containing antioxidant(s) with a minimum oxidation induction time (OIT) according to ASTM D3895 of not less than 40 min shall be used. The OIT test shall be performed on samples take from the finished product. The duct shall be manufactured with a dimensional ratio (*DR*) of 17.0 as established by either ASTM D3350 or ASTM F714 as appropriate for the manufacturing process used.

10.8.4—Duct Area

The provisions of Article 5.4.6.2, "Size of Ducts," of the *AASHTO LRFD Bridge Design Specifications*, 2007, shall apply.

10.8.5—Duct Fittings

Coupling and transition fittings for ducts formed by sheathing shall be of either ferrous metal polyolefin (polyethylene or polypropylene), and shall be air and watertight and of sufficient strength to prevent distortion or displacement of the ducts during concrete placement and/or tendon grouting.

All ducts or anchorage assemblies shall be provided with pipes or other suitable connections at each end of the duct for the injection of grout after prestressing. As specified in Article 10.4.1.2, "Duct Inlets and Outlets," ducts shall also be provided with ports for venting or grouting at high points and for draining at intermediate low points.

Vent and drain pipes shall be at least 0.75-in. diameter for strand and at least 0.5-in. diameter for single-bar tendons and three or four strand flat duct tendons. Connection to ducts shall be made with metallic or plastic structural fasteners. The vents and drains shall be mortar tight, taped as necessary, and constructed with either mechanical or shrink wrap connections. Vents and drains shall provide means for injection of grout through the vents and for sealing to prevent leakage of grout.

10.9—GROUT

Post-tensioning grout shall meet the grout physical properties stated in Article 10.9.3, "Grout Physical Properties." Grouts may be either a unique design for the project or supplied in a prebagged form by a grout Manufacturer. For uniquely designed grouts, the cement and admixtures utilized in the laboratory trial batches of the proposed grout shall not be changed during the construction without retesting. Freshness of the cement should be in accordance with AASHTO M 85 (ASTM C150), except as specified herein. Daily field testing of the grout for the following properties shall be required:

- Fluidity,
- Bleed at 3 h, and
- Permeability.

A preapproved, prebagged grout supplied by a grout Manufacturer may be used as an alternative to the required field testing. These grouts shall be prebagged in plastic lined or coated containers, stamped with date of manufacture, lot number, and mixing instructions. Any change of materials or material sources shall require retesting and certification of the conformance of the grout with the physical properties requirements. A copy of the Quality-Control Data Sheet for each lot number and shipment sent to the job site shall be provided to the Contractor by the grout supplier and shall be furnished to the Engineer. Materials with a total time from manufacture to usage in excess of six months shall be retested and certified by the supplier before use, or shall be removed from the job site and replaced.

10.9.1—Approval

Manufacturers of post-tensioning grout shall submit for approval certified test reports from an audited and independent Cement Concrete Research Laboratory (CCRL) which shows the material meets all the requirements specified herein.

10.9.2—Mixing

The material shall be mixed in accordance with the Manufacturer's recommendations.

The water used in the grout shall be potable, clean, and free of injurious quantities of substances known to be harmful to Portland Cement or prestressing steel.

C10.9.2

See Article 10.11.4, "Mixing of Grout."

10.9.3—Grout Physical Properties

Grouts shall achieve a nonbleeding characteristic. Grout shall contain no aluminum powder or gas generating system that produces hydrogen, carbon dioxide, or oxygen. Cementitious grout shall meet or exceed the specified physical properties stated herein as determined by the following standard and modified ASTM test methods.

Grout Classes shall be taken as specified in Table 10.9.3-1 and Grout Properties shall be as specified in Table 10.9.3-2.

C10.9.3

Bleed characteristics of cementitious grouts are documented in numerous reports, as well as the report titled *Development of High Performance Grouts for Bonded Post-Tensioned Structures* by A. J. Schokker, B. D. Koester, J. E. Brean, and M. E. Kreger.

Table 10.9.3-1—Grout Classes

Class	Exposure	Constituent Materials								Required Testing
		Cement, lb	Fly Ash (Type F), % [a]	Slag, % [a]	Silica Fume (dry), % [a]	Water/ Cementitious Material Ratio [$W/(c+m)$]	High-Range Water Reducer [b] (Type F or G), oz/100 lb	Calcium Nitrite [c], lb/yd^3	Other Admixtures [d]	
A	Nonaggressive: Indoor or nonaggressive outdoor	220	0	0	0	0.45 max.	0	0	—	10.9.3
B	Aggressive: Subject to wet/dry cycles, marine environment, deicing salts	220	0 min. 25 max.	0 min. 55 max.	0 min. 15 max.	0.45 max.	0 min. 46 max.	0 min. 46 max.	As per Manufacturer's recommendation	10.9.3
C Prepackaged	Aggressive or nonaggressive	—	—	—	—	0.45 max.	—	—	—	See Note [e]
D Special	Determined by the Specifying Designer									10.9.3

Notes:

[a] Percent of cement substitute.

[b] Type D and E admixtures may be permitted with approval of the Designer.

[c] Alternate corrosion inhibitors may be used.

[d] Other admixtures such as anti-bleed admixtures, pumping aids, and air entraining agents.

[e] No testing is required if material is prepackaged and the Manufacturer has conducted performance testing.

Table 10.9.3-2—Grout Properties for Volume Change at 28 Days

Property	Test Value	Test Method
Total Chloride Ions	Max. 0.08% by weight of cementitious material	ASTM C1152/C1152M
Fine Aggregate (if utilized)	Max. Size ≤No. 50 Sieve	ASTM C33
Volume Change at 28 days	0.0% to +0.2% at 24 h and 28 days	ASTM C1090*
Expansion	≤2.0% for up to 3 h	ASTM C940
Compressive Strength 28 day (average of 3 cubes)	≥6 ksi	ASTM C942
Initial Set of Grout	Min. 3 h Max. 12 h	ASTM C953
Fluidity Test** Efflux Time from Flow Cone a) Immediately after Mixing b) 30 min after Mixing with Remixing for 30 s	Min. 11 s Max. 30 s or Min. 9 s Max. 20 s Max. 30 s or Max. 30 s	 ASTM C939 ASTM C939*** ASTM C939 ASTM C939***
Bleeding at 3 h	Max. 0.0%	ASTM C940****
Permeability at 28 days	Max. 2500 coulombs at 30 volts for 6 h	AASHTO T 277 (ASTM C1202)

Notes:

* Modify ASTM C1090 to include verification at both 24 h and 28 days.

** Adjustments to flow rates will be achieved by strict compliance with the Manufacturer’s recommendations.

*** Grout fluidity shall meet either the standard ASTM C939 flow cone test or the modified test described herein. Modify the ASTM C939 test by filling the cone to the top instead of to the standard level. The efflux time is the time to fill a 1.0-L container placed directly under the flow cone.

**** Modify ASTM C940 to conform with the wick induced bleed test described below:

a) Condition dry ingredients, mixing water, prestressing strand and test apparatus overnight at 70 to 77°F.

b) Insert 800 mL of mixed conditioned grout with conditioned water into the 1000 mL graduated cylinder. Mark the level of the top of the grout.

c) Wrap the strand with 2.0-in. wide duct or electrical tape at each end prior to cutting to avoid splaying of the wires when it is cut. Degrease (with acetone or hexane solvent) and wire brush to remove any surface rust on the strand before temperature conditioning. Insert completely a 20.0-in. length of conditioned, cleaned, ASTM A416/A416M seven wire strand 0.5-in. diameter into the 1000 mL graduated cylinder. Center and fasten the strand so it remains essentially parallel to the vertical axis of the cylinder (possibly using a centralizer). Mark the level of the top of the grout.

d) Store the mixed grout at the temperature range listed above in (a).

e) Measure the level of the bleed water every 15 min for the first hour and hourly afterward for 2 h.

f) Calculate the bleed water, if any, at the end of the 3-h test period and the resulting expansion per the procedures outlined in ASTM C940, with the quantity of bleed water expressed as a percent of the initial grout volume. Note if the bleed water remains above or below the top of the grout.

10.10—TENSIONING

10.10.1—General Tensioning Requirements

Prestressing steel shall be tensioned by hydraulic jacks so as to produce the forces shown in the contract documents or on the approved working drawing with appropriate allowances for all losses. Losses to be provided for shall be as specified in Article 5.9.5, “Loss of Prestress,” of the *AASHTO LRFD Bridge Design Specifications*, 2007. For post-tensioned work, the losses shall also include the anchor set loss appropriate for the anchorage system employed.

C10.10.1

The proposed changes result from changes made to clarify provisions of *AASHTO LRFD Bridge Design Specifications*, 2007, Article 5.9.3, “Stress Limitations for Prestressing Tendons.”

For pretensioned members, the strand stress prior to seating (jacking stress) shall not exceed 80 percent of the minimum ultimate tensile strength of the prestressing steel ($0.80 f'_s$). This allowable stress, which slightly exceeds the values allowed in *AASHTO LRFD Bridge Design Specifications*, Article 5.9.3, may be permitted to offset seating losses and to accommodate compensation for temperature differences specified in Article 10.10.2, "Pretensioning Requirements."

For post-tensioned members, the standard stress prior to seating (jacking stress) and the stress in the steel immediately after seating shall not exceed the values allowed in *AASHTO LRFD Bridge Design Specifications*, Article 5.9.3.

Tensioning may be accomplished by pretensioning, post-tensioning, or the combined method; as specified in the contract documents, or on the approved working drawings, or approved in writing by the Engineer.

During stressing of strand, individual wire failures may be accepted by the Engineer, provided not more than one wire in any strand is broken and the area of broken wires does not exceed two percent of the total area of the prestressing steel in the member.

For pretensioned members, the existing second paragraph of this Article has been expanded to include the permitted increase in strand stress prior to seating. This limit is placed here after being removed from *AASHTO LRFD Bridge Design Specifications*, Article 5.9.3. The wording "shall not exceed 80 percent of the minimum ultimate tensile strength of the prestressing steel" is taken directly from Article 10.10.2. A reference is added regarding compensation for temperature differences which is specified in Article 10.10.2.

For post-tensioned members, the existing second paragraph of this Article is revised to be consistent with the revised wording of *AASHTO LRFD Bridge Design Specifications*, Article 5.9.3.

The methods of tensioning may be defined as follows:

- Pretensioning—The prestressing strand or tendons are stressed prior to being embedded in the concrete placed for the member. After the concrete has attained the required strength, the prestressing force is released from the external anchorages and transferred, by bond, into the concrete.

- Post-tensioning—The reinforcing tendons are installed in voids or ducts within the concrete and are stressed and anchored against the concrete after the development of the required concrete strength. As a final operation under this method, the voids or ducts are pressure-grouted.

- Combined Method—Part of the reinforcement is pretensioned and part post-tensioned. Under this method, all applicable requirements for pretensioning and for post-tensioning shall apply to the respective reinforcing elements using these methods.

10.10.1.1—Concrete Strength

Prestressing forces shall not be applied or transferred to the concrete until the concrete has attained the strength specified for initial stressing. In addition, cast-in-place concrete for other than segmentally constructed bridges shall not be post-tensioned until at least ten days after the last concrete has been placed in the member to be post-tensioned.

10.10.1.2—Prestressing Equipment

Hydraulic jacks used to stress tendons shall be capable of providing and sustaining the necessary forces and shall be equipped with either a pressure gage or a load cell for determining the jacking stress. The jacking system shall provide an independent means by which the tendon elongation can be measured. The pressure gage shall have an accurately reading dial at least 6.0 in. in diameter or a digital display, and each jack and its gage shall be calibrated as a unit with the cylinder extension in the approximate position that it will be at final jacking force, and shall be accompanied by a certified calibration chart or curve. The load cell shall be calibrated and shall be provided with an indicator by means of which the prestressing force in the tendon may be determined. The range of the load cell shall be such that the lower ten percent of the Manufacturer's rated capacity will not be used in determining the jacking stress. When approved by the Engineer, calibrated proving rings may be used in lieu of load cells.

Recalibration of gages shall be repeated at least annually and whenever gage pressures and elongations indicate materially different stresses.

Only oxygen flame or mechanical cutting devices shall be used to cut strand after installation in the member or after stressing. Electric arc welders shall not be used.

10.10.1.3—Sequence of Stressing

When the sequence of stressing individual tendons is not otherwise specified in the contract documents or on the approved working drawings, the stressing of post-tensioning tendons and the release of pretensioned tendons shall be done in a sequence that produces a minimum of eccentric force in the member.

10.10.1.4—Measurement of Stress

A record of gage pressures and tendon elongations for each tendon shall be provided by the Contractor for review and approval by the Engineer. Elongations shall be measured to an accuracy of 0.0625 in. Stressing tails of post-tensioned tendons shall not be cut off until the stressing records have been approved.

The stress in tendons during tensioning shall be determined by the gage or load-cell readings and shall be verified with the measured elongations. Calculations of anticipated elongations shall utilize the modulus of elasticity, based on nominal area, as furnished by the Manufacturer for the lot of steel being tensioned, or as determined by a bench test of strands used in the work.

All tendons shall be tensioned to a preliminary force as necessary to eliminate any take-up in the tensioning system before elongation readings are started. This preliminary force shall be between 5 and 25 percent of the final jacking force. The initial force shall be measured by a dynamometer or by other approved method, so that its amount can be used as a check against elongation as computed and as measured. Each strand shall be marked prior to final stressing to permit measurement of elongation and to ensure that all anchor wedges set properly.

It is anticipated that there may be discrepancy in indicated stress between jack gage pressure and elongation. In such event, the load used as indicated by the gage pressure shall produce a slight overstress rather than understress. When a discrepancy between gage pressure and elongation of more than five percent in tendons over 50.0 ft long or seven percent in tendons of 50.0 ft or less in length occurs, the entire operation shall be carefully checked and the source of error determined and corrected before proceeding further. When provisional ducts are provided for addition of prestressing force in event of an apparent force deficiency in tendons over 50.0 ft long, the discrepancy between the force indicated by gage pressure and elongation may be increased to seven percent before investigation into the source of the error.

10.10.2—Pretensioning Requirements

Stressing shall be accomplished by either single-strand stressing or multiple-strand stressing. The amount of stress to be given to each strand shall be as shown in the contract documents or on the approved working drawings.

All strands to be stressed in a group (multiple-strand stressing) shall be brought to a uniform initial tension prior to being given their full pretensioning. The amount of the initial tensioning force shall be within the range specified in Article 10.10.1.4, "Measurement of Stress," and shall be the minimum required to eliminate all slack and to equalize the stresses in the tendons as determined by the Engineer. The amount of this force will be influenced by the length of the casting bed and the size and number of tendons in the group to be tensioned.

Draped pretensioned tendons shall either be tensioned partially by jacking at the end of the bed and partially by uplifting or depressing tendons, or they shall be tensioned entirely by jacking, with the tendons being held in their draped positions by means of rollers, pins, or other approved methods during the jacking operation.

Approved low-friction devices shall be used at all points of change in slope of tendon trajectory when tensioning draped pretensioned strands, regardless of the tensioning method used.

If the load for a draped strand, as determined by elongation measurements, is more than five percent less than that indicated by the jack gages, the strand shall be tensioned from both ends of the bed, and the load as computed from the sum of elongation at both ends shall agree within five percent of that indicated by the jack gages.

When ordered by the Engineer, prestressing steel strands in pretensioned members, if tensioned individually, shall be checked by the Contractor for loss of prestress not more than 3 h prior to placing concrete for the members.

The method and equipment for checking the loss of prestress shall be subject to approval by the Engineer. All strands that show a loss of prestress in excess of three percent shall be retensioned to the original computed jacking stress.

Stress on all strands shall be maintained between anchorages until the concrete has reached the compressive strength required at time of transfer of stress to concrete.

When prestressing steel in pretensioned members is tensioned at a temperature more than 25°F lower than the estimated temperature of the concrete and the prestressing steel at the time of initial set of the concrete, the calculated elongation of the prestressing steel shall be increased to compensate for the loss in stress due to the change in temperature, but in no case shall the jacking stress exceed 80 percent of the specified minimum ultimate tensile strength of the prestressing steel.

Strand splicing methods and devices shall be approved by the Engineer. When single-strand jacking is used, only one splice per strand will be permitted. When multi-strand jacking is used, either all strands shall be spliced or no more than ten percent of the strands shall be spliced. Spliced strands shall be similar in physical properties, from the same source, and shall have the same “twist” or “lay.” All splices shall be located outside of the prestressed units.

Side and flange forms that restrain deflection shall be removed before release of pretensioning reinforcement.

Except when otherwise shown in the contract documents, all pretensioned prestressing strands shall be cut off flush with the end of the member, and the exposed ends of the strand and a 1.0-in. strip of adjoining concrete shall be cleaned and painted. Cleaning shall be by wire brushing or abrasive blast cleaning to remove all dirt and residue that is not firmly bonded to the metal or concrete surfaces. The surfaces shall be coated with one thick coat of zinc-rich paint conforming to the requirements of U.S. Military Specification MIL-P-24441/20. The paint shall be thoroughly mixed at the time of application, and shall be worked into any voids in the strands.

10.10.3—Post-Tensioning Requirements

Prior to post-tensioning any member, the Contractor shall demonstrate to the satisfaction of the Engineer that the prestressing steel is free and unbonded in the duct.

All strands in each tendon, except for those in flat ducts with not more than four strands, shall be stressed simultaneously with a multi-strand jack.

Tensioning shall be accomplished so as to provide the forces and elongations specified in Article 10.5.1, “Pretensioning Tendons.”

Except as provided herein or when specified in the contract documents or on the approved working drawings, tendons in continuous post-tensioned members shall be tensioned by jacking at each end of the tendon. For straight tendons and when one end stressing is shown in the contract documents, tensioning may be performed by jacking from one-end or both ends of the tendon at the option of the Contractor.

10.10.4—Record of Stressing Operation

A record of the following post-tensioning operations shall be kept for each tendon installed:

- Project name, number,
- Contractor and/or subcontractor,
- Tendon location, size and type,
- Date tendon was first installed in ducts,
- Coil/reel number for strands or wires and heat number for bars and wire,
- Assumed and actual cross-sectional area,
- Assumed and actual modulus of elasticity,
- Date stressed,
- Jack and gage numbers per end of tendon,
- Required jacking force,
- Gage pressures,
- Elongations (anticipated and actual),
- Anchor sets (anticipated and actual),
- Stressing sequence (i.e., tendons before and after this tendon),
- Stressing mode (one end/two ends/simultaneous),
- Witnesses to stressing operation (Contractor and Inspector),
- Date grouted, days from stressing to grouting, grouting pressure applied, and injection end,
- Record any other relevant information including pourback and bitumastic installation dates,

The Engineer shall be provided with a complete copy of all stressing operations, and the jack calibration forms.

10.10.5—Protection of Tendon

Within 4 h after stressing and prior to grouting, tendons shall be protected against corrosion or harmful effects of debris by temporarily plugging or sealing all openings and vents; cleaning rust and other debris from all metal surfaces which will be covered by the grout cap; and placing the grout cap, including a seal, over the wedge plate until the tendon is grouted.

10.11—GROUTING

10.11.1—General

When the post-tensioning method is used, the prestressing steel shall be provided with permanent protection and shall be bonded to the concrete by completely filling the void space between the duct and the tendon with grout. Grout should be injected from low points pumping toward the high-point vent. For segmental, span-by-span construction, grout shall be injected through a grout inlet at midspan.

All grouting operations shall be carried out by experienced superintendents and foremen that have received instructional training and have at least 3 yr of experience on previous projects involving grouting of similar type and magnitude.

A grouting operation plan shall be submitted for approval at least 45 days in advance of any scheduled grouting operations. Written approval of the grouting operation plan by the Engineer shall be required before any grouting of the permanent tendons in the structure takes place.

At a minimum, the following items shall be provided in the grouting operation plan:

- Provide names, and proof of training and experience records for the grouting crew and the crew supervisor in conformance with this Specification;
- Type, quantity, and brand of materials used in grouting including all required certifications;
- Type of equipment furnished, including capacity in relation to demand and working condition, as well as back-up equipment and spare parts;
- General grouting procedure;
- Duct pressure test and repair procedures;
- Method to be used to control the rate of flow within ducts;
- Theoretical grout volume calculations;
- Mixing and pumping procedures;
- Direction of grouting;
- Sequence of use of the inlets and outlet pipes;
- Procedures for handling blockages; and
- Procedures for possible post grouting repair.

Before grouting operations begin, a joint meeting of the Contractor, grouting crew and the Engineer shall be conducted. At the meeting, the grouting operation plan, required testing, corrective procedures, and any other relevant issues shall be discussed.

10.11.2—Preparation of Ducts

Each duct shall be air pressure tested prior to the installation of the prestressing steel into the ducts. If leaks are indicated during the test, the duct shall be repaired to eliminate the leakage or minimize the consequences of the leakage.

All ducts shall be clean and free of deleterious materials that would impair bonding or interfere with grouting procedures.

Ducts with concrete walls (cored ducts) shall be flushed to ensure that the concrete is thoroughly wetted. Metal ducts shall be flushed if necessary to remove deleterious material.

Water used for flushing ducts may contain slack lime (calcium hydroxide) or quicklime (calcium oxide) in the amount of 0.1 lb/gal.

After flushing, all water shall be blown out of the duct with oil-free compressed air.

C10.11.2

The purpose of the test is to find the location of any leaks.

10.11.3—Equipment

The grouting equipment for Type B grout material shall include a high-speed shear mixer capable of continuous mechanical mixing which will produce a grout free of lumps and undispersed cement, a grout pump, and stand-by flushing equipment with water supply. The equipment shall be able to pump the mixed grout in a manner which will comply with all requirements.

Accessory equipment which will provide for accurate solid and liquid measures shall be provided to batch all materials.

The pump shall be a positive displacement type and be able to produce an outlet pressure of at least 0.150 ksi. The pump should have seals adequate to prevent introduction of oil, air, or other foreign substance into the grout, and to prevent loss of grout or water.

A pressure gage having a full-scale reading of no greater than 0.300 ksi shall be placed at some point in the grout line between the pump outlet and the duct inlet.

The grouting equipment shall contain a screen having clear openings of 0.125-in. maximum size to screen the grout prior to its introduction into the grout pump. If a grout with a thixotropic additive is used, a screen opening of 0.1875 in. is satisfactory. This screen shall be easily accessible for inspection and cleaning.

The grouting equipment shall utilize gravity feed to the pump inlet from a hopper attached to and directly over it. The hopper must be kept at least partially full of grout at all times during the pumping operation to prevent air from being drawn into the post-tensioning duct.

Under normal conditions, the grouting equipment shall be capable of continuously grouting the largest tendon on the project in no more than 20 min.

10.11.4—Mixing of Grout

Water shall be added to the mixer first, followed by cement grout.

Grout shall be mixed in accordance with the Manufacturer's instructions using a colloidal mixer to obtain homogeneous mixture. A fluidity test shall be performed on the mixed grout prior to beginning the injection process. Target flow rates as a function of mixer type used and ambient temperatures shall be obtained from the grout Manufacturer. The grouting process shall not be started until the proper grout properties have been obtained.

Mixing shall be of such duration as to obtain a uniform, thoroughly blended grout, without excessive temperature increase or loss of expansive properties of the admixture. The grout shall be continuously agitated until it is pumped.

Water shall not be added to increase grout flowability which has been decreased by delayed use of the grout.

Other methods may be used to determine fluidity such as U.S. Army Corps of Engineers Method CRD C79.

10.11.5—Injection of Grout

All grout vents shall be opened before grouting starts. Injection and ejection vents with positive shut-offs shall be provided. Grout shall be allowed to flow from the first injection vent until any residual flushing water or entrapped air has been removed prior to closing that vent. Remaining vents shall be closed in sequence in the same manner. A continuous flow of grout at a rate between 35.0 and 50.0 ft of duct per min shall be maintained.

The pumping pressure at the injection vent should not exceed 0.150 ksi. Normal operations shall be performed at approximately 0.075 ksi. If the actual grouting pressure exceeds the maximum allowed, the injection vent shall be closed and the grout shall be injected at the next vent that has been, or is ready to be closed, as long as a one-way flow is maintained. Grout shall not be injected into a succeeding vent from which grout has not yet flowed.

Grout shall be pumped through the duct and continuously wasted at the ejection vent until no visible slugs of water or air are ejected. A fluidity test shall be performed on each tendon in accordance with Article 10.9.3, "Grout Physical Properties," measuring the grout fluidity from the discharge outlet. The measured grout efflux time shall not be faster than the efflux time measured at the inlet or the minimum efflux time established in Article 10.9.3. If the grout efflux time is not acceptable, additional grout shall be discharged from the discharge outlet. Grout efflux time shall be tested. This cycle shall be continued until acceptable grout fluidity is

C10.11.5

To minimize potential for bleed water, especially on vertical ducts, a secondary shot of grout can be given to the duct approximately 2 to 3 min after initial completion of the grouting.

achieved. To ensure that the tendon remains filled with grout, the ejection and injection vents shall be closed in sequence, respectively, under pressure when the tendon duct is completely filled with grout. The positive shut-offs at the injection and ejection vents shall not be removed or vents opened until the grout has set.

10.11.6—Temperature Considerations

In temperatures below 32°F, ducts shall be kept free of water to avoid damage due to freezing. The temperature of the concrete shall be 35°F or higher from the time of grouting until job cured 2.0-in. cubes of grout reach a minimum compressive strength of 0.800 ksi. Grout shall not be above 90°F during mixing or pumping. If necessary, cool the mixing water.

10.11.7—Vertical Grouting

In lieu of a positive shut-off, vertical or near vertical tendon ducts for grouting shall terminate in reservoirs at the upper-most point. The reservoir shall have sufficient capacity to store excess grout bleed water. Visible grout level shall be maintained in the reservoirs. The reservoirs shall be maintained until the grout has set.

Grout shall be injected at a rate of 15.0 ft of duct per min.

10.11.8—Post-Grouting Inspection

Vacuum grouting shall be used to fill any voids that expose strands discovered in the grouting process.

Where possible, all anchorages and high-point vents shall be drilled and probed 48 h after grouting, until the Engineer is assured that no bleed water or subsidence (settlement) voids exist. After the Engineer is assured that voids do not exist, only one or two anchorages per span shall be drilled and probed to ensure quality grouting. Any voids discovered should be filled immediately with the approved grout.

10.11.9—Finishing

The following requirements apply:

- Valves, caps and vent pipes shall not be removed or opened until the grout has set.
- The ends of vents shall be removed at least 1.0 in. below the concrete surface after the grout has set.
- The void shall be filled with epoxy grout. All miscellaneous material used for sealing grout caps shall be removed before carrying out further work to protect end anchorages.

C10.11.9

Miscellaneous materials include paper, tie wire, duct tape, etc.

10.11.10—Protection of End Anchorages

Permanent grout caps constructed from either stainless steel or polymer shall be specified.

The following requirements apply:

- Within seven days upon completion of the grouting, the anchorage of post-tensioning bars and tendons shall be protected as indicated in the contract documents. The application of the elastomeric coating may be delayed up to 90 days after grouting. Plastic or stainless steel threaded caps shall be used to plug all grout inlets/outlets. A sand-filled epoxy grout suitable for machinery base plate shall be used to construct all pour-backs located at anchorages of expansion joints or other areas exposed to the elements.

- All laitance, grease, curing compounds, surface treatments, coatings, and oils shall be removed by grit blasting or water blasting using a minimum 10.0 ksi nozzle pressure. The surface shall be flushed with water and blown dry. Surfaces shall be clean, sound, and without any standing water. In case of dispute, ACI 503 shall be followed for substrate testing and a minimum of 0.175 ksi tension (pull-off value) be developed.

- Epoxy shall be mixed and applied as per Manufacturer's current standard technical guidelines. All pour-backs shall be in leak proof forms creating neat lines. The pumping of epoxy grout shall be permitted for proper installation. Forms shall be constructed to maintain a liquid head to insure intimate contact with the concrete surface. Vents shall be used as needed to provide for the escape of air to insure complete filling of the forms.

- The exposed surfaces of pour-backs or grout caps, except on transverse tendons, shall be coated with an elastomeric coating system having a thickness of 30.0 to 45.0 mils. Concrete, grout caps or other substrates shall be structurally sound, clean, and dry. Concrete shall be a minimum of 28 days old. Laitance, grease, curing compounds, surface treatments, coatings, and oils shall be removed by grit blasting or water blasting using a minimum 10.0 ksi nozzle pressure to establish the anchor pattern. Surfaces shall be blown with compressed air to remove the dust or water.

C10.11.10

Permanent polymer grout caps, with or without fibers, are adequate for use.

- A 2.0 × 4.0-ft concrete test block shall be constructed with a similar surface texture to the surfaces to be coated and a vertical face shall be coated with the elastomeric coating system chosen. The number of coats required to achieve a coating thickness between 30.0 to 45.0 mils without runs and drips shall be determined. The elastomeric coating shall be mixed and applied as per Manufacturer's current standard technical specifications. Spray or roller application may be permitted; spray application is preferred. Coatings shall be applied using approved and experienced personnel with a minimum of 3 yr experience applying similar polyurethane systems. Credentials of these persons shall be submitted to the Engineer for review and consideration for approval.

10.11.11—Construction Traffic and Operations Causing Vibrations

For the designated period of time after grouting of a tendon begins, vibrations from all sources such as moving vehicles, jackhammers, compressors, generators, etc., that are operating within the affected bridge superstructure shall be eliminated and pile driving and soil compaction within 300 ft of the affected superstructure shall not be permitted. The designated period of time shall be taken as 4 h for prequalified anti-bleed grout and 24 h for plain grout. The affected bridge superstructure shall be taken as that portion of the total superstructure that is 300 ft up or down-station of the ends of the span in which grouting is taking place.

C10.11.11

Recent (2003) research has demonstrated that traffic-induced vibrations (5 Hz) of post-tensioning tendons 4 h after grouting with prequalified anti-bleed grout does not have detrimental effects in generating bleed water in the tendon, and does not have significant effect on the bond between the grout and tendon following hardening of the grout. Accordingly, there is no need for restriction of traffic for more than 4 h after grouting of tendons with prequalified anti-bleed grout on any type of post-tensioned bridge. Research also shows that traffic-induced vibrations (5 Hz) of post-tensioning tendons following grouting with plain grout (water-cement) significantly increases bleed, and 24-h traffic restrictions are necessary when plain grout is used.

10.12—MEASUREMENT AND PAYMENT

10.12.1—Measurement

The prestressing of cast-in-place concrete will be measured by the lump sum for each item or location listed in the contract documents.

10.12.2—Payment

No separate payment will be made for prestressing precast concrete members. Payment for prestressing precast concrete members shall be considered as included in the contract price paid for the precast members as provided for in Section 8, "Concrete Structures."

The contract lump-sum price paid for prestressing cast-in-place concrete shall include full compensation for furnishing all labor, materials, tools, equipment, and incidentals; and for doing all work involved in furnishing, placing, and tensioning the prestressing steel in cast-in-place concrete structures, complete in place, as specified in the contract documents and in these Specifications, and as directed by the Engineer.

Full compensation for furnishing and placing additional concrete and deformed bar reinforcing steel required by the particular system used; ducts, anchoring devices, distribution plates, or assemblies and incidental parts; for furnishing samples for testing, working drawings, and for pressure grouting ducts shall be considered as included in the contract lump-sum price paid for prestressing cast-in-place concrete or in the contract price for furnishing precast members, and no additional compensation will be allowed therefore.

10.13—REFERENCES

AASHTO. 1987. *Guide Specifications for Design and Construction of Segmental Concrete Bridges*, 1st Edition, GSCB-1, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 1999. *Guide Specifications for Design and Construction of Segmental Concrete Bridges*, 2nd Edition, GSCB-2, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2009. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 29th Edition, HM-29, American Association of State and Highway Transportation Officials, Washington, DC.

DeSalvo Jessica, and Andrea Schokker. "Guide Specification for Grouting of Post-Tensioned Structures," PTI Committee on Grouting Specifications, Influence of Vibration during Setting of Post-Tensioned Grout: Effects of Bleed and Bond.

DOD. U.S. Military Specification MIL-P-24441/20 for zinc-rich paint, U.S. Department of Defense, Washington, DC. See https://assist.daps.dla.mil/quicksearch/.

FIB. 2000. "Corrugated Plastic Ducts for Internal Bonded Post-Tensioning," *Bulletin No. 7*, Task Group 9.6 Plastic Ducts of FIB Commission 9, Federale Internationale du Beton (International Federation of Structural Concrete), Lausanne, Switzerland.

FL DOT. 2002. *New Direction for Florida Post-Tensioned Bridges*, Florida Department of Transportation, Tallahassee, FL, June 2002.

FL DOT. 2003. *New Direction for Florida Post-Tensioned Bridges: Final Phase of Implementation*, Florida Department of Transportation, Tallahassee, FL, July 2003.

Hsuan, Grace Y. "Protocol for 100 Years Service Life of Corrugated High Density Polyethylene Pipes, Part II – Stress Crack Resistance, Oxidation Resistance and Viscoelastic Properties of Finished Corrugated Pipes".

PTI. 1998. *Acceptable Standards for Post-Tensioning Systems*, Post-Tensioning Institute, Phoenix, AZ.

PTI. 2001. *Guide Specification for Grouting of Post-Tension Structures*, Post-Tensioning Institute, Phoenix, AZ.

Schokker, A.J., et al. 1999. *Development of High Performance Grouts for Bonded Post-Tensioned Structures*, Research Report 1405-2, University of Texas, Center for Transportation Research, Austin, TX.

USACE. U.S. Army Corps of Engineers Method CRD C79 for grout fluidity, USACE Publication Dept, Hyattsville, MD.

Back to 2010 Edition

SECTION 10: PRESTRESSING

Revise the article as follows:

10.3.1.2—Wire

Uncoated ~~stress-relieved~~ <u>low-relaxation</u> steel wire shall conform to the requirements of AASHTO M 204M/M 204 (ASTM A421/A421M).

2016 Revision

SECTION 11: STEEL STRUCTURES

TABLE OF CONTENTS

SECTION 11

STEEL STRUCTURES

11.1—GENERAL

11.1.1—Description

2014 Revision 2015 Revision

This work shall consist of furnishing, fabricating, and erecting steel structures and structural steel portions of other structures in accordance with these Specifications, and in the contract documents.

Unless otherwise specified, the structural steel fabricating plant shall be certified under the AISC Quality Certification Program, Category I. The fabrication of fracture-critical members shall be Category III.

Details of design which are permitted to be selected by the Contractor shall conform to the *AASHTO LRFD Bridge Design Specifications*, 2007.

Painting shall conform to the provisions of Section 13, "Painting."

Falsework used in the erection of structural steel shall conform to the provisions of Section 3, "Temporary Works."

Structural components designated in the contract documents as "fracture-critical" shall conform to the provisions of the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*, Section 12, "Fracture Control Plan (FCP) for Nonredundant Members."

Welding and weld qualification tests shall conform to the provisions of the current AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code.*

11.1.2—Notice of Beginning of Work

The Contractor shall give the Engineer ample notice of the beginning of work at the mill or in the shop so that inspection may be provided. No material shall be manufactured or work done in the shop before the Engineer has been so notified.

C11.1.2

The term "mill" means any rolling mill or foundry where material for the work is to be manufactured.

11.1.3—Inspection

Structural steel will be inspected at the fabrication site.

The Contractor shall furnish to the Engineer a copy of all mill orders and certified mill test reports. Mill test reports shall show the chemical analysis and physical test results for each heat of steel used in the work. With the approval of the Engineer, Certificates of Compliance shall be furnished in lieu of mill test reports for material that normally is not supplied with mill test reports and for items such as fills, minor gusset plates, and similar material when quantities are small and the material is taken from stock.

Certified mill test reports for steels with specified impact values shall include the results of Charpy V-Notch impact tests in addition to other test results. When fine grain practice is specified, the test report shall confirm the material that was so produced. Copies of mill orders shall be furnished at the time orders are placed with the Manufacturer. Certified mill test reports and Certificates of

2010 Edition

Compliance shall be furnished prior to the start of fabrication of material covered by these reports. The Certificate of Compliance shall be signed by the Manufacturer and shall certify that the material is in conformance with the specifications to which it has been manufactured.

Material to be used shall be made available to the Engineer so that each piece can be examined. The Engineer shall have free access at all times to any portion of the fabrication site where the material is stored or where work on the material is being performed.

11.1.4—Inspector's Authority

The Inspector shall have the authority to reject materials or work which does not fulfill the requirements of these Specifications. In cases of dispute, the Contractor may appeal to the Engineer, whose decision shall be final.

Inspection at the mill and shop is intended as a means of facilitating the work and avoiding errors and it is expressly understood that it shall not relieve the Contractor of any responsibility in regard to defective material or work and the necessity for replacing the same at the Contractor's cost.

The acceptance of any material or finished members by the Inspector shall not be a bar to their subsequent rejection, if found defective. Rejected materials and work shall be replaced as soon as practical or corrected by the Contractor.

11.2—WORKING DRAWINGS

The Contractor shall expressly understand that the Engineer's approval of the working drawings submitted by the Contractor covers the requirements for "strength and detail," and that the Engineer assumes no responsibility for errors in dimensions.

Working drawings must be approved by the Engineer prior to performance of the work involved and such approval shall not relieve the Contractor of any responsibility under the contract for the successful completion of the work.

11.2.1—Shop Drawings

The Contractor shall submit copies of the detailed shop drawings to the Engineer for approval. Shop drawings shall be submitted sufficiently in advance of the start of the affected work to allow time for review by the Engineer and corrections by the Contractor, if any, without delaying the work.

Shop drawings for steel structures shall give full, detailed dimensions and sizes of component parts of the structure and details of all miscellaneous parts, such as pins, nuts, bolts, drains, etc.

Where specific orientation of plates is required, the direction of rolling of plates shall be shown.

Unless otherwise specified in the contract documents, shop drawings shall identify each piece that is to be made of steel which is to be other than AASHTO M 270M/M 270 (ASTM A709/A709M), Grade 36 (Grade 250) steel.

11.2.2—Erection Drawings

The Contractor shall submit drawings illustrating fully the proposed method of erection. The drawings shall show details of all falsework bents, bracing, guys, dead-men, lifting devices, and attachments to the bridge members: sequence of erection, location of cranes and barges, crane capacities, location of lifting points on the bridge members, and weights of the members. The drawings shall be complete in detail for all anticipated phases and conditions during erection. Calculations may be required to demonstrate that factored resistances are not exceeded and that member capacities and final geometry will be correct.

11.2.3—Camber Diagram

A camber diagram shall be furnished to the Engineer by the Fabricator, showing the camber at each panel point in the cases of trusses or arch ribs, and at the location of field splices and fractions of span length (quarter points minimum) in the cases of continuous beam and girders or rigid frames. The camber diagram shall show calculated cambers to be used in preassembly of the structure in accordance with Article 11.5.3, "Preassembly of Field Connections."

11.3—MATERIALS

11.3.1—Structural Steel

11.3.1.1—General

Steel shall be furnished according to the following specifications. The grade or grades of steel to be furnished shall be as specified in the contract documents.

All steel for use in main load-carrying member components subject to tensile stress shall conform to the applicable Charpy V-Notch impact test requirements of AASHTO M 270M/M 270 (ASTM A709/A709M).

Welded girders made of AASHTO M 270M/M 270 (ASTM A709/A709M), Grade HPS 70W (HPS 485W) shall be fabricated in accordance with the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code* and shall be supplemented by the AASHTO *Guide Specifications for Highway Bridge Fabrication with HPS70W Steel.*

11.3.1.2—Carbon Steel

Unless otherwise specified in the contract documents, structural carbon steel for bolted or welded construction shall conform to: Structural Steel for Bridges, AASHTO M 270M/M 270 (ASTM A709/A709M), Grade 36 (Grade 250).

11.3.1.3—High-Strength, Low-Alloy Structural Steel

High-strength, low-alloy steel shall conform to Structural Steel for Bridges, AASHTO M 270M/M 270 (ASTM A709/A709M), Grades 50, 50S, 50W, or HPS 50W (Grades 345, 345S, 345W, or HPS 345W).

11.3.1.4—High-Strength, Low-Alloy, Quenched, and Tempered Structural Steel Plate

High-strength, low-alloy, quenched, and tempered steel plate shall conform to Structural Steel for Bridges, AASHTO M 270M/M 270 (ASTM A709/A709M), Grade HPS 70W (Grade HPS 485W).

11.3.1.5—High-Yield-Strength, Quenched, and Tempered Alloy-Steel Plate

High-yield-strength, quenched, and tempered alloy-steel plate shall conform to:

- Structural Steel for Bridges AASHTO M 270M/M 270 (ASTM A709/A709M), Grade HPS 100W (Grade HPS 690W).

- Quenched-and-tempered alloy-steel structural shapes and seamless mechanical tubing meeting all of the mechanical and chemical requirements of AASHTO M 270M/M 270 (ASTM A709/A709M), Grade HPS 100W (Grade HPS 690W), except that the specified maximum tensile strength may be 140 ksi for structural shapes and 145 ksi for seamless mechanical tubing, shall be considered as AASHTO M 270M/M 270 (ASTM A709/A709M), Grade HPS 100W (Grade HPS 690W).

11.3.1.6—Eyebars

Steel for eyebars shall be of a weldable grade. These grades include structural steel conforming to:

- Structural Steel for Bridges, AASHTO M 270M/M 270 (ASTM A709/A709M), Grade 36 (Grade 250).

- Structural Steel for Bridges, AASHTO M 270M/M 270 (ASTM A709/A709M), Grades 50, 50W, or HPS 50W (Grades 345, 345W, or HPS 345W).

11.3.1.7—Structural Tubing

Structural tubing shall be either cold-formed welded or seamless tubing conforming to ASTM A500, Grade B or Grade C, or ASTM A 847; or hot-formed welded or seamless tubing conforming to ASTM A501 or ASTM A618.

C11.3.1.7

ASTM A500 cautions that structural tubing manufactured to that specification may not be suitable for applications involving dynamically loaded elements in welded structures where low-temperature notch-toughness properties may be important. As such, the use of this material should be carefully examined with respect to its specific application in consultation with the Owner. Where this material is contemplated for use in applications where low-temperature notch-toughness properties are deemed important, consideration should be given to requiring that the material satisfy the Charpy V-notch fracture toughness requirements specified in Article 6.6.2 of the *AASHTO LRFD Bridge Design Specifications.*

11.3.2—High-Strength Fasteners

11.3.2.1—Material

2014 Revision

High-strength bolts for structural steel joints shall conform to either AASHTO M 164 (ASTM A325) or AASHTO M 253 (ASTM A490). When high-strength bolts are used with unpainted weathering grades of steel, the bolts shall be Type 3.

The supplier shall provide a lot of number appearing on the shipping package and a certification noting when and where all testing was done, including rotational capacity tests, and zinc thickness when galvanized bolts and nuts are used.

The maximum hardness for AASHTO M 164 (ASTM A325) bolts shall be 33 HRC.

Proof-load tests (ASTM F606, Method 1) shall be required for the bolts. Wedge tests of full-size bolts are required in accordance with Section 8.3 of AASHTO M 164 (ASTM A325). Galvanized bolts shall be wedge tested after galvanizing. Proof-load tests of AASHTO M 291 (ASTM A563) are required for the nuts. The proof-load tests for nuts to be used with galvanized bolts shall be performed after galvanizing, overtapping, and lubricating.

Except as noted below:

- Nuts for AASHTO M 164 (ASTM A325) bolts shall conform to AASHTO M 291 (ASTM A563), Grades DH, DH3, C, C3, and D (Property Class 8S, 8S3, 10S, or 10S3).

- Nuts for AASHTO M 253 (ASTM A490) bolts shall conform to the requirements of AASHTO M 291 (ASTM A563), Grades DH and DH3 (Property Class 10S or 10S3).

The exceptions are:

- Nuts to be galvanized (hot-dip or mechanically galvanized) shall be Grade DH (Property Class 10S).

C11.3.2.1

Type 2 bolts have been withdrawn from AASHTO M 164 (ASTM A325) and AASHTO M 253 (ASTM A490) and, therefore, are no longer manufactured. However, Type 2 bolts manufactured before this discontinuation may still be in inventory and are considered acceptable.

Grade DH (Property Class 10S) nuts are recommended for all Type 1 and Type 2 bolts. Grade DH3 (Property Class 10S3) nuts are not recommended for Type 1 and Type 2 bolts. Grade DH3 (Property Class 10S3) nuts must be used for Type 3 bolts.

- Nuts to be used with AASHTO M 164 (ASTM A325) Type 3 bolts shall be Grade C3 or DH3 (Property Class 8S3 or 10S3). Nuts to be used with AASHTO M 253 (ASTM A490), Type 3 bolts shall be Grade DH3 (Property Class 10S3).

All galvanized nuts shall be lubricated with a lubricant containing a visible dye. Black bolts must be oily to touch when delivered and installed.

Washers shall be hardened steel washers conforming to the requirements of AASHTO M 293 (ASTM F436) and Article 11.5.6.4.3, "Requirements for Washers."

11.3.2.2—Identifying Marks 2014 Revision

AASHTO M 164 (ASTM A325) for bolts and the specifications referenced therein for nuts require that bolts and nuts manufactured to the specification be identified by specific markings on the top of the bolt head and on one face of the nut. Head markings must identify the grade by the symbol "A325", the Manufacturer, and the type, if Type 3. Nut markings must identify the property class, the Manufacturer, and, if Type 3, the type. Markings on direct tension indicators (DTI, ASTM F959) must identify the Manufacturer and Type "325" (Class "8.8"). Other washer markings must identify the Manufacturer, and, if Type 3, the type.

AASHTO M 253 (ASTM A490) for bolts and the specifications referenced therein for nuts require that bolts and nuts manufactured to the specifications be identified by specific markings on the top of the bolt head and on one face of the nut. Head markings must identify the grade by the symbol "A490", the Manufacturer, and the type, if Type 3. Nut markings must identify the property class, the Manufacturer and if Type 3, the type. Markings on direct tension indicators must identify the Manufacturer and Type "490" (Class "10.9"). Other washer markings must identify the Manufacturer, and, if Type 3, the type.

11.3.2.3—Dimensions

Bolt and nut dimensions shall conform to the requirements for heavy hexagon structural bolts and heavy semi-finished hexagon nuts given in ANSI Standards B18.2.1 and B18.2.2 (B18.2.3.7M and B18.2.4.6M), respectively.

11.3.2.4—Galvanized High-Strength Fasteners

AASHTO M 253 (ASTM A490) bolts shall not be galvanized.

When fasteners are galvanized, they shall be specified to be hot-dip galvanized in accordance with AASHTO M 232M/M 232 (ASTM A153/A153M), Class C or mechanically galvanized in accordance with AASHTO M 298 (ASTM B695), Class 50 (Class 345). Bolts to be galvanized shall be either AASHTO M 164 (ASTM A325) Type 1. Galvanized bolts shall be tension tested after

C11.3.2.4 2014 Revision

AASHTO M 164 (ASTM A325) Type 2 bolts shall be mechanically galvanized only.

galvanizing. Washers, nuts, and bolts of any assembly shall be galvanized by the same process. The nuts should be overtapped to the minimum amount required for the fastener assembly and shall be lubricated with a lubricant containing a visible dye so a visual check can be made for the lubricant at the time of field installation.

11.3.2.5—Alternative Fasteners 2014 Revision

Other fasteners or fastener assemblies, such as those conforming to the requirements of ASTM F1852, which meet the materials, manufacturing, and chemical composition requirements of AASHTO M 164 (ASTM A325) or AASHTO M 253 (ASTM A490) which meet the mechanical property requirements of the same specification in full-size tests, and which have body diameter and bearing areas under the head and nut, or their equivalent, not less than those provided by a bolt and nut of the same nominal dimensions prescribed in Article 11.3.2.3, may be used, subject to the approval of the Engineer. Such alternate fasteners may differ in other dimensions from those of the specified bolts and nuts.

Subject to the approval of the Engineer, high-strength steel lock-pin and collar fasteners may be used as an alternate for high-strength bolts as shown in the contract documents. The shank and head of high-strength steel lock-pin and collar fasteners shall meet the requirements of Article 11.3.2.3. Each fastener shall provide a solid-shank body of sufficient diameter to provide tensile and shear strength equivalent to or greater than that of the bolt specified in the contract documents and shall have a cold-forged head on one end, of type and dimensions as approved by the Engineer; a shank length suitable for material thickness fastened; locking grooves; breakneck groove; and pull grooves (all annular grooves) on the opposite end. Each fastener shall provide a steel locking collar of proper size for shank diameter used which, by means of suitable installation tools, is cold-swaged into the locking grooves forming head for the grooved end of the fastener after the pull groove section has been removed. The steel locking collar shall be a standard product of an established Manufacturer of lock-pin and collar fasteners, as approved by the Engineer.

11.3.2.6—Load-Indicator Devices 2010 Revision 2011 Revision 2014 Revision

Load-indicating devices may be used in conjunction with bolts, nuts and washers specified in Article 11.3.2.1. Load-indicating devices shall conform to the requirements of ASTM Specification for Compressible-Washer Type Direct Tension Indicators for Use with Structural Fasteners, ASTM F959, except as provided in the following paragraph.

Subject to the approval of the Engineer, alternative design direct tension indicating devices may be used provided they satisfy the requirements of Article 11.5.6.4.6 or other requirements detailed in specifications provided by the Manufacturer and subject to the approval of the Engineer.

11.3.3—Welded Stud Shear Connectors

11.3.3.1—Materials

Stud shear connectors shall conform to the requirements of Cold-Finished Carbon Steel Bars and Shafting, AASHTO M 169 (ASTM A108), cold-drawn bars, Grades 1015, 1018, or 1020, either semi- or fully-killed. If flux retaining caps are used, the steel for the caps shall be of a low-carbon grade suitable for welding and shall comply with Cold-Rolled Carbon Steel Strip, ASTM A109/A109M.

Tensile properties, as determined by tests of bar stock after drawing or of finished studs, shall conform to the requirements in Table 11.3.3.1-1 in which the yield strength is as determined by a 0.2-percent offset method.

Table 11.3.3.1-1—Tensile Properties of Stud Shear Connectors

Tensile Strength	60 ksi
Yield Strength	50 ksi
Elongation	20% in 2.0 in.
Reduction of Area	50%

11.3.3.2—Test Methods

Tensile properties shall be determined in accordance with the applicable sections of AASHTO T 244 (ASTM A370), Mechanical Testing of Steel Products. Tensile tests of finished studs shall be made on studs welded to test plates using a test fixture similar to that shown in Figure 7.2 of the current AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*. If fracture occurs outside of the middle half of the gage length, the test shall be repeated.

11.3.3.3—Finish

Finished studs shall be of uniform quality and condition, free from injurious laps, fins, seams, cracks, twists, bends, or other injurious defects. Finish shall be as produced by cold-drawing, cold-rolling, or machining.

11.3.3.4—Certification

The Manufacturer shall certify that the studs as delivered are in accordance with the material requirements of this section. Certified copies of in-plant quality-control test reports shall be furnished to the Engineer upon request.

11.3.3.5—Check Samples

The Engineer may select, at the Contractor's expense, studs of each type and size used under the contract documents as necessary for checking the requirements of this Section.

11.3.4—Steel Forgings and Steel Shafting

11.3.4.1—Steel Forgings

Steel forgings shall conform to the Specifications for Steel Forgings Carbon and Alloy for General Use, AASHTO M 102M/M 102 (ASTM A668/A668M), Class C, D, F, or G.

11.3.4.2—Cold-Finished Carbon Steel Shafting

Cold-finished carbon steel shafting shall conform to the specifications for Cold-Finished Carbon Steel Bars Standard Quality, AASHTO M 169 (ASTM A108). Grades 10160 through 10300, inclusive, shall be furnished unless otherwise specified in the contract documents.

11.3.5—Steel Castings

11.3.5.1—Mild Steel Castings

Steel castings for use in highway bridge components shall conform to Standard Specifications for Steel Castings for Highway Bridges, ASTM A781/A781M, Class 70 (Class 485), or Standard Specifications for Steel Castings, Carbon, for General Application, AASHTO M 103M/M 103 (ASTM A27/A27M), Class 70 or Grade 70-36 (Class 485 or Grade 485-250), unless otherwise specified.

11.3.5.2—Chromium Alloy-Steel Castings

Chromium alloy-steel castings shall conform to the Specification for Corrosion-Resistant Iron-Chromium, Iron-Chromium-Nickel, and Nickel-Based Alloy Castings for General Application, AASHTO M 163M/M 163 (ASTM A743/A743M). Grade CA 15 (Grade CA 15M) shall be furnished unless otherwise specified.

11.3.6—Iron Castings

11.3.6.1—Materials

- Gray Iron Castings—Gray iron castings shall conform to the Specification for Gray Iron Castings, AASHTO M 105 or ASTM A48/A48M, Class 30, unless otherwise specified in the contract documents.

- Ductile Iron Castings—Ductile iron castings shall conform to the Specifications for Ductile Iron Castings, ASTM A536, Grade 60-40-18 (Grade 414-276-18), unless otherwise specified in the contract documents. In addition to the specified test coupons, test specimens from parts integral with the castings, such as risers, shall be tested for castings with a weight (mass) more than 1.0 kip to determine that the required quality is obtained in the castings in the finished condition.

- Malleable Castings—Malleable castings shall conform to the Specification for Ferritic Malleable Iron Castings, ASTM A47/A47M. Grade 35018 (Grade 24118) shall be furnished unless otherwise specified in the contract documents.

11.3.6.2—Work Quality and Finish

Iron castings shall be true to pattern in form and dimensions, free from pouring faults, sponginess, cracks, blow holes, and other defects in positions affecting their strength and value for the service intended.

Castings shall be boldly filleted at angles and the arises shall be sharp and perfect.

11.3.6.3—Cleaning

All castings must be sandblasted or otherwise effectively cleaned of scale and sand so as to present a smooth, clean, and uniform surface.

11.3.7—Galvanizing

When galvanizing is specified in the contract documents, ferrous metal products, other than fasteners and hardware items, shall be galvanized in accordance with the Specifications for Zinc (Hot-Galvanized) Coatings on Products Fabricated from Rolled, Pressed, and Forged Steel Shape Plates, Bars, and Strip, AASHTO M 111M/M 111 (ASTM A123/A123M). Fasteners and hardware items shall be galvanized in accordance with the Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware, AASHTO M 232M/M 232 (ASTM A153/A153M), except as noted in Article 11.3.2.4, "Galvanized High-Strength Fasteners."

11.4—FABRICATION

11.4.1—Identification of Steels During Fabrication

The Contractor's system of assembly-marking individual pieces, and the issuance of cutting instructions to the shop shall be such as to maintain identity of the original piece.

The Contractor may furnish material that can be identified by heat number and mill test report from stock.

During fabrication, up to the point of assembling members, each piece of steel, other than Grade 36 (Grade 250) steel, shall show clearly and legibly its specification.

Any piece of steel, other than Grade 36 (Grade 250) steel, which will be subject to fabricating operations such as blast cleaning, galvanizing, heating for forming, or painting which might obliterate marking prior to assembling into members, shall be marked for grade by steel die stamping or by a substantial tag firmly attached. Steel die stamps shall be low stress-type.

C11.4.1 2014 Revision

Assembly-marking individual pieces and the issuance of cutting instructions to the shop are generally done by cross-referencing of the assembly marks shown on the shop drawings with the corresponding item covered on the mill purchase order.

Color codes for steels, as noted in previous versions of the AASHTO M 160M/M 160 (ASTM A6/A6M) Specifications, may also be used for identification purposes. This method, which will no longer be shown in either of these Specifications, is also being eliminated by many Owners due to the complexity of the code with the many new material grades.

Upon request by the Engineer, the Contractor shall furnish an affidavit certifying that throughout the fabrication operation the identification of steel has been maintained in accordance with this specification.

11.4.2—Storage of Materials

Structural material, either plain or fabricated, shall be stored above the ground on platforms, skids, or other supports. It shall be kept free from dirt, grease, and other foreign matter, and shall be protected as far as practicable from corrosion. Storage of high-strength fasteners shall conform to Article 11.5.6.4, "Installation."

11.4.3—Plates

11.4.3.1—Direction of Rolling

2015 Revision

Unless otherwise specified in the contract documents, steel plates for main members and splice plates for flanges and main tension members, not secondary members, shall be cut and fabricated so that the primary direction of rolling is parallel to the direction of the main tensile and/or compressive stresses.

11.4.3.2—Plate-Cut Edges

11.4.3.2.1—Edge Planing

Sheared edges of plate more than 0.625 in. in thickness and carrying calculated stress shall be planed, milled, ground, or thermal-cut to a depth of 0.25 in.

11.4.3.2.2—Oxygen Cutting

Oxygen cutting of structural steel shall conform to the requirements of the current AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code.*

11.4.3.2.3—Visual Inspection and Repair of Plate-Cut Edges

Visual inspection and repair of plate-cut edges shall be in accordance with the current AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code.*

11.4.3.3—Bent Plates

11.4.3.3.1—General

2012 Revision 2014 Revision

Cold-bending of fracture-critical steels and fracture-critical members is prohibited. Perform cold-bending of other steels or members, in accordance with the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code* and Table 11.4.3.3.2-1 and in a manner such that no cracking occurs.

11.4.3.3.2—Cold-Bending 2012 Revision

Unless otherwise approved, the minimum bend radii for cold-forming (at room temperature), measured to the concave face of the plate, are given in Table 11.4.3.3.2-1. If a smaller radius is required, heat may need to be applied as a part of the bending procedure. Provide the heating procedure for review by the Engineer. For grades not included in Table 11.4.3.3.2-1, follow minimum bend radii recommendations of the plate Producer.

If possible, orient bend lines perpendicular to the direction of final rolling of the plate. If the bend line is parallel to the direction of final rolling, multiply the suggested minimum radii in Table 11.4.3.3.2-1 by 1.5.

C11.4.3.3.2

For bent plates, the bend radius and the radius of the male die should be as liberal as the finished part will permit. The width across the shoulders of the female die should be at least eight times the plate thickness for Grade 36 (Grade 250) steel. Higher-strength steels require larger die openings. The surface of the dies in the area of radius should be smooth.

Where the concave face of a bent plate must fit tightly against another surface, the male die should be sufficiently thick and have the proper radius to ensure that the bent plate has the required concave surface.

Since cracks in cold-bending commonly originate from the outside edges, shear burrs and gas-cut edges should be removed by grinding. Sharp corners on edges and on punched or gas-cut holes should be removed by chamfering or grinding to a radius.

Table 11.4.3.3.2-1—Minimum Cold-Bending Radii

	Thickness, in. (*t*)			
AASHTO M 270M/M 270 (ASTM A709/A709M) Grades, ksi	Up to 0.75	Over 0.75 to 1.0, incl.	Over 1.0 to 2.0, incl.	Over 2.0
36	1.5*t*	1.5*t*	1.5*t*	2.0*t*
50, 50S, 50W, or HPS 50W	1.5*t*	1.5*t*	2.0*t*	2.5*t*
HPS 70W	1.5*t*	1.5*t*	2.5*t*	3.0*t*
HPS 100W	1.75*t*	2.25*t*	4.5*t*	5.5*t*

11.4.3.3.3—Hot-Bending 2012 Revision

If a radius shorter than the minimum specified for cold-bending is essential, the plates shall be bent hot at a temperature not greater than 1200°F, except for AASHTO M 270M/M 270 (ASTM A709/A709M), Grades HPS 70W and HPS 100W (Grades HPS 485W and HPS 690W) for which plates shall be bent hot at a temperature not greater than 1100°F.

C11.4.3.3.3

The temperature limitation for hot bending, along with the subsequent temperature limitations for heat straightening and heat curving, of quenched and tempered steel plate are based on the traditional practice of limiting such heating to 50°F below the minimum required tempering temperature so as not to adversely affect the strength of the material. In the absence of other data, the temperature limitation for AASHTO M 270M/M 270 (ASTM A709/A709M) Grade HPS 70W (Grade HPS 485W) would be specified as 1050°F since the minimum tempering temperature is 1100°F; however, Grade HPS 70W (Grade HPS 485W) steels have been tested by the New York State Thruway Authority (NYSTA) at short-time heating temperatures as high as 1250°F, High Steel Structures (FHWA, 1999), with minimal effect on the strength properties of this grade. Therefore, the limitation of 1100°F was selected for Grade HPS 70W (Grade HPS 485W) to be consistent with the limitation of 1100°F specified for AASHTO M 270M/M 270 (ASTM A709/A709M) Grade HPS 100W (Grade HPS 690W), which is based on the traditional 50°F limit below its minimum tempering temperature of 1150°F.

11.4.4—Fit of Stiffeners

End bearing stiffeners for girders and stiffeners intended as supports for concentrated loads shall have full bearing (either milled, ground, or on weldable steel in compression areas of flanges, welded as specified in the contract documents) on the flanges to which they transmit load or from which they receive load. Intermediate stiffeners not intended to support concentrated loads, unless specified in the contract documents otherwise, shall have a tight fit against the compression flange.

11.4.5—Abutting Joints

2014 Revision

Abutting ends in compression members of trusses and columns shall be milled or saw-cut to give a square joint and uniform bearing. At other joints, not required to be faced, the opening shall not exceed 0.375 in.

11.4.6—Facing of Bearing Surfaces

The surface finish of bearing, base plates, and other bearing surfaces that are to come in contact with each other or with concrete shall meet the ANSI surface roughness requirements as defined in ANSI B46.1, Surface Roughness, Waviness, and Lay, Part I:

Steel slabs	ANSI 2000 μin. (50 μm) (RMS)
Heavy plates in contact in shoes to be welded	ANSI 1000 μin. (25 μm) (RMS)
Milled ends of compression members, milled or ground ends of stiffeners and fillers	ANSI 500 μin. (12.5 μm) (RMS)
Bridge rollers and rockers	ANSI 250 μin. (6.3 μm) (RMS)
Pins and pin holes	ANSI 125 μin. (3.2 μm) (RMS)
Sliding bearings	ANSI 125 μin. (3.2 μm) (RMS)

11.4.7—Straightening Material

The straightening of plates, angles, other shapes, and built-up members, when permitted by the Engineer, shall be done by methods that will not produce fracture or other injury to the metal. Distorted members shall be straightened by mechanical means or, if approved by the Engineer, by carefully planned procedures and supervised application of a limited amount of localized heat, except that heat-straightening of AASHTO M 270M/M 270 (ASTM A709/A709M) Grades HPS 70W and HPS 100W (Grades HPS 485W and HPS 690W) steel members shall be done only under rigidly controlled procedures, each application subject to the approval of the Engineer. In no case shall the maximum temperature exceed values in Table 11.4.7-1.

Table 11.4.7-1—Maximum Straightening Temperature

AASHTO M 270M/M 270 (ASTM A709/A709M) Grades	Temperature
HPS 70W	1100°F
HPS 100W	1100°F

In all other steels, the temperature of the heated area shall not exceed 1200°F as controlled by temperature indicating crayons, liquids, or bimetal thermometers. Heating in excess of the limits shown shall be cause for rejection, unless the Engineer allows testing to verify material integrity.

Parts to be heat-straightened shall be substantially free of stress and from external forces, except stresses resulting from mechanical means used in conjunction with the application of heat.

Evidence of fracture following straightening of a bend or buckle will be cause for rejection of the damaged piece.

11.4.8—Bolt Holes

11.4.8.1—Holes for High-Strength Bolts and Unfinished Bolts

11.4.8.1.1—General 2014 Revision

All holes for bolts shall be either punched or drilled, except as noted herein. The width of each standard hole shall be the nominal diameter of the bolt plus 0.0625 in. The standard hole diameter for metric bolts M24 and smaller shall be the nominal diameter of the bolt plus 2 mm. For metric bolts M27 and larger, the standard hole diameter shall be the nominal diameter of the bolt plus 3 mm.

Except as noted in the articles below, material forming parts of a member composed of not more than five thicknesses of metal may be punched full-size.

When more than five thicknesses of material are joined or, as required by Article 11.4.8.5, material shall be subdrilled or subpunched and then reamed full-size, or drilled full-size while in assembly.

When required, all holes shall be either subpunched or subdrilled 0.1875 in. smaller and, after assembling, reamed or drilled to full size.

Holes in cross frames, lateral bracing components, and the corresponding holes in connection plates between girders and cross frames or lateral components may be punched full size. Holes in longitudinal main load-carrying members, transverse floorbeams, and any components designated as fracture critical (FCMs) shall not be punched full-size.

When shown in the contract documents, enlarged or slotted holes are allowed with high-strength bolts.

C11.4.8.1.1 2015 Revision 2012 Revision

Previous punching restrictions whenever the thickness of the material was not greater than 0.75 in. for structural steel, 0.625 in. for high strength steel, or 0.5 in. for quenched-and-tempered alloy steel, are upper limits but punching equipment may be more restrictive.

For other dimensional criteria assumed in the design of bolted details, e.g., oversize holes, slotted holes, edge distances, and end distances, see Article 6.13.2, "Bolted Connections," of the *AASHTO LRFD Bridge Design Specifications.*

With the owner's approval, round or slotted holes for non-main members in thin plate may be thermally cut by plasma, laser, or oxygen-acetylene methods subject to the requirements herein.

With the owner's approval, round or slotted holes for non-main members in thin plate may successfully be thermally cut by plasma, laser, or oxygen-acetylene means. The maximum surface roughness of ANSI 1000 μin. and the conical taper of the hole must be maintained within tolerance. See references AISC *Steel Construction Manual*, 13th Edition, Section M2.5; RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, Section 3.3; and NSBA *Steel Bridge Fabrication*, S2.1.

11.4.8.1.2—Punched Holes

If any holes must be enlarged to admit the bolts, such holes shall be reamed. Holes must be clean-cut without torn or ragged edges. The slightly conical hole that naturally results from punching operations shall be considered acceptable.

11.4.8.1.3—Reamed or Drilled Holes

Reamed or drilled holes shall be cylindrical, perpendicular to the member, and shall comply with the requirements of Article 11.4.8.1.1 as to size. Where practical, reamers shall be directed by mechanical means. Burrs on the outside surfaces shall be removed. Reaming and drilling shall be done with twist drills, twist reamers, or rotobroach cutters. Connecting parts requiring reamed or drilled holes shall be assembled and securely held while being reamed or drilled and shall be match-marked before disassembling.

11.4.8.1.4—Accuracy of Holes 2014 Revision

Holes not more than 0.03125 in. larger in diameter than the true decimal equivalent of the nominal diameter that may result from a drill or reamer of the nominal diameter shall be considered acceptable. The width of slotted holes which are produced by thermal cutting or a combination of drilling or punching and thermal cutting should be not more than 0.03125 in. greater than the nominal width. The thermally-cut surface shall be ground smooth to obtain a maximum surface roughness of ANSI 1000 μin.

11.4.8.2—Accuracy of Hole Group

11.4.8.2.1—Accuracy before Reaming 2014 Revision

All holes punched full-size, subpunched, or subdrilled shall be so accurately punched that after assembling (before any reaming is done) a cylindrical pin 0.125 in. smaller in diameter than the nominal size of the punched hole may be entered perpendicular to the face of the member, without drifting, in at least 75 percent of the contiguous holes in the same plane. If the requirement is not fulfilled, the badly punched pieces shall be rejected. If any hole will not pass a pin 0.1875 in. smaller in diameter than the nominal size of the punched hole, this shall be cause for rejection.

11.4.8.2.2—Accuracy after Reaming 2014 Revision

When holes are reamed or drilled, 85 percent of the holes in any contiguous group shall, after reaming or drilling, show no offset greater than 0.03125 in. between adjacent thicknesses of metal.

All steel templates shall have hardened steel bushings in holes accurately dimensioned from the centerlines of the connection as inscribed on the template. The centerlines shall be used in locating accurately the template from the milled or scribed ends of the members.

11.4.8.3—Numerically-Controlled Drilled Field Connections

In lieu of subsized holes and reaming while assembled, or drilling holes full-size while assembled, the Contractor shall have the option to drill or punch bolt holes full-size in unassembled pieces and/or connections including templates for use with matching subsized and reamed holes, by means of suitable numerically-controlled (N/C) drilling or punching equipment. Full-size punched holes shall meet the requirements of Article 11.4.8.1.

If N/C drilling or punching equipment is used, the Contractor shall be required to demonstrate the accuracy of this drilling or punching procedure in accordance with the provisions of Article 11.5.3.3, "Check Assembly—Numerically-Controlled Drilling," by means of check assemblies.

Holes drilled or punched by N/C equipment shall be drilled or punched to appropriate size either through individual pieces or drilled through any combination of pieces held tightly together.

11.4.8.4—Holes for Ribbed Bolts, Turned Bolts, or Other Approved Bearing-Type Bolts 2014 Revision

All holes for ribbed bolts, turned bolts, or other approved bearing-type bolts shall be subpunched or subdrilled 0.1875 in. smaller than the nominal diameter of the bolt and reamed when assembled, or drilled to a steel template or, after assembling, drilled from the solid at the option of the Fabricator. In any case, the finished holes shall provide a driving fit as specified in the contract documents.

11.4.8.5—Preparation of Field Connections

Holes in all field connections and field splices of main member of trusses, arches, continuous-beam spans, bents, towers (each face), plate girders, and rigid frames shall be subpunched or subdrilled and subsequently reamed while assembled or drilled full-size through a steel template while assembled. Holes in cross frames, lateral bracing components, and the corresponding holes in connection plates between girders and cross frames or lateral components may be punched full size. Holes in longitudinal main load-carrying members, transverse floorbeams, and any components designated as fracture critical (FCMs) shall not be punched full-size. Holes for field splices of rolled beam stringers continuous over floor beams or cross frames may be drilled full-size unassembled to a steel template. All holes for floorbeams or cross frames may be drilled full-size unassembled to a steel template, except that all holes for floor beam and stringer field end connections shall be subpunched and reamed while assembled or drilled full-size to a steel template. Reaming or drilling full-size of field-connection holes through a steel template shall be done after the template has been located with utmost care as to position and angle and firmly bolted in place. Templates used for reaming matching members or the opposite faces of a single member shall be exact duplicates. Templates used for connections on like parts or members shall be so accurately located that the parts or members are duplicates and require no match-marking.

For any connection, in lieu of subpunching and reaming or subdrilling and reaming, the Fabricator may, at the Fabricator's option, drill holes full-size with all thicknesses or material assembled in proper position.

11.4.9—Pins and Rollers

11.4.9.1—General

2010 Revision

Pins and rollers shall be accurately turned to the dimensions shown on the drawings and shall be straight, smooth, and free from flaws. Pins and rollers more than 9.0 in. in diameter shall be forged and annealed. Pins and rollers 9.0 in. or less in diameter may be either forged and annealed or cold-finished carbon-steel shafting.

In pins larger than 9.0 in. in diameter, a hole not less than 2.0 in. in diameter shall be bored full-length along the axis after the forging has been allowed to cool to a temperature below the critical range, under suitable conditions to prevent injury by too rapid cooling, and before being annealed.

11.4.9.2—Boring Pin Holes

2014 Revision

Pin holes shall be bored true to the specified diameter, smooth and straight, at right angles with the axis of the member and parallel with each other unless otherwise required. The final surface shall be produced by a finishing cut.

The diameter of the pin hole shall not exceed that of the pin by more than 0.02 in. for pins 5.0 in. or less in diameter, or by 0.03125 in. for larger pins.

The distance outside to outside of end holes in tension members and inside to inside of end holes in compression members shall not vary from that specified more than 0.03125 in. Boring of pin holes in built-up members shall be done after the member has been assembled.

11.4.9.3—Threads for Bolts and Pins

Threads for all bolts and pins for structural steel construction shall conform to the United Standard Series UNC ANSI B1.1, Class 2A for external threads and Class 2B for internal threads, except that pin ends having a diameter of 1.375 in. or more shall be threaded six threads to the inch (metric screw threads—M Profile ANSI B1.13M with a tolerance Class 6G for external threads and 6H for internal threads).

11.4.10—Eyebars

Pin holes may be flame-cut at least 2.0 in. smaller in diameter than the finished pin diameter. All eyebars that are to be placed side by side in the structure shall be securely fastened together in the order that they will be placed on the pin and bored at both ends while so clamped. Eyebars shall be packed and match-marked for shipment and erection. All identifying marks shall be stamped with steel stencils on the edge of one head of each member after fabrication is completed so as to be visible when the bars are nested in place on the structure. Steel die stamps shall be low stress-type. No welding is allowed on eyebars or to secure adjacent eyebars.

The eyebars shall be straight and free from twists and the pin holes shall be accurately located on the centerline of the bar. The inclination of any bar to the plane of the truss shall not exceed a slope of 0.5 percent. The edges of eyebars that lie between the transverse centerline of their pin holes shall be cut simultaneously with two mechanically operated torches abreast of each other, guided by a substantial template, in such a manner as to prevent distortion of the plates.

11.4.11—Annealing and Stress Relieving

Structural members which are indicated in the contract documents to be annealed or normalized shall have finished machining, boring, and straightening done subsequent to heat treatment. Normalizing and annealing (full annealing) shall be as specified in ASTM A941. The temperatures shall be maintained uniformly throughout the furnace during the heating and cooling so that the temperature at no two points on the member will differ by more than 100°F at any one time.

Members of AASHTO M 270M/M 270 (ASTM A709/A709M) Grades HPS 70W and HPS 100W (Grades HPS 485W and HPS 690W) steels shall not be annealed or normalized and shall be stress relieved only with the approval of the Engineer after consultation with the material producers.

A record of each furnace charge shall identify the pieces in the charge and show the temperatures and schedule actually used. Proper instruments, including recording pyrometers, shall be provided for determining at any time the temperatures of members in the furnace. The records of the treatment operation shall be available to and meet the approval of the Engineer. The holding temperature for stress relieving shall be in accordance with Section 4.4 of the current AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*.

Members, such as bridge shoes, pedestals, or other parts that are built up by welding sections of plate together shall be stress relieved in accordance with the procedure of Section 4.4 of the current AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*, when required by the contract documents.

11.4.12—Curved Girders

11.4.12.1—General 2014 Revision

Flanges of curved, welded girders may be cut to the radii specified in the contract documents or curved by applying heat as specified in the succeeding articles providing the radii is not less than allowed by Article 10.15.2, "Minimum Radius of Curvature," of the AASHTO *Standard Specifications for Highway Bridges*, 17th Edition, Design Specifications.

11.4.12.2—Heat-Curving Rolled Beams and Welded Girders

11.4.12.2.1—Materials

Structural steels conforming to AASHTO M 270M/M 270 (ASTM A709/A709M), Grade 36, 50, 50S, 50W, HPS 50W, HPS 70W, or HPS 100W (Grade 250, 345, 345S, 345W, HPS 345W, HPS 485W, or HPS 690W) may be heat-curved.

C11.4.12.2.1

See Article C11.4.3.3.3.

11.4.12.2.2—Type of Heating 2014 Revision

Beams and girders may be curved by either continuous or V-type heating as approved by the Engineer. For the continuous method, a strip or intermittent strips along the edge of the top and bottom flange shall be heated approximately simultaneously depending on flange widths and thicknesses; the strip shall be of sufficient width and temperature to obtain the required curvature. For the V-type heating, the top and bottom flanges shall be heated in truncated triangular or wedge-shaped areas having their base along the flange edge and spaced at regular intervals along each flange; the spacing and temperature shall be as required to obtain the required curvature and heating shall progress along the top and bottom flange at approximately the same rate.

For the V-type heating, the apex of the truncated triangular area applied to the inside flange surface shall terminate just before the juncture of the web and the flange is reached. To avoid unnecessary web distortion, special care shall be taken when heating the inside flange surfaces (the surfaces that intersect the web) so that heat is not applied directly to the web. When the radius of curvature is 1000 ft or more, the apex of the truncated triangular heating pattern applied to the outside flange surface shall extend to the juncture of the flange and web. When the radius of curvature is less than 1000 ft, the apex of the

C11.4.12.2.2

Additional references on heat-curved girders include:

- ASCE. 1970. "Experimental Stresses and Strains from Heat Curving," *Journal of the Structural Division*, Volume 96, No. ST7, American Society of Civil Engineers, New York.
- ASCE. 1970. "Theoretical Stresses and Strains from Heat Curving," *Journal of the Structural Division*, Volume 96, No. ST7, American Society of Civil Engineers, New York.
- ASCE. 1970. "Criteria for Heat Curving Steel Beams and Girders," *Journal of the Structural Division*, Volume 96, No. ST7, American Society of Civil Engineers, New York.
- U.S. Steel. 2001. *Fabrication Aids for Continuously Heat-Curved Girders*, ADUSS 88-5538-01, United States Steel Corporation, Pittsburgh.
- U.S. Steel. 2002. *Fabrication Aids for Girders Curved with V-Heats*, ADUSS 88-5539-02, United States Steel Corporation, Pittsburgh.

truncated triangular heating pattern applied to the outside flange surface shall extend past the web for a distance equal to one-eighth of the flange width or 3.0 in., whichever is less. The truncated triangular pattern shall have an included angle of approximately 15 to 30 degrees but the base of the triangle shall not exceed 10.0 in. Variations in the patterns prescribed above may be made with the approval of the Engineer.

For both types of heating, the flange edges to be heated are those that will be on the inside of the horizontal curve after cooling. Heating both inside and outside flange surfaces is only mandatory when the flange thickness is 1.25 in. or greater, in which case, the two surfaces shall be heated concurrently. The maximum temperature shall be prescribed as follows.

11.4.12.2.3—Temperature

The heat-curving operation shall be conducted in such a manner that the temperature of the steel does not exceed 1200°F for Grades 36, 50, 50S, 50W, and HPS 50W (Grades 250, 345, 345S, 345W, and HPS 345W); and 1100°F for Grades HPS 70W and HPS 100W (Grades HPS 485W and HPS 690W) as measured by temperature-indicating crayons or other suitable means. The girder shall not be artificially cooled until after naturally cooling to 600°F. The method of artificial cooling shall be subject to the approval of the Engineer.

11.4.12.2.4—Position for Heating

The girder may be heat-curved with the web in either a vertical or a horizontal position. When curved in the vertical position, the girder shall be braced or supported in such a manner that the tendency of the girder to deflect laterally during the heat-curving process will not cause the girder to overturn.

When curved in the horizontal position, the girder shall be supported near its ends and at intermediate points, if required, to obtain a uniform curvature; the bending stress in the flanges due to the dead load of the girder and externally applied loads shall not exceed the usual allowable design stress. When the girder is positioned horizontally for heating, intermediate safety catch blocks shall be maintained at the mid-length of the girder within 2.0 in. of the flanges at all times during the heating process to guard against a sudden sag due to plastic flange buckling.

11.4.12.2.5—Sequence of Operations

The girder shall be heat-curved in the fabrication shop before it is painted. The heat-curving operation may be conducted either before or after all the required welding of transverse intermediate stiffeners is completed. However, unless provisions are made for girder shrinkage, connection plates and bearing stiffeners shall be located and attached after heat-curving. If longitudinal stiffeners are required, they shall be heat-curved or oxygen-cut separately and then welded to the curved girder. When cover plates are to be attached to rolled beams, they may be attached before heat-curving if the total thickness of one flange and cover plate is less than 2.5 in. and the radius of curvature is greater than 1000 ft. For other rolled beams with cover plates, the beams shall be heat-curved before the cover plates are attached; cover plates shall be either heat-curved or oxygen-cut separately and then welded to the curved beam.

11.4.12.2.6—Camber 2010 Revision

Girders shall be cambered before heat-curving. Camber for rolled beams may be obtained by heat-cambering methods approved by the Engineer. For plate girders, the web shall be cut to the prescribed camber with suitable allowance for shrinkage due to cutting, welding, and heat-curving. However, subject to the approval of the Engineer, moderate deviations from specified camber may be corrected by a carefully supervised application of heat.

11.4.12.2.7—Measurement of Curvature and Camber

Horizontal curvature and vertical camber shall be measured for final acceptance after all welding and heating operations are completed and the flanges have cooled to a uniform temperature. Horizontal curvature shall be checked with the girder in the vertical position.

11.4.13—Orthotropic-Deck Superstructures

11.4.13.1—General

Dimensional tolerance limits for orthotropic-deck bridge members shall be applied to each completed but unloaded member and shall be as specified in paragraph 3.5 of the current AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*, except as follows:

- The deviation from detailed flatness, straightness, or curvature at any point shall be the perpendicular distance from that point to a template edge which has the detailed straightness or curvature and which is in contact with the element at two other points.

- The template edge may have any length not exceeding the greatest dimension of the element being examined and, for any panel, not exceeding 1.5 times the least dimension of the panel; it may be placed anywhere within the boundaries of the element.

C11.4.13.1

The term element as used herein refers to individual panels, stiffeners, flanges, or other pieces.

- The deviation shall be measured between adjacent points of contact of the template edge with the element; the distance between these adjacent points of contact shall be used in the formulas to establish the tolerance limits for the segment being measured whenever this distance is less than the applicable dimension of the element specified for the formula.

11.4.13.2—Flatness of Panels

2014 Revision

The maximum deviation, δ, from detailed flatness or curvature of a panel shall not exceed the greater of 0.1875 in. or:

$$\delta \leq \frac{D}{144\sqrt{T}} \qquad (11.4.13.2\text{-}1)$$

where:

D = the least dimension along the boundary of the panel, in.

T = the minimum thickness of the plate comprising the panel, in.

C11.4.13.2

The term “panel” as used in this Article means a clear area of steel plate surface bounded by stiffeners, webs, flanges, or plate edges and not further subdivided by any such elements. The provisions of this Article apply to all panels in the bridge; for plates stiffened on one side only such as orthotropic-deck plates or flanges of box girders, this includes the total clear width on the side without stiffeners as well as the panels between stiffeners on the side with stiffeners.

11.4.13.3—Straightness of Longitudinal Stiffeners Subject to Calculated Compressive Stress, Including Orthotropic-Deck Ribs

The maximum deviation, δ, from detailed straightness or curvature in any direction perpendicular to its length of a longitudinal web stiffener or other stiffener subject to calculated compressive stress shall not exceed:

$$\delta \leq \frac{L}{480} \qquad (11.4.13.3\text{-}1)$$

where:

L = the length of the stiffener or rib between cross members, webs, or flanges, in.

11.4.13.4—Straightness of Transverse Web Stiffeners and Other Stiffeners Not Subject to Calculated Compressive Stress

The maximum deviation, δ, from detailed straightness or curvature in any direction perpendicular to its length of a transverse web stiffener or other stiffener not subject to calculated compressive stress shall not exceed:

$$\delta \leq \frac{L}{240} \qquad (11.4.13.4\text{-}1)$$

where:

L = the length of the stiffener between cross members, webs, or flanges, in.

11.4.14—Full-Size Tests

When full-size tests of fabricated structural members or eyebars are required in the contract documents, the Contractor shall provide suitable facilities, material, supervision, and labor necessary for making and recording the required tests. The members tested in accordance with the contract documents shall be paid for in accordance with Article 11.7.2, "Basis of Payment."

11.4.15—Marking and Shipping

Each member shall be painted or marked with an erection mark for identification and an erection diagram showing these marks shall be furnished to the Engineer.

The Contractor shall furnish to the Engineer as many copies of material orders, shipping statements, and erection diagrams as the Engineer may direct. The weight (mass) of the individual members shall be shown on the statements. Members having a weight (mass) of more than 3.0 tons shall have the weight (mass) marked thereon. Structural members shall be loaded on trucks or cars in such a manner that they may be transported and unloaded at their destination without being damaged.

Bolts, nuts, and washers (where required) from each rotational-capacity lot shall be shipped in the same container. If there is only one production lot number for each size of nut and washer, the nuts and washers may be shipped in separate containers. Pins; small parts; and packages of bolts, washers, and nuts shall be shipped in boxes, crates, kegs, or barrels but the gross weight (mass) of any package shall not exceed 300 lb. A list and description of the contained materials shall be plainly marked on the outside of each shipping container.

11.5—ASSEMBLY

11.5.1—Bolting

Surfaces of metal in contact shall be cleaned before assembling. The parts of a member shall be assembled, well pinned, and firmly drawn together before drilling, reaming, or bolting is commenced. Assembled pieces shall be taken apart, if necessary, for the removal of burrs and shavings produced by the operation. The member shall be free from twists, bends, and other deformation.

The drifting done during assembling shall be only such as to bring the parts into position and not sufficient to enlarge the holes or distort the metal.

11.5.2—Welded Connections

Surfaces and edges to be welded shall be smooth, uniform, clean, and free of defects which would adversely affect the quality of the weld. Edge preparation shall be done in accordance with the current AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*.

11.5.3—Preassembly of Field Connections

11.5.3.1—General

Field connections of main members of trusses, arches, continuous beams, plate girders, bents, towers, and rigid frames shall be preassembled prior to erection as necessary to verify the geometry of the completed structure or unit and to verify or prepare field splices. Attaining accurate geometry is the responsibility of the Contractor and the Contractor shall propose an appropriate method of preassembly for approval by the Engineer. The method and details of preassembly shall be consistent with the erection procedure shown on the erection plans and camber diagrams prepared by the Contractor and approved by the Engineer. As a minimum, the preassembly procedure shall consist of assembling three contiguous panels accurately adjusted for line and camber. Successive assemblies shall consist of at least one section or panel of the previous assembly (repositioned if necessary and adequately pinned to assure accurate alignment) plus two or more sections or panels added at the advancing end. In the case of structures longer than 150.0 ft, each assembly shall be not less than 150.0 ft long regardless of the length of individual continuous panels or sections. At the option of the Fabricator, sequence of assembly may start from any location in the structure and proceed in one or both directions so long as the preceding requirements are satisfied.

11.5.3.2—Bolted Connections

For bolted connections, holes shall be prepared as outlined in Article 11.4.8, "Bolt Holes." Where applicable, major components shall be assembled with milled ends of compression members in full bearing and then shall have their subsized holes reamed to the specified size while the connections are assembled.

11.5.3.3—Check Assembly—Numerically-Controlled Drilling

When the Contractor elects to use numerically-controlled drilling, a check assembly shall be required for each major structural type of each project, unless otherwise designated in the contract documents, and shall consist of at least three contiguous shop sections or, in a truss, all members in at least three contiguous panels but not less than the number of panels associated with three contiguous chord lengths (i.e., length between field splices).

Check assemblies should be based on the proposed order of erection, joints in bearings, special complex points, and similar considerations.

C11.5.3.3

Special complex points could be the portals of skewed trusses, for example.

If the check assembly fails in some specific manner to demonstrate that the required accuracy is being obtained, further check assemblies may be required by the Engineer for which there shall be no additional cost to the Owner.

Each assembly, including camber, alignment, accuracy of holes, and fit of milled joints, shall be approved by the Engineer before reaming is commenced or before an N/C drilled check assembly is dismantled.

The check assemblies shall preferably be the first sections of each major structural type to be fabricated.

Shop assemblies other than the check assemblies will not be required.

11.5.3.4—Field-Welded Connections

For field-welded connections the fit of members including the proper space between abutting flanges shall be prepared or verified with the segment preassembled in accordance with Article 11.5.3.1.

11.5.4—Match-Marking

Connecting parts preassembled in the shop to assure proper fit in the field shall be match-marked, and a diagram showing such marks shall be furnished to the Engineer.

11.5.5—Connections Using Unfinished, Turned, or Ribbed Bolts

11.5.5.1—General

When unfinished bolts are specified, the bolts shall be unfinished, turned, or ribbed bolts conforming to the requirements for Grade A bolts of standard specification for carbon steel bolts and studs, ASTM A307, 60.0-ksi tensile strength (carbon and alloy-steel externally threaded metric fasteners, ASTM F568M, Property Class 4.6, 400-MPa tensile strength). Bolts shall have single self-locking nuts or double nuts unless otherwise specified in the contract documents. Beveled washers shall be used where bearing faces have a slope of more than 1:20 with respect to a plane normal to the bolt axis. The specifications of this Article do not pertain to the use of high-strength bolts. Bolted connections fabricated with high-strength bolts shall conform to Article 11.5.6, "Connections Using High-Strength Bolts."

11.5.5.2—Turned Bolts

The surface of the body of turned bolts shall meet the ANSI B46.1 roughness rating value of 125 μin. Heads and nuts shall be hexagonal with standard dimensions for bolts of the nominal size specified or the next larger nominal size. Diameter of threads shall be equal to the body of the bolt or the nominal diameter of the bolt specified. Holes for turned bolts shall be carefully reamed with bolts furnished to provide for a light driving fit. Threads shall be entirely outside of the holes. A washer shall be provided under the nut.

11.5.5.3—Ribbed Bolts 2014 Revision

The body of ribbed bolts shall be of an approved form with continuous longitudinal ribs. The diameter of the body measured on a circle through the points of the ribs shall be 0.078125 in. greater than the nominal diameter specified for the bolts.

Ribbed bolts shall be furnished with round heads conforming to ANSI B18.5 (ANSI B18.5.2.2M or B18.5.2.3M) as specified. Nuts shall be hexagonal, either recessed or with a washer of suitable thickness. Ribbed bolts shall make a driving fit with the holes. The hardness of the ribs shall be such that the ribs do not distort to permit the bolts to turn in the holes during tightening. If for any reason the bolt twists before drawing tight, the hole shall be carefully reamed and an oversized bolt used as a replacement.

11.5.6—Connections Using High-Strength Bolts

11.5.6.1—General 2014 Revision

This Article covers the assembly of structural joints using AASHTO M 164 (ASTM A325) or AASHTO M 253 (ASTM A490) high-strength bolts or equivalent fasteners, installed so as to develop the minimum required bolt tension specified in Table 11.5.6.4.1-1. The bolts are used in holes conforming to the requirements of Article 11.4.8, “Bolt Holes.”

C11.5.6.1

Information is found in the *Structural Bolting Handbook*, SBH-1 (1996).

11.5.6.2—Bolted Parts

All material within the grip of the bolt shall be steel; there shall be no compressible material such as gaskets or insulation within the grip. Bolted steel parts shall fit solidly together after the bolts are snugged and may be coated or uncoated. The slope of the surfaces of parts in contact with the bolt head or nut shall not exceed 1:20 with respect to a plane normal to the bolt axis.

11.5.6.3—Surface Conditions 2014 Revision

At the time of assembly, all joint surfaces, including surfaces adjacent to the bolt head and nut, shall be free of scale, except tight mill scale, and shall be free of dirt or other foreign material. Burrs that would prevent solid seating of the connected parts in the snug condition shall be removed.

Paint is permitted on the faying surface including slip-critical joints when designed in accordance with Article 6.13.2, “Bolted Connections,” of the *AASHTO LRFD Bridge Design Specifications*.

The faying surfaces of slip-critical connections shall meet the requirements of the following paragraphs, as applicable:

- In noncoated joints, paint, including any inadvertent overspray, shall be excluded from areas closer than one bolt diameter but not less than 1.0 in. from the edge of any hole and all areas within the bolt pattern.

C11.5.6.3

Surface conditions refers to Article 6.13.2, “Bolted Connections,” and Article 6.13.2.8, “Slip Resistance,” of the *AASHTO LRFD Bridge Design Specifications*.

- Joints specified to have painted faying surfaces shall be blast cleaned and coated with a paint which has been qualified in accordance with requirements of Article 6.13.2.8, "Slip Resistance," of the *AASHTO LRFD Bridge Design Specifications.*

- Coated joints shall not be assembled before the coating has cured for the minimum time used in the qualifying test.

- Faying surfaces specified to be galvanized shall be hot-dip galvanized in accordance with AASHTO M 111M/M 111 (ASTM A123/A123M) and shall subsequently be roughened by means of hand wire brushing. Power wire brushing is not permitted.

"Surface conditions" refers to Article 6.13.2, "Bolted Connections," and Article 6.13.2.8, "Slip Resistance," of the *AASHTO LRFD Bridge Design Specifications*, 2007.

11.5.6.4—Installation

11.5.6.4.1—General 2010 Revision

C11.5.6.4.1 2014 Revision 2015 Revision

Fastener components shall be assigned lot numbers, including rotational-capacity lot numbers, prior to shipping and components shall be assembled when installed. Such components shall be protected from dirt and moisture at the job site. Only the number of anticipated components to be installed and tensioned during a work shift shall be removed from protective storage. Components not used shall be returned to protective storage at the end of the shift. Assemblies for slip-critical connections which accumulate rust or dirt resulting from job site conditions shall be cleaned, relubricated, and tested for rotational capacity prior to installation. All galvanized nuts shall be lubricated with a lubricant containing a visible dye. Plain bolts must be oily to touch when delivered and installed. Lubricant on exposed surfaces shall be removed prior to painting.

A bolt-tension measuring device (a Skidmore-Wilhelm Calibrator or other acceptable bolt-tension indicating device) shall be at all job sites where high-strength bolts are being installed and tensioned. The tension-measuring device shall be used to perform the rotational-capacity test and to confirm:

- the suitability to satisfy the requirements of Table 11.5.6.4.1-1 of the complete fastener assembly, including lubrication if required to be used in the work,

- calibration of the wrenches, if applicable, and

- the understanding and proper use by the bolting crew of the installation method.

To perform the calibrated wrench verification test for short grip bolts, direct tension indicators (DTI) with solid plates may be used in lieu of a tension-measuring device. The DTI lot shall be first verified with a longer grip bolt in the Skidmore-Wilhelm Calibrator or an acceptable equivalent device. The frequency of confirmation testing, the number of tests to be performed, and the test procedure shall be as specified in Articles 11.5.6.4.4 through 11.5.6.4.7, as applicable. The accuracy of the tension-measuring device shall be confirmed by an approved testing agency at least annually.

Bolts and nuts together with washers of size and quality specified in the contract documents, located as required below, shall be installed in properly aligned holes and tensioned and inspected by any of the installation methods described in Articles 11.5.6.4.4 through 11.5.6.4.7 to at least the minimum tension specified in Table 11.5.6.4.1-1. Tensioning may be done by turning the bolt while the nut is prevented from rotating when it is impractical to turn the nut. Impact wrenches, if used, shall be of adequate capacity and sufficiently supplied with air to tension each bolt in approximately ten seconds.

AASHTO M 253 (ASTM A490) fasteners and galvanized AASHTO M 164 (ASTM A325) fasteners shall not be reused. Other AASHTO M 164 (ASTM A325) bolts may be reused if approved by the Engineer. Touching up or retorquing previously tensioned bolts which may have been loosened by the tensioning of adjacent bolts shall not be considered as reuse provided the torquing continues from the initial position and does not require greater rotation, including the tolerance, than that required by Table 11.5.6.4.1-2.

Bolts shall be installed in all holes of the connection and the connection brought to a snug condition.

Snug is defined as having all plies of the connection in firm contact.

Snugging shall progress systematically from the most rigid part of the connection to the free edges. The snugging sequence shall be repeated until the full connection is in a snug condition.

Table 11.5.6.4.1-1—Minimum Required Bolt Tension, kips

Bolt Size	AASHTO M 164 ASTM A325	AASHTO M 253 ASTM A490
0.5	12	15
0.625	19	24
0.75	28	35
0.875	39	49
1.0	51	64
1.125	56	80
1.25	71	102
1.375	85	121
1.5	103	148

The minimum bolt tension shall be taken as 70 percent of specified minimum tensile strength of bolts (as specified in ASTM Specifications for tests of full-size A325 and A490 bolts with UNC threads (metric coarse thread series, ANSI B1.13M), loaded in axial tension) rounded to the nearest kip.

Table 11.5.6.4.1-2—Nut Rotation from the Snug Condition for Inch Series Fasteners

	Geometry of Outer Faces of Bolted Parts		
Bolt length measured from underside of head to end of bolt	Both faces normal to bolt axis	One face normal to bolt axis and other face sloped not more than 1:20. Bevel washer not used.	Both faces sloped not more than 1:20 from normal to bolt axis. Bevel washers not used.
Up to and including four diameters	$^{1}/_{3}$ turn	$^{1}/_{2}$ turn	$^{2}/_{3}$ turn
Over four diameters, but not exceeding eight diameters	$^{1}/_{2}$ turn	$^{2}/_{3}$ turn	$^{5}/_{6}$ turn
Over eight diameters, but not exceeding 12 diameters	$^{2}/_{3}$ turn	$^{5}/_{6}$ turn	1 turn

Rotation, as used in Table 11.5.6.4.1-2, shall be taken as relative to the bolt, regardless of the element (nut or bolt) being turned. The tolerances are minus 0, plus 30° for bolts installed by 1/2 turn or less; for bolts installed by 2/3 turn or more, the tolerances are minus 0, plus 45°.

The values, given in Table 11.5.6.4.1-2, shall be applicable only to connections in which all material within grip of the bolt is steel.

For situations in which the bolt length measured from the underside of the head to the end of the bolt exceeds 12 diameters, the required rotation shall be determined by actual tests in a suitable tension device simulating the actual conditions.

No research work has been performed by the Research Council Riveted and Bolted Structural Joints to establish the turn-of-nut procedure when bolt lengths exceed 12 diameters.

11.5.6.4.2—Rotational-Capacity Tests 2010 Revision

C11.5.6.4.2 2014 Revision

Rotational-capacity testing is required for all fastener assemblies. Assemblies specified as galvanized shall be tested after galvanizing. Washers shall be required as part of the test even though they may not be required as part of the installation procedure. The following shall apply:

- Except as modified herein, the rotational-capacity test shall be performed in accordance with the requirements of AASHTO M 164 (ASTM A325).
- Each combination of bolt production lot, nut lot, and washer lot shall be tested as an assembly. Where washers are not required by the installation procedures, they need not be included in the lot identification.

- A rotational-capacity lot number shall have been assigned to each combination of lots tested.
- The minimum frequency of testing shall be two assemblies per rotational-capacity lot.
- For bolts that are long enough to fit in a Skidmore-Wilhelm Calibrator, the bolt, nut, and washer assembly shall be assembled in a Skidmore-Wilhelm Calibrator or an acceptable equivalent device.
- Bolts that are too short to test in a Skidmore-Wilhelm Calibrator may be tested in a steel joint. The tension requirement, in the section below, need not apply. The maximum torque requirement, torque $\leq 0.250PD$, shall be computed using a value of P equal to the turn test tension taken as 1.15 times the bolt tension in Table 11.5.6.4.1-1.
- The tension reached at the rotation below, i.e., the turn-test tension, shall be equal to or greater than 1.15 times the required fastener tension, i.e., installation tension, shown in Table 11.5.6.4.1-1.
- In a tension-measuring device, the minimum rotation from an initial tension of ten percent of the minimum required tension shall be two times the required number of turns indicated in Table 11.5.6.4.1-2 without stripping or failure.

The ten percent of the minimum required tension is assumed to bring the connection to a snug condition.

- After the required installation tension listed above has been exceeded, one reading of tension and torque shall be taken and recorded. The torque value shall conform to the following:

$$\text{Torque} \leq 0.250PD \qquad (11.5.6.4.2\text{-}1)$$

where:

Torque = measured torque, ft·lb
P = measured bolt tension, lb
D = bolt diameter, ft

11.5.6.4.3—Requirement for Washers 2014 Revision

C11.5.6.4.3

Where the outer face of the bolted parts has a slope greater than 1:20 with respect to a plane normal to the bolt axis, a hardened bevelled washer shall be used to compensate for the lack of parallelism.

Hardened bevelled washers for American Standard Beams and Channels shall be required and shall be square or rectangular, shall conform to the requirements of AASHTO M 293 (ASTM F436), and shall taper in thickness.

American Standard Beams and Channels can be located in the AISC *LRFD Manual of Steel Construction* (2003).

Where necessary, washers may be clipped on one side to a point not closer than 0.875 times the bolt diameter from the center of the washer.

Hardened washers are not required for connections using AASHTO M 164 (ASTM A325) and AASHTO M 253 (ASTM A490) bolts, except as follows:

- Hardened washers shall be used under the turned element when tensioning is to be performed by calibrated wrench method.
- Irrespective of the tensioning method, hardened washers shall be used under both the head and the nut when AASHTO M 253 (ASTM A490) bolts are to be installed in material having a specified yield point less than 40 ksi.
- Where AASHTO M 164 (ASTM A325) bolts of any diameter or AASHTO M 253 (ASTM A490) bolts equal to or less than 1.0 in. in diameter are to be installed in oversize or short-slotted holes in an outer ply, a hardened washer conforming to AASHTO M 293 (ASTM F436) shall be used.
- Where AASHTO M 253 (ASTM A490) bolts over 1.0 in. in diameter are to be installed in an oversize or short-slotted hole in an outer ply, hardened washers conforming to AASHTO M 293 (ASTM F436), except with 0.3125-in. minimum thickness, shall be used under both the head and the nut in lieu of standard thickness hardened washers. Multiple hardened washers with combined thickness equal to or greater than 0.3125 in. shall not be considered as satisfying this requirement.
- Where AASHTO M 164 (ASTM A325) bolts of any diameter or AASHTO M 253 (ASTM A490) bolts equal to or less than 1.0 in. in diameter are to be installed in a long slotted hole in an outer ply, a plate washer or continuous bar of at least 0.3125-in. thickness with standard holes shall be provided. These washers or bars shall have a size sufficient to completely cover the slot after installation and shall be of structural grade material, but need not be hardened, except as follows.
- Where AASHTO M 253 (ASTM A490) bolts over 1.0 in. in diameter are to be used in long slotted holes in external plies, a single hardened washer conforming to AASHTO M 293 (ASTM F436), but with 0.3125-in. minimum thickness shall be used in lieu of washers or bars of structural grade material. Multiple hardened washers with combined thickness equal to or greater than 0.3125 in. shall not be considered as satisfying this requirement.

Alternate design fasteners meeting the requirements of Article 11.3.2.6, "Load Indicator Devices," with a geometry which provides a bearing circle on the head or nut with a diameter equal to or greater then the diameter of hardened washers meeting the requirements of AASHTO M 293 (ASTM F436) satisfy the requirements for washers specified herein and may be used without washers.

11.5.6.4.4—Turn-of-Nut Installation Method

When the turn-of-nut installation method is used, hardened washers are not required, except as may be specified in Article 11.5.6.4.3.

Verification testing using a representative sample of not less than three fastener assemblies of each diameter, length, and grade to be used in the work shall be performed at the start of work in a device capable of indicating bolt tension. This verification test shall demonstrate that the method used by the bolting crew to develop a snug condition and to control the turns from a snug condition develops a tension not less than five percent greater than the tension required by Table 11.5.6.4.1-1. Periodic retesting shall be performed when ordered by the Engineer.

After snugging, the applicable amount of rotation specified in Table 11.5.6.4.1-2 shall be achieved. During the tensioning operation there shall be no rotation of the part not turned by the wrench. Tensioning shall progress systematically from the most rigid part of the joint to its free edges.

11.5.6.4.5—Calibrated Wrench Installation Method

The calibrated wrench method may be used only when wrenches are calibrated on a daily basis and when a hardened washer is used under the turned element. Standard torques determined from tables or from formulas which are assumed to relate torque to tension shall not be acceptable.

When calibrated wrenches are used for installation, they shall be set to deliver a torque which has been calibrated to produce a tension not less than five percent in excess of the minimum tension specified in Table 11.5.6.4.1-1. The installation procedures shall be calibrated by verification testing at least once each working day for each fastener assembly that is being installed in the work that day. This verification testing shall be accomplished in a tension-measuring device capable of indicating actual bolt tension by testing three typical fastener assemblies from each lot. Bolts, nuts, and washers under the turned element shall be sampled from production lots. Wrenches shall be recalibrated when a significant difference is noted in the surface condition of the bolts, threads, nuts, or washers. It shall be verified during actual installation in the assembled steel work that the wrench adjustment selected by the calibration does not produce a nut or bolt head rotation from snug condition greater than that permitted in Table 11.5.6.4.1-2. If manual torque wrenches are used, nuts shall be torqued in the tensioning direction when torque is measured.

When calibrated wrenches are used to install and tension bolts in a connection, bolts shall be installed with hardened washers under the turned element. Following snugging, the connection shall be tensioned using the calibrated wrench. Tensioning shall progress systematically from the most rigid part of the joint to its free edges. The wrench shall be returned to "touch up" previously tensioned bolts which may have been relaxed as a result of the subsequent tensioning of adjacent bolts until all bolts are tensioned to the prescribed amount.

11.5.6.4.6—Alternative Design Bolt Installation Method

When fasteners which incorporate a design feature intended to indirectly indicate that the applied torque develops the required bolt tension or to automatically develop the tension required by Table 11.5.6.4.1-1 and which have been qualified under Article 11.3.2.5, "Alternative Fasteners," are to be installed, verification testing using a representative sample of not less than three fastener assemblies of each diameter, length and grade to be used in the work shall be performed at the job site in a device capable of indicating bolt tension. The test assembly shall include flat-hardened washers, if required in the actual connection, arranged as in the actual connections to be tensioned. The verification test shall demonstrate that each bolt develops a tension not less than five percent greater than the tension required by Table 11.5.6.4.1-1. The Manufacturer's installation procedure shall be followed for installation of bolts in the calibration device and in all connections. Periodic retesting shall be performed when ordered by the Engineer.

When alternative design fasteners which are intended to control or indicate bolt tension of the fasteners are used, bolts shall be installed in all holes of the connection and initially snugged sufficiently to bring all plies of the joint into firm contact, but without yielding or fracturing the control or indicator element of the fasteners. All fasteners shall then be further tensioned, progressing systematically from the most rigid part of the connection to the free edges in a manner that will minimize relaxation of previously tensioned bolts. In some cases, proper tensioning of the bolts may require more than a single cycle of systematic partial tensioning prior to final yielding or fracturing of the control or indicator element of individual fasteners. If yielding or fracture occurs prior to the final tensioning cycle, the individual fastener assembly shall be replaced with a new one.

11.5.6.4.7—Direct Tension Indicator Installation Method

2010 Revision

When Direct Tension Indicators (DTIs) meeting the requirements of Article 11.3.2.6 are used with high-strength bolts to indicate bolt tension, they shall be subjected to the verification testing described below and installed in accordance with the method specified below.

C11.5.6.4.7

The procedures contained in ASTM F959 are designed to provide an easy method for using Direct Tension Indicators to indicate proper bolt tension. DTIs measure load by compressing the protrusions on the DTI with a corresponding reduction in the gap in the spaces between

Unless otherwise approved by the Engineer-of-Record, the DTIs shall be installed under the head of the bolt and the nut turned to tension the bolt. The Manufacturer's recommendations shall be followed for the proper orientation of the DTI and additional washers, if any, required for the correct use of the DTI. Installation of a DTI under the turned element may be permitted if a washer is used to separate the turned element from the DTI.

the protrusions. The method of measurement is based upon the criteria that a DTI with half or more of its spaces less than 0.005 in. indicates a bolt tension above the required minimum tension. In order to verify that DTI will provide this performance, the DTI is checked at 1.05 times the required installation tension. One less than half of the number of spaces must be greater than 0.005 in. at this load. Consequently, in the structure, if half or more of the spaces are less than 0.005 in. (number of refusals greater than half the number of spaces), the fastener is properly installed at a tension above the required minimum installation tension.

An upper limit on the DTI deformation allowed in the structure is imposed to ensure that bolt failure will not occur during installation. A visible gap must remain in any space after installation. This requirement is necessary since ASTM F959 accepts DTIs with an average gap of 0.015 in. and loads higher than the required minimum tension. Consequently, the load required to reduce the gaps to less than 0.005 in. may exceed the minimum tensile strength of the fastener assembly. The capability of the fastener to tolerate this small gap is determined in one of two ways. The simplest is to turn the nut by hand down the thread. If the nut can be assembled for the length of the thread, the bolt has not undergone significant inelastic deformation and, therefore, the tension required is much less than the installation strength of the fastener assembly. If the bolt does not pass this inelastic deformation test, the load must be less than 95 percent of the average load measured at the end of the rotational capacity test for the rotational capacity lot of the fastener assembly. Note that the tension capacity of the fastener assembly may be less than the minimum required tensile strength of the bolts due to the reduction in tensile strength from the torque applied to the bolt during tightening.

11.5.6.4.7a—Verification 2011 Revision

Verification testing shall be performed in a calibrated bolt-tension measuring device. A special flat insert shall be used in place of the normal bolt head holding insert. Three verification tests shall be required for each combination of fastener assembly rotational-capacity lot, DTI lot, and DTI position relative to the turned element (bolt head or nut) to be used on the project. The fastener assembly shall be installed in the tension-measuring device with the DTI located in the same position as in the work. The element intended to be stationary (bolt or nut) shall be restrained from rotation.

The verification tests shall be conducted in two stages. The bolt nut and DTI assembly shall be installed in a manner so that at least three and preferably not more than five threads are located between the bearing face of the nut and the bolt head. The bolt shall be tensioned first to the load equal to that listed in Table 11.5.6.4.7a-1 under Verification Tension for the grade and diameter of the bolt. If an impact wrench is used, the tension developed using the impact wrench shall be no more than two-thirds of the required tension. Subsequently, a manual wrench shall be

C11.5.6.4.7a

The purpose of the verification testing is to ensure that the fastener will be at or above the desired installation tension when half or more of the spaces in the DTI have a gap less than 0.005 in. and that the bolt will not have excessive plastic deformation at the minimum gap allowed on the project.

used to attain the required tension. The number of refusals of the 0.005-in. tapered feeler gage in the spaces between the protrusions shall be recorded. The number of refusals for uncoated DTIs under the stationary or turned element, or coated DTIs under the stationary element, shall not exceed the number listed under Maximum Verification Refusals in Table 11.5.6.4.7a-1 for the grade and diameter of bolt used. The maximum number of verification refusals for coated DTIs (galvanized, painted, or epoxy-coated), when used under the turned element, shall be no more than the number of spaces on the DTI less one. The DTI lot shall be rejected if the number of refusals exceeds the values in the table or, for coated DTIs if the gage is refused in all spaces.

After the number of refusals is recorded at the verification load, the bolt shall be further tensioned until the 0.005-in. feeler gage is refused at all the spaces and a visible gap exists in at least one space. The load at this condition shall be recorded and the bolt removed from the tension-measuring device. The nut shall be able to be run down by hand for the complete thread length of the bolt excluding thread run-out. If the nut cannot be run down for this thread length, the DTI lot shall be rejected unless the load recorded is less than 95 percent of the average load measured in the rotational capacity test of the fastener lot as specified in Article 11.5.6.4.2, "Rotational-Capacity Tests."

If the bolt is too short to be tested in the calibration device, the DTI lot shall be verified on a long bolt in a calibrator to determine the number of refusals at the verification tension listed in Table 11.5.6.4.7a-1. The number of refusals shall not exceed the values listed under maximum verification refusals in Table 11.5.6.4.7a-1. Another DTI from the same lot shall then be verified with the short bolt in a convenient hole in the work. The bolt shall be tensioned until the 0.005-in. feeler gage is refused in all spaces and a visible gap exists in at least one space. The bolt shall then be removed from the tension-measuring device and the nut shall be able to be run down by hand for the complete thread length of the bolt excluding thread run-out. The DTI lot shall be rejected if the nut cannot be run down this thread length.

Table 11.5.6.4.7a-1—Direct Tension Indicator Requirements

Bolt Size, in.	Verification Tension, kips		Maximum Verification Refusals		DTI Spaces		Minimum Installation Refusals	
	A325	A490	A325	A490	A325	A490	A325	A490
0.5	13	16	1	2	4	5	2	3
0.625	20	25	1	2	4	5	2	3
0.75	29	37	2	2	5	6	3	3
0.875	41	51	2	2	5	6	3	3
1.0	54	67	2	3	6	7	3	4
1.125	59	84	2	3	6	7	3	4
1.25	75	107	3	3	7	8	4	4
1.375	89	127	3	3	7	8	4	4
1.5	108	155	3	4	8	9	4	5

11.5.6.4.7b—Installation 2011 Revision

Installation of fastener assemblies using DTIs shall be performed in two stages. The stationary element shall be held against rotation during each stage of the installation. The connection shall be first snugged with bolts installed in all the holes of the connection and tensioned sufficiently to bring all the plies of the connection into firm contact. The number of spaces in which a 0.005-in. feeler gage is refused in the DTI after snugging shall not exceed those listed under Maximum Verification Refusals in Table 11.5.6.4.7a-1. If the number exceeds the values in the table, the fastener assembly shall be removed and another DTI installed and snugged.

For uncoated DTIs used under a stationary or turned element and for coated DTIs used under stationary element, the bolts shall be further tensioned until the number of refusals of the 0.005-in. feeler gage shall be equal to or greater than the number listed under Minimum Installation Refusals in Table 11.5.6.4.7a-1. If the bolt is tensioned so that no visible gap in any space remains, the bolt and DTI shall be removed and replaced by a new properly tensioned bolt and DTI.

When coated DTIs (galvanized, painted, or epoxy-coated) are used under a turned element, the 0.005-in. feeler gage shall be refused in all spaces.

C11.5.6.4.7b

Turning of the element against the DTI will reduce the gap at a given tension and cause the DTI to indicate a load higher than the actual load in the fastener.

Since the fastener tension may have relaxed during the snugging of adjacent fasteners, the number of refusals after snugging is required not to exceed the maximum allowed during verification.

A DTI does not rebound when the fastener tension is reduced. If a DTI which exceeds the number of refusals is not replaced, it would give a false indication of bolt tension.

A visible gap must remain in any space after installation to ensure that the fasteners are not tensioned to their ultimate strength.

11.5.6.4.8—Lock-Pin and Collar Fasteners

The installation of lock-pin and collar fasteners shall be by methods and procedures approved by the Engineer.

11.5.6.4.9—Inspection

11.5.6.4.9a—General

The Engineer shall determine that the requirements of Articles 11.5.6.4.9.b and 11.5.6.4.9.c, following, are met in the work.

11.5.6.4.9b—Responsibilities of the Engineer

Before the installation of fasteners in the work, the Engineer shall:

- check the marking, surface condition, and storage of bolts, nuts, washers, and DTIs, if used, and the faying surfaces of joints for compliance with the requirements of Articles 11.3.2, 11.5.6.1, and 11.5.6.4.1 and

- observe calibration and/or testing procedures required in Articles 11.5.6.4.4 through 11.5.6.4.7, as applicable, to confirm that the selected procedure is properly used and that, when so used with the fastener assemblies supplied, the tensions specified in Table 11.5.6.4.1-1 are developed.

The Engineer shall monitor the installation of fasteners in the work to assure that the selected installation method, as demonstrated in the initial testing to develop the specified tension, is routinely followed.

11.5.6.4.9c—Inspection Procedures

Either the Engineer or the Contractor, in the presence of the Engineer at the Engineer's option, shall inspect the tensioned bolts using an inspection torque wrench, unless alternate fasteners or direct tension indicator devices are used, allowing verification by other methods. Inspection tests should be conducted prior to possible loss of lubrication or corrosion influence on tightening torque.

Three fastener assembly lots in the same condition as those under inspection shall be placed individually in a device calibrated to measure bolt tension. This calibration operation shall be done at least once each inspection day.

There shall be a washer under the turned element in tensioning each bolt if washers are used on the structure. If washers are not used on the structure, the material used in the tension-measuring device which abuts the part turned shall be of the same specification as that used on the structure. In the calibrated device, each bolt shall be tensioning by any convenient means to the specified tension. The inspecting wrench shall then be applied to the tensioned bolt to determine the torque required to turn the nut or head five degrees (approximately 1.0 in. at a 12.0-in. radius) in the tensioning direction. The average of the torque required for all three bolts shall be taken as the job-inspection torque.

Ten percent (at least two) of the tensioned bolts on the structure represented by the test bolts shall be selected at random in each connection. The job-inspection torque shall then be applied to each with the inspecting wrench turned in the tensioning direction. If this torque turns no bolt head or nut, the bolts in the connection shall be considered to be properly tensioned. If the torque turns one or more boltheads or nuts, the job-inspection torque shall then be applied to all bolts in the connection. Any bolt whose head or nut turns at this stage shall be retorqued and reinspected. The Contractor may, however, retorque all the bolts in the connection and resubmit it for inspection, so long as DTIs are not overtensioned and fastener assemblies are not damaged.

C11.5.6.4.9c

The 24-hour time lapse for inspection tests on installed bolts required by earlier editions of Division I of the Standard Specifications was thought to be too restrictive. Since the effects of loss of lubricant or initiation of corrosion on the internal threads of installed fasteners is dependent on environmental conditions at the site, it was decided to leave the lapse time between installation and verification to the discretion of the Engineer/Owner rather than specifying one fixed time for all sites.

Where the elapsed time between installation (bolt insertion) and final tensioning is more than ten days, sample fastener assemblies should be removed from joints that are to be tensioned. At least three assemblies for each size and length should be used to verify that the loss of lubricant will not hinder achieving the required tension. These assemblies should be used to establish the inspection torque. The Rotational Capacity test on the sample assemblies can be used to demonstrate whether or not the assembly will tension satisfactorily.

After assemblies have been either found acceptable based on additional RoCap testing or replaced, all should be brought to the snug condition and then tightened by the selected method. The Contractor should then apply the appropriate inspection torque to at least 10 percent of bolts which had delayed tightening while the owner's representative observes. Any fastener assembly rotated by the torque wrench indicates improper tightening, and that fastener should be removed and inspected for galling or other signs of improper lubrication, and more bolts should be checked with the torque wrench until the owner's representative is satisfied the bolts are properly tightened. All bolts in a common connection should have approximately equal exposed tip lengths. Any bolts with significantly more or less stick-through than others should be removed and replaced unless the Contractor can explain the variation.

11.5.7—Welding

Welding, welder qualifications, prequalification of weld details, and inspection of welds shall conform to the requirements of the current AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code.*

Brackets, clips, shipping devices, or other material not required by the contract documents shall not be welded or tacked to any member unless specified in the contract documents and approved by the Engineer.

11.6—ERECTION

11.6.1—General

The Contractor shall provide all tools, machinery, and equipment necessary to erect the structure.

Falsework and forms shall be in accordance with the requirements of Section 3, "Temporary Works."

C11.6.1

See also the *Steel Bridge Erection Guide Specification*, S10.1, NSBEGS-1 (2007).

11.6.2—Handling and Storing Materials

Material to be stored at the job site shall be placed on skids above the ground. It shall be kept clean and properly drained. Girders and beams shall be placed upright and shored. Long members, such as columns and chords, shall be supported on skids placed near enough together to prevent injury from deflection. If the contract documents are for erection only, the Contractor shall check the material received against the shipping lists and report promptly in writing any shortage or injury discovered. After material is received by the Contractor, the Contractor shall be responsible for any damage to or loss of material.

11.6.3—Bearings and Anchorages

Bridge bearings shall be furnished and installed in conformance with Section 18, "Bearing Devices."

If the steel superstructure is to be placed on a substructure that was built under a separate contract, the Contractor shall verify that the masonry has been constructed in the right location and to the correct lines and elevations before ordering materials.

11.6.4—Erection Procedure

11.6.4.1—Conformance to Drawings

The erection procedure shall conform to the erection drawings submitted in accordance with Article 11.2.2. Any modifications to or deviations from this erection procedure shall require revised drawings and verification of stresses and geometry.

11.6.4.2—Erection Stresses

Any erection stresses induced in the structure as a result of using a method of erection which differs from the contract documents shall be accounted for by the Contractor. Erection design calculations for such changed methods shall be prepared at the Contractor's expense and submitted to the Engineer. The calculations shall indicate any change in stresses or change in behavior for the temporary and final structures. Additional material required to keep both the temporary and final stresses within the allowable limits used in design shall be provided at the Contractor's expense.

The Contractor shall be responsible for providing temporary bracing or stiffening devices to accommodate handling stresses in individual members or segments of the structure during erection.

11.6.4.3—Maintaining Alignment and Camber

During erection, the Contractor shall be responsible for supporting segments of the structure in a manner that will produce the proper alignment and camber in the completed structure. Cross frames and diagonal bracing shall be installed as necessary during the erection process to provide stability and assure correct geometry. Temporary bracing, if necessary at any stage of erection, shall be provided by the Contractor.

11.6.5—Field Assembly

The parts shall be accurately assembled as specified in the contract documents or erection drawings and any match-marks shall be followed. The material shall be carefully handled so that no parts will be bent, broken, or otherwise damaged. Hammering which will injure or distort the members shall not be done. Bearing surfaces and surfaces to be in permanent contact shall be cleaned before the members are assembled. Splices and field connections shall have one-half of the holes filled with bolts and cylindrical erection pins (half bolts and half pins) before installing and tightening the balance of high-strength bolts. Splices and connections carrying traffic during erection shall have three-fourths of the holes so filled.

Fitting-up bolts may be the same high-strength bolts used in the installation. If other fitting-up bolts are used, they shall be of the same nominal diameter as the high-strength bolts and cylindrical erection pins shall be 0.03125 in. larger.

11.6.6—Pin Connections

Pilot and driving nuts shall be used in driving pins. They shall be furnished by the Contractor without charge. Pins shall be so driven that the members will take full bearing on them. Pin nuts shall be screwed up tight and the threads burred at the face of the nut with a pointed tool.

11.6.7—Misfits

The correction of minor misfits involving minor amounts of reaming, cutting, grinding, and chipping shall be considered a legitimate part of the erection. However, any error in the shop fabrication or deformation resulting from handling and transporting shall be cause for rejection.

The Contractor shall be responsible for all misfits, errors, and damage and shall make the necessary corrections and replacements.

11.7—MEASUREMENT AND PAYMENT

11.7.1—Method of Measurement 2014 Revision

Pay quantities for each type of steel and iron will be measured by the pound, computed from dimensions shown in the contract documents using the following rules and assumptions in Table 11.7.1-1.

Table 11.7.1-1—Unit Weights of Steel and Iron

Unit Weights, lb/ft^3	
Cast Iron	445
Malleable Iron	470
Wrought Iron	487
Steel—Rolled or Cast	490

The weight (mass) of rolled shapes shall be computed on the basis of their nominal weight per foot as specified in the contract documents or as listed in handbooks.

The weight (mass) of plates shall be computed on the basis of the nominal weight (mass) for their width and thickness as specified in the contract documents, plus an estimated overrun computed as one-half the "Permissible Variation in Thickness and Weight" as tabulated in "General Requirements for Delivery of Rolled Steel Plates, Shapes, Steel Piling, and Bars for Structural Use," AASHTO M 160M/M 160 (ASTM A6/A6M).

The weight (mass) of castings shall be computed from the dimensions shown on the approved shop drawings, deducting for open holes. To this weight (mass) shall be added five percent allowance for fillets and overrun. Scale weight (mass) may be substituted for computed weight (mass) in the case of castings or of small complex parts for which accurate computations of weight (mass) would be difficult.

The weight (mass) of temporary erection bolts; shop and field paint; boxes, crates, and other containers used for shipping; and materials used for supporting members during transportation and erection, will not be included.

The weight (mass) of any additional material required by Article 11.6.4.2, "Erection Stresses," to accommodate erection stresses resulting from the Contractor's choice of erection methods will not be included.

In computing pay weight (mass) on the basis of computed net weight (mass) the following stipulations in addition to those in the foregoing paragraphs shall apply:

- The weight (mass) shall be computed on the basis of the net finished dimensions of the parts as specified in the contract documents, deducting for copes, cuts, clips, and all open holes, except bolt holes.

- The weight (mass) of heads, nuts, single washers, and threaded stick-through of all high tensile strength bolts, both shop and field, shall be included on the basis of the following weight (mass) as specified in Table 11.7.1-2.

Table 11.7.1-2—Weight per 100 Bolts

Diameter of Bolt	Weight per 100 Bolts, lb
0.5	19.7
0.625	31.7
0.75	52.4
0.875	80.4
1.0	116.7
1.125	165.1
1.25	212.0
1.375	280.0
1.5	340.0

- The weight (mass) of fillet welds shall be as specified in Table 11.7.1-3.

Table 11.7.1-3—Weight of Fillet Welds

Size of Fillet Weld, in.	Weight, lb/ft
0.1875	0.08
0.25	0.14
0.3125	0.22
0.375	0.30
0.5	0.55
0.625	0.80
0.75	1.10
0.875	1.50
1.0	2.00

- To determine the pay quantities of galvanized metal, the weight (mass) to be added to the calculated weight (mass) of base metal for the galvanizing will be determined from the weight (mass) of zinc coatings specified by AASHTO M 111M/M 111 (ASTM A123/A123M).

- No allowance will be made for the weight (mass) of paint.

11.7.2—Basis of Payment

The contract-documents price for fabrication and erection of structural steel shall be considered to be full compensation for the cost of all labor, equipment, materials, transportation, and shop and field painting, if not otherwise provided for, necessary for the proper completion of the work in accordance with the contract documents. The contract-documents price for fabrication without erection shall be considered to be full compensation for the cost of all labor, equipment, and materials necessary for the proper completion of the work, other than erection and field assembly, in accordance with the contract documents.

Under contract documents containing an item for structural steel, all metal parts other than metal reinforcement for concrete, such as anchor bolts and nuts, shoes, rockers, rollers, bearing and slab plates, pins and nuts, expansion dams, roadway drains and scuppers, weld metal, bolts embedded in concrete, cradles and brackets, railing, and railing pots shall be paid for as structural steel unless otherwise stipulated.

Payment will be made on a pound- (mass-) unit-price or a lump-sum basis as required by the terms of the contract documents but, unless stipulated otherwise, it shall be on a pound- (mass-) unit-price basis. For members comprising both carbon steel and other special steel or material, when separate unit prices are provided for same, the weight (mass) of each class of steel in each such member shall be separately computed and paid for at the contract-documents unit price therefore.

Full-size members which are tested in accordance with the contract documents, when such tests are required by the contract documents, shall be paid for at the same rate as for comparable members for the structure. The cost of testing including equipment, labor, and incidentals shall be included in the contract-documents price for structural steel. Members which fail to meet the contract-documents requirements and members rejected as a result of tests will not be paid for by the Owner.

11.8—ADDITIONAL PROVISIONS FOR CURVED STEEL GIRDERS

11.8.1—General

These provisions apply to horizontally curved girders and shall be considered in addition to other provisions of these Specifications. However, when there is a conflict between these provisions for curved girders and other provisions of these Specifications, these provisions shall prevail.

These provisions shall be used by the Contractor to create a construction plan as required herein.

11.8.1.1—Scope

These provisions shall apply to the superstructure including fabrication, shipping, erection of the girders, cross-frames and other structural steel elements, and placement of the concrete deck.

11.8.2—Contractor's Construction Plan for Curved Girder Bridges

The Engineer's construction plan, if shown in the contract documents, shall not be taken as supplanting or implying any supplantation of the Contractor's responsibility for the fabrication, erection, or construction of any part of the bridge.

The Contractor shall provide a construction plan which details fabrication, procedures for erection, and deck placement, and which shall be referred to herein as the

C11.8.2

The Engineer's construction plan, if provided in the contract documents, shows the construction plan considered in design of the bridge and is only one possible means of construction of the bridge. However, the Contractor/Fabricator/Erector is not required to build the bridge according to this plan, subject to approval of the Engineer.

A construction plan provided on the contract documents indicates what considerations were made in the

Contractor's construction plan. This plan may be based on the plan shown in the contract documents, if one is provided, or may be developed entirely by the Contractor. In either event, it shall demonstrate the general stability of the structure and individual components during each stage of construction, including while supported on any temporary jacks. The Contractor's construction plan shall be stamped by a Professional Engineer and be accepted by the Owner. Where the Contractor's construction plan causes a difference in camber of the girders for dead load from that presented on the contract documents, acceptance by the Owner shall be obtained prior to commencement of fabrication. The Contractor's construction plan shall include:

- Fabrication procedures, including method of curving the girders,
- Shipping weights, lengths, widths, and heights, and means of shipping,
- Erection plan, including sequence of erection; crane capacities; and the location, capacity and elevation of any temporary supports, and
- Deck placement sequence, including the time between placements; and the magnitude and position of any temporary load required to prevent girder lift-off at bearings.

Computations which show that the factored construction stresses satisfy the requirements of Articles 6.10.3 or 6.11.3, "Constructibility," of the *AASHTO LRFD Bridge Design Specifications*, as applicable, shall be provided.

design. In most instances, the actual plan of construction of the bridge would not be expected to differ significantly enough from the Engineer's construction plan that a redesign of the bridge would be required.

Although a construction plan may be provided in the contract documents, the Contractor is responsible for the Contractor's construction plan even if there are no modifications of the construction plan provided in the contract documents. The Contractor's construction plan will be considerably more detailed than the construction plan provided in the contract documents.

Because of the complexity of curved steel bridges, the Contractor's construction plan must be stamped and signed by a Professional Engineer.

11.8.3—Fabrication

11.8.3.1—General

The Fabricator shall ensure that the steel can be fit up in the no-load condition unless specified otherwise in the construction plan.

11.8.3.2—Handling

Rolled shapes and plates composing the flanges and webs shall be handled in such a manner as to prevent visible deformations or other incidental damage.

11.8.3.3—Girders

11.8.3.3.1—Rolled I-Beams

Horizontal curvature may be obtained by heat-curving. Heat-curving of beams shall be performed in accordance with the provisions of Article 6.7.7, "Heat-Curved Rolled Beams and Welded Plate Girders," of the *AASHTO LRFD Bridge Design Specifications* and Article 11.4.12.2 of these Specifications. The Owner shall determine the necessity of

C11.8.3.3.1

Research has indicated that heat-curving has no deleterious effect on the fatigue strength of curved beams or girders (Daniels and Bacheler, 1979).

Experience has shown that additional camber may not be needed to compensate for camber loss after heat-curving (Hilton, 1984).

providing the additional camber specified in Article 6.7.7, "Camber," of the *AASHTO LRFD Bridge Design Specifications.*

11.8.3.3.2—Welded I-Girders

In addition to heat-curving as specified in Article 11.8.3.3.1, welded girders may be fabricated from cut-curved flanges.

Vertical camber shall be obtained by cutting the web plate to the necessary contour.

11.8.3.3.3—Welded Box and Tub Girders

Box flanges shall be cut-curved.

Top flanges of tub girders may be curved according to the provisions of Article 11.8.3.3.2.

C11.8.3.3.3

Top flanges of tub girders are treated as I-girder flanges and may be heat-curved after they are welded to the webs.

11.8.3.4—Web Attachments

11.8.3.4.1—Transverse Stiffeners

Transverse stiffeners shall be bolted to the web or welded to the web with continuous fillet welds on both sides of the stiffener. These welds shall be terminated between $4t_w$ and the lesser of $6t_w$ and 4.0 in. from the near edge of the adjacent flange or longitudinal web stiffener welds. If transverse stiffeners or transverse web elements serving as stiffeners are interrupted by a longitudinal stiffener, the transverse stiffeners shall be attached to the longitudinal stiffener to develop the flexural and axial resistance of the transverse stiffener. If single-sided transverse stiffeners are used, the end of the stiffener should be attached to both flanges. Fillet welds on both sides of the stiffener may be used.

When pairs of transverse stiffeners are used, they shall be fitted tightly to both flanges.

C11.8.3.4.1

Transverse stiffeners are required to be attached to longitudinal stiffeners because the full depth of the web is considered for shear and transverse stiffeners are designed to stiffen the web over that depth. Bearing stiffeners and connection plates also are designed as a single element over the web depth.

In regions of reverse bending, the stiffeners may have to be attached to both flanges. A bolted detail may be substituted for the end welds on tension flanges.

11.8.3.4.2—Connection Plates

Connection plates, otherwise identified as transverse stiffeners at cross-frames and diaphragms, shall be attached to both flanges by either welding or bolting. If a bolted connection is shown on the contract documents, a welded connection may not be substituted without permission from the Engineer.

C11.8.3.4.2

Connection plates are connected to the flanges to prevent flange rotation or raking relative to the web and to transfer lateral forces from the cross-frames or diaphragms directly to the flanges. By direct connection to the flanges, through-thickness bending stresses in the web are minimized (Wilson et al., 1988).

11.8.3.4.3—Longitudinal Stiffeners

Wherever practical, longitudinal stiffeners shall extend uninterrupted over their specified length unless otherwise permitted in the contract documents. Longitudinal stiffeners shall be bolted to the web or welded to the web with continuous fillet welds on both sides of the stiffener.

Where longitudinal stiffeners are interrupted, they should be fitted to, and attached to, the continuous element sufficiently to develop the required strength and stiffness of the longitudinal stiffener.

C11.8.3.4.3

Longitudinal web stiffeners are designed as continuous members. Therefore, except as noted herein, the longitudinal stiffeners must be made continuous wherever practical. Base metal at ends of longitudinal stiffeners should be investigated for fatigue. Copies should always be provided to avoid intersecting welds.

11.8.3.4.4—Cross-frames and Diaphragms

Cross-frames and diaphragms shall be detailed to fit under the no-load condition unless otherwise specified.

11.8.3.5—Bolt Holes

Unless permitted otherwise in the contract documents, bolt holes in girder splices and primary load-carrying members shall be standard size. Exceptions shall be approved by the Owner's Engineer.

11.8.3.6—Tolerances

11.8.3.6.1—Welded Web Flatness

Webs shall meet dimensional tolerances specified in the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*. Flatness shall be measured with respect to a straightedge oriented perpendicular to the flanges.

C11.8.3.6.1

Web flatness can be measured from a straightedge oriented along the shortest line between flange-to-web welds. The tolerance for unstiffened webs is determined using the vertical distance along the web between flanges.

11.8.3.6.2—Camber

Cambers provided on the contract documents shall be adhered to unless steel erection or deck placement is to be performed in a manner that will lead to deflections different from those used to determine the camber specified. If the Contractor or the Contractor's Fabricator/Detailer or Erector intends other procedures or outcomes, the approach shall be confirmed with the Owner's Engineer, and any documentation required for approval shall be provided at no cost to the Owner. Any required submissions should be made and approved prior to the submission of the shop drawings for approval.

C11.8.3.6.2

The provisions of Article 6.7.2, "Dead Load Camber," of the *AASHTO LRFD Bridge Design Specifications* contain general instructions to the Engineer regarding the camber of girders in straight skewed I-girder bridges and horizontally curved I-girder bridges with and without skewed supports.

Camber is difficult to measure on curved girders. The vertical camber of I-girders may best be measured by laying the girder sections on their sides with the webs horizontal if twist camber is not provided. Camber may also be checked by blocking the girders into their no-load positions with their webs plumb. If a twist is cambered into the girders, the vertical camber can best be checked by blocking the girders to the near vertical position while maintaining the no-load out-of-plumb orientation of the webs along the girder length.

11.8.3.6.3—Sweep

Sweep tolerance shall meet the requirements of the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*. Sweep tolerance shall be measured radially from the theoretical curve of the girder.

C11.8.3.6.3

The theoretical curve may be a constant radius, a compound radius, or a spiral. The theoretical offsets from a chord can be computed so that deviations from the theoretical offsets can be compared to the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code* (2002) permissible values for sweep of a straight girder. These measurements should be made with the girder vertical and in the no-load condition.

11.8.3.6.4—Girder Lengths 2015 Revision

Girder lengths shall be determined based on an ambient temperature of 68°F. Girder length shall be measured along the arc.

C11.8.3.6.4

Girder length is important with respect to location of anchor bolts. If a laser instrument, which is free from temperature effect, is used to survey either the anchor bolt locations and/or the girder length, it is important that compensation be made for temperature of the girder.

11.8.3.7—Fit-Up

11.8.3.7.1—General 2014 Revision

Fit-up of girder sections shall meet the provisions of Article 11.5.3, "Preassembly of Field Connections," of these Specifications. Unless otherwise specified in the Contractor's construction plan, fit-up shall be assumed to be performed under the no-load condition.

When numerically controlled drilling is employed, trial fit-up of cross-frames or diaphragms between properly positioned girder sections shall be performed as specified in Article 11.5.3.3 of these specifications.

Shop fit-up of bolted connections in load-carrying connections of cross-frames and diaphragms to the girders may be required for structures with complex geometry or stiff elements.

C11.8.3.7.1

Article 11.5.3.1 requires that girder sections be fit up to ensure proper fit in the field. This requirement generally is applied only to girder splices on multistringer bridges. If numerically controlled drilling is used, only trial fit-up is usually required of cross-frames or diaphragms unless the construction plan specifies that full fit-up is required. In special instances, three sections of the bridge may need to be fitted at one time. This requirement would normally be applied to structures with particularly rigid or complex framing, or if numerically controlled drilling is not used.

11.8.3.7.2—Girder Section Field Splices

Field splices may be fit up in either the vertical or horizontal position. Girder splices may be fit up prior to heat-curving.

C11.8.3.7.2

Fit-up of girder splices can be performed in the same manner as for straight girders in most cases if the girder is to be heat-curved. The web can have either a vertical or a horizontal orientation. If the flanges are cut-curved, fit-up of I-girder splices is usually performed with the flanges horizontal after welding of the flanges to the web.

The design vertical camber must be in the girder when girder splices are fit.

11.8.4—Transportation Plan

A Transportation Plan may be required by the Owner for complex or large structures. The type of girder supports required and their locations shall be identified. The types, size, and locations of tie-downs shall be shown. A sufficient number of tie-downs shall be specified to provide adequate redundancy.

Girder stresses due to selfweight while being shipped shall be computed with a dynamic load allowance of 100 percent.

The computed girder stresses shall satisfy the provisions of Article 6.10.3.2 or 6.11.3.2 of the *AASHTO LRFD Bridge Design Specifications*, as applicable.

Fatigue stresses shall not exceed the constant-amplitude fatigue threshold for the appropriate categories in Table 6.6.1.2.5-3 of the *AASHTO LRFD Bridge Design Specifications*.

Wherever practical, girder sections should be shipped in the same orientation as in the completed structure. Girders shall be supported in such a manner that their cross-section shape is maintained and through-thickness stresses are minimized.

Supports should be such to ensure that dynamic lateral bending stresses are controlled.

Temporary stiffening trusses or beams, if required to meet the requirements of this section, shall be specified in the Transportation Plan.

C11.8.4

A Transportation Plan may be required if the girder sections are heavier, wider, deeper, or longer than normally permitted by the selected transportation mode.

During transportation, the girders should not be subjected to stresses that could damage them by either overstressing or by fatigue. Fatigue can be caused by longitudinal stresses in the girders or by through-thickness stresses due to raking of the section. Where the stress range is less than the constant-amplitude fatigue threshold, the detail will theoretically provide infinite life. A 100 percent dynamic load allowance is provided to account for dropping the girders on rigid supports.

In lieu of any alternative criteria, for control of dynamic lateral bending stresses during shipment, the following limit on the length, L_c, in ft, of a single unbraced curved I-girder cantilever may be considered:

$$L_c \le 36.0\left(\frac{I}{w}\right)^{0.25} \quad \text{(C11.8.4-1)}$$

where:

I = moment of inertia of the I-girder cross-section about the vertical axis in the plane of the web (ft^4)

w = I-girder weight per unit length (kip/ft)

This suggested limit is provided to ensure that the first mode of vibration of the unbraced cantilever about the vertical axis in the plane of the web is greater than 5 Hz (Young and Budynas, 2002). Critical lengths for other frequencies may be determined by changing the constant in Eq. C11.8.4-1 from 36.0 to $36.0(5/f)^{1/2}$, where f is the desired lower-bound natural frequency in Hz.

11.8.5—Steel Erection

11.8.5.1—General

Erection shall be performed in accordance with the Contractor's construction plan as approved by the Engineer.

Factored stresses due to selfweight of the steel and wind at each stage of erection shall not exceed those computed according to the provisions of Article 6.10.3 or 6.11.3, as applicable, of the *AASHTO LRFD Bridge Design Specifications* and Article 11.6.4.2 of these Specifications.

Reaming of bolt holes during erection shall be permitted only with the approval of the Engineer.

The bolted girder splices shall be field assembled according to the provisions of Article 11.6.5 of these Specifications.

C11.8.5.1

Article 11.2.2 requires that the Contractor supply erection drawings showing how the bridge will be erected, including falsework. Calculations are required to show that allowable stresses in the steel are not exceeded during erection. Steel that does not fit in the field implies that stresses in the steel are inconsistent with those computed in design. Excessive stresses may be relieved by temporary supports. If reaming is necessary, it should be done only after the resulting stress state and the deflections have been investigated.

The use of an adequate number of pins and bolts in girder splices during fit-up, as specified in Article 11.6.5 of these Specifications, is important.

11.8.5.2—Falsework

Falsework shall be designed to carry vertical and lateral loads that are specified in the construction plan. The elevation of falsework shall be such as to support the girders at the cambered no-load elevation. Jacks used in conjunction with the falsework shall have a stroke adequate to permit full unloading. Unloading of temporary supports should be performed such that all temporary supports at each cross-section are unloaded uniformly.

Where appropriate, cranes may be substituted for falsework.

C11.8.5.2

Temporary supports are more often employed for curved girders than for straight girders of similar span because of the need to provide stability to the curved girders.

The elevation of the temporary supports must allow for deflection of the erected steel after the temporary support(s) are removed. All jacks at a location should be released at once through a manifold arrangement to minimize twisting of the steel.

11.8.5.3—Bearings

Computed bearing rotation during construction shall not exceed the rotational capacity of the bearing. Bearings shall be installed such that, after dead load has been applied, sufficient rotation capacity shall be available to accommodate rotations due to environmental loads and live load. Expansion bearings shall be installed so that they will be in the center of the permitted travel at the ambient temperature of 60°F unless otherwise specified by the Owner.

C11.8.5.3

During erection, the girder may be rotated beyond the rotational capacity of the bearing even if the load at the time is within permitted limits. Skewed structures are particularly susceptible to twisting about the longitudinal axis of the girders.

11.8.5.4—I-Girders

The Contractor shall ensure that girders are stable throughout the erection process. The stage of completeness of the bolted connections shall be considered when evaluating the strength and stability of the steel during erection.

C11.8.5.4

Torsional restraint of curved I-girders is required at all times. Instability is manifest in greater lateral movement and rotation about the longitudinal axis of the girder that cannot be maintained by internal bracing.

Stability of curved I-girders with large unbraced length is not well predicted with present theories. Stability can be determined simply by test lifting of the girder. Where practical, it is best to keep the unbraced length of curved I-girders within the limits specified in Article 6.7.4.2, "Straight I-Sections," of the *AASHTO LRFD Bridge Design Specifications*.

11.8.5.5—Closed Box and Tub Girders

The Contractor shall ensure that the cross-section shape of each box is maintained during erection.

Top lateral bracing should be installed in tub girders prior to shipping and erection of the field pieces.

C11.8.5.5

Erection of closed box and tub girders is complicated by their large torsional stiffness. Shop fit-up of external diaphragms and cross-frames is important because the torsional stiffness of the box makes field adjustment difficult.

11.8.6—Deck

11.8.6.1—Forms

11.8.6.1.1—General

Plywood, permanent metal forms, or concrete panels may be used as deck forms as approved by the Owner. Proprietary forms shall be placed in accordance with the Manufacturer's specifications incorporating any modifications to those Specifications approved by the Engineer. Form work shall be supported by the superstructure.

C11.8.6.1.1

Deck forms should be attached firmly to the top flange in a manner approved by the Engineer. The forms should not be considered to have adequate stiffness to act as bracing for curved flanges.

11.8.6.1.2—Overhangs

Overhang forms shall be removed after the deck has cured. Wherever practical, overhang brackets should bear near the bottom flange and be attached to the top flange. If overhang brackets bear against the web, the Contractor's Engineer shall ensure that precautions have been taken to prevent permanent deformation of the web and excessive deflection of the wet slab and forms. The lateral force on the top flange due to overhang brackets shall be investigated to ensure that the flange is adequate as specified in Article 6.10.3.4, "Deck Placement," of the *AASHTO LRFD Bridge Design Specifications*.

Loads applied on the overhang brackets shall be considered in determining lateral flange bending stresses, cross-frame forces, and associated web and top flange deformations. If the loads or their application are to be different than those provided for in the contract documents, an additional analysis shall be made by the Contractor and approved by the Engineer.

C11.8.6.1.2

Overhang forms are usually removed when the concrete has hardened.

Concrete and other loads on the overhangs cause eccentric loading on the girder. The result is additional torsional forces on the exterior girder.

11.8.6.1.3—Tub Girders

Deck forms shall not be supported at locations other than girder flanges unless specifically considered in the design.

C11.8.6.1.3

Since it is extremely difficult to remove deck forms from inside tub girders, permanent deck forms are desirable.

Debris should not be allowed to remain in the box because it obstructs subsequent inspection.

11.8.6.2—Placement of Concrete

Concrete placements shall be made in the sequence specified in the approved construction plan. The time between placements shall be such that the concrete in prior pours has reached an age or strength specified in the construction plan. Any accelerating or retarding agents to be used in the concrete mix shall be specified.

The duration of each placement shall be specified in the construction plan. Placements that include both negative and positive dead load moment regions should be placed such that the positive moment region is poured first.

C11.8.6.2

When concrete is placed in a span adjacent to a span that already has a hardened deck, negative moment in the adjacent span causes tensile stresses and torsional shear stress in the cured concrete.

If long placements are made such that the negative moment region is poured first, it is possible that this region will harden and be stressed in tension during the remainder of the placement. This may cause early cracking of the deck.

It has been determined that placed concrete obtains composite action in a matter of hours. Therefore, the appropriate age and strength of the freshly placed concrete should be determined in part by the stress that will be induced during subsequent deck section placements.

11.8.7—Reports

Any modifications to the construction plan in the field from the original plan shall be documented with appropriate approvals noted.

11.9—Ultrasonic Impact Treatment (UIT)

11.9.1—General

C11.9.1

Typical post-weld treatment includes ultrasonic impact treatment, heat treatment, grinding, weld toe remelting (GTAW dressing or plasma arc dressing), hammer peening, and shot peening.

Based on research performed at Lehigh University, the fatigue strength of welded connections can be improved by post weld Ultrasonic Impact Treatment (S. Roy, et al., 2005; S. Roy, and J.W. Fisher, 2005). In general, the objective of UIT is to plastically deform the material at the weld toes and introduce residual compressive stresses substantially greater than the largest anticipated tensile stresses to a depth of no less than 0.02 in.

Treatment is necessary only at the weld toe. The weld toe is defined as the interface of the weld metal and base metal. Due to the finite size of the UIT needles and the recommended treatment procedure, some base metal adjacent to the weld toe will get treated.

To successfully accomplish the treatment, the following sample procedure should be included in the special provisions of the contract documents, unless otherwise recommended by the UIT equipment Manufacturer:

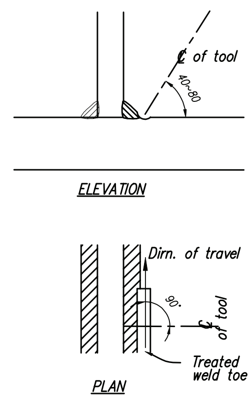


Figure C11.9.1-1—UIT Treatment along Fillet Weld

1. Position the UIT tool tip at the weld toe in a manner that both the weld metal and base metal are equally treated. This will normally be achieved by holding the tool needles in contact with and centered about the weld toe and gently rocking the tool back and forth normal to the longitudinal weld axis up to 10° from the mean position. The inclination of the tool will depend on the condition of the as-welded toe, and in most cases, the tool axis should lie between 40° and 80° with respect to the base metal surface in the plane of the cross-section to satisfy this requirement, as shown in the elevation view.

2. Sufficient force must be applied to keep the hand-held tool in position and to prevent jumping or unsteady movement when the vibrating needles come in contact with the weld toe. The self-weight of the tool is sufficient for this purpose and it is not necessary for the operator to exert undue force to achieve the required treatment.

3. The treatment shall progress at a travel speed of 1.0 to 5.0 ft/min in multiple short passes. At least five passes shall be made to ensure completeness of the treatment. In addition, the tool axis must be maintained at 90° with the direction of travel.

4. The diameter of the indenters influences the resulting appearance of the treated surface. In general, the smaller the diameter, the greater is the likelihood of eliminating the original weld toe. Thus, the treatment at the weld toe shall be carried out using 0.12 in. diameter pins. Pin holders accommodating four pins at various inclinations are generally used, except at narrow re-entrant corners, where a special pin holder having only one 0.2 in. diameter pin must be used.

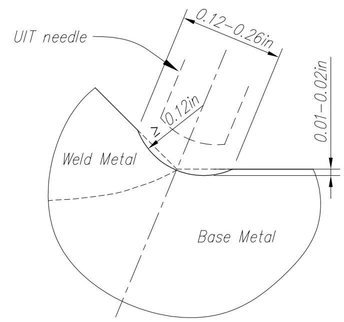


Figure C11.9.1-2—At Toe of Fillet

5. The treatment shall result in a uniform groove appearance without a trace of the original weld toe and shall be free from indentation marks produced by individual needle to toe impact. All sharp transitions, undercuts, and re-entrant corners along the weld toe shall be removed. The groove shall have a bright metallic surface and ideally have a notch radius of 0.12 in. or greater. Dimensional limits have been recommended by the Manufacturer on the width and depth of the groove cross-section, which are essentially related to the groove radius. Since it is difficult to make any accurate measurement of the treated toe, a visual determination of the depth of indentation below the plate surface of 0.010 to 0.020 in. is recommended as a guide, but not as a requirement of sufficient treatment.

6. The extent of treatment and general uniformity of appearance must be checked visually with the aid of a 10× magnifying glass.

In general, it is not possible to verify whether UIT has been performed correctly by visual inspection alone. Although features such as removal of original weld toe or surface finish can be described qualitatively as indications of complete treatment, the extent of plastic deformation, which is responsible for inducing the beneficial compressive residual stresses, cannot be ascertained. An acceptable treatment procedure should be established by performing trials on materials to be treated, preferably containing a representative weld. It is important to ensure that the equipment is functioning properly and operating at the correct settings. For a UIT tool operating at 27000 Hz, and for steel materials having nominal yield stress less than or equal to 100 ksi, the intensity of treatment should be maintained by setting the output power level of the generator to produce electronically displayed oscillation amplitudes of the transducer under an unloaded condition and during impact to 0.0014 and 0.0012 in., respectively.

Since the benefits of UIT are primarily derived from the introduction of compressive residual stresses, it is critical that other processes such as heat treatment including hot dipped galvanizing or overloading that might relieve these stresses are avoided after the toe treatment is performed. If the structure is going to be subjected to a significant dead load when in use, it is advisable that this be taken into consideration and the structure be loaded prior to treatment. An enhancement corresponding to the lower stress ratio may be obtained in that case, depending on the live load stress ratio of the nominal stress cycles applied after the treatment.

11.9.2—Procedure

UIT shall be accomplished in accordance with the equipment Manufacturer's recommendations, except as specifically described in the contract documents.

The use of UIT shall be limited to base metal with a specified minimum yield strength of 100 ksi or less.

Prior to UIT, the weld toe to receive UIT shall be visually inspected and magnetic particle-tested for conformance to the quality standards specified in the contract documents. The instrument shall be calibrated against the maximum flaw size allowed. Discontinuities greater than 1/32 in. shall be repaired satisfactorily prior to initiating the work.

UIT shall be done along the toe of the weld in a manner that will cause the center of the resultant treatment groove to be at the weld toe, with equal treatment and smooth transition to the adjacent weld and base metal.

UIT shall be performed so as to result in a uniform groove with a distinct and uniform bright metallic surface, as verified by visual inspection immediately after completion. When viewed under a 10x magnifying glass, the groove shall be free from any visible indications of the untreated base metal or weld metal. If such untreated indications exist within the treated area, such area shall be retreated to obtain a uniform, bright metallic surface across the entire surface of the UIT groove.

11.9.3—Other Weld Metal and Base Metal Treatments

Other weld or base metal treatments after UIT, including heat treatment of the treated UIT, shall not be allowed.

11.10—REFERENCES

2010 Revision 2011 Revision 2012 Revision 2014 Revision 2015 Revision

AASHTO. 2002. *Standard Specifications for Highway Bridges*, 17th Edition, HB-17, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2003. *Guide Specifications for Highway Bridge Fabrication with HPS70W Steel*, HBF-2, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2009. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 29th Edition, HM-29, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO and AWS. 2008. AASHTO/AWS D1.5M/D1.5:2008 *Bridge Welding Code*, Fifth Edition, BWC-5, American Welding Society, Washington, DC.

AISC. 2003. *LRFD Manual of Steel Construction*, 3rd Edition, American Institute of Steel Construction, Chicago, IL.

AISC. 2005. *Steel Construction Manual*, 13th Edition, American Institute of Steel Construction, Chicago, IL.

AISC Quality Certification Program, American Institute of Steel Construction, Chicago, IL, Category I: Structural Steel and Category III: Fracture-Critical. See http://www.aisc.org.

ASCE. 1970. "Experimental Stresses and Strains from Heat Curving," *Journal of the Structural Division*, Volume 96, No. ST7, American Society of Civil Engineers, New York. Journal published since 1983 (vol. 109) under the title *Journal of Structural Engineering*.

ASCE. 1970. "Theoretical Stresses and Strains from Heat Curving," *Journal of the Structural Division*, Volume 96, No. ST7, American Society of Civil Engineers, New York. Journal published since 1983 (vol. 109) under the title *Journal of Structural Engineering*.

ASCE. 1970. "Criteria for Heat Curving Steel Beams and Girders," *Journal of the Structural Division*, Volume 96, No. ST7, American Society of Civil Engineers, New York. Journal published since 1983 (vol. 109) under the title *Journal of Structural Engineering*.

ASME. Daniels, J. H., and R. P. Bacheler. 1979. *Fatigue of Curved Steel Bridge Elements: Effect of Heat Curving on the Fatigue Strength of Plate Girders*, Report No. FHWA-RD-79-136, Federal Highway Administration, August 1979, Washington, DC.

FHWA. 1999. *Summary of High-Performance Steel Grade 70W Studies*, Demonstration Project No. TE-50, High-Performance for Bridges, Turner-Fairbank Highway Laboratories, Federal Highway Administration, McLean, VA, October 1999.

Hilton, M. H. 1984. "Deflections and Camber Loss in Heat-Curved Girders." *Transportation Research Record 950*, Vol. 2, Transportation Research Board, Washington, DC, pp. 51–59.

NSBA. 2007. *Steel Bridge Erection Guide Specification*, S10.1, National Steel Bridge Alliance, Chicago, IL. See also AASHTO NSBASBEGS-1, 2007.

NSBA. 2008. *Steel Bridge Fabrication Guide Specifications*, S2.1, National Steel Bridge Alliance, Chicago, IL. See also AASHTO NSBASBF-2, 2008.

Research Council on Structural Connections (RCSC). 2004. *Specification for Structural Joints Using ASTM A325 or A490 Bolts.* American Institute of Steel Construction, Chicago, IL.

S. Roy, J. W. Fisher, and B. T. Yen. 2005. *Post-weld Enhancement in Fatigue Strength of Transverse Stiffeners and Cover-plates using Ultrasonic Impact Treatment (UIT)*, Draft Report of FHWA. Report No. 05-12, ATLSS Engineering Research Center, Lehigh University, Bethlehem, PA.

S. Roy, J. W. Fisher. 2005. "Enhancing Fatigue Strength by Ultrasonic Impact Treatment." *International Journal of Steel Structures,* Vol. 5, pp. 241–252.

SSTC. 1996. *Structural Bolting Handbook*, SBH-1, Steel Structures Technology Center, Inc., Novi, MI.

U.S. Steel. 2001. *Fabrication Aids for Continuously Heat-Curved Girders*, ADUSS 88-5538-01, United States Steel Corporation, Pittsburgh, PA.

U.S. Steel. 2002. *Fabrication Aids for Girders Curved with V-Heats*, ADUSS 88-5539-02, United States Steel Corporation, Pittsburgh, PA.

Wilson, P. J., R. R. Duncan, III, and J. W. Fisher. 1988. Repair of Fatigue Cracks in Steel Box Girder Bridges on I-110. *Proceedings of the 5th Annual International Bridge Conference*. Paper IBC-88-44, Pittsburgh, PA, pp. 234–241.

W. C. Young, and R. G. Budynas. 2002. *Roark's Formulas for Stress and Strain,* 7th Edition, McGraw-Hill, New York, NY.

Back to 2010 Edition

SECTION 11: STEEL STRUCTURES

11.3.2.6—Load-Indicator Devices

2011 Revision 2014 Revision

Add the following to the end of paragraph 1 of this Article:

Load-indicating devices which are incorporated into assemblies with hardened heavy hex AASHTO M 291 (ASTM A563) (AASHTO M 291M (ASTM A563M)) Grade DH nuts shall be considered permissible for use, provided both the load-indicating device and heavy hex nut meet the mechanical property requirements of their respective ASTM standards.

Insert new Commentary to this Article:

C11.3.2.6

An assembly comprised of a load-indicating device affixed to a hardened heavy hex structural nut by the fastener manufacturer is referred to herein as a captive DTI/nut.

Page left intentionally blank.

11.4.9.1—General

Revise the Article as follows.

Pins and rollers shall be accurately turned to the dimensions shown on the drawings and shall be straight, smooth, and free from flaws. Pins and rollers more than 9.0 in. ~~(225 mm)~~ in diameter shall be forged and annealed <u>or normalized and tempered</u>. Pins and rollers 9.0 in. ~~(225 mm)~~ or less in diameter may be either forged and annealed or normalized and tempered or may be cold-finished carbon-steel shafting.

In <u>annealed</u> pins larger than 9.0 in. ~~(225 mm)~~ in diameter, a hole not less than 2.0 in. ~~(50 mm)~~ in diameter shall be bored full-length along the axis after the forging has been allowed to cool to a temperature below the critical range, under suitable conditions to prevent injury by too rapid cooling, and before being annealed.

<u>Normalized and tempered pins do not require the bore hole along the full-length of the axis.</u>

Insert new commentary to this Article:

<u>C11.4.9.1</u>

<u>The ASTM A668 Class C, D, and G material has the same requirements for strength, ductility, and stress-relieved state in the normalized and tempered condition as in the annealed condition. Normalizing improves the grain size and toughness for larger diameter forgings and tempering relieves residual stresses incurred from the normalizing or quenching processes.</u>

<u>Boring a full-length hole along the axis will not result in any benefit in properties for normalized and tempered pins since any residual stresses that remain after normalizing are released by the tempering process.</u>

Page left intentionally blank.

11.4.12.2.6—Camber

Revise the Article as follows:

Girders shall be cambered before heat-curving. Camber for rolled beams may be obtained by cold cambering (cold bending) or heat-cambering methods approved by the Engineer. For plate girders, the web shall be cut to the prescribed camber with suitable allowance for shrinkage due to cutting, welding, and heat-curving. However, subject to the approval of the Engineer, moderate deviations from specified camber may be corrected by a carefully supervised application of heat.

Insert new commentary to this Article:

C11.4.12.2.6

Cold cambering, or introduction of camber by cold bending, is a customary means of achieving camber in rolled beams. While all steel bending operations, heated and cold (ambient temperature), alter steel base metal properties to some extent, the relatively small strains associated with cold cambering result in minimal effect. To avoid impact damage to the steel, it is appropriate to introduce bending pressure in a controlled fashion. For a useful reference, see "Cold Bending of Wide-Flange Shapes for Construction", Reidar Bjorhovde, Engineering Journal, Fourth Quarter, 2006.

Page left intentionally blank.

11.5.6.4.1—General 2014 Revision 2015 Revision

Revise paragraph 3, sentences 1 and 2 as follows:

To perform the calibrated wrench verification test for short grip bolts, direct tension indicators (DTI) <u>or captive DTI/nuts</u> with solid plates may be used in lieu of a tension-measuring device. The DTI <u>or captive DTI/nut</u> lot shall be first verified with a longer grip bolt in the Skidmore-Wilhelm Calibrator or an acceptable equivalent device.

Revise paragraph 4, sentence 1 as follows:

Bolts and nuts together with washers <u>or captive DTI/nuts</u> of size and quality specified in the contract documents, located as required below; shall be installed in properly aligned holes and tensioned; and inspected by any of the installation methods described in Articles 11.5.6.4.4 through 11.5.6.4.7 to at least the minimum tension specified in Table 11.5.6.4.1-1.

2010 Revision

Page left intentionally blank.

Back to 2010 Edition

11.5.6.4.2—Rotational-Capacity Tests 2014 Revision

Revise the Article as follows:

Rotational-capacity testing is required for all fastener assemblies, except for lock-pin and collar type fastener assemblies. Assemblies specified as galvanized shall be tested after galvanizing. Washers shall be required as part of the test even though they may not be required as part of the installation procedure. The following shall apply:

- Except as modified herein, the rotational-capacity test shall be performed in accordance with the requirements of AASHTO M 164 (ASTM A325) (AASHTO M 164M (ASTM A325M)).
- Each combination of bolt production lot, nut lot (or captive DTI/nut lot), and washer lot comprise one "assembly lot" for the purposes of testing and shall be tested as an assembly. Where washers are not required by the installation procedures, they need not be included in the assembly lot testing or lot identification.
- A rotational-capacity lot number shall have been assigned to each combination of component lots tested. Each qualified combination of component lots shall be assigned a unique rotational capacity assembly lot number.
- The minimum frequency of testing shall be two assemblies per rotational-capacity assembly lot.
- For bolts that are long enough to fit in a Skidmore-Wilhelm Calibrator, the bolt, nut (or captive DTI/nut), and washer assembly shall be assembled in a Skidmore-Wilhelm Calibrator or an equivalent device.
- Bolts that are too short to test in a Skidmore-Wilhelm Calibrator may be tested in a steel joint. The tension requirement, in the section below, need not apply. The maximum torque requirement, torque $\leq 0.250PD$, shall be computed using a value of P equal to the turn test tension taken as 1.15 times the bolt tension in Table 11.5.6.4.1-1. The assembly shall be tightened two times the required number of turns indicated in Table 11.5.6.4.1-2 after snugging. After the required turn has been completed, it shall be verified that the maximum torque requirement has not been exceeded.
- The tension reached at the rotation below, i.e., the turn-test tension, shall be equal to or greater than 1.15 times the bolt ~~required fastener~~ tension~~, i.e., installation tension, shown~~ in Table 11.5.6.4.1-1.
- In a tension-measuring device, the minimum rotation from an initial tension of ten percent of the minimum required tension shall be two times the required number of turns indicated in Table 11.5.6.4.1-2 without stripping or failure. Minimum rotation for bolt and captive DTI/nut combination assemblies shall be the required number of turns indicated in Table 11.5.6.4.1-2 beyond the turn after the gaps in the DTI are closed.
- After the required installation tension listed above has been exceeded, one reading of tension and torque shall be taken and recorded. The torque value shall conform to the following:

$$\text{Torque} \leq 0.250PD \qquad (11.5.6.4.2\text{-}1)$$

where:

Torque = measured torque ft·lb (N·mm)
P = measured bolt tension, lb (N)
D = bolt diameter, ft (mm)

C11.5.6.4.2

Add the following paragraph at the beginning of this Article:

An assembly lot is defined as a combination of fastener components of different types which are configured as they are to be installed in the steel. An example would be a bolt, nut, and washer. Each component in an assembly lot will have come from a production lot of similar components. Any change in component lots warrants additional testing of the assembly lots into which the component lots are integrated.

2010 Revision

Page left intentionally blank.

11.5.6.4.7—Direct Tension Indicator Installation Method

Revise the last sentence of this Article as follows:

Installation of a DTI under the turned element may only be permitted if either a washer is used to separate the turned element from the DTI or verification testing in accordance with the provisions of this Article and the Manufacturer's instructions demonstrates satisfactory performance without the washer under the turned element.

Page left intentionally blank.

11.10—REFERENCES

2011 Revision 2012 Revision 2014 Revision 2015 Revision

Add the following reference:

Reidar, B, 2006. "Cold Bending of Wide-Flange Shapes for Construction," *AISC Engineering Journal*. American Institute of Steel Construction, Chicago, IL, Vol. 4.

Back to 2010 Edition

SECTION 11: STEEL STRUCTURES

11.3.2.6—Load-Indicator Devices

2010 Revision 2014 Revision

Revise paragraph 1 of this Article:

Load-indicating devices may be used in conjunction with bolts, nuts and washers specified in Article 11.3.2.1. All load-indicating devices shall conform to the requirements of ASTM Specification for Compressible-Washer Type Direct Tension Indicators for Use with Structural Fasteners, ASTM F 959, except as provided in the following paragraph. Load-indicating devices which are incorporated into assemblies with hardened heavy hex AASHTO M 291 (ASTM A 563) (AASHTO M 291M (ASTM A 563M)) Grade DH nuts shall be considered permissible for use, provided both the load-indicating device and heavy hex nut meet the mechanical property requirements of their respective ASTM standards. Load-indicating devices which incorporate a self-indicating feature to signal sufficient bump compression shall also be considered permissible for use.

C11.3.2.6

Add the following sentence to the end of this Article:

An assembly which incorporates a self-indicating feature to signal sufficient bump compression is referred to herein as a self-indicating DTI.

11.5.6.4.7a—Verification

Back to 2010 Edition

Add the following sentence to the end of paragraph 2 of this Article:

In addition to the feeler gage test, the visual appearance of self-indicating DTIs shall be noted and compared with the Manufacturer's written instructions to verify the self-indicating feature of the DTI.

11.5.6.4.7b—Installation

Back to 2010 Edition

Revise paragraph 2 of this Article:

For uncoated DTIs used under a stationary or turned element and for coated DTIs used under a stationary element, the bolts shall be further tensioned until the number of refusals of the 0.005-in. feeler gage shall be equal to or greater than the number listed under Minimum Installation Refusals in Table 11.5.6.4.7a-1. If the bolt is tensioned so that no visible gap in any space remains, the bolt and DTI shall be removed and replaced by a new properly tensioned bolt and DTI. For self-indicating DTIs, if the visual appearance of the installed DTI is also not in accordance with the Manufacturer's written instructions, the bolt and DTI shall be removed and replaced by a new properly tensioned bolt and DTI.

11.10—REFERENCES

Back to 2010 Edition 2010 Revision 2014 Revision 2015 Revision
2012 Revision

Add the following reference:

Szewczyk, M. 2009. "High Strength Bolt Tensioning Using ASTM F 959 Direct Tension Indicating Washers with Silicone Rubber as a Visual Indicator." A project submitted in conformity with the requirements of the degree of Masters in Engineering, Graduate Department of Civil Engineering, University of Toronto, Toronto, Ontario, Canada, 63 pp.

Back to 2010 Edition

SECTION 11: STEEL STRUCTURES

11.4.3.3.1—General

Revise the Article as follows:

~~Cold-bending of fracture-critical steels and fracture-critical members is prohibited. Perform cold-bending of other steels or members,~~ Fracture-critical and nonfracture-critical plates and bars shall be cold bent, unless otherwise permitted according to the provisions of Article 11.4.3.3.3. The bending shall be performed in accordance with the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code* and the provisions of Article 11.4.3.3.2~~Table 11.4.3.3.2-1 and in a manner such that no cracking occurs~~.

Insert new commentary to this Article:

C11.4.3.3.1

Research (Keating, 2012) which focused on the influence of heat and bending strain upon the toughness and mechanical properties of steel revealed that increasing the bend radius to 5.0 times the thickness of the plate and bending the material at room temperature did not significantly degrade the material properties of the plate. The previous bend requirements were based on preventing cracking of the plate from the bending operation. Heating of the plate is not necessary and was found to reduce the ductility of the plate after bending and increase the likelihood of cracking.

11.4.3.3.2—Cold-Bending

Back to 2010 Edition

Revise the Article as follows:

Unless otherwise permitted herein or approved by the Engineer, the minimum bend radii for cold-~~forming~~ bending (at room temperature), measured to the concave face of the plate, ~~are given~~ shall be taken as 5.0t for all grades and thicknesses of steel conforming to AASHTO M 270M/M 270 (ASTM A709/A709M) where t is the thickness of the plate in inches~~in Table 11.4.3.3.2-1~~. For cross-frame or diaphragm connection plates up to 0.75 in., the minimum bending radii may be taken as 1.5t. ~~If a smaller radius is required, heat may need to be applied as a part of the bending procedure. Provide the heating procedure for review by the Engineer.~~ For all other grades of steel~~not included in Table 11.4.3.3.2-1~~, ~~follow~~ the minimum bend radii recommendations ~~of~~ from the plate producer shall be followed but the radii shall not be less than the minimums specified herein.

~~If~~ Wherever possible, ~~orient~~ bend lines shall be oriented perpendicular to the direction of final rolling of the plate. If the bend lines ~~is~~ are parallel to the direction of final rolling, ~~multiply~~ the ~~suggested~~ minimum bend radii ~~in Table 11.4.3.3.2-1~~ shall be increased to 7.5t ~~by 1.5~~.

~~Table 11.4.3.3.2-1—Minimum Cold-Bending Radii~~

	~~Thickness, in. (*t*)~~			
~~AASHTO M 270M/M 270 (ASTM A709/A709M) Grades, ksi~~	~~Up to 0.75~~	~~Over 0.75 to 1.0, incl.~~	~~Over 1.0 to 2.0, incl.~~	~~Over 2.0~~
~~36~~	~~1.5*t*~~	~~1.5*t*~~	~~1.5*t*~~	~~2.0*t*~~
~~50, 50S, 50W, or HPS 50W~~	~~1.5*t*~~	~~1.5*t*~~	~~2.0*t*~~	~~2.5*t*~~
~~HPS 70W~~	~~1.5*t*~~	~~1.5*t*~~	~~2.5*t*~~	~~3.0*t*~~
~~HPS 100W~~	~~1.75*t*~~	~~2.25*t*~~	~~4.5*t*~~	~~5.5*t*~~

2012 Revision

11.4.3.3.3—Hot-Bending

Revise the Article as follows:

~~If a radius shorter than the minimum specified for cold bending is essential, the plates shall~~Fracture-critical and nonfracture-critical plates and bars may be bent hot subject to the approval of the Engineer. If hot-bending is to be employed, the heating and bending procedure shall be submitted for review and approval by the Engineer. ~~at a~~The plates and bars shall be bent hot at a temperature above the blue brittle temperature of steel (700°F) but not greater than l200°F, except for AASHTO M 270M/M 270 (ASTM A709/A709M), Grades HPS 70W and HPS 100W (Grades HPS 485W and HPS 690W) for which plates and bars shall be bent hot at a temperature not greater than l100°F. The minimum radii of the hot bend shall satisfy the requirements of Article 11.4.3.3.2.

Back to 2010 Edition

11.4.8.1.1—General 2014 Revision 2015 Revision

Revise paragraph 1 of this Article as follows:

Previous punching restrictions whenever the thickness of the material was not greater than 0.75 in. for structural steel, 0.625 in. for high strength steel, or 0.5 in. for quenched-and-tempered alloy steel, ~~are~~ were upper limits. Thickness of material that can be punched is often controlled by equipment capacity~~but punching equipment may be more restrictive~~.

Page left intentionally blank.

11.10—REFERENCES

2010 Revision 2011 Revision 2014 Revision 2015 Revision

Add the following reference:

Keating, P. B. and L. C. Christian. 2012. "Effects of Bending and Heat on the Ductility and Fracture Toughness of Flange Plate," FHWA/TX-10/0-4624-2. Texas Transportation Institute, Texas A&M University, Austin, TX.

Back to 2010 Edition

SECTION 11: STEEL STRUCTURES

11.1.1—Description 2015 Revision

Revise paragraph 3 of this Article:

Details of design which are permitted to be selected by the Contractor shall conform to the provisions of the current *AASHTO LRFD Bridge Design Specifications*~~, 2007~~.

Page left intentionally blank.

11.3.2.1—Material

Delete AASHTO M 160, M 164, M 253, and M 291 citations in the Article and Commentary as follows:

High-strength bolts for structural steel joints shall conform to either ~~AASHTO M 164 (~~ASTM A325~~)~~ or ~~AASHTO M 253 (~~ASTM A490~~)~~. When high-strength bolts are used with unpainted weathering grades of steel, the bolts shall be Type 3.

Type 2 bolts have been withdrawn from ~~AASHTO M 164 (~~ASTM A325~~)~~ and ~~AASHTO M 253 (~~ASTM A490~~)~~ and, therefore, are no longer manufactured. However, Type 2 bolts manufactured before this discontinuation may still be in inventory and are considered acceptable.

The supplier shall provide a lot of number appearing on the shipping package and a certification noting when and where all testing was done, including rotational capacity tests, and zinc thickness when galvanized bolts and nuts are used.

The maximum hardness for ~~AASHTO M 164 (~~ASTM A325~~)~~ bolts shall be 33 HRC.

Proof-load tests (ASTM F606, Method 1) shall be required for the bolts. Wedge tests of full-size bolts are required in accordance with Section 8.3 of ~~AASHTO M 164 (~~ASTM A325~~)~~. Galvanized bolts shall be wedge tested after galvanizing. Proof-load tests of ~~AASHTO M 291 (~~ASTM A563~~)~~ are required for the nuts. The proof-load tests for nuts to be used with galvanized bolts shall be performed after galvanizing, overtapping, and lubricating.

Except as noted below:

- Nuts for ~~AASHTO M 164 (~~ASTM A325~~)~~ bolts shall conform to ~~AASHTO M 291 (~~ASTM A563~~)~~, Grades DH, DH3, C, C3, and D (Property Class 8S, 8S3, 10S, or 10S3).

- Nuts for ~~AASHTO M 253 (~~ASTM A490~~)~~ bolts shall conform to the requirements of ~~AASHTO M 291 (~~ASTM A563~~)~~, Grades DH and DH3 (Property Class 10S or 10S3).

Grade DH (Property Class 10S) nuts are recommended for all Type 1 and Type 2 bolts. Grade DH3 (Property Class 10S3) nuts are not recommended for Type 1 and Type 2 bolts. Grade DH3 (Property Class 10S3) nuts must be used for Type 3 bolts.

The exceptions are:

- Nuts to be galvanized (hot-dip or mechanically galvanized) shall be Grade DH (Property Class 10S).

- Nuts to be used with ~~AASHTO M 164 (~~ASTM A325~~)~~ Type 3 bolts shall be Grade C3 or DH3 (Property Class 8S3 or 10S3). Nuts to be used with ~~AASHTO M 253 (~~ASTM A490~~)~~, Type 3 bolts shall be Grade DH3 (Property Class 10S3).

All galvanized nuts shall be lubricated with a lubricant containing a visible dye. Black bolts must be oily to touch when delivered and installed.

Washers shall be hardened steel washers conforming to the requirements of ~~AASHTO M 293 (~~ASTM F436~~)~~ and Article 11.5.6.4.3, "Requirements for Washers."

2014 Revision

Back to 2010 Edition

11.3.2.2—Identifying Marks

Delete AASHTO M 164 and M 253 citations in paragraphs 1 and 2 as follows:

~~AASHTO M 164 (~~ASTM A325~~)~~ for bolts and the specifications referenced therein for nuts require that bolts and nuts manufactured to the specification be identified by specific markings on the top of the bolt head and on one face of the nut. Head markings must identify the grade by the symbol "A325", the Manufacturer, and the type, if Type 3. Nut markings must identify the property class, the Manufacturer, and, if Type 3, the type. Markings on direct tension indicators (DTI, ASTM F959) must identify the Manufacturer and Type "325" (Class "8.8"). Other washer markings must identify the Manufacturer, and, if Type 3, the type.

~~AASHTO M 253 (~~ASTM A490~~)~~ for bolts and the specifications referenced therein for nuts require that bolts and nuts manufactured to the specifications be identified by specific markings on the top of the bolt head and on one face of the nut. Head markings must identify the grade by the symbol "A490", the Manufacturer, and the type, if Type 3. Nut markings must identify the property class, the Manufacturer and if Type 3, the type. Markings on direct tension indicators must identify the Manufacturer and Type "490" (Class "10.9"). Other washer markings must identify the Manufacturer, and, if Type 3, the type.

11.3.2.4—Galvanized High-Strength Fasteners

Delete AASHTO M 164 and M 253 citations in the Article and Commentary as follows:

~~AASHTO M 253 (~~ASTM A490~~)~~ bolts shall not be galvanized.

When fasteners are galvanized, they shall be specified to be hot-dip galvanized in accordance with ~~AASHTO M 232M/M 232 (~~ASTM A153/A153M~~)~~, Class C or mechanically galvanized in accordance with ~~AASHTO M 298 (~~ASTM B695*)*, Class 50 (Class 345). Bolts to be galvanized shall be either ~~AASHTO M 164 (~~ASTM A325) Type 1. Galvanized bolts shall be tension tested after galvanizing. Washers, nuts, and bolts of any assembly shall be galvanized by the same process. The nuts should be overtapped to the minimum amount required for the fastener assembly and shall be lubricated with a lubricant containing a visible dye so a visual check can be made for the lubricant at the time of field installation.

~~AASHTO M 164 (~~ASTM A325~~)~~ Type 2 bolts shall be mechanically galvanized only.

11.3.2.5—Alternative Fasteners

Back to 2010 Edition

Delete AASHTO M 164 and M 253 citations in paragraph 1 as follows:

Other fasteners or fastener assemblies, such as those conforming to the requirements of ASTM F1852, which meet the materials, manufacturing, and chemical composition requirements of ~~AASHTO M 164 (~~ASTM A325~~)~~ or ~~AASHTO M 253 (~~ASTM A490~~)~~ which meet the mechanical property requirements of the same specification in full-size tests, and which have body diameter and bearing areas under the head and nut, or their equivalent, not less than those provided by a bolt and nut of the same nominal dimensions prescribed in Article 11.3.2.3, may be used, subject to the approval of the Engineer. Such alternate fasteners may differ in other dimensions from those of the specified bolts and nuts.

11.3.2.6—Load-Indicator Devices

2010 Revision 2011 Revision

Delete AASHTO M 291 citation in paragraph 1as follows:

Load-indicating devices may be used in conjunction with bolts, nuts and washers specified in Article 11.3.2.1. All load-indicating devices shall conform to the requirements of ASTM Specification for Compressible-Washer Type Direct Tension Indicators for Use with Structural Fasteners, ASTM F959, except as provided in the following paragraph. Load-indicating devices which are incorporated into assemblies with hardened heavy hex ~~AASHTO M 291 (~~ASTM A 563~~) (AASHTO M 291M (~~ASTM A 563M)~~)~~ Grade DH nuts shall be considered permissible for use, provided both the load-indicating device and heavy hex nut meet the mechanical property requirements of their respective ASTM standards. Load-indicating devices which incorporate a self-indicating feature to signal sufficient bump compression shall also be considered permissible for use.

C11.4.1

Delete AASHTO M 160 citation in paragraph 2 as follows:

Color codes for steels, as noted in previous versions of the ~~AASHTO M 160M/M 160 (~~ASTM A6/A6M~~)~~ Specifications, may also be used for identification purposes. This method, which will no longer be shown in either of these Specifications, is also being eliminated by many Owners due to the complexity of the code with the many new material grades.

Page left intentionally blank.

11.4.5—Abutting Joints

Revise the Article as follows:

Abutting ends in compression members of trusses and columns shall be milled or saw-cut to give a square joint and uniform bearing. At other joints, not required to be faced, the opening shall not exceed ~~0.375~~ $\underline{^{3}/_{8}}$ in.

Page left intentionally blank.

11.4.8.1.1—General 2015 Revision 2012 Revision

Revise paragraph 1, sentence 2 as follows:

The width of each standard hole shall be the nominal diameter of the bolt plus ~~0.0625~~ $^{1}/_{16}$ in.

Revise paragraph 4 of this Article as follows:

When required, all holes shall be either subpunched or subdrilled ~~0.1875~~ $^{3}/_{16}$ in. smaller and, after assembling, reamed or drilled to full size.

C11.4.8.1.1

Revise paragraph 3, sentence 3 as follows:

See references AISC *Steel Construction Manual,* ~~13th Edition,~~ Section M2.5; RCSC *Specification for Structural Joints Using* ~~*ASTM A325 or A490*~~ *High-Strength Bolts,* Section 3.3; and NSBA *Steel Bridge Fabrication*, S2.1.

Page left intentionally blank.

11.4.8.1.4—Accuracy of Holes

Revise the Article as follows:

Holes not more than ~~0.03125~~ $^1/_{32}$ in. larger in diameter than the true decimal equivalent of the nominal diameter that may result from a drill or reamer of the nominal diameter shall be considered acceptable. The width of slotted holes which are produced by thermal cutting or a combination of drilling or punching and thermal cutting should be not more than ~~0.03125~~ $^1/_{32}$ in. greater than the nominal width. The thermally-cut surface shall be ground smooth to obtain a maximum surface roughness of ANSI 1000 μin.

Page left intentionally blank.

11.4.8.2.1—Accuracy before Reaming

Revise the Article as follows:

All holes punched full-size, subpunched, or subdrilled shall be so accurately punched that after assembling (before any reaming is done) a cylindrical pin ~~0.125~~ $^1/_8$ in. smaller in diameter than the nominal size of the punched hole may be entered perpendicular to the face of the member, without drifting, in at least 75 percent of the contiguous holes in the same plane. If the requirement is not fulfilled, the badly punched pieces shall be rejected. If any hole will not pass a pin ~~0.1875~~ $^3/_{16}$ in. smaller in diameter than the nominal size of the punched hole, this shall be cause for rejection.

Page left intentionally blank.

11.4.8.2.2—Accuracy after Reaming

Revise paragraph 1 as follows:

When holes are reamed or drilled, 85 percent of the holes in any contiguous group shall, after reaming or drilling, show no offset greater than ~~0.03125~~ $\underline{^{1}/_{32}}$ in. between adjacent thicknesses of metal.

Page left intentionally blank.

11.4.8.4—Holes for Ribbed Bolts, Turned Bolts, or Other Approved Bearing-Type Bolts

Revise the Article as follows:

All holes for ribbed bolts, turned bolts, or other approved bearing-type bolts shall be subpunched or subdrilled ~~0.1875~~ $^{3}/_{16}$ in. smaller than the nominal diameter of the bolt and reamed when assembled, or drilled to a steel template or, after assembling, drilled from the solid at the option of the Fabricator. In any case, the finished holes shall provide a driving fit as specified in the contract documents.

Page left intentionally blank.

11.4.9.2—Boring Pin Holes

Revise paragraph 2 as follows:

The diameter of the pin hole shall not exceed that of the pin by more than ~~0.02~~ $^{1}/_{50}$ in. for pins 5.0 in. or less in diameter, or by ~~0.03125~~ $^{1}/_{32}$ in. for larger pins.

Revise paragraph 3 as follows:

The distance outside to outside of end holes in tension members and inside to inside of end holes in compression members shall not vary from that specified more than ~~0.03125~~ $^{1}/_{32}$ in. Boring of pin holes in built-up members shall be done after the member has been assembled.

Page left intentionally blank.

11.4.12.1—General

Revise this Article as follows:

Flanges of curved, welded girders may be cut to the radii specified in the contract documents or curved by applying heat as specified in the succeeding articles~~ providing the radii is not less than allowed by Article 10.15.2, "Minimum Radius of Curvature," of the *AASHTO Standard Specifications for Highway Bridges*, 17th Edition, Design Specifications~~.

Page left intentionally blank.

Back to 2010 Edition

11.4.12.2.2—Type of Heating

11.4.12.2.3—Temperature

11.4.12.2.4—Position for Heating

11.4.12.2.5—Sequence of Operations

11.4.12.2.6—Camber

11.4.12.2.7—Measurement of Curvature and Camber

Renumber Articles 11.4.12.2.2 through 11.4.12.2.7 as 11.4.12.2.3 through 11.4.12.2.8.

Insert the following new Article:

11.4.12.2.2—Geometric Limitations

11.4.12.2.2a—Cross-Sectional Criteria

Rolled beams and constant depth welded I-section plate girders satisfying all of the following criteria may be heat curved to obtain a horizontal curvature:

- $R > 1{,}000$ ft if ($t_f > 3.0$ in.) or ($b > 30.0$ in.), otherwise $R > 150$ ft
- $\Psi \leq 2.0$
- $\Psi_f \geq 0.20$
- $t_{nf} \leq t_f$

in which:

$$\psi = \frac{b_{nf}t_{nf} + bt_f + D_w t_w}{b_{nf}t_{nf} + bt_f} \quad (11.4.12.2.2a\text{-}1)$$

$$\psi_f = \frac{b_{nf}t_{nf}}{bt_f} \leq 1.0 \quad (11.4.12.2.2a\text{-}2)$$

where:

R	=	horizontal radius of curvature measured to the centerline of the girder web (in., unless noted otherwise)
b	=	width of wider flange (in.)
b_{nf}	=	width of narrower flange (in.)
D_w	=	clear distance between flanges (in.)
t_f	=	thickness of wider flange (in.)
t_{nf}	=	thickness of narrower flange (in.)
t_w	=	web thickness (in.)

Insert the following new Article:

C11.4.12.2.2a

The development of the minimum radius limits in Articles 11.4.12.2.2b and 11.4.12.2.2c was limited to cases with ψ less than or equal to 2, ψ_f greater than or equal to 0.2, and t_{nf} less than or equal to t_f (Sause et al., 2013). The 150 ft limit is traditional.

Guidance on shop implementation of continuous and V-heating to heat curve girders is given in U.S. Steel (2001) and (2002).

Insert the following new Article:

11.4.12.2.2b—Minimum Radius for Doubly-Symmetric Beams and Girders

For heat-curved doubly-symmetric beams and girders, the horizontal radius of curvature measured to the centerline of the girder web, in inches, shall not be less than the following:

- $If\ \frac{D_w}{t_w} > \frac{592}{\sqrt{F_{yw}}}, \text{then :}$

$$R = 0.0365\frac{b}{\psi}\left(\frac{D_w}{t_w}\right)^2 \qquad (11.4.12.2.2b\text{-}1)$$

- Otherwise:

$$R = \frac{12{,}800b}{F_{yw}\psi} \qquad (11.4.12.2.2b\text{-}2)$$

where:

F_{yw} = specified minimum yield strength of a web (ksi)

C11.4.12.2.2b

The stress analysis performed by Brockenbrough (1970) to develop the equations in earlier editions of AASHTO specifications was based on the following assumptions:

- The heat curving process introduces heat continuously along the girder length, resulting in a heated portion of the flange that is the same at every cross-section along the length.
- The girder cross-section is a doubly-symmetric I-shaped section.
- A tensile shrinkage force, *P*, develops in the heated portion of each flange near the flange edge on the inside of the curve. *P* is eccentric to the girder centroid.
- The heated width of the flange is 0.20*b*, i.e. 20 percent of the flange width.
- The entire cross-section of the girder resists *P*, resulting in transverse bending stresses associated with the horizontal curvature about a vertical axis and axial compressive stresses on the cross-section. The entire cross-section remains elastic and plane sections remain plane.

The resulting equation for the compressive stress in the web due to heat curving, normalized to the yield stress, was

$$\frac{\sigma_w}{F_{yw}} = \left(\frac{-6{,}000}{F_{yw}}\right)\left(\frac{1}{\psi}\right)\left(\frac{1}{R/b}\right) \qquad (C11.4.12.2.2b\text{-}1)$$

The equations in this Article are based on numerical simulations that overcame three limitations of the stress analysis by Brockenbrough (Sause et al., 2013). These three limitations are:

1. The stress in the cross-section was permitted to exceed the yield stress,
2. The stress analysis was limited to doubly-symmetric cross-sections, and
3. The heated width was fixed at 0.20*b*, rather than varying with *R*.

The studies, which varied the heated flange width with *R* and considered the effect of yielding in the flanges, found that the compressive stress in the web due to heat curving could be adequately represented by the empirical equation of Eq. C11.4.12.2.2b-1, with the constant 6,000 replaced by 6,670 resulting in Eq. C11.4.12.2.2b-2 (Sause et al., 2013).

$$\frac{\sigma_w}{F_{yw}} = \left(\frac{-6,670}{F_{yw}}\right)\left(\frac{1}{\psi}\right)\left(\frac{1}{R/b}\right) \qquad \text{(C11.4.12.2.2b-2)}$$

With the stress in the web from the heat curving operation now quantified, limits on *R* were established following the process developed by Brockenbrough. The basis for Eq. C11.4.12.2.2b-1 is limiting stress in web to the buckling stress instead of the post-buckling strength previously used. Eq. C11.4.12.2.2b-2 is based on Von Mises yield criteria but revised assuming greater web shear stresses ($0.425F_y$) under current design practice and an allowable stress of $0.90F_y$, while the original development assumed a web shear stress of $0.33F_y$ and an allowable stress of F_y.

Insert the following new Article:

11.4.12.2.2c—Minimum Radius of Curvature for Singly-Symmetric Girders

For heat-curved singly-symmetric beams or girders, the horizontal radius of curvature measured to the centerline of the girder, in inches, shall not be less than the values calculated from Eqs. 11.4.12.2.2b-1 and 11.4.12.2.2b-2. Additionally, for singly-symmetric girders with ψ greater than or equal to 1.46 and with ψ_f less than ψ_{fo}, the radius shall not be less than that determined as:

$$R = \left[1.43\psi\left(1-\frac{\psi_f}{\psi_{fo}}\right)^2+1\right]\left(\frac{12800\ b}{F_{yw}\ \psi}\right) \qquad \text{(11.4.12.2.2c-1)}$$

in which:

$$\psi_{fo} = 0.68\psi - 0.79 \qquad \text{(11.4.12.2.2c-2)}$$

C11.4.12.2.2c

Singly-symmetric cross-sections were included in the study described in Article C11.4.12.2.2b with the limitation that the moment of the heated area of the narrower flange about the elastic neutral axis is equal to the moment of the heated area of the wider flange about the elastic neutral axis. Parametric studies demonstrated that Eq. C11.4.12.2.2b-2 was adequate for the singly-symmetric case as well, provided that the flange width in the equation is taken as the width of the wider flange.

The stress analysis of singly-symmetric heat-curved girders that was used to develop Eq. C11.4.12.2.2b-1 and Eq. C11.4.12.2.2b-2 is valid when *R* is greater than the radius at which the heated width of the narrower flange equals the flange half width. This limit on *R* is provided by Eq. 11.4.12.2.2c-1, which is also considered to be a practical limit on the radius of singly-symmetric heat curved beams and girders. For many cases, this limit on heated flange width is reached when the web stresses are quite high so it is not of practical concern, as the radius will be limited by the two equations in Article 11.4.12.2.2b. However, for highly unsymmetrical cases, this limit will be reached when the web stresses are not large, so that a limit on the radius of heat curved girders based on this limit on heated flange width is needed.

Insert the following new Article:

11.4.12.2.2d—Minimum Radius of Curvature for Hybrid Girders

Hybrid girders which meet the following criteria may be heat curved:

- $\eta_w \leq \eta_f$, and
- $\eta_w = \eta_f$ if $\eta_f < 1$
- $b_{yf} \geq b_{yfw}$ if $\eta_f < 1$

in which:

$$\eta_f = \frac{F_{yfw}}{F_{yf}} \quad \text{(11.4.12.2.2d-1)}$$

$$\eta_w = \frac{F_{yw}}{F_{yf}} \quad \text{(11.4.12.2.2d-2)}$$

where:

F_{yfw} = yield stress of flange with lower yield stress (ksi)
F_{yf} = yield stress of flange with higher yield stress (ksi)
F_{yw} = yield stress of web (ksi)
b_{yfw} = width of flange with lower yield stress (in.)
b_{yf} = width of flange with higher yield stress (in.)

For hybrid sections with $\eta_f = 1$ and $\eta_w < 1$, the horizontal radius of curvature measured to the centerline of the girder, in inches, shall not be less than the minimum radius determined from Articles 11.4.12.2.2b and 11.4.12.2.2c with F_{yf} substituted for F_{yw} in Eq. 11.4.12.2.2c-1.

For hybrid sections with $\eta_f < 1$, the horizontal radius of curvature measured to the centerline of the girder, in inches, shall not be less than the minimum radius determined from Article 11.4.12.2.2b. Additionally, for girders with ψ_{fo} greater than or equal to $0.2\sqrt{\eta_f}$ and with ψ_f less than $\frac{\psi_{fo}}{\sqrt{\eta_f}}$, the radius shall not be less than that determined as:

$$R = \left(1.43\left(1 - \frac{\psi_f\sqrt{\eta_f}}{\psi_{fo}}\right)^2 \cdot \psi + 1\right)\left(\frac{12800\, b}{F_{yw}\psi}\right) \quad \text{(11.4.12.2.2d-3)}$$

C11.4.12.2.2d

Hybrid cross-sections were also included in the study described in Article C11.4.12.2.2b with the limitation that the moment of the heated area of the weaker flange about the elastic neutral axis is equal to the moment of the heated area of the stronger flange about the elastic neutral axis.

The results of the study showed that Eqs. 11.4.12.2.2b-1 and 11.4.12.2.2b-2 are also valid for hybrid sections, subject to the limitations outlined in this Article. The stress analysis of hybrid singly-symmetric heat-curved girders that was used to validate Eqs. 11.4.12.2.2b-1 and 11.4.12.2.2b-2 is valid when *R* is greater than the radius at which the heated width of the weaker flange equals the flange half width. This limit on *R* is provided by Eq. 11.4.12.2.2d-3.

2014 Revision

11.4.13.2—Flatness of Panels

Revise paragraph as follows:

The maximum deviation, δ, from detailed flatness or curvature of a panel shall not exceed the greater of ~~0.1875~~ $\underline{^{3}/_{16}}$ in. or:

Page left intentionally blank.

11.5.5.3—Ribbed Bolts

Revise paragraph 1 as follows:

The body of ribbed bolts shall be of an approved form with continuous longitudinal ribs. The diameter of the body measured on a circle through the points of the ribs shall be ~~0.078125~~ $\underline{^{5}/_{64}}$ in. greater than the nominal diameter specified for the bolts.

Page left intentionally blank.

Back to 2010 Edition

11.5.6.1—General

Delete AASHTO M 164 and M 253 citations in paragraph 1 as follows:

This Article covers the assembly of structural joints using ~~AASHTO M 164 (~~ASTM A325~~)~~ or ~~AASHTO M 253 (~~ASTM A490~~)~~ high-strength bolts or equivalent fasteners, installed so as to develop the minimum required bolt tension specified in Table 11.5.6.4.1-1. The bolts are used in holes conforming to the requirements of Article 11.4.8, “Bolt Holes.”

2014
Revision

Page left intentionally blank.

11.5.6.3—Surface Conditions

Revise the last paragraph as follows:

"Surface conditions" refers to Article 6.13.2, "Bolted Connections," and Article 6.13.2.8, "Slip Resistance," of the *AASHTO LRFD Bridge Design Specifications*~~, 2007~~.

Page left intentionally blank.

11.5.6.4.1—General 2010 Revision 2015 Revision

Delete AASHTO M 164 and M 253 citations in paragraph 5 and Table 11.5.6.4.1-1 as follows:

AASHTO ~~M 253 (~~ASTM A490~~)~~ fasteners and galvanized ~~AASHTO M 164 (~~ASTM A325~~)~~ fasteners shall not be reused. Other ~~AASHTO M 164 (~~ASTM A325~~)~~ bolts may be reused if approved by the Engineer. Touching up or retorquing previously tensioned bolts which may have been loosened by the tensioning of adjacent bolts shall not be considered as reuse provided the torquing continues from the initial position and does not require greater rotation, including the tolerance, than that required by Table 11.5.6.4.1-2.

Table 11.5.6.4.1-1—Minimum Required Bolt Tension, kips

Bolt Size	~~AASHTO M 164~~ ASTM A325	~~AASHTO M 253~~ ASTM A490
0.5	12	15
0.625	19	24
0.75	28	35
0.875	39	49
1.0	51	64
1.125	56	80
1.25	71	102
1.375	85	121
1.5	103	148

11.5.6.4.2—Rotational-Capacity Tests Back to 2010 Edition 2010 Revision

Delete AASHTO M 164 citation in paragraph 1, bullet 1as follows:

Rotational-capacity testing is required for all fastener assemblies, except for lock-pin and collar type fastener assemblies. Assemblies specified as galvanized shall be tested after galvanizing. Washers shall be required as part of the test even though they may not be required as part of the installation procedure. The following shall apply:

- Except as modified herein, the rotational-capacity test shall be performed in accordance with the requirements of ~~AASHTO M 164 (~~ASTM A325~~)~~.

11.5.6.4.3—Requirement for Washers Back to 2010 Edition

Delete AASHTO M 164 and M 253 citations in paragraph 4 and bullets 2 through 6 as follows:

Hardened washers are not required for connections using ~~AASHTO M 164 (~~ASTM A325~~)~~ and ~~AASHTO M 253 (~~ASTM A490~~)~~ bolts, except as follows:

- Irrespective of the tensioning method, hardened washers shall be used under both the head and the nut when ~~AASHTO M 253 (~~ASTM A490~~)~~ bolts are to be installed in material having a specified yield point less than 40 ksi.

- Where ~~AASHTO M 164 (~~ASTM A325~~)~~ bolts of any diameter or ~~AASHTO M 253 (~~ASTM A490~~)~~ bolts equal to or less than 1.0 in. in diameter are to be installed in oversize or short-slotted holes in an outer ply, a hardened washer conforming to AASHTO M 293 (ASTM F436) shall be used.

2014 Revision

- Where ~~AASHTO M 253 (~~ASTM A490~~)~~ bolts over 1.0 in. in diameter are to be installed in an oversize or short-slotted hole in an outer ply, hardened washers conforming to AASHTO M 293 (ASTM F436), except with 0.3125-in. minimum thickness, shall be used under both the head and the nut in lieu of standard thickness hardened washers. Multiple hardened washers with combined thickness equal to or greater than 0.3125 in. shall not be considered as satisfying this requirement.

- Where ~~AASHTO M 164 (~~ASTM A325~~)~~ bolts of any diameter or ~~AASHTO M 253 (~~ASTM A490~~)~~ bolts equal to or less than 1.0 in. in diameter are to be installed in a long slotted hole in an outer ply, a plate washer or continuous bar of at least 0.3125-in. thickness with standard holes shall be provided. These washers or bars shall have a size sufficient to completely cover the slot after installation and shall be of structural grade material, but need not be hardened, except as follows.

- Where ~~AASHTO M 253 (~~ASTM A490~~)~~ bolts over 1.0 in. in diameter are to be used in long slotted holes in external plies, a single hardened washer conforming to AASHTO M 293 (ASTM F436), but with 0.3125-in. minimum thickness shall be used in lieu of washers or bars of structural grade material. Multiple hardened washers with combined thickness equal to or greater than 0.3125 in. shall not be considered as satisfying this requirement.

11.7.1—Method of Measurement

Delete AASHTO M 160 citation in paragraph 3 as follows:

The weight (mass) of plates shall be computed on the basis of the nominal weight (mass) for their width and thickness as specified in the contract documents, plus an estimated overrun computed as one-half the “Permissible Variation in Thickness and Weight” as tabulated in “General Requirements for Delivery of Rolled Steel Plates, Shapes, Steel Piling, and Bars for Structural Use,” ~~AASHTO M 160M/M 160 (~~ASTM A6/A6M~~)~~.

Page left intentionally blank.

11.8.3.7.1—General

Eliminate the blank line in the middle of the text in both Article and Commentary.

Fit-up of girder sections shall meet the provisions of Article 11.5.3, "Preassembly of Field Connections," of these Specifications. Unless otherwise specified in the Contractor's construction plan, fit-up shall be assumed to be performed under the no-load condition.

When numerically controlled drilling is employed, trial fit-up of cross-frames or diaphragms between properly positioned girder sections shall be performed as specified in Article 11.5.3.3 of these specifications.

Shop fit-up of bolted connections in load-carrying connections of cross-frames and diaphragms to the girders may be required for structures with complex geometry or stiff elements.

Article 11.5.3.1 requires that girder sections be fit up to ensure proper fit in the field. This requirement generally is applied only to girder splices on multistringer bridges. If numerically controlled drilling is used, only trial fit-up is usually required of cross-frames or diaphragms unless the construction plan specifies that full fit-up is required. In special instances, three sections of the bridge may need to be fitted at one time. This requirement would normally be applied to structures with particularly rigid or complex framing, or if numerically controlled drilling is not used.

Page left intentionally blank.

11.10—REFERENCES

2010 Revision 2011 Revision 2012 Revision 2015 Revision

Revise the following references:

AASHTO. ~~2007~~ 2012. *AASHTO LRFD Bridge Design Specifications*, ~~Fourth Edition, LRFDUS-4-M or LRFDSI-4~~ ~~PE,~~ Sixth Edition, LRFDUS-6. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO and AWS. ~~2008~~ 2010. AASHTO/AWS D1.5M/D1.5:~~2008~~2010 *Bridge Welding Code*, ~~Fifth~~ Sixth Edition, BWC-~~5~~6, American Welding Society, Washington, DC.

AISC. ~~2005~~ 2010. *Steel Construction Manual*, ~~13th~~ 14th Edition, American Institute of Steel Construction, Chicago, IL.

Research Council on Structural Connections (RCSC). ~~2004~~ 2009. *Specification for Structural Joints Using ~~ASTM A325 or A490~~ High-Strength Bolts.* American Institute of Steel Construction, Chicago, IL.

Add the following references:

Brockenbrough, R. L. 1970. "Criteria for Heat Curving Steel Beams and Girders," *Journal of the Structural Division*. Vol. 96, October 1970. American Society of Civil Engineers, New York, NY.

Brockenbrough, R. L. 1970a. "Experimental Stresses and Strains from Heat Curving," *Journal of the Structural Division*, Volume 96, No. ST7, American Society of Civil Engineers, New York, NY.

Brockenbrough, R. L. 1970b. "Theoretical Stresses and Strains from Heat Curving," *Journal of the Structural Division*, Volume 96, No. ST7, American Society of Civil Engineers, New York, NY.

Sause, R., H. Ma, and J. M. Kulicki. 2013. "Residual Stresses in Heat-Curved I-Girders and Associated Limits on Radius of Curvature," ATLSS Report No. 13-01, Center for Advanced Technology for Large Structural Systems, Lehigh University, Bethlehem, PA, April 2013.

Delete the following references:

~~ASCE. 1970. "Experimental Stresses and Strains from Heat Curving," *Journal of the Structural Division*, Volume 96, No. ST7, American Society of Civil Engineers, New York. Journal published since 1983 (vol. 109) under the title *Journal of Structural Engineering*.~~

~~ASCE. 1970. "Theoretical Stresses and Strains from Heat Curving," *Journal of the Structural Division*, Volume 96, No. ST7, American Society of Civil Engineers, New York. Journal published since 1983 (vol. 109) under the title *Journal of Structural Engineering*.~~

~~ASCE. 1970. "Criteria for Heat Curving Steel Beams and Girders," *Journal of the Structural Division*, Volume 96, No. ST7, American Society of Civil Engineers, New York. Journal published since 1983 (vol. 109) under the title *Journal of Structural Engineering*.~~

2014 Revision

Back to 2010 Edition

SECTION 11: STEEL STRUCTURES

11.1.1—Description

2014 Revision

Revise paragraph 2 of this Article:

This work shall consist of furnishing, fabricating, and erecting steel structures and structural steel portions of other structures in accordance with these Specifications, and in the contract documents.

~~Unless otherwise specified, the structural steel fabricating plant shall be certified under the AISC Quality Certification Program, Category I. The fabrication of fracture-critical members shall be Category III.~~ Unless otherwise specified, certification from the AISC Quality Certification Program shall be required for Fabricators to the standard and supplemental requirements appropriate for the type of work being performed. Details of design which are permitted to be selected by the Contractor shall conform to the provisions of the current *AASHTO LRFD Bridge Design Specifications*.

Painting shall conform to the provisions of Section 13, “Painting.”

Falsework used in the erection of structural steel shall conform to the provisions of Section 3, “Temporary Works.”

Structural components designated in the contract documents as “fracture-critical” shall conform to the provisions of the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*, Section 12, “Fracture Control Plan (FCP) for Nonredundant Members.”

Welding and weld qualification tests shall conform to the provisions of the current AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code.*

2015 Revision

Page left intentionally blank.

Back to 2010 Edition

11.4.3.1—Direction of Rolling

Revise Article 11.4.3.1 as follows:

Unless otherwise specified in the contract documents, steel plates for ~~main~~ primary members and flange splices ~~plates for flanges~~ ~~and main tension members,~~ ~~not secondary members~~ shall be cut and fabricated so that the primary direction of rolling is parallel to the direction of the main tensile stress and/or compressive stress~~es~~. Steel web splice plates, fillers, gusset plates not serving as chord splices, connection plates, and web stiffeners shall not be included in this rolling direction requirement.

2015
Revision

Page left intentionally blank.

Back to 2010 Edition

11.4.8.1— Holes for High-Strength Bolts and Unfinished Bolts

11.4.8.1.1—General

2014 Revision 2012 Revision

Revise paragraph 5 of Article 11.4.8.1.1. as follows:

Holes in fillers, cross frames, lateral bracing components, and the corresponding holes in connection plates between girders and cross frames or lateral components may be punched full size. Holes in longitudinal main load-carrying members, transverse floorbeams, and any components designated as fracture critical (FCMs) shall not be punched full-size~~.~~; this restriction shall not apply to fillers used in connections of FCMs.

11.5.6.4.1—Installation

Back to 2010 Edition 2010 Revision 2014 Revision

Add the following to the end of Article 11.5.6.4.1:

11.5.6.4.1—General

For situations in which the bolt length measured from the underside of the head to the end of the bolt exceeds 12 diameters, the required rotation shall be determined by actual tests in a suitable tension device simulating the actual conditions.

The bolt length used shall be such that the end of the bolt is flush with or extends beyond the outer face of the nut when properly installed.

C11.5.6.4.1

No research work has been performed by the Research Council Riveted and Bolted Structural Joints to establish the turn-of-nut procedure when bolt lengths exceed 12 diameters.

The requirement in the last paragraph of this article related to the minimum bolt length is taken from Section 2.3.2 of RCSC (2009). Contract documents sometimes include a stick-through length requirement or minimum protrusion of the bolt point beyond the nut. However, because the threaded length for any given bolt diameter is constant regardless of the bolt length, an excessive stick-through length requirement, which may require a longer bolt, increases the risk of jamming the nut on the thread run-out. Because a stick-through length requirement does not enhance the performance of the bolt and can reduce the rotational ductility of the fastener, a minimum stick-through requirement should not be specified. Note that there is no specified maximum limitation on bolt stick-through. However, in order to provide the rotational ductility of the fastener required for proper tensioning of high-strength bolts, sufficient threads in the grip must be available. Three full threads located within the grip of the bolt is sufficient to provide the required ductility. The use of an additional flat washer under the bolt head is a common solution to provide the additional threads within the grip or when there is a risk of jamming the nut on the thread run-out. For standard holes, up to two washers may be used under either or both the head and the nut to accommodate variations in bolt and thread length.

Page left intentionally blank.

11.8.3.6—Tolerances

Revise the first sentence of this Article:

11.8.3.6.4—Girder Lengths

Girder lengths shall be determined based on an ambient temperature of 68°F, unless otherwise specified by the Owner. Girder length shall be measured along the arc.

C11.8.3.6.4

Girder length is important with respect to location of anchor bolts. If a laser instrument, which is free from temperature effect, is used to survey either the anchor bolt locations and/or the girder length, it is important that compensation be made for temperature of the girder.

Page left intentionally blank.

11.10—References 2010 Revision 2011 Revision 2012 Revision 2014 Revision

Revise the following reference:

AISC Quality Certification Program, American Institute of Steel Construction, Chicago, IL~~, Category I: Structural Steel and Category III: Fracture-Critical~~. See http://www.aisc.org.

TABLE OF CONTENTS

SECTION 12

STEEL GRID FLOORING

12.1—GENERAL

12.1.1—Description

This work shall consist of furnishing and installing steel grid flooring of the open type or of the concrete-filled type, as specified in the contract documents. When the Contractor is allowed to select any details of the design, said details shall meet the requirements for the design of steel grid floors specified in Articles 4.6.2.1 and 9.8.2 of the *AASHTO LRFD Bridge Design Specifications.*

C12.1.1

Steel grid flooring requirements refers to Article 4.6.2.1, "Approximate Methods of Analysis for Decks," and Article 9.8.2, "Metal Grid Decks," of the *AASHTO LRFD Bridge Design Specifications*, 2007.

12.1.2—Working Drawings

The Contractor shall submit complete working drawings with assembly details to the Engineer for approval. Fabrication or construction of the flooring shall not be started until the drawings have been approved. Such approval shall not relieve the Contractor of any responsibility under the contract documents for the successful completion of the work.

12.2—MATERIALS

12.2.1—Steel

All steel shapes, plates, and bars shall conform to AASHTO M 270M/M 270 (ASTM A709/A709M) Grade 36, 50, or 50W (Grade 250, 345, or 345W). Unless the material is galvanized or epoxy-coated, it shall have a copper content of 0.2 percent.

Reinforcing steel shall conform to the requirements of Section 9, "Reinforcing Steel."

12.2.2—Protective Treatment

Open-type floors, unless otherwise specified, shall be galvanized in accordance with the requirements of AASHTO M 111M/M 111 (ASTM A123/A123M).

Filled or partially filled types, specified in the contract documents, shall be either galvanized, painted, epoxy-coated, or supplied in unpainted weathering steel.

If painted, the paint shall be applied according to the specifications for Section 13, "Painting," except that dipping will be permitted. The paint shall be as specified for metal structures unless paint or coating of another type is required by the contract documents. When painting is specified, those areas of steel grid flooring completely encased in concrete may remain unpainted, unless otherwise specified in the contract documents.

12.2.3—Concrete

All concrete in filled steel grid floors shall conform to the requirements of Section 8, "Concrete Structures." The concrete and the size of aggregate shall be as specified for Class C (AE) concrete.

12.2.4—Skid Resistance

The upper edges of all members forming the wearing surface of open-type grid flooring shall be serrated to give the maximum skid resistance.

Concrete-filled or overlayed grid floors shall be given a skid-resistant texture as specified in Article 8.10.2, "Roadway Surface Finish."

12.3—ARRANGEMENT OF SECTIONS

Where the main elements are normal to centerline of roadway, the units generally shall be of such length as to extend over the full width of the roadway for roadways up to 40.0 ft but in every case the units shall extend over at least three panels. Where joints are required, the ends of the main floor members shall be welded at the joints over their full cross-sectional area or otherwise connected to provide full continuity.

Where the main elements are parallel to centerline of roadway, the sections shall extend over not less than three panels and the ends of abutting units shall be welded over their full cross-sectional area or otherwise connected to provide full continuity in accordance with the design.

12.4—PROVISION FOR CAMBER

Unless otherwise specified in the contract documents, provision for camber shall be made as follows:

- Steel units so rigid that they will not readily follow the camber required shall be cambered in the shop. For grid flooring types other than those employing a field placed full depth concrete filling attached to the deck with welded shear connectors, the stringers shall be canted or provided with shop-welded beveled bearing bars to provide a bearing surface parallel to the crown of the roadway. If beveled bars are used, they shall be continuous and fillet-welded along the centerline of the stringer flange, in which case, the design span length shall be governed by the width of the bearing bar instead of the width of the stringer flange.

- Longitudinal stringers, except as provided in the following paragraph, shall be mill cambered or provided with bearing strips so that the completed floor after dead load deflection will conform to the longitudinal camber specified in the contract documents.

- Vertical adjustment of full-depth-filled grid floors that are to be connected to supporting members with shear connectors may be accomplished by use of adjusting bolts operating through nuts welded to the grid and bearing on the top flange of framing members. Alternatively, shims may be used, and shims must be used if construction vehicles are to be allowed on the floor prior to final attachment.

12.5—FIELD ASSEMBLY

Areas of considerable size shall be placed and, if necessary, adjusted to proper fit before the floor is connected to its supports. Care shall be taken during lifting and placing to avoid overstressing the grid units. The main elements shall be made continuous as specified in Article 12.3, “Arrangement of Sections,” and sections shall be connected together along their edges by welding or bolting in accordance with the contract documents.

12.6—CONNECTION TO SUPPORTS

Except when other connection methods are specified in the contract documents or approved, the floor shall be connected to its steel supports by welding every fourth main element to the supporting member; however, welds shall be spaced no greater than 15.0 in. on centers. Before any welding is done, the floor shall either be temporarily loaded or it shall be clamped down to make a tight joint with full bearing. To minimize the stresses induced through clamping down, any differential elevation of 0.25 in. or more over a 4.0-ft supporting member shall be shimmed before welding the shim, the grid, and the supporting member. The location, length, and size of the welds shall be subject to the approval of the Engineer.

Around the perimeter of continuous units of grid flooring, the ends of all the main steel members of the flooring shall be securely fastened together by means of steel plates or angles welded to the ends of the main members or by thoroughly encasing the ends with concrete.

When specified in the contract documents, approved methods other than welding may be used for attaching steel grid floors (both open and concrete-filled types) to framing members. In such cases, welded headed shear connectors can be employed for concrete-filled grids and open steel grids can be connected to framing members by bolting.

12.7—WELDING

All shop and field welding shall be done in accordance with the current edition of AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code.*

12.8—REPAIRING DAMAGED GALVANIZED COATINGS

Galvanized surfaces that are abraded or damaged at any time after the application of the zinc coating shall be repaired by thoroughly wire brushing the damaged areas and removing all loose and cracked coating, after which the cleaned areas shall be painted with two applications of unthinned, commercial quality zinc-rich primer (organic-vehicle type). Spray cans shall not be used.

12.9—PLACEMENT OF CONCRETE FILLER

12.9.1—Forms

Concrete-filled types of flooring with bottom flanges not in contact with each other shall be provided with bottom forms of metal or wood to retain the concrete filler without excessive leakage. Forms shall be removed after the concrete has been cured, except that metal forms conforming to the following paragraph may be left in place.

If metal form strips are used, they shall fit tightly on the bottom flanges or protrusions of the grid members and be placed in noncontinuous lengths so as to extend not more than 1.0 in. onto the edge of each support, but in all cases the forms shall be such as will result in adequate bearing of slab on the support. If metal forms are to be left in place, they shall either be galvanized or protective-treated by the same method that is required for the grid flooring.

12.9.2—Placement

When the contract documents indicate that the concrete filling does not extend to the bottom of the steel grid, the concrete, except concrete for cells in which shear connectors are to be installed, may be placed with the grid in an inverted position prior to installation, or the portion of the grid to remain unfilled may be blocked out by the use of a temporary inert filling material, such as sand or polystyrene board filler which is later removed, or by the use of metal lath form strips or other approved methods. The method used shall permit full embedment of the tertiary bars and the shear connector studs, if used.

When the contract documents indicate that filled or partially filled grids or reinforced concrete slabs incorporating steel grids are to act compositely with their supporting members, all shear connecting studs shall be fully encased in concrete and the entire area between the top flange of the supporting member and the bottom of the grid filling shall be filled with concrete.

The concrete for filled grid floors shall be mixed, placed, and cured in accordance with the requirements of Section 8, "Concrete Structures." The concrete shall be thoroughly compacted by vibrating the steel grid floor. The vibrating device and the manner of operating it shall be subject to the approval of the Engineer.

12.10—MEASUREMENT AND PAYMENT

Steel grid flooring shall be measured by the square foot. The number of square feet shall be based on the dimensions of the flooring in place and approved by the Engineer in the completed work.

Steel grid flooring shall be paid for at the contract document price per square foot. Such payment for steel grid floor, open, or concrete-filled types, shall be considered to be full compensation for the cost of furnishing of all materials, equipment, tools, and labor necessary for the satisfactory completion of the work.

12.11—REFERENCES

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4, American Association of State and Highway Transportation Officials, Washington, DC.

AASHTO. 2009. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 29th Edition, HM-29, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO and AWS. 2008. AASHTO/AWS D1.5M/D1.5:2008 *Bridge Welding Code*, Fifth Edition, BWC-5, American Welding Society, Washington, DC.

TABLE OF CONTENTS

PAINTING

13.1—GENERAL

13.1.1—Description

This work shall consist of the painting of surfaces specified in the contract documents to be painted. The work shall be taken to include, but is not limited to, the preparation of surfaces to be painted; application and curing of the paint; protection of the work; protection of existing facilities, vehicles, and the public from damage due to this work; and the furnishing of all labor, equipment, and materials needed to perform the work.

13.1.2—Protection of Public and Property

The Contractor shall comply with all applicable environmental protection and occupational safety and health standards, rules, regulations, and orders. Failure to comply with these standards, rules, regulations, and orders shall be sufficient cause for suspension or disqualification.

All reasonable precautions shall be taken to contain waste materials (used blasting material and old paint) classified as hazardous. Disposal of hazardous waste material shall be performed in accordance with all applicable Federal, State, and Local laws.

The Contractor shall provide protective devices such as tarps, screens, or covers as necessary to prevent damage to the work and to other property or persons from all cleaning and painting operations. The Contractor shall be responsible for all damage caused by the painting project to vehicles, persons, or property.

Paint or paint stains that result in an unsightly appearance on surfaces not designated to be painted shall be removed or obliterated by the Contractor at the Contractor's expense.

13.1.3—Protection of the Work

The Contractor shall take all precautions necessary to protect the surface from contamination prior to or during the application process.

The Contractor shall protect all parts of the work against disfigurement by splatters, splashes, and smirches of paint materials.

All painted surfaces that are marred or damaged as a result of operations of the Contractor shall be repaired by the Contractor, at the Contractor's expense, with materials and to a condition equal to that of the coating specified herein.

If traffic causes an objectionable amount of dust, the Contractor, when directed by the Engineer, shall sprinkle the adjacent roadbed and shoulders with water or dust palliative for a sufficient distance on each side of the location where painting is being done.

Upon completion of all painting operations and of any other work that would cause dust, grease, or other foreign materials to be deposited on the painted surfaces, the painted surfaces shall be thoroughly cleaned. At the time of opening structures to public traffic, the painting shall be completed and the surfaces shall be undamaged and clean.

13.1.4—Thickness and Color

The dry film thickness of each coat and total thickness of the finished product shall be in accordance with the contract documents. The thickness of previously applied coatings or of an existing coating that is to be top coated shall be determined in accordance with SSPC-PA 2, *Measurement of Dry Paint Thickness with Magnetic Gages*, prior to applying the next coat.

Each coat of paint should be separately colored to ensure complete coverage, and such that the previous coat can be hidden by a single coat of the next application.

C13.1.4

These provisions are taken from the *AASHTO Guide for Painting Steel Structures*, AASHTO, 1997, which has been discontinued. Please see the AASHTO–NSBA steel bridge collaboration document, *Guide Specifications for Application of Coating Systems with Zinc-Rich Primer to Steel Bridges.* [SSPC is the Society for Protective Coatings, formerly known as the Steel Structures Painting Council.]

13.2—PAINTING METAL STRUCTURES

13.2.1—Coating Systems and Paints

The coating system and paints to be applied shall consist of the system in the contract documents.

C13.2.1

Please see the AASHTO–NSBA steel bridge collaboration document, *Guide Specifications for Application of Coating Systems with Zinc-Rich Primer to Steel Bridges.*

13.2.2—Weather Conditions

Paint shall be applied only on thoroughly dry surfaces. Painting shall not be permitted under any of the following circumstances:

- the atmospheric temperature, paint or the surface to be painted is at or below 40°F or above 100°F,
- metal surfaces are less than 5°F above the dew point,
- the humidity exceeds 85 percent at the site of the work,
- freshly painted surfaces may become damaged by rain, fog, or dust, or
- it can be anticipated that the atmospheric temperature will drop below 40°F during the drying period, except as provided herein for painting in enclosures.

Metal surfaces which are hot enough to cause the paint to blister, to produce a porous paint film, or to cause the vehicle to separate from the pigment shall not be painted.

Subject to approval of the Engineer, the Contractor may provide a suitable enclosure to permit painting during inclement weather. Provisions shall be made to artificially control atmospheric conditions inside the enclosure within limits suitable for painting throughout the painting operation.

Surfaces painted under cover in damp or cold weather shall remain under cover until the paint dries or weather conditions permit open exposure. Full compensation for providing and maintaining such enclosures shall be considered as included in the prices paid for the various contract items of work involving painting and, therefore, no additional compensation will be allowed.

All blast cleaning, except that performed with enclosed buildings, and all painting shall be performed during daylight hours unless otherwise specified in the contract documents.

13.2.3—Surface Preparation

All exposed surfaces of structural steel, except galvanized or metalized surfaces, shall be cleaned and painted.

All surfaces of new structural steel shall be cleaned by the blast-cleaning method unless otherwise specified in the contract documents or approved in writing by the Engineer.

In repainting existing steel structures, the method of cleaning shall be as specified in the contract documents. Any damage to sound paint, on areas not designated for treatment, resulting from the Contractor's operations shall be repaired by the Contractor at the Contractor's expense to the satisfaction of the Engineer.

The methods used in the cleaning of metal surfaces shall conform to the specifications herein.

C13.2.3

Please see the AASHTO–NSBA steel bridge collaboration document, *Guide Specifications for Application of Coating Systems with Zinc-Rich Primer to Steel Bridges.*

13.2.3.1—Blast Cleaning

Abrasives used for blast cleaning shall be either clean dry sand, mineral grit, steel shot, or steel grit, at the option of the Contractor, and shall have a suitable grading to produce satisfactory results. The use of other abrasives shall not be permitted unless approved in writing by the Engineer.

Unwashed beach sand containing salt or excessive amounts of silt will not be allowed.

All dirt, mill scale, rust, paint, and other foreign material shall be removed from exposed steel surfaces in accordance with the requirements of the Steel Structures Painting Council Surface Preparation Specification No. 10, SSPC-SP 10, *Near-White Blast Cleaning*. Blast cleaning shall leave all surfaces with a dense and uniform anchor pattern of not less than 1 mil or more than 3 mils as measured with an approved surface profile comparator.

When blast cleaning is being performed near machinery, all journals, bearings, motors, and moving parts shall be sealed against entry of abrasive dust before blast cleaning begins.

Blast-cleaned surfaces shall be primed or treated the same day blast cleaning is done, unless otherwise authorized by the Engineer. If cleaned surfaces rust or are contaminated with foreign material before painting is accomplished, they shall be re-blast-cleaned by the Contractor at the Contractor's expense.

C13.2.3.1

Removal of foreign material shall conform to Steel Structures Painting Council Surface Preparation Specification No. 10, SSPC-SP 10, June 1991.

13.2.3.2—Steam Cleaning

All dirt, grease, loose chalky paint, or other foreign material that has accumulated on the previously painted or galvanized surfaces shall be removed with a steam cleaning apparatus which shall precede all other phases of cleaning. It is not intended that sound paint be removed by this process. Any paint which becomes loose, curled, lifted, or loses its bond with the preceding coat or coats after steam cleaning shall be removed as directed by the Engineer to sound paint or metal surface by the Contractor at the Contractor's expense.

A biodegradable detergent shall be added to the feed water of the steam generator or applied to the surface to be cleaned. The detergent shall be of such composition and shall be added or applied in such quantity that the cleaning as described in the above paragraph is accomplished.

Any residue, detergent, or other foreign material that may accumulate on cleaned surfaces shall be removed by flushing with fresh water.

Steam cleaning shall not be performed more than two weeks prior to painting or other phases of cleaning.

Subsequent painting shall not be performed until the cleaned surfaces are thoroughly dry and in no case in less than 24 h after cleaning and flushing.

13.2.3.3—Solvent Cleaning

Unless otherwise prohibited by the contract documents, solvents shall be used to remove oil, grease, and other soluble contaminants in accordance with the requirements of SSPC-SP 1, *Solvent Cleaning*. Solvent cleaning shall be performed prior to blast cleaning. If contamination remains after blasting, the area shall be recleaned with solvent.

C13.2.3.3

Solvent cleaning shall conform to Steel Structures Painting Council, SSPC-SP 1, November 1982.

13.2.3.4—Hand Cleaning

Wire brushes, either hand or powered, hand scraping tools, power grinders, or sandpaper shall be used to remove all dirt, loose rust and mill scale, or paint which is not firmly bonded to the metal surfaces.

Pneumatic chipping hammers shall not be used unless authorized in writing by the Engineer.

13.2.3.5—Power Washing

Power washing shall utilize water at a pressure between 0.8 to 1.5 ksi, applied with the nozzle no further than 1.0 ft from the surface of the steel.

C13.2.3.5

Please see the AASHTO–NSBA steel bridge collaboration document, *Guide Specifications for Application of Coating Systems with Zinc-Rich Primer to Steel Bridges.*

13.2.4—Application of Paints

The Contractor shall notify the Engineer, in writing, at least one week in advance of the date that cleaning and painting operations are to begin.

C13.2.4

Please see the AASHTO–NSBA steel bridge collaboration document, *Guide Specifications for Application of Coating Systems with Zinc-Rich Primer to Steel Bridges.*

Painting shall be done in a neat manner. Unless otherwise specified in the contract documents, paint shall be applied by brush, spray, or roller, or any combination thereof peculiar to the paint being applied.

Each application of paint shall be thoroughly cured and any skips, holidays, thin areas, or other deficiencies corrected before the succeeding application. The surface of the paint being covered shall be free from moisture, dust, grease, or any other deleterious materials that would prevent the bond of the succeeding applications. In spot painting, old paint which lifts after the first application shall be removed by scraping and the area repainted before the next application.

Paints that are specified "formulated ready for application" and "no thinning" shall be allowed unless otherwise provided in the applicable materials specification for the paint being used.

Brushes, when used, shall have sufficient body and length of bristle to spread the paint in a uniform film. Round, oval-shaped brushes, or flat brushes not wider than 4.5 in. shall be used. Paint shall be evenly spread and thoroughly brushed out.

On all surfaces that are inaccessible for painting by regular means, the paint shall be applied by sheepskin daubers, bottle brushes, or by any other means approved by the Engineer.

Rollers, when used, shall be of a type that do not leave a stippled texture in the paint film. Rollers shall be used only on flat, even surfaces to produce a paint film of even thickness with no skips, runs, sags, or thin areas.

Paint may be applied with airless or conventional spray equipment.

Suitable traps or separators acceptable to the Engineer shall be furnished and installed in the airline to each spray pot to exclude oil and water from the air.

Any spray method which produces excessive paint build-up, runs, sags, or thin areas in the paint film, or skips and holidays, shall be considered unsatisfactory and the Engineer may require modification of the spray method or prohibit its use and require brushing instead.

Mechanical mixers shall be used to mix paint. Prior to application, paint shall be mixed a sufficient length of time to thoroughly mix the pigment and vehicle together, and shall be kept thoroughly mixed during its application.

The dry film thickness of the paint shall be measured in place with a calibrated magnetic film thickness gage according to Steel Structures Painting Council SSPC-PA 2.

Dry film thickness shall be measured in accordance with the Steel Structures Painting Council, SSPC-PA 2, August 1991.

The thickness of each application shall be limited to that which will result in uniform drying throughout the paint film.

Succeeding applications of paint shall be of such shade as to contrast with the paint being covered.

Structures shall be blast-cleaned and painted with the total thickness of undercoats before erection. After erection and before applying subsequent paint, all areas where paint has been damaged or has deteriorated and all exposed unpainted surfaces shall be thoroughly cleaned and spot painted with the specified undercoats to the specified thickness.

Surfaces exposed to the atmosphere and that would be inaccessible for painting after erection shall be painted the full number of applications prior to erection.

Vinyl wash primer, if required, shall not be applied more than 12 h before application of the succeeding coat of paint. The vinyl wash primer shall be applied by spraying to produce a uniform wet film on the surface. The dry film thickness shall be between 0.3 and 0.5 mils.

The painting of areas under joint connection and splice plates shall conform to Article 11.5.6.3, "Surface Conditions."

13.2.4.1—Application of Zinc-Rich Primers

Zinc-rich primers, which include organic and inorganic zinc primers, shall be applied by spray methods. On areas inaccessible to spray application, the paint may be applied by brush or daubers.

Mechanical mixers shall be used in mixing the primer. After mixing, zinc-rich primers shall be strained through a metal No. 30 to No. 60 (250 to 600 μm) mesh screen or a double layer of cheesecloth immediately prior to or during pouring into the spray pot.

An agitating spray pot shall be used in all spray application of zinc-rich primers. The agitator or stirring rod shall reach to within 2.0 in. of the bottom of the spray pot and shall be in motion at all times during primer application. Such motion shall be sufficient to keep the primer well mixed.

Spray equipment shall provide the proper pot pressure and atomization pressure to produce a coating, the composition of which shall comply in all respects to the specifications for zinc paint. The hose from pot to nozzle in the contract documents shall not be more than 75.0 ft long, nor be used more than 15.0 ft above or below the pot.

Cured, zinc-rich primer shall be free from dust, dirt, salt, or other deleterious deposits and thoroughly dry before applying vinyl wash primer.

In addition, the application of inorganic zinc paints shall conform to the following:

- Succeeding applications of inorganic zinc paints shall be applied within 24 h, but not less than 30 min after prior application of such paint.

- In areas where mud-cracking occurs in the inorganic zinc paint, it shall be blast-cleaned back to soundly bonded paint and recoated to the same thickness by the same methods specified for the original coat.

- Paint shall be cured for 48 h at a relative humidity of at least 45 percent before the application of vinyl wash primer. The cured inorganic zinc paint shall be hosed down with water and be in a surface dry condition before the application of vinyl wash primer if the vinyl wash primer is not applied within three weeks after the inorganic zinc paint is applied, or if there is evidence of dust, dirt, salt, or other deleterious deposits on the inorganic zinc paint.

13.2.5—Measurement and Payment

Cleaning and painting structural steel will be paid for on the basis of lump-sum prices, unless otherwise specified in the contract documents.

The lump-sum prices paid for clean structural steel and for paint structural steel or the lump-sum price paid for clean and paint structural steel shall include full compensation for furnishing all labor, materials, tools, equipment, and incidentals, and for doing all the work involved in cleaning and painting structural steel as specified in the contract documents, these Specifications, and the special provisions and as directed by the Engineer.

13.3—PAINTING GALVANIZED SURFACES

All galvanized surfaces that are to be painted shall first be cleaned by washing with mineral spirit solvent sufficient to remove any oil, grease, or other materials foreign to the galvanized coating.

After cleaning, vinyl wash primer shall be applied to such surfaces. The vinyl wash primer shall be applied by spraying to produce a uniform wet film on the surface. The dry film thickness shall be between 0.3 and 0.5 mils.

Finish paint to be applied to primed galvanized surfaces shall be specified in the contract documents. If not otherwise specified, the finish paint shall be the same as that used on adjacent metal work or shall be as directed by the Engineer.

No separate payment shall be made for preparing and painting galvanized surfaces. Full compensation for furnishing all labor, materials, tools, equipment, and incidentals, and for doing all the work involved in preparing and painting galvanized surfaces as specified in the contract documents, these Specifications, and as directed by the Engineer will be considered as included in the prices paid for the various contract items of work involving the galvanized surfaces.

13.4—PAINTING TIMBER

13.4.1—General

Unless otherwise specified in the contract documents, all new timber requiring painting shall be painted with three applications of paint. The paint used for various applications will be as specified in these Specifications or as specified in the contract documents.

The painting of previously painted surfaces shall be as required by the contract documents and Specifications herein.

13.4.2—Preparation of Surfaces

All cracked or peeled paint, loose chalky paint, dirt, and other foreign material shall be removed by wire brushing, scraping, or other means immediately prior to painting. The moisture content of the timber shall not be more than 20 percent at the time of the first application.

13.4.3—Paint

Paint for timber structures, except as otherwise provided herein, shall be as specified in the contract documents. The paint as specified is intended for use in covering previously painted surfaces. When it is applied to unpainted timber, turpentine and linseed oil shall be added as required by the character of the surface in an amount not to exceed one-eighth of the paint as specified. The paint shall be either white or tinted as directed by the Engineer.

If a black finish paint is specified in the contract documents, the first or prime coat shall be as specified above.

13.4.4—Application

When permitted in writing by the Engineer, the first application of paint may be applied prior to erection.

After the first application has dried and the timber is in place, all cracks, checks, nail holes, or other depressions shall be puttied flush with the surface and allowed to dry before the second application of paint.

Paint shall be applied by brush, air spray, or roller; spread evenly; and worked thoroughly into all seasoning cracks, corners, and recesses. No later coat shall be applied until the full thickness of the previous coat has dried.

Final brush strokes with aluminum paint shall be made in the same direction to ensure that powder particles “leaf” evenly.

13.4.5—Painting Treated Timber

Timber treated with creosote or oil-borne pentachlorophenol preservatives shall not be painted.

Timber treated with water-borne preservatives shall be clean and be reduced to no more than 20 percent moisture content before it is painted. Any visible salt crystals on the wood surface shall be washed and brushed away, and the moisture content reduced again to the specified level before painting. Stored timber awaiting painting shall be covered and stacked with spreaders to ensure air circulation.

13.4.6—Payment

No separate payment shall be made for preparing surfaces and for painting new timber. The painting of existing timber shall be paid for on the basis of lump-sum prices. Full compensation for furnishing all labor, materials, tools, equipment, and incidentals, and for doing all the work involved in preparing surfaces and painting timber, as specified in these Specifications and the contract documents, and as directed by the Engineer shall be considered as included in the prices paid for the various contract items of work involving new timber or the prices paid for painting existing timber.

13.5—PAINTING CONCRETE

13.5.1—Surface Preparation

Prior to painting concrete surfaces, laitance and curing compounds shall be removed from the surface by abrasive blast cleaning in accordance with Article 13.2.3.1, "Blast Cleaning."

Concrete surfaces shall be thoroughly dry and free of dust at the time the paint is to be applied.

Any artificial drying procedures and methods shall be subject to approval by the Engineer.

13.5.2—Paint

Unless otherwise specified in the contract documents, paint to be applied to concrete surfaces shall be acrylic emulsion and shall comply in all respects to the following Master Painters Institute Detailed Performance Standards:

- MPI—#10 for flat paint,
- MPI—#11 for semi-gloss, or
- MPI—#119 for gloss.

This paint may be tinted by using "universal" or "all purpose" concentrates.

C13.5.2

Prior to 2005, these Specifications referred to Federal Specification TT-P-19, which was superseded in April 2000. Federal Specification TT-P-19 specified a 60-degree specular gloss between 0 and 20 units. The current MPI specifications have 60-degree specular glosses as follows:

- MPI—#10 Flat (Gloss Level 1) 0–5 units
- MPI—#11 Semi-Gloss (Gloss Level 5) 35–70 units
- MPI—#119 Gloss (Gloss Level 6) 70–85 units

13.5.3—Application

Acrylic emulsion paint shall be applied in not less than two applications to produce a uniform appearance.

The paint shall be applied only when the ambient temperature is 50°F or above. Painting shall not be permitted when it can be anticipated that the ambient temperature will drop below 50°F during the application and drying of the paint.

13.5.4—Measurement and Payment

Preparing and painting concrete will be measured either by the lump sum or by the square foot as listed in the contract documents. When measured by the square foot, measurement will be determined along the surface of the actual area painted.

The contract price paid per lump sum or square foot to prepare and paint concrete shall include full compensation for furnishing all labor, materials, tools, equipment, and incidentals, and for doing all the work involved in preparing the concrete and applying the paint to concrete surfaces, as specified in these specifications and the contract documents, and as directed by the Engineer.

13.6—REFERENCES

GSA. *60-Degree Specular Gloss*, Federal Specification TT-P-19, U.S. General Services Administration. Cancelled; see MPI #10, #11, #15, and #119 listed below.

MPI. 2001. *MPI Detailed Performance Standard—#10*. Master Painters Institute, Burnaby, BC, Canada.

MPI. 2001. *MPI Detailed Performance Standard—#11*. Master Painters Institute, Burnaby, BC, Canada.

MPI. 2001. *MPI Detailed Performance Standard—#15*. Master Painters Institute, Burnaby, BC, Canada.

MPI. 2001. *MPI Detailed Performance Standard—#119*. Master Painters Institute, Burnaby, BC, Canada.

NSBA. 2006. *Guide Specifications for Application of Coating Systems with Zinc-Rich Primer to Steel Bridges*, S8.1,. National Steel Bridge Alliance, Chicago, IL. See also AASHTO NSBASBCS-2, 2006.

SSPC. 1982. *Solvent Cleaning*, SSPC-SP 1,. Steel Structures Painting Council, [now the Society for Protective Coatings], Pittsburgh, PA.

SSPC. 1991. *Measurement of Dry Coating Thickness with Magnetic Gages*, SSPC-PA 2, Steel Structures Painting Council, [now the Society for Protective Coatings], Pittsburgh, PA.

SSPC. 1991. *Near-White Blast Cleaning*, SSPC-SP 10, Steel Structures Painting Council, [now the Society for Protective Coatings], Pittsburgh, PA.

SECTION 14: STONE MASONRY

TABLE OF CONTENTS

SECTION 14

STONE MASONRY

14.1—DESCRIPTION

This work shall consist of the construction of stone masonry structures and the stone masonry portions of composite structures, in accordance with these Specifications and in reasonably close conformity with the lines and grades shown in the contract documents or established by the Engineer.

14.1.1—Rubble Masonry

Rubble masonry, as specified herein, shall include various classes of roughly squared and dressed stone laid in cement mortar.

14.1.2—Ashlar Masonry

Ashlar masonry shall consist of first-class cut stone masonry laid in regular courses and shall include all work in which, as distinguished from rubble masonry, the individual stones are dressed or tooled to exact dimensions.

14.2—MATERIALS

14.2.1—General

Stone for masonry shall be tough, dense, sound, and durable, and free of seams, cracks, inclusions, or other structural defects. Stone shall be of the type and quality specified in the contract documents. Prior to shipment of stone to the job site, the Contractor shall obtain approval of the proposed source and shall submit a representative sample of stone to the Engineer for inspection and, if necessary, testing. The sample shall be dressed and finished as specified for use in the work and shall not be less than 6.0 in. in any dimension. All stone used in the work shall be of a quality comparable to that of the sample submitted.

14.2.1.1—Rubble Stone

Stone for mortar rubble masonry shall be free from rounded, worn, or weathered surfaces. All weathered stone shall be rejected.

14.2.1.2—Ashlar Stone

Stone for ashlar masonry shall be reasonably fine grained and uniform in color. Preferably, stone shall be from a quarry, the product of which is known to be of satisfactory quality. Stone shall be of such character that it can be brought to such lines and surfaces, whether curved or plane, as may be required. Any stone having defects that have been repaired with cement or other materials shall be rejected.

C14.1

The *AASHTO LRFD Bridge Design Specifications* do not cover stone masonry. The provisions of the 17th Edition of the *AASHTO Standard Specification for Highway Bridges* shall be used.

14.2.2—Shipment and Storage of Stone

Quarry operations and delivery of stone to the point of use shall be organized to ensure deliveries well ahead of masonry operations. A sufficiently large stock of the specified stone shall be kept on the site at all times to permit adequate selection of stone by the masons.

The stone shall be kept free from dirt, oil, or any other injurious material that may prevent the proper adhesion of the mortar or detract from the appearance of the exposed surfaces.

14.2.3—Mortar

The ingredients used in making mortar shall conform to the following requirements:

- Portland cement, Admixtures and Water; Section 8, “Concrete Structures”
- Masonry cement; ASTM C91
- Hydrated lime; ASTM C207
- Quicklime used to make lime putty; ASTM C5
- Sand aggregate; AASHTO M 45, (ASTM C144)

The proportions of materials shall be such that the volume of sand in a damp, loose condition is between 2.25 and 3 times the volume of the cementitious materials. The cementitious materials shall consist of either one part of portland cement to between 0.25 and 0.50 parts of hydrated lime or lime putty, or one part of portland cement to between one and two parts of masonry cement. Premixed materials conforming to these requirements may be used.

Admixtures shall be used only when specified in the contract documents or approved by the Engineer.

14.3—MANUFACTURE OF STONE FOR MASONRY

14.3.1—General

Each stone shall be free from depressions and projections that might weaken it or prevent it from being properly bedded, and shall be of a shape to meet the requirements for the class of masonry specified.

When no dimensions are specified in the contract documents, the stones shall be furnished in the sizes and face areas necessary to produce the general characteristics and appearance as indicated in the contract documents.

The thickness of courses, if varied, shall diminish regularly from bottom to top of wall. The size of ring stones in arches shall be as specified in the contract documents.

When headers are required, their lengths shall be not less than the width of bed of the widest adjacent stretcher, plus 12.0 in.

14.3.2—Surface Finishes of Stone

For the purpose of this Specification, the surface finishes of stone are defined as follows:

- Smooth-Finished—Having a surface in which the variations from the pitch line do not exceed 0.0625 in.
- Fine-Finished—Having a surface in which the variations from the pitch line do not exceed 0.25 in.
- Rough-Finished—Having a surface in which the variations from the pitch line do not exceed 0.5 in.
- Scabbled—Having a surface in which the variations from the pitch line do not exceed 0.75 in.
- Rock-Faced—Having an irregular projecting face without indications of tool marks. The projections beyond the pitch line shall not exceed 3.0 in. and no part of the face shall recede back of the pitch line.

14.3.3—Rubble Masonry

14.3.3.1—Size

Individual stones shall have a thickness of not less than 8.0 in. and a width of not less than 1.5 times the thickness. No stones, except headers, shall have a length less than 1.5 times their width.

14.3.3.2—Shape

The stones shall be roughly squared on joints, beds, and faces. Selected stone, roughly squared and pitched to line, shall be used at all angles and ends of walls. If specified in the contract documents, all corners or angles in exterior surfaces shall be finished with a chisel draft.

Bed surfaces of face stones shall be normal to the faces of the stones for about 3.0 in. and from this point may depart from normal not more than 2.0 in. in 12.0 in. Joint surfaces of face stones shall form an angle with the bed surfaces of not less than 45 degrees.

All shaping or dressing of stone shall be done before the stone is laid in the wall, and no dressing or hammering which will loosen the stone will be permitted after it is placed.

14.3.3.3—Dressing

Stone shall be dressed to remove any thin or weak portions. Face stones shall be dressed to provide bed and joint lines with a maximum variation from true line of 1.5 in., unless otherwise indicated in the contract documents.

14.3.4—Ashlar Masonry

14.3.4.1—Size

The individual stones shall be large and well proportioned. They shall not be less than 12.0 in. nor more than 30.0 in. in thickness.

14.3.4.2—Dressing

Stones shall be dressed to exact sizes and shapes before being laid and shall be cut to lie on their natural beds with top and bottom truly parallel. Hollow beds will not be permitted. The bottom bed shall be the full size of the stone, and no stone shall have an over-hanging top. In rock-face construction, the face side of any stone shall not present an undercut contour adjacent to its bottom arris giving a top-heavy, unstable appearance when laid.

Beds of face stone shall be fine-finished for a depth of not less than 12.0 in.

Vertical joints of face stone shall be fine-finished and full to the square for a depth of not less than 9.0 in.

Exposed surfaces of the face stone shall be given the surface finish indicated in the contract documents, with edges pitched to true lines and exact batter. Chisel drafts 1.5 in. wide shall be cut at all exterior corners. Face stone forming the starling or nosing of piers shall be rough-finished unless otherwise specified in the contract documents.

Holes for stone hooks shall not be permitted to show in exposed surfaces.

14.3.4.3—Stretchers

Stretchers shall have a width of bed of not less than 1.50 times their thickness. They shall have a length of bed not less than twice nor more than 3.5 times their thickness, and not less than 3.0 ft.

14.3.5—Arch Ring Stones

Arch ring stone joint surfaces shall be radial and at right angles to the front faces of the stones. They shall be dressed for a distance of at least 3.0 in. from the front faces and the soffits, from which points they may depart from a plane normal to the face not to exceed 0.75 in. to 12.0 in. The back surface in contact with the concrete of the arch barrel shall be parallel to the front face and shall be dressed for a distance of 6.0 in. from the intrados. The top shall be cut perpendicular to the front face and shall be dressed for a distance of at least 3.0 in. from the front.

When concrete is to be placed after the masonry has been constructed, adjacent ring stones shall vary at least 6.0 in. in depth.

Stratification in arch ring stones shall be parallel to the radial joints and in other stones shall be parallel to the beds.

When specified in the contract documents, a full-size template of the arch ring shall be laid out near the quarry site, showing face dimensions of each ring stone and thickness of joints. The template shall be approved by the Engineer before the shaping of any ring stone is started, and no ring stone shall be placed in the structure until all ring stones have been shaped, dressed, and approved by the Engineer.

14.4—CONSTRUCTION

14.4.1—Weather Conditions

Stone masonry shall not be constructed in freezing weather or when the stone contains frost, except by written permission of the Engineer and subject to such conditions which the Engineer may require.

14.4.2—Mixing Mortar

The mortar shall be hand- or machine-mixed, as may be required by the Engineer. In the preparation of hand-mixed mortar, the sand and cement shall be thoroughly mixed together in a clean, tight mortar box until the mixture is of uniform color, after which clean water shall be added in such quantity as to form a stiff plastic mass. Machine-mixed mortar shall be prepared in an approved mixer and shall be mixed not less than 3 min nor more than 10 min. Mortar shall be used within 1.5 h after mixing and before final set begins. Retempering of mortar shall be done as necessary to maintain proper consistency during placement.

14.4.3—Selection and Placing of Stone

14.4.3.1—General

When masonry is placed on a prepared foundation bed, the bed shall be firm and normal to, or in steps normal to, the face of the wall, and approved by the Engineer before any stone is placed. When it is placed on foundation masonry, the bearing surface of the foundation masonry shall be cleaned thoroughly and in a saturated-surface dry condition when the mortar bed is spread.

All masonry shall be constructed by experienced workers. Face stones shall be set in random bond to produce the effect specified in the contract documents.

Care shall be taken to prevent the bunching of small stones or stones of the same size. When weathered or colored stones or stones of varying texture are being used, care shall be exercised to distribute the various kinds of stones uniformly throughout the exposed faces of the work. Large stones shall be used for the bottom courses and large, selected stones shall be used in the corners. In general, the stones shall decrease in size from the bottom to the top of work.

Each stone shall be cleaned and thoroughly saturated with water before being set, and the bed that is to receive it shall be clean and well moistened. All stones shall be well bedded in freshly made mortar. The mortar joints shall be full and the stones carefully settled in place before the mortar has set. No spalls shall be permitted in the beds. No pinning up of stones with spalls shall be permitted in beds.

Stone shall not be dropped upon or slid over the wall, nor will hammering, rolling, or turning of stones on the wall be allowed. They shall be carefully set without jarring the stone already laid, and they shall be handled with a lewis or other appliance that will not cause disfigurement.

In case any stone is moved or the joint broken, the stone shall be taken up, the mortar thoroughly cleaned from bed and joints, and the stone reset in fresh mortar.

14.4.3.2—Rubble Masonry

Rubble masonry shall be laid to line and in courses roughly leveled up. The bottom or foundation courses shall be composed of large, selected stones, and all courses shall be laid with bearing beds parallel to the natural bed of the material. The vertical joints in each course of rubble masonry shall break with those in adjoining courses at least 6.0 in. In no case shall a vertical joint be so located as to occur directly above or below a header.

14.4.3.3—Ashlar Masonry

The stones in any one course of ashlar masonry shall be placed so as to form bonds of not less than 12.0 in. with the stones of adjoining courses. Headers shall be placed over stretchers and, in general, the headers of each course shall equally divide the spaces between the headers of adjoining courses, but no header shall be placed over a joint and no joint shall be made over a header.

14.4.4—Beds and Joints

Beds and joints in rubble masonry shall have an average thickness of not more than 1.0 in. Beds and joints in ashlar masonry shall be not less than 0.375 in. nor more than 0.5 in. in thickness; and the thickness of the joint or bed shall be uniform throughout.

The thickness of beds in ashlar masonry may vary as shown from the bottom to the top of the work. However, in each course the beds shall be of uniform thickness throughout.

Beds shall not extend in an unbroken line through more than five stones.

Joints in ashlar masonry shall be vertical. In all other masonry, joints may be at angles with the vertical from 0 degrees to 45 degrees.

Each face stone shall bond with all contiguous face stones at least 6.0 in. longitudinally and 2.0 in. vertically. Ring stone joints on the faces and soffits shall be not less than 0.25 in. nor more than 1.5 in. in thickness.

Cross beds for vertical walls shall be level and for battered walls may vary from level to normal to the batter line of the face of the wall. All joints shall be completely filled with mortar.

14.4.5—Headers

Headers shall hold in the heart of the wall the same size shown in the face and shall extend not less than 12.0 in. into the core or backing. They shall occupy not less than one-fifth of the face area of the wall and shall be evenly distributed.

Headers in rubble masonry walls 2.0 ft or less in thickness shall extend entirely through the wall.

Headers in ashlar masonry shall be placed in each course and shall have a width of not less than 1.5 times their thickness. In walls having a thickness of 4.0 ft or less, the headers shall extend entirely through the wall. In walls of greater thickness, the length of headers shall be not less than 2.5 times their thickness when the course is 18.0 in. or less in height, and not less than 4.0 ft in courses of greater height. Headers shall be spaced not further apart than 8.0 ft center-to-center. There shall be at least one header to every two stretchers.

14.4.6—Cores and Backing

14.4.6.1—General

Cores and backing shall consist either of roughly bedded and jointed headers and stretchers, as specified above, or of Class B or C concrete, as may be specified in the contract documents.

The headers and stretchers in walls having a thickness of 3.0 ft or less shall have a width or length equal to the full thickness of the wall. Backing shall not be permitted.

14.4.6.2—Stone

When stone is used for cores or backing, at least one-half of the stone shall be of the same size and character as the face stone, and with parallel ends. No course shall be less than 8.0 in. thick.

Stone backing shall be laid in the same manner as specified above for face stone, with headers interlocking with face headers when the thickness of the wall will permit. Backing shall be laid to break joints with the face stone. Stone cores shall be laid in full mortar beds so as to bond not less than 12.0 in. with face and backing stone and with each other. Bed joints in cores and backing shall not exceed 1.0 in., and vertical joints shall not exceed 4.0 in. in thickness.

14.4.6.3—Concrete

Concrete used for cores and backing shall conform to the requirements specified in Section 8, "Concrete Structures."

The operations involved in the handling and placing of concrete used in cores and backing shall conform to the requirements specified in Section 8, "Concrete Structures." However, the puddling and compacting of concrete adjacent to the ashlar masonry facing shall be done in a manner that will ensure the filling of all spaces around the stones and secure full contact and efficient bond with all stone surfaces.

14.4.6.4—Leveling Courses

Stone cores and backing shall be carried up to the approximate level of the face course before the succeeding course is started.

The construction joints produced in concrete cores or backing by the intermittent placing of concrete shall be located, in general, not less than 6.0 in. below the top bed of any course of masonry.

14.4.7—Facing for Concrete

Unless otherwise specified in the contract documents, the stone masonry shall be constructed before placing concrete.

Steel anchors as shown in the contract documents shall be used. To improve the bond between the stone masonry and the concrete backing, the back of the masonry shall be made as uneven as the stones will permit.

After the stone facing has been laid and the mortar has attained sufficient strength, all surfaces against which concrete is to be placed shall be cleaned carefully and all dirt, loose material, and accumulations of mortar droppings removed.

When placing concrete, all interstices of the masonry shall be filled and the concrete thoroughly spaded and worked until it is brought into intimate contact with every part of the back of the masonry.

14.4.8—Copings

14.4.8.1—Stone

Stones for copings of wall, pier, and abutment bridge seats shall be carefully selected and fully dimensioned stones. On piers, not more than two stones shall be used to make up the entire width of coping. The copings of abutment bridge seats shall be of sufficient width to extend at least 4.0 in. under the backwall. Each step forming the coping of a wingwall shall be formed by a single stone that shall overlap the stone forming the step immediately below it at least 12.0 in.

Tops of copings shall be given a bevel cut at least 2.0 in. wide, and beds, bevel cuts, and tops shall be fine-finished. The vertical joints shall be smooth-finished, and the copings shall be laid with joints not more than 0.25 in. in thickness. The undersides of projecting copings, preferably, shall have a drip bead.

Joints in copings shall be located so as to provide not less than a 12.0-in. bond with the stones of the undercourse and so that no joint will come directly under the superstructure masonry plates.

14.4.8.2—Concrete

Copings, bridge seats, and backwalls shall be of the material specified in the contract documents and when not otherwise specified shall be of Class A concrete which shall conform to the requirements of Section 8, “Concrete Structures.”

Concrete copings shall be made in sections extending the full width of the wall, not less than 12.0 in. in thickness, and from 5.0 ft to 10.0 ft long. The sections may be cast-in-place or precast and set in place in full mortar beds.

14.4.9—Dowels and Cramps

Where required, coping stone, stone in the wings of abutments, and stone in piers shall be secured with wrought-iron cramps or dowels as indicated in the contract documents.

Dowel holes shall be drilled through each stone before the stone is placed and, after it is in place, such dowel holes shall be extended by drilling into the underlying course not less than 6.0 in.

Cramps shall be of the shapes and dimensions shown in the contract documents or approved by the Engineer. They shall be inset in the stone so as to be flush with the surfaces.

Cramps and dowels shall be set in lead, care being taken to completely fill the surrounding spaces with the molten metal, or shall be rigidly anchored by other means approved by the Engineer.

14.4.10—Weep Holes

All walls and abutments shall be provided with weep holes. Unless otherwise specified in the contract documents or directed by the Engineer, the weep holes shall be placed at the lowest points where free outlets can be obtained and shall be spaced not more than 10.0 ft center-to-center. A minimum of 2.0 ft^3 of permeable material encapsulated with filter fabric shall be placed at each weep hole.

14.4.11—Pointing

Pointing shall not be done in freezing weather or when the stone contains frost.

Whenever possible the face joints shall be properly pointed before the mortar becomes set. Joints which cannot be so pointed shall be prepared for pointing by raking them out to a depth of 2.0 in. before the mortar has set. The face surfaces of stones shall not be smeared with the mortar forced out of the joints or that used in pointing.

Joints not pointed at the time the stone is laid shall be thoroughly wet with clean water and filled with mortar. The mortar shall conform to Article 14.2.3, "Mortar," except that the proportion of hydrated lime putty shall be increased to one-half to two times the volume of the cement, or the cement shall be all masonry-type cement. The mortar shall be well driven into the joints and finished with an approved pointing tool. The wall shall be kept wet while pointing is being done and in hot or dry weather the pointed masonry shall be protected from the sun and kept wet for a period of at least three days after completion.

After the pointing is completed and the mortar set, the wall shall be thoroughly cleaned and left in a neat condition.

14.4.12—Arches

The number of courses and the depth of voussoirs shall be as shown in the contract documents. Voussoirs shall be placed in the order indicated in the contract documents, shall be full-size throughout, and shall have bond not less than thickness of the stone. Beds shall be roughly pointed to bring them to radial planes. Radial joints shall be in planes parallel to the transverse axis of the arch and, when measured at the intrados, shall not exceed 0.75 in. in thickness. Joints perpendicular to the arch axis shall not exceed 1.0 in. in thickness when measured at the intrados. The intrados face shall be dressed sufficiently to permit the stone to rest properly upon the centering. Exposed faces of the arch ring shall be rock-faced with edges pitched to true lines.

The work shall be carried up symmetrically about the crown, the stone being laid in full mortar beds, and the joints grouted where necessary. Pinning by the use of stone spalls shall not be permitted.

Backing may consist of Class B concrete or of large stones shaped to fit the arch, bonded to the spandrels, and laid in full beds of mortar. The extrados and interior faces of the spandrel walls shall be given a finished coat of 1:2.5 cement mortar which shall be trowelled smooth to receive the waterproofing.

Arch centering, waterproofing, draining, and filling shall be as specified in Section 3, "Temporary Works," Section 8, "Concrete Structures," and Section 21, "Waterproofing."

14.5—MEASUREMENT AND PAYMENT

Stone masonry shall be measured by either the cubic yard or the square foot as listed in the contract documents. The volume or area shall be that actually placed to the limiting dimensions specified in the contract documents, or the contract document dimensions as may have been revised by the Engineer.

Stone masonry, as measured above, shall be paid for by the contract price per cubic yard or square foot. Such payment shall be considered to be full compensation for the cost of all labor, tools, materials, and other items incidental to the satisfactory completion of the work.

Concrete used in connection with stone masonry shall be measured and paid for in the same manner as concrete for structures.

14.6—REFERENCES

AASHTO. 2002. *Standard Specifications for Highway Bridges*, 17th Edition, HB-17, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2009. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 29th Edition, HM-29, American Association of State Highway and Transportation Officials, Washington, DC.

TABLE OF CONTENTS

SECTION 15

STONE CONCRETE BLOCK AND BRICK MASONRY

15.1—DESCRIPTION

Concrete block and brick masonry shall consist of concrete blocks or brick laid in cement mortar and may be unreinforced or reinforced with steel reinforcing. Block or brick pavements are not included under this designation.

C15.1

The *AASHTO LRFD Bridge Design Specifications* do not cover concrete block and brick masonry. The provisions of the 17th Edition of the *AASHTO Standard Specifications for Highway Bridges* shall be used.

15.2—MATERIALS

15.2.1—Concrete Block

Unless otherwise specified in the contract documents or approved in writing by the Engineer, all concrete block for masonry construction shall be Type I moisture controlled units (Grade N-I) that meet the requirements of ASTM C90. The value of f'_m shall be as shown in the contract documents.

Concrete block units should be protected from rain, snow, or other moisture during storage on or off the job site to ensure that they will meet the Type I moisture requirements at the time they are placed in the construction.

15.2.2—Brick

Brick for masonry construction shall conform to the Specification for Building Brick (solid masonry units made from clay or shale) AASHTO M 114 (ASTM C62), Concrete Building Brick (ASTM C55), or Solid Load-Bearing Concrete Masonry Units (ASTM C90). The type and grade of brick to be furnished shall be as specified in the contract documents.

The bricks shall have a fine-grained, uniform, and dense structure, free from lumps of lime, laminations, cracks, checks, soluble salts, or other defects which may in any way impair their strength, durability, appearance, or usefulness for the purpose intended. Bricks shall emit a clear, metallic ring when struck with a hammer.

15.2.3—Reinforcing Steel

Reinforcing steel used in the construction of concrete block or brick masonry structures shall conform to the requirements for uncoated reinforcing in Section 9, "Reinforcing Steel."

15.2.4—Mortar

Mortar used shall conform, as regards materials, proportions, and mixing, to the mortar specified in Articles 14.2.3, "Mortar," and 14.4.2, "Mixing Mortar."

15.2.5—Grout

Grout for filling voids in hollow masonry units shall either conform to the requirements of ASTM C476 or to the requirements of the following paragraph.

As an alternative to the requirements of ASTM C476, the materials for grout shall conform to the requirements of Section 8, "Concrete Structures," for cement, aggregates, water, and admixtures and to the requirements of Article 14.2.3, "Mortar," for lime. Coarse aggregate shall be of either 0.5-in. or 0.375-in. maximum gradation. For fine grout, if proportioned by volume, the cementitious materials shall consist of one part portland cement to no more than 0.1 part hydrated lime or lime putty and the aggregates shall consist of sand in the amount of 2.25 to three times the total volume of cementitious materials. For coarse grout, the proportions shall be the same as for fine grout except that coarse aggregate in the amount of one to two times the total volume of cementitious materials shall be added. If proportioned by weight (mass), the weight (mass) used shall be equivalent to those which would be obtained by volumetric methods.

Adjustments in mix proportions, within the limits allowed, shall be made as necessary to satisfy workability and strength requirements.

Admixtures shall be used only when specified in the contract documents or approved by the Engineer.

15.2.6—Sampling and Testing

15.2.6.1—Mortar

Unless otherwise specified in the contract documents, mortar shall have a minimum 28-day compressive strength of 1.8 ksi, based on the average of three specimens tested in accordance with the requirements of ASTM C780. Field samples shall be obtained using the following procedure:

- Spread 0.5 in. or the thickness of the mortar joint of mortar on masonry units.
- After 1 min, remove mortar and compress into 2.0 in. × 4.0in. cylinder in two layers using flat end of a rod or fingers, being sure to see that mold is solidly filled.
- Lightly tap cylinder immediately and maintain in damp condition.
- After 48 h, remove mold and store in fog room until testing.

15.2.6.2—Grout

When required by the contract documents or requested by the Engineer, the Contractor shall manufacture grout prisms for testing. Prisms shall be manufactured at the site during construction using the following procedure:

- Place masonry units, having same moisture condition as those being placed, on nonabsorptive base to form a void for a square prism with a height twice the side and a minimum side of 3.0 in.

- Line the side faces of the prism with permeable paper or porous separator to allow water passage through liner into masonry units.

- Fill prism with a fully representative grout sample in two layers. Puddle each layer to eliminate air voids.

- Level off specimen and maintain in a damp condition.

- Remove prisms from masonry units after 48 h and deliver to Engineer.

Grout prisms shall be tested in accordance with the provisions of AASHTO T 22 (ASTM C39/C39M). Grout shall have attained a compressive strength of 2.0 ksi at 28 days unless otherwise specified in the contract documents.

15.3—CONSTRUCTION

15.3.1—Weather Conditions

Block or brick masonry shall not be constructed in freezing weather or when the block or brick contains frost, except by written permission of the Engineer and subject to such conditions which the Engineer may require.

15.3.2—Laying Block and Brick

The blocks or bricks shall be laid in such manner as will thoroughly bond them into the mortar by means of the "shove-joint" method; "buttered" or plastered joints shall not be permitted. All clay or shale brick shall be thoroughly saturated with water before being laid. Dampening of concrete masonry units before or during construction shall not be permitted unless approved by the Engineer. The arrangement of headers and stretchers shall be such as will thoroughly bond the mass and, unless otherwise specified, work shall be of alternate headers and stretchers with consecutive courses breaking joints. Other types of bonding, as for ornamental work, shall be as specified in the contract documents.

All joints shall be completely filled with mortar. They shall not be less than 0.25 in. and not more than 0.625 in. in thickness, and the thickness shall be uniform throughout. All joints shall be finished properly as the work progresses, and on exposed faces they shall be neatly struck, using the "weather" joint.

No spalls or bats shall be used except for shaping around irregular openings or when unavoidable to finish out a course, in which case full bricks shall be placed at the corners, the bats being placed in the interior of the course.

Each masonry unit shall be adjusted to its final position while mortar is still soft and plastic. Units which are disturbed after mortar has stiffened shall be removed and relaid in fresh mortar.

Vertical cells to be filled with grout shall be aligned to provide a continuous unobstructed opening.

Piers and walls may be built of solid brick work, or may consist of a brick or block shell backed with concrete or other suitable material as specified in the contract documents. All details of the construction shall be in accordance with approved practice and to the satisfaction of the Engineer.

15.3.3—Placement of Reinforcement

Prior to and during grouting, the reinforcing steel shall be securely held in position at the top and bottom and at intermediate points not exceeding 200 bar diameters or 10.0 ft apart. Bars shall be maintained clear of the cell walls and within ±0.5 in. of their planned position transverse to the wall and within ±2.0 in. of their planned position longitudinal to the wall.

15.3.4—Grouting of Voids

Grouted masonry shall be constructed in such a manner that all elements of the masonry act together as a structural element.

Prior to grouting, the grout space shall be clean so that all spaces to be filled with grout do not contain mortar projections greater than 0.5 in., mortar droppings, or other foreign material. Grout shall be placed so that all spaces to be grouted do not contain voids.

Grout materials and water content shall be controlled to provide adequate fluidity for placement without segregation.

Size and height limitations of the grout space or cell on the average shall not be less than shown in Table 15.3.4-1. Higher grout pours or smaller cavity widths or cell size than shown in Table 15.3.4-1 may be used when approved by the Engineer, if it is demonstrated that grout spaces are properly filled.

When required by Table 15.3.4-1, cleanouts shall be provided in the bottom course at every vertical bar, but shall not be spaced more than 32.0 in. on center for solidly grouted masonry. Cleanouts shall be of sufficient size to allow removal of debris.

Units may be laid to the full height of the grout pour, and grout shall be placed in a continuous pour in grout lifts not exceeding 6.0 ft. If construction joints are used in columns of grout, they shall be located at least 1.5 in. below the level of a mortar bed joint.

Segregation of the grout materials and damage to the masonry shall be avoided during the grouting process.

Grout shall be consolidated before loss of plasticity in a manner to fill the grout space. Grout pours greater than 12.0 in. in height shall be mechanically reconsolidated to minimize voids due to water loss. Grout not mechanically vibrated shall be puddled.

In nonstructural elements, mortar of pouring consistency may be substituted for grout when the masonry is constructed and grouted in pours of 12.0 in. or less.

Vertical barriers of masonry may be built across the grout space. The grouting of any section of wall between barriers shall be completed in one day with no interruption longer than 1 h.

Table 15.3.4-1—Grouting Limitations

		Least Clear Dimensions		
Grout Type	Grout Pour Maximum Height, ft	Width of Grout Space, in.	Cell Dimensions, in. × in.	Cleanouts Required
Fine	1.0	0.75	1.5 × 2.0	No
Fine	5.0	1.5	1.5 × 2.0	No
Fine	8.0	1.5	1.5 × 3.0	Yes
Fine	12.0	1.5	1.75 × 3.0	Yes
Fine	24.0	2.0	3.0 × 3.0	Yes
Coarse	1.0	1.5	1.5 × 3.0	No
Coarse	5.0	2.0	2.5 × 3.0	No
Coarse	8.0	2.0	3.0 × 3.0	Yes
Coarse	12.0	2.5	3.0 × 3.0	Yes
Coarse	24.0	3.0	3.0 × 4.0	Yes

15.3.5—Copings, Bridge Seats, and Backwalls

The tops of retaining walls, abutment wingwalls, and similarly exposed brick or block work shall be provided, in general, with either a stone or concrete coping. The underside of the coping shall have a batter or drip bead at least 1.0 in. beyond the face of the block or brick work wall. The coping upon an abutment backwall shall commonly have no projection beyond its bridge seat face. When concrete is used, it shall conform to the requirements for Class A concrete specified in Section 8, “Concrete Structures.” For thin copings, mortar of the same proportions as used for laying the block or brick may be used to produce precast sections not less than 3.0 ft nor more than 5.0 ft in length. No coping shall be less than 4.0 in. thick.

Copings of piers and abutment bridge seats shall be of Ashlar stone work or of Class A concrete and shall conform to the requirements for Article 14.1.2, “Ashlar Masonry,” or for concrete as specified in Section 8, “Concrete Structures,” as the contract documents may indicate. Unless otherwise shown in the contract documents, concrete shall be used.

C15.3.5

Ashlar masonry is composed of rectangular stones laid with mortar in horizontal courses.

15.4—MEASUREMENT AND PAYMENT

Concrete block and brick masonry shall be measured by the number of cubic yards or the number of square feet of the type of masonry actually placed in the structure, in accordance with the contract documents or as modified by written instructions from the Engineer. The units of measure for the various types of masonry shall be as listed in the contract documents.

Concrete block and stone masonry, as measured above, shall be paid for by the contract price per cubic yard or square foot. Such payment shall be considered to be full compensation for the cost of all labor, equipment, materials, and other expenses incidental to the satisfactory completion of the work. Filling material for the interior of the wall, reinforcing steel, and concrete or mortar copings shall be considered as included in the price paid for number of cubic yards or square feet of block or brick masonry actually placed.

15.5—REFERENCES

AASHTO. 2002. *Standard Specifications for Highway Bridges*, 17th Edition, HB-17, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2009. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 29th Edition, HM-29, American Association of State Highway and Transportation Officials, Washington, DC.

SECTION 16: TIMBER STRUCTURES

TABLE OF CONTENTS

SECTION 16

TIMBER STURCTURES

16.1—GENERAL

This work shall consist of constructing timber structures and the timber portions of composite structures, in accordance with these Specifications and in reasonably close conformity with the details specified in the contract documents or established by the Engineer.

It shall include furnishing, preparing, fabricating, erecting, treating, and painting of timber. All timber, treated or untreated, shall be of the specified species, grades, and dimensions. Also included shall be any required yard lumber of the sizes and grades specified and all hardware required for timber connections and ties.

16.1.1—Related Work

Other work involved in the construction of timber structures shall be as specified in the applicable sections of this Specification including, but not limited to:

- Section 4, "Driven Foundation Piles,"
- Section 13, "Painting,"
- Section 17, "Preservative Treatment of Wood," and
- Section 20, "Railings."

16.2—MATERIALS

16.2.1—Lumber and Timber (Solid Sawn or Glue Laminated)

Sawn lumber and timber shall conform to the Specifications for Wood Products, AASHTO M 168.

Structural glue laminated timber shall conform to the American National Standard ANSI/AITC A-190.1, Specification for Structural Glue Laminated Timber. The separate laminations may not exceed 2.0 in. in net thickness. They may be comprised of pieces end-joined to form any length, of pieces placed or glued edge to edge to make wider ones, or of pieces bent to curved form during gluing. On glue laminated structural members that are not to be preservatively treated, an approved end sealer shall be applied after end trimming of each completed member.

The grades of timber used for various structural purposes shall be as specified in the contract documents.

Structural lumber and timber, solid sawn or glue laminated, in exposed permanent structures, other than running planks on decks, shall be treated in conformance with the requirements of Section 17, "Preservative Treatment of Wood." Preservative treatment may be omitted for temporary structures or lumber and timber of certain species with adequate heartwood requirements, as listed in AASHTO M 168, when permitted by the contract documents.

C16.2.1

Structural glue laminated timber, as employed in ANSI/AITC A-190.1, is an engineered, stress-rated product of a timber laminating plant, comprising assemblies of suitably selected and prepared wood laminations securely bonded together with wet-use adhesives. The grain of all laminations is approximately parallel longitudinally.

When the contract documents require certification of quality for timber or lumber, the Contractor shall furnish the following Certificates of Compliance to the Engineer, as appropriate, upon delivery of the materials to the job site:

- For timber and lumber, a certification by an agency certified by the American Lumber Standards Committee that the timber or lumber conforms to the grade, species, and any other specified requirements.
- For glue laminated timber, a certification by a qualified inspection and testing agency that the glue laminated timber complies with the grade, species, and other requirements outlined in ANSI/AITC A-190.1.
- If the wood is to be treated with a preservative, a Certificate of Compliance, as specified in Article 17.3.3, "Certificate of Compliance," shall be furnished.

16.2.2—Steel Components

Rods, plates, eyebars, and shapes shall conform to the requirements of AASHTO M 270M/M 270 (ASTM A709/A709M), Grade 36 (Grade 250), unless otherwise specified in the contract documents.

16.2.3—Castings

Castings shall be cast steel or gray-iron, as specified in the contract documents, conforming to the requirements of Article 11.3.5, "Steel Castings," or 11.3.6, "Iron Castings."

16.2.4—Hardware

Bolts, nuts, drift-bolts, and dowels may be of mild steel. Washers may be cast iron ogee or malleable iron castings, or they may be cut from mild steel plate, as specified in the contract documents.

Bolts shall have either standard square, hex or dome heads, or economy type (washer) heads. Nails shall be cut or round wire of standard form. Spikes shall be cut or wire spikes, or boat spikes, as specified. Unless otherwise specified, bolts shall comply with ASTM A307, and shall have coarse threads, Class 2 tolerance conforming to ANSI B18.2.2, Square and Hex Nuts (Inch Series) (ANSI B18.2.4.6M, Metric Heavy Hex Nuts).

All fasteners, including nails, spikes, bolts, dowels, washers, and lag screws shall be galvanized, unless otherwise specified in the contract documents or permitted.

16.2.5—Galvanizing

16.2.5.1—General

Unless otherwise specified in the contract documents, all hardware for timber structures shall be galvanized in accordance with AASHTO M 232M/M 232 (ASTM A153/A153M) or cadmium plated in accordance with AASHTO M 299 (ASTM B696). All steel components, timber connectors, and castings, other than malleable iron, shall be galvanized in accordance with AASHTO M 111M/M 111 (ASTM A123/A123M).

16.2.6—Timber Connectors

16.2.6.1—Dimensions

The various types of timber connectors should generally conform to the dimensions shown in Tables 16.2.6.1-1a and 16.2.6.1-1b and to the dimensions specified in this Article.

Table 16.2.6.1-1a—Typical Dimensions of Timber Connectors

Split Rings		
	2.5 in.	4.0 in.
Split Ring		
Inside Diameter at Center When Closed	2.500	4.00
Thickness of Metal at Center	0.163	0.193
Depth of Metal (Width of Ring)	0.750	1.00
Groove		
Inside Diameter	2.56	4.08
Width	0.18	0.21
Depth	0.375	0.50
Bolt Diameter	0.5	0.75
Hole Diameter	0.5625	0.8125
Washers, Standard		
Round, Cast or Malleable Iron		
Diameter	2.625	3.0
Round, Mild Steel		
Diameter	1.375	2.0
Thickness	0.09375	0.15625
Square Plate, Mild Steel		
Length of Side	2.0	3.0
Thickness	0.125	0.1875

Table 16.2.6.1-1b—Typical Dimensions of Timber Connectors

Shear Plates, in.				
	2.625	2.625	4.0	4.0
Shear Plate:				
Material	Pressed Steel	Light Gage	Malleable	Malleable
Diameter of Plate	2.62	2.62	4.03	4.03
Diameter of Bolt Hole	0.81	0.81	0.81	0.94
Thickness of Plate	0.172	0.12	0.20	0.20
Depth of Flange	0.42	0.35	0.64	0.64
Steel straps or shapes for use with shear plates shall be designed in accordance with accepted engineering practices.				
Hole Diameter in Straps or Shapes for Bolts	0.8125	0.8125	0.8125	0.9375
Circular Dap—Dimensions:				
A	2.63	2.63	4.03	4.03
B	—	1.07	1.55	1.55
C	0.81	0.81	0.81	0.94
D	—	0.65	0.97	0.97
E	0.19	0.13	0.27	0.27
F	0.45	0.38	0.64	0.64
G	0.25	0.14	0.22	0.22
H	—	0.34	0.50	0.50
I	2.25	2.37	3.49	3.49
Bolt Hole—Diameter in Timber	0.8125	0.8125	0.8125	0.9375
Washers, Standard:				
Round, Cast, or Malleable Iron Diameter	3.0	3.0	3.0	3.5
Round, Medium Steel, minimum				
Diameter	2.0	2.0	2.0	2.25
Thickness	0.15625	0.15625	0.15625	0.171875
Square Plate:				
Length of Side	3.0	3.0	3.0	3.0
Thickness	0.25	0.25	0.25	0.25

16.2.6.2—Split Ring Connectors

Split rings of 2.5-in. inside diameter and 4.0-in. inside diameter shall be manufactured from hot-rolled carbon steel conforming to the Society of Automotive Engineers Specification SAE 1010. Each ring shall form a closed true circle with the principal axis of the cross section of the ring metal parallel to the geometric axis of the ring. The metal section shall be beveled from the central portion toward the edges to a thickness less than the midsection. It shall be cut through in one place in its circumference to form a tongue and slot.

C16.2.6.2

Split ring connector dimensions refer to SAE 1010 carbon steel located in the *Society of Automotive Engineers Specification Manual*, Volume 1, 1995.

16.2.6.3—Shear-Plate Connectors

Pressed steel shear-plates of 2.625-in. diameter shall be manufactured from hot-rolled carbon steel conforming to the *Society of Automotive Engineers* Specification SAE 1010. Each plate shall be a true circle with a flange around the edge, extending at right angles to the face of the plate and extending from one face only, the plate portion having a central bolt hole and two small perforations on opposite sides of the hole and midway from the center and circumference.

Malleable iron shear-plates of 4.0-in. diameter shall be manufactured according to ASTM A47/A47M, Grade 32510 (Grade 22010), for malleable iron casting. Each casting shall consist of a perforated round plate with a flange around the edge extending at right angles to the face of the plate and projecting from one face only, the plate portion having a central bolt hole reamed to size with an integral hub concentric to the bolt hole and extending from the same face as the flange.

C16.2.6.3

Split ring connector dimensions refer to SAE 1010 carbon steel located in the *Society of Automotive Engineers Specification Manual*, Volume 1, 1995.

16.2.6.4—Spike-Grid Connectors

Spike-grid timber connectors shall be manufactured according to ASTM A47/A47M, Grade 32510 (Grade 22010), for malleable iron casting.

Square grids shall consist of four rows of opposing spikes forming a 4.125-in. square grid with 16 teeth that are held in place by fillets. Fillets for the flat grid in cross section shall be diamond shaped. Fillets for the single curve grids shall be increased in depth to allow for curvature and shall maintain a thickness between the sloping faces of the fillets equal to the width of the fillet.

Circular grids of 3.25-in. diameter shall consist of eight opposing spikes equally spaced around the outer circumference and held in place by connecting fillets around the outer diameter and radial fillets projecting to a central circular fillet that forms a bolt hole opening of 1.25 in. Fillets in cross section shall be diamond shaped, except that the inner circular fillet may be flattened on one side to provide for manufacturer identification.

16.3—FABRICATION AND CONSTRUCTION

16.3.1—Quality

Quality-of-work shall be first class throughout, and all framing shall be true and exact. Unless otherwise specified in the contract documents, nails and spikes shall be driven with just sufficient force to set the heads flush with the surface of the wood. Deep hammer marks in wood surfaces shall be considered evidence of poor quality and sufficient cause for removal of the worker causing them.

16.3.2—Storage of Material

Lumber and timber stored at the construction site shall be kept in orderly piles or stacks. Untreated material shall be open-stacked on supports at least 12.0 in. above the ground surface to avoid absorption of ground moisture and permit air circulation, and it shall be so stacked and stickered as to permit free circulation of air between the tiers and courses. In particular cases required by the Engineer, the Contractor shall provide protection from the weather by a suitable covering. The ground underneath and in the vicinity of the timber shall be cleared of weeds and rubbish. The storage area shall be chosen or constructed so that water will not collect under or near the stored timber.

16.3.3—Treated Timber

16.3.3.1—Handling

Treated timber shall be carefully handled without sudden dropping, breaking of outer fibers, bruising, or penetrating the surface with tools. It shall be handled with web slings. Cant hooks, peaveys, pikes, or hooks shall not be used. When metal bands are used to bundle members, corner protectors shall be provided to prevent damage to the treated timber.

16.3.3.2—Framing and Boring

All cutting, framing, and boring of treated timbers shall be done before treatment insofar as is practicable. When treated timbers are to be placed in waters infested by marine borers, untreated cuts, borings, or other joint framings below high water elevation shall be avoided.

16.3.3.3—Cuts and Abrasions

All cuts and all recesses formed by countersinking in creosote treated piles or timbers, and all abrasions, after having been carefully trimmed, shall be field-treated as specified either in this paragraph or the following paragraph. Cuts and recesses shall be covered with two applications of a mixture of 60 percent creosote oil and 40 percent roofing pitch or brush-coated with at least two applications of hot creosote oil and covered with hot roofing pitch. Recesses likely to collect injurious materials shall be filled with hot roofing pitch. Unless specified otherwise in the contract documents, hot preservatives shall be heated to a temperature between 150°F and 200°F. Where particularly heavy coatings are required, a suitable plastic compound can be prepared by mixing 10 to 20 percent of creosote and 80 to 90 percent of coal-tar roofing pitch.

For timbers originally treated with pentachlorophenol, creosote, creosote solutions, or water-borne preservatives, all cuts, abrasions, and recesses that occur after treatment shall be field-treated by two liberal applications of a compatible preservative in accordance with the requirements of the *American Wood Preservers Association Standard* M 4, “Standard for the Care of Pressure Treated Wood Products.”

16.3.3.4—Bored Holes

All holes bored after treatment shall be treated by filling the holes with the preservative used for field treatment. After treatment, any holes not filled with bolts or other items shall be plugged with preservative-treated plugs.

16.3.3.5—Temporary Attachment

With the approval of the Engineer, whenever forms or temporary braces are attached to treated timber with nails or spikes, the resulting holes shall be treated as required for bored holes and shall be filled by driving galvanized nails, spikes, or preservative-treated plugs flush with the surface.

16.3.4—Installation of Connectors

As specified in the contract documents, timber connectors shall be one of the following types: split ring, shear plate, or spike grid. The split ring and the shear plate types shall be installed in precut grooves of dimensions as given herein or as recommended by the Manufacturer. Spike grids shall be forced into the wood so that timbers will be in firm contact. Pressure equipment that does not damage the wood shall be utilized. When high-strength bolts or rods fitted with low-friction ball-bearing washers made for this purpose are used to install the connectors, the high-strength bolt shall be replaced with specified bolts for the final installation. All connectors of this type at a joint shall be embedded simultaneously and uniformly.

Connector grooves in timber shall be cut concentric with the bolt hole, shall conform to the cross-sectional shape of the rings, and shall provide a snug fit. Inside groove diameter shall be larger than nominal ring diameter as specified in Tables 16.2.6.1-1a and 16.2.6.1-1b in order that the ring will expand slightly during installation.

Fabrication of all structural members using connectors shall be done prior to preservative treatment. When prefabricated from templates or shop details, bolt holes shall not be more than 0.0625 in. from required placement. Bolt holes shall be 0.0625 in. larger than the finished bolt diameter. Bolt holes shall be bored perpendicular to the face of the timber.

Timber after fabrication shall be stored in a manner that will prevent changes in the dimensions of the members before assembly. Timber should be cured before fabrication so that it will remain stable in its dimensions. Timber that shrinks during storage, causing predrilled grooves for split rings or plates to become elliptical or causing bolt hole spacing to change, will be sufficient reason for rejection.

16.3.5—Holes for Bolts, Dowels, Rods, and Lag Screws

Holes for round drift-bolts and dowels shall be bored with a bit 0.0625 in. less in diameter than the bolt or dowel to be used. The diameter of holes for square drift-bolts or dowels shall be equal to the least dimension of the bolt or dowel.

Holes for machine bolts shall be bored with a bit the same diameter as the finished bolt, except as otherwise provided for bolts in connectors.

Holes for rods shall be bored with a bit 0.0625 in. greater in diameter than the finished rod.

Holes for lag screws shall be bored with a bit not larger than the body of the screw at the base of the thread. To prevent splitting or stripping the threads, the hole for the shank shall be bored the same diameter and to the same depth as the shank. The depth of holes for lag screws shall be approximately 1.0 in. less than the length under the head.

16.3.6—Bolts and Washers

A washer of the size and type specified shall be used under all bolt heads (except for timber bolts with economy-type heads) and nuts which would otherwise come in contact with wood.

The nuts of all bolts shall be effectually locked after they have been finally tightened.

16.3.7—Countersinking

Countersinking shall be done where smooth or flush surfaces are required. All recesses in treated timber, formed for countersinking, shall be treated as specified in Article 16.3.3.3, "Cuts and Abrasions." Recesses likely to collect injurious materials shall be filled with hot roofing pitch.

16.3.8—Framing

All lumber and timber shall be accurately cut and framed to a close fit in such manner that the joints will have even bearing over the entire contact surfaces. Mortises shall be true to size for their full depth and tenons shall fit snugly. No shimming shall be permitted in making joints, nor shall open joints be accepted.

16.3.9—Framed Bents

16.3.9.1—Mud Sills

Mud sills shall be firmly and evenly bedded to solid bearing and tamped in place. Mud sills shall be pressure preservative treated for ground contact. Where untreated timber is permitted for mud sills, it shall be of heart cedar, heart cypress, redwood, or other durable timber as approved by the Engineer.

16.3.9.2—Concrete Pedestals

Concrete pedestals for the support of framed bents shall be carefully finished so that the sills or posts will take even bearing. Dowels for anchoring sills or posts shall be not less than 0.75 in. in diameter and project at least 6.0 in. above the tops of the pedestals. These dowels shall be cast in the concrete pedestals. Concrete and reinforcing steel shall conform to the requirements of Section 8, "Concrete Structures," and Section 9, "Reinforcing Steel," respectively.

16.3.9.3—Sills

Sills shall have true and even bearing on mud sills, piles, or pedestals. They shall be drift-bolted to mud sills or piles with bolts of not less than 0.75-in. diameter and extending into the mud sills or piles at least 6.0 in., or by other types of connectors as detailed in the contract documents. When possible, all earth shall be removed from contact with sills so that there will be free air circulation around the sills.

16.3.9.4—Posts

Posts shall be fastened to pedestals with dowels of not less than 0.75-in. diameter, extending at least 6.0 in. into the posts, or by other types of connectors as detailed in the contract documents.

Posts shall be fastened to sills by one of the following methods, as specified in the contract documents:

- By dowels of not less than 0.75-in. diameter, extending at least 6.0 in. into posts and sills.
- By drift-bolts of not less than 0.75-in. diameter driven diagonally through the base of the post and extending at least 9.0 in. into the sill. Drift-bolts shall be driven in holes as required by Article 16.3.5, "Holes for Bolts, Dowels, Rods, and Lag Screws," at a 45-degree angle and shall enter the post at least 6.0 in. above the post base.
- By other types of connectors as detailed in the contract documents.

16.3.9.5—Caps

Timber caps shall be placed, with ends aligned, in a manner to secure an even and uniform bearing over the tops of the supporting posts or piles. All caps shall be secured by drift-bolts of not less than 0.75-in. diameter, extending at least 9.0 in. into the posts or piles, or by other types of connectors as detailed in the contract documents. The drift-bolts shall be approximately in the center of the post or pile.

16.3.9.6—Bracing

Bracing shall be bolted through the pile, post, or cap at the ends and at all intermediate intersections using a bolt of not less than 0.625 in. in diameter. Bracing shall be of sufficient length to provide a minimum distance of 8.0 in. between the outside bolt and the end of the brace.

16.3.10—Stringers

Stringers shall be sized at bearings and shall be placed in position so that knots near edges will be in the top portions of the stringers.

Outside stringers may have butt joints with the ends cut on a taper, but interior stringers shall be lapped to take bearing over the full width of the floorbeam or cap at each end. The lapped ends of untreated stringers shall be separated at least 0.5 in. for the circulation of air and shall be securely fastened by drift-bolting where specified in the contract documents. When stringers are two panels in length, the joints shall be staggered.

Unless otherwise specified in the contract documents, cross-bridging or blocking shall be placed at the center of each span. Cross-bridging between stringers shall be neatly and accurately framed and securely toe-nailed with at least two nails in each end. All cross-bridging members shall have full bearing at each end against the sides of stringers. Blocking shall be snug-fit and held in place by either prefabricated galvanized steel beam hangers or by tie-rods, as detailed in the contract documents.

16.3.11—Plank Floors

Unless otherwise specified in the contract documents, planks for flooring shall be surfaced four sides (S 4 S).

Single-plank floors shall consist of a single thickness of plank supported by stringers or joists. The planks shall be laid heart side down, with 0.25-in. openings between them for seasoned material and with tight joints for unseasoned material. Each plank shall be securely spiked to each joist. The planks shall be carefully graded as to thickness and so laid that no two adjacent planks shall vary in thickness by more than 0.125 in.

Two-ply timber floors shall consist of two layers of flooring supported on stringers or joists. The top course shall be laid either diagonal or parallel to the centerline of roadway, as specified in the contract documents, and each floor piece shall be securely fastened to the lower course. Joints shall be staggered at least 3.0 ft. If the top flooring is placed parallel to the centerline of the roadway, special care shall be taken to securely fasten the ends of the flooring. At each end of the bridge these members shall be beveled.

16.3.12—Nail-Laminated or Strip Floors

The strips shall be placed on edge, at right angles to the centerline of roadway. Each strip shall be nailed to the preceding strip as shown in Figure 16.3.12-1. The spikes shall be of sufficient length to pass through two strips and at least halfway through the third strip.

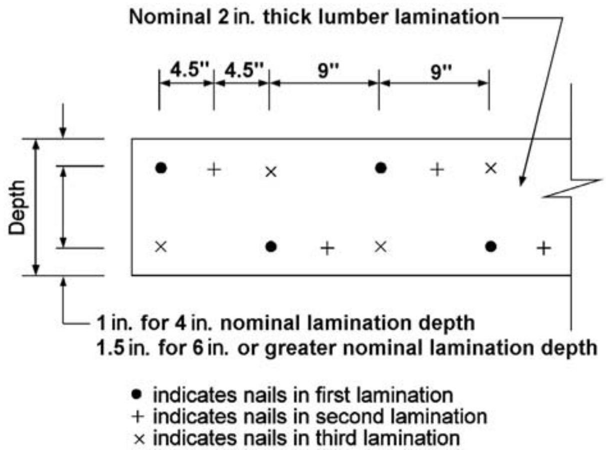


Figure 16.3.12-1—Nail Placement Pattern

If timber supports are used, every other strip shall be toe-nailed to every other support. The size of the spikes shall be as shown in the contract documents. When specified in the contract documents, the strips shall be securely attached to steel supports by the use of approved galvanized metal clips. Care shall be taken to have each strip vertical and tight against the preceding strip, and bearing evenly on all the supports.

16.3.13—Glue Laminated Panel Decks

Unless otherwise specified in the contract documents, deck panels shall be pressure preservative treated with creosote or pentachlorophenol with Type A, C, or D carrier. When it is not possible to complete the fabrication and drilling of glulam members for field connections before treating, a preservative treatment shall be applied to cut or drilled areas in the field, in accordance with Articles 16.3.3.3, "Cuts and Abrasions," and 16.3.3.4, "Bored Holes."

Panels shall not be dragged or skidded. Glue laminated deck panels shall be handled and transported in a way to prevent bending the panels, especially transverse to the laminated pieces. When lifted, they shall be supported at a sufficient number of points to avoid overstressing, and the edges shall be protected from damage.

When dowels are shown on the drawings between deck panels, a template or drilling jig shall be used to ensure that dowel holes are accurately spaced. The holes shall be drilled to a depth 0.25 in. greater than one-half the dowel length and of the same diameter as the dowel unless otherwise shown in the contract documents. A temporary dowel shall be used as a check for snug fit prior to production drilling. The dowels shall be of the size shown in the contract documents with the tips slightly tapered or rounded. A lubricant may be used to facilitate the connection process.

The tips of the dowels shall be partially and equally started into the holes of the two panels being joined. The panels shall be drawn together, keeping the edges parallel, until the panels abut tightly. Each panel shall be securely fastened to each stringer or girder as shown in the contract documents.

16.3.14—Composite Wood-Concrete Decks

Shear connectors needed to resist shear and provide hold-down capacity between timber and concrete elements that are designed for composite action shall be furnished and installed in conformance with the details shown in the contract documents. If no such details are provided and the construction is described in the contract documents as being composite, the Contractor shall submit working drawings for such details and devices for approval by the Engineer before the subject work is begun.

16.3.15—Wheel Guards and Railing

Wheel guards and railing shall be accurately framed in accordance with the contract documents and erected true to line and grade. Unless otherwise specified in the contract documents, wheel guards, rails, and rail posts shall be surfaced four sides (S 4 S). Wheel guards shall be laid in sections not less than 12.0 ft long, except where necessary to match expansion joints or end joints.

Railings shall conform to the requirements in Section 20, "Railings."

16.3.16—Trusses

Trusses, when completed, shall show no irregularities of line. Chords shall be straight and true from end to end in horizontal projection and, in vertical projection, shall show a smooth curve through panel points conforming to the correct camber. All bearing surfaces shall fit accurately. Uneven or rough cuts at the points of bearing shall be cause for rejection of the piece containing the defect.

16.4—PAINTING

Rails and rail posts of timber and any other parts designated in the contract documents to be painted shall be painted with three coats of specification paint. Paint and its application shall conform to the requirements in Section 13, "Painting."

Metal parts, except for hardware, galvanized or cadmium-plated metal, and malleable iron, shall be given one coat of shop paint and, after erection, two coats of field paint as specified in Section 13, "Painting."

16.5—MEASUREMENT

Measurements of lumber and timber shall be computed from the nominal dimensions and actual lengths. The cross-sectional dimensions in the contract documents shall be interpreted as standard sizes. The standard cross-sectional dimensions shall be used in the computations even though the actual size is less in the dimension specified.

Timber in wheel guards shall be included. Timber in piling, railing, and other items for which separate payment is provided shall not be included.

Measurements for glue laminated girders and beams shall be computed from the applicable finished dimensions and actual lengths. Quantities for glue laminated girders and beams to be paid for by the linear foot for each size and stress combination.

The measurement of lumber and timber and of glue laminated girders and beams shall include only such material as is a part of the completed and accepted work, and shall not include materials used for erection purposes, such as falsework, bracing, and sheeting.

The quantities to be paid for shall be the number of thousand feet board measure [abbreviated MBM in the lumber trade] (cubic meters) of each species and grade of lumber and timber listed in the contract documents, complete in place and accepted.

16.6—PAYMENT

Payment for timber, lumber, and glue laminated girders and beams shall be considered to be full compensation for all costs of furnishing of materials, including hardware and timber connectors, preservative treatment, equipment, tools, and labor for the fabrication, erection, and painting necessary to complete all of the work in compliance with the contract documents in a satisfactory manner.

Metal parts, other than hardware and timber connectors, shall be measured and paid for as provided in Section 23, "Miscellaneous Metal."

Railings and concrete shall be measured and paid for as provided in Sections 20, "Railings," and 8, "Concrete Structures," respectively.

16.7—REFERENCES

AASHTO. 2009. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 29th Edition, HM-29, American Association of State Highway and Transportation Officials, Washington, DC.

ANSI and AITC. 2002. *American National Standard for Structural Glued Laminated Timber*, ANSI/AITC A190.1, American Institute of Timber Construction, Englewood, CO.

ASME. 1979. *Metric Heavy Hex Nuts*, B18.2.4.6M, American Society of Mechanical Engineers, Fairfield, NJ. Reaffirmed 1998.

ASME. 1987. *Square and Hex Nuts*, B18.2.2, American Society of Mechanical Engineers, Fairfield, NJ. Inch series. Reaffirmed 1999.

AWPA. 2002. *Standard for the Care of Preservative-Treated Wood Products*, AWPA M4-02, American Wood-Preservers' Association, Selma, AL.

SAE. 1995. *Society of Automotive Engineers Specification Manual*, Volume 1, Society of Automotive Engineers, Warrendale, PA.

TABLE OF CONTENTS

SECTION 17

PRESERVATIVE TREATMENT OF WOOD

17.1—GENERAL

This work shall consist of treating wood, including lumber, timber, piles, and poles, with designated preservatives in accordance with these Specifications. It shall include furnishing, preparing, and treating all materials, and performing all work to complete treating the wood products required for the project.

The type of preservative treatment required shall be as specified in the contract documents.

When a specific type of preservative is not called for, the kind of preservative to be used shall be adopted for its suitability to the conditions of exposure to which it shall be subjected and shall be subject to approval of the Engineer.

The handling and care of treated woods shall conform to the requirements of Sections 4, "Driven Foundation Piles," and 16, "Timber Structures."

17.2—MATERIALS

17.2.1—Wood

Piling shall conform to the requirements of Section 4, "Driven Foundation Piles." Timber and lumber shall conform to the requirements of Section 16, "Timber Structures."

17.2.2—Preservatives and Treatments

Timber preservatives and treatment methods shall conform to AASHTO M 133. The type of preservative furnished shall be in accordance with that specified in the contract documents.

Unless otherwise specified in the contract documents, timber railings and posts and timber that are to be painted shall be treated with pentachlorophenol with a Type C solvent or with a water-borne preservative of either Type CCA or ACZA.

C17.2.2

AASHTO M 133 designates the preservatives and retentions recommended for coastal waters and in marine structures and further that timber for use in "ground or water contact" has requirements that differ from timbers for use "not in ground or water contact." In some instances, there is a range of retentions offered which provides for different degrees of exposure based on climate or degree of insect infestation. Unless the higher retentions are specified in the contract documents, not less than the minimum retention is required.

17.2.3—Coal-Tar Roofing Cement

For purposes of these Specifications, pitch, coal-tar pitch, coal-tar roofing pitch, or coal-tar roofing compound shall mean coal-tar roofing cement wherever the terms are used.

C17.2.3

Coal-tar roofing cement is a residue of the manufacturing of coke and creosote from bituminous coal. It shall be a thick, heavy-bodied, and paste-like material. When called for, it can be mixed with creosote. It may or may not contain fibrous material.

17.3—IDENTIFICATION AND INSPECTION

17.3.1—Branding and Job Site Inspection

Each piece of treated timber shall bear a legible brand, mark, or tag indicating the name of the treater and the specification symbol or specification requirements to which the treatment conforms. Treated wood products bearing the quality mark of the American Wood Preservers Bureau (AWPB) shall be acceptable. The Engineer shall be provided adequate facilities and free access to the necessary parts of the treating plant for inspection of material and work quality to determine that the contract document requirements are met. The Engineer reserves the right to retest all materials after delivery to the job site and to reject all materials which do not meet the requirements of the contract documents, provided that, at the job site reinspection, conformance within five percent of contract document requirements shall be acceptable. Reinspection at the job site may include assay to determine retention of preservatives and extraction and analysis of preservative to determine its quality.

17.3.2—Inspection at Treatment Plant

Unless otherwise specified in the contract documents, inspection of materials and preservative treatment shall be the responsibility of the Contractor and the supplier of treated wood products. Inspections shall be conducted in accordance with AASHTO M 133 by the treater or an independent commercial inspection agency approved by the American Wood Preservers Bureau (AWPB) and the Engineer.

The inspection agency shall be engaged by the Contractor directly or through the Contractor's supplier. No direct compensation will be made for these inspection costs, it being understood that the costs of inspection shall be included in the contract bid prices for treated wood products or construction items of work.

C17.3.2

The AWPB "FDN" grademark is applied only to wood treated under the quality control provisions of the American Wood Preservers Bureau, P.O. Box 6085, 2772 South Randolph Street, Arlington, VA 22206. This grademark provides assurance that the products meet the treatment level required by the standards of the AWPB.

17.3.3—Certificate of Compliance

Whenever specified in the contract documents or requested by the Engineer, a Certificate of Compliance with copies of the inspection reports attached shall be furnished to the Engineer with each shipment of material. Such certificates shall identify the type of preservative used and the quantity in pounds per cubic foot (assay method) and shall be signed by the treater or the qualified independent inspection agency.

17.4—MEASUREMENT AND PAYMENT

No separate measurement and payment shall be made for preservative treatment, as such work is a part of the work included in furnishing preservative-treated materials.

17.5—REFERENCES

AASHTO. 2009. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 29th Edition, HM-29, American Association of State Highway and Transportation Officials, Washington, DC.

SECTION 18: BEARING DEVICES

TABLE OF CONTENTS

SECTION 18

BEARING DEVICES

18.1—GENERAL

This work shall consist of furnishing and installing bridge bearings and the bedding of materials used under masonry plates.

Bearings shall be constructed in accordance with the details shown in the contract documents. When complete details are not provided, bearings shall be furnished that conform to the limited details shown in the contract documents and shall provide the design capacities for loads and movements shown or specified and the performance characteristics specified.

C18.1

Bearing types include, but are not limited to, elastomeric pad, rocker, roller, pot, spherical, disc, and sliding plate bearings. Included as components of bearings are masonry, sole and shim plates, bronze or copper-alloyed bearing and expansion plates, anchor bolts, guide devices, Polytetrafluorethylene (PTFE) sheets or surfacing, lubricants, and adhesives.

The Designer and the Contractor must provide sufficient information to permit manufacture and certification of the bearing. This requires additional information when a higher level certification is required. The design load is required because it is needed in some of the test procedures.

Designing bearings for replacement is important because even high-quality bearings have in some cases been known to fail because of unanticipated forces or other conditions. Setting the bearing in a shallow recess in the masonry plate is a simple way of making replacement easy.

The mating parts of each bearing, especially machined metal bearings such as self-lubricating bronze bearings, should be furnished by a single Manufacturer in order to ensure proper fitting of the mating surfaces.

18.1.1—Working Drawings

Whenever complete details for bearings and anchorages are not shown in the contract documents, the Contractor shall prepare and submit working drawings for the bearings. Such drawings shall show all details of the bearings and of the materials proposed for use and shall be approved by the Engineer prior to the fabrication of the bearings. Such approval shall not relieve the Contractor of any responsibility under the contract documents for the successful completion of the work.

The following shall be specified on the working drawings:

- The total quantity of each kind of bearing required (fixed, guided expansion, or nonguided expansion), grouped first according to type (load range), and then by actual design capacity.
- The plan view and section elevation view showing all relative dimensions of each type of bearing.
- The maximum design coefficient of friction as noted in the contract documents.
- The type of materials to be used for all bearing elements.
- If applicable, a clear description and details for any welding process used in the bearing manufacture that does not conform to the approved processes of the current AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*.

- The vertical and horizontal load, rotation, and movement capacity.
- Painting or coating requirements.
- Alignment plans.
- Installation scheme.
- Complete design calculations verifying conformance with these Specifications, if required by the Engineer.
- Anchorage details.
- Bearing preset details, if applicable.
- The location of the fabrication plant.
- The Manufacturer's name and the name of the representative who will be responsible for coordinating production, inspection, sampling, and testing.

18.1.2—Materials

18.1.2.1—Rolled Steel

Rolled steel shall be of the type specified in the contract documents and shall satisfy the testing requirements of the standard to which it conforms. Unless otherwise specified, steel shall conform to AASHTO M 270M/M 270 (ASTM A709/A709M), Grade 36 (Grade 250), and shall cause no adverse electrolytic or chemical reaction with other components of the bearing and shall be free of all rust and mill scale.

18.1.2.2—Steel Laminates

Steel laminates shall meet the requirements of AASHTO M 251.

18.1.2.3—Cast Steel

Cast steel shall satisfy the requirements of ASTM A802/A802M and be free of all blow-holes and impurities larger than 0.125 in. The inside wall of the pot in pot bearings and the contact surface of metal rocker or roller bearings shall be free of blow-holes or impurities of any size.

18.1.2.4—Forged Steel

Forged steel shall satisfy the requirements of ASTM A788.

18.1.2.5—Stainless Steel

Stainless steel shall conform to the requirements of ASTM A167, Type 304 or ASTM A240/A240M, Type 304 and shall have a minimum thickness of 20 gage (0.91 mm) and a surface finish in the finished bearing better than or equal to 8.0 μin. Stainless steel in contact with PTFE sheet shall be polished to a finish no less than 20 μin.

C18.1.2.5

Stainless steel welding is sometimes used to create a continuous overlay over carbon steel plate, for example, in spherical sliding bearings. The stainless steel layer so created is then machined to give a smooth surface.

18.1.3—Packaging, Handling, and Storage

Prior to shipment from the point of manufacture, bearings shall be packaged in such a manner to ensure that during shipment and storage the bearings will be protected against damage from handling, weather, or any normal hazard. Each completed bearing shall have its components clearly identified, be securely bolted, strapped, or otherwise fastened to prevent any relative movement, and be marked on its top as to location and orientation in each structure in the project in conformity with the contract documents.

All bearing devices and components shall be stored at the work site in an area that provides protection from environmental and physical damage. When installed, bearings shall be clean and free of all foreign substances.

Dismantling of bearings at the site shall not be done unless absolutely necessary for inspection or installation. Bearings shall not be opened or dismantled at the site except under the direct supervision of or with the approval of the Manufacturer.

C18.1.3

Small amounts of grit, dirt, or other contamination can seriously detract from the good performance which could otherwise be obtained from a bearing. It is, therefore, very important that the bearing should not be opened up on site, except under the supervision of the Manufacturer or his agent.

18.1.4—Manufacture or Fabrication

18.1.4.1—General

The Manufacturer shall certify that each bearing satisfies the requirements of the contract documents and these Specifications, and shall supply the Engineer a certified copy of material test results. Each reinforced bearing shall be marked in indelible ink or flexible paint. The marking shall consist of the orientation, the order number, lot number, bearing identification number, and elastomer type and grade number. Unless otherwise specified in the contract documents, the marking shall be on the face that is visible after erection of the bridge.

Unless otherwise specified in the contract documents, the surface finish of bearing components that come into contact with each other or with concrete, but are not embedded in concrete, shall conform to the requirements of Section 11, "Steel Structures."

Bearing assemblies shall be preassembled in the shop by the supplier and checked for proper completeness and geometry before shipping to the site.

Unless otherwise specified in the contract documents, steel bearing components other than stainless steel, including anchor bolts, shall be galvanized in accordance with Article 11.3.7, "Galvanizing."

C18.1.4.1

In the short term, marking simplifies the identification of the correct bearings and establishing which way up they should be placed at the job site. In the long term, it may permit the removal of bearings after a number of years of service to check the change in material properties over time. It also helps in settling disputes.

18.1.4.2—Fabrication Tolerances

2010 Revision

Plain elastomeric pads and laminated bearings shall be built as specified in the contract documents within the tolerances of AASHTO M 251.

Other fabrication tolerances are given in Table 18.1.4.2-1.

Table 18.1.4.2-1—Fabrication Tolerances

Item		Thickness Tolerance, in.	Dimension Tolerance, in.	Flatness or Out-of-Round Tolerance, in.	Surface Finish, μin.(rms)
Metal Rocker & Roller Bearings		—			
Single Roller:	Diameter	—	–0.063, +0.063	-0.001, +0.001	63
Nested Roller:	Diameter	—	–0.002, +0.002	–0.001, +0.001	63
Rockers:	Diameter	—	–0.125, +0.125	–0.001, +0.001	125
Pins:	Diameter	—	–0.005, +0.000	–0.002, +0.002	32
Bushings:	Diameter	—	–0.000, +0.005	–0.002, +0.002	32
Pot Bearings					
Overall Dimensions		–0.000, +0.250	–0.000, +0.125	—	—
Pot Depth (Inside)		—	–0.000, +0.025	—	—
Pot Wall: Thickness and Average Inside Diameter		–0.000, +0.125	–0.003, +0.003	–0.001, +0.001	32
Pot Base: Top and Bottom Surfaces		–0.000, +0.025	—	Class C	63
Piston: Rim		–0.000, +0.063	–0.003, +0.003	–0.001, +0.001	32
Piston: Top and Bottom Surfaces		–0.000, +0.025	—	Class C	63
Elastomeric Disc (Unstressed)		–0.000, +0.125	–0.000, +0.063	—	—
Disc Bearings					
Overall Dimensions		–0.000, +0.250	–0.000, +0.125	—	—
Shear-Restricting Element		—	–0.000, +0.005	Class A	32
Other Machined Parts		–0.000, +0.063	–0.000, +0.063	Class B	63
Urethane Disc		–0.000, +0.063	–0.000, +0.125	Class B	63
Flat PTFE Sliding Bearings					
PTFE		–0.000, +0.063	–0.000, +0.030	Class A	—
Stainless Steel		–0.000, +0.063	–0.000, +0.125	Class A	#8 Mirror
Flat Bronze and Copper Alloy Sliding Bearings					
Sliding Surfaces		–0.000, +0.125	–0.000, +0.125	Class A	32
Curved PTFE Sliding Bearings					
Convex Radius		—	–0.010, +0.000	–0.002, +0.002	#8 Mirror
Concave Radius		—	–0.000, +0.010	–0.002, +0.002	125
Curved Bronze and Copper Alloy Sliding Bearings					
Convex Radius		—	–0.010, +0.000	–0.002, +0.002	32
Concave Radius		—	–0.000, +0.010	–0.002, +0.002	32
Guides					
Contact Surface		—	–0.000, +0.125	Class A	32
Distance between Guides		—	–0.000, +0.030	—	—
Parallelism of Guides		—	±0.005 rad	—	—
Load Plates					
Overall Dimensions		–0.063, +0.063	–0.250, +0.250	Class A	125
Bevel Slope		±0.002 rad	—	—	—

18.1.5—Testing and Acceptance

18.1.5.1—General

18.1.5.1.1—Scope

Testing and acceptance criteria for bearings shall conform to the minimum requirements specified in Article 18.1.5.2. The Engineer may require more stringent standards.

When bearings are manufactured from a number of components, each component shall satisfy the testing requirements from the applicable article.

The Engineer shall be given free access to inspect the manufacture of the bearings at all times.

C18.1.5.1.1

The purpose of testing is to ensure a good quality finished bearing. The obvious way to achieve this is to conduct rigorous tests on every bearing. However, this is economically infeasible and resorting to other methods may be necessary for certain tests.

18.1.5.1.2—Adapter Plates and Attachments

Each bearing that is sampled for testing with a tapered sole plate or other attachment that impedes the application of a perpendicular loading to the bearing device shall be delivered to the test site accompanied by an unattached adapter plate or attachment. For bearings with a tapered sole plate, the adapter plate shall have a single bevel and be so constructed that, when placed in contact with the tapered sole plate, the two shall form a single body, rectangular in shape and uniform in thickness. For bearings with other attachments, the adapter plate or attachment shall be so constructed as to permit the accurate application of perpendicular loading to the bearing device.

18.1.5.2—Tests

18.1.5.2.1—General

The tests specified herein shall be carried out at the Manufacturer's expense. Unless otherwise agreed by the Engineer, they shall be supervised by an independent testing agency.

Additional tests for specific types of bearings, as specified in other articles of these Specifications, shall also be conducted.

18.1.5.2.2—Material Certification Tests

Material certification tests to determine the physical and chemical properties of all materials shall be conducted in accordance with the appropriate specification governing the material. The test certificates shall be provided to the Engineer.

18.1.5.2.3—Material Friction Test—Sliding Surfaces Only

The coefficient of friction between the two mating surfaces shall be measured. Tests shall be made on samples taken from the same batch of materials as those used in the bearings. Only new materials shall be used and no material that has been previously tested shall be used.

The surfaces shall be thoroughly cleaned with a degreasing solvent. No lubrication other than that specified for the bearings shall be used. The mating surfaces for the test pieces shall have a common area no less than the smaller of the bearing area or 7.0 in.^{2}.

The test pieces shall be loaded in compression to a stress corresponding to the service limit state design capacity of the bearings, which shall be held constant for 1 h prior to and throughout the duration of the sliding test. At least 100 cycles of sliding, each consisting of at least ±1.0 in. of movement, shall then be applied at a temperature of 68°F ± 2°F. The uniform sliding speed shall be 2.5 in. per min.

The sliding friction coefficient shall be computed for each direction of each cycle. The initial static breakaway coefficient of friction for the first cycle shall not exceed twice the design coefficient of friction. The maximum sliding coefficient of friction for all subsequent cycles shall not exceed the design coefficient of friction. Failure of a single sample shall result in rejection of the entire lot.

Following the 100 cycles of testing, the sliding coefficient of friction shall be determined again and shall not exceed the initial value. The bearing or specimen shall show no appreciable sign of wear, bond failure, or other defects.

C18.1.5.2.3

It is important that the material tested here is identical to that used in the finished bearing. In particular, no lubricant whatsoever should be applied during the test unless it is also required in the finished bearing and the stainless steel mating surface should be new for every piece of material tested. Thus, the same piece of stainless steel should not be used for more than one PTFE specimen in the PTFE material tests. The friction coefficients which constitute the performance criteria for the tests are directly related to the values used in design.

The friction coefficient decreases with increasing compressive stress, while it increases with velocity. As such, testing at the design capacity (which is typically greater than the design load and provided by the Owner) will tend to underestimate the coefficient of friction. A testing speed of 2.5 in. per min. is faster than what would be expected in service. Consequently, the biasing effects caused by the compressive stress and speed at which this test is conducted may be somewhat offset.

Static friction is larger than dynamic or sliding friction, and the dynamic coefficient of friction is larger for the first cycle of movement than it is for later cycles.

18.1.5.2.4—Dimension Check

The dimensions of all bearings shall be checked by the Manufacturer and shall be recorded and provided to the Engineer. Failure of a bearing dimension to satisfy any dimensional tolerance shall be cause, at the discretion of the Engineer, for rejection of the bearing or a lot. Flatness shall be checked by placing a precision straightedge on the surface to be checked and by inserting feeler gages between the two. The straightedge shall be placed at different orientations and the worst condition shall be established. No more than three feeler gages may be stacked on top of one another. The straightedge shall be as long as the largest dimension of the flat surface.

All dimension checks shall satisfy the tolerances of Article 18.1.4.2.

C18.1.5.2.4

An accumulation of dimensional tolerances may still result in a bearing not functioning properly. The clearance test, as provided in Article 18.1.5.2.5, provides an additional safeguard for these cases.

18.1.5.2.5—Clearance Test

The components of the bearing shall be moved through their design displacements or rotations to verify that the required clearances exist. If the test is conducted on a rotational component which is not under simultaneous full vertical load, allowance shall be made for the displacements which would be caused by that load.

18.1.5.2.6—Bearing Friction Test—Sliding Surfaces Only

No lubrication shall be applied except that used for the whole lot of bearings. The bearing shall be loaded in compression at its service limit state design capacity, which shall be held constant for 1 h prior to and throughout the duration of the sliding test. At least 12 cycles of sliding, each consisting of the smaller of the design displacement and ±1.0 in. of movement, shall then be applied. The average sliding speed shall be between 0.1 and 1.0 in. per min. When the test is applied to curved sliding bearings, the design rotation shall be used in place of the displacement.

For flat sliding bearings, the sliding friction coefficient shall be computed for each direction of each cycle, and its mean and standard deviation shall be computed for the sixth through twelfth cycles. Neither the friction coefficient for the first movement nor the mean plus two standard deviations for the sixth through twelfth cycles shall exceed the value used in design, and the mean value for the sixth through the twelfth cycles shall not exceed two-thirds of the value used in design.

In curved sliding surfaces, the moment corresponding to the design rotation shall be established at each peak movement (positive and negative) during the first and last six full cycles of testing. The corresponding load eccentricity shall be calculated by dividing the moment by the total compressive load acting. The eccentricity shall be small enough that the allowable stresses on the PTFE used in design are not violated.

C18.1.5.2.6

The purpose of the bearing friction test is to verify that the friction values achieved in the material friction tests are adequate predictors of the friction in the finished bearing.

Because this test is conducted at the service limit state design capacity at slower speeds than those specified in Article 18.1.5.2.3, the measured coefficient of friction may be lower. As such, the pass/fail criteria for this test can be more refined than that given in Article 18.1.5.2.3. See also Article C18.1.5.2.3 for further information.

18.1.5.2.7—Long-Term Deterioration Test

At the discretion of the Owner, the contract documents shall specify if the long-term deterioration test is to be conducted on a per lot basis or if the test is satisfied by pre-qualification. The Owner's pre-qualification criteria or contract documents, as applicable, shall specify whether the test is to be conducted on full size bearings, scaled down versions of the bearings, components of bearings, samples of the materials used in the bearings, or a combination of these. Other pre-qualification criteria associated with the long-term deterioration test shall be as specified by the Owner.

The samples shall have an area not less than 7.0 in.^{2}. The test piece shall first be loaded in compression to a stress corresponding to the service limit state design capacity of the bearings. Flat sliding systems shall then be displaced through at least 1000 cycles with an amplitude of at least ±1.0 in. (2.0 in. peak to peak). Curved sliding systems and rotational systems that depend on deformation of an elastomeric element shall be subjected to 5000 cycles or displacements corresponding to a rotation of plus or minus the design amplitude. The sliding may take place at up to 10.0 in. per min, except when readings of the coefficient of friction are taken, at which time the sliding speed shall be 2.5 in. per min.

C18.1.5.2.7

The purpose of the long-term deterioration test is to verify the long-term resistance of the materials to creep, wear, and deterioration. The long-term deterioration test may be conducted on a pair of bearings placed back-to-back.

Because the long-term tests require more complicated test machinery and a longer test time, they are inevitably more expensive than short-term ones. Therefore, the frequency of and manner in which each of these tests are conducted may be determined separately by the Owner. The considerations Owners should take into account include, but are not limited to:

- Schedule of project,
- Pre-qualification, and
- Appropriate bearing test configuration.

Changes in the design configuration of a pre-qualified bearing will necessitate re-qualification via re-testing. At the Owner's discretion, however, the requirement to re-test may be waived if the change or changes to a pre-qualified bearing design may not fundamentally alter its long term performance.

The following shall be cause for rejection of the bearing:

- Damage visible to the naked eye on disassembly of the bearing, such as excessive wear, cracks, or splits in the material.
- A coefficient of friction which exceeds the value used in design.

18.1.5.2.8—Bearing Horizontal Force Capacity—Fixed or Guided Bearings Only

One or more loading combinations, consisting of a horizontal and vertical service-load which could exist simultaneously in the structure, shall be selected. The vertical load shall be applied first, at 1.0 times its nominal value. The horizontal load shall be applied in stages, up to 1.5 times its nominal value. Failure or excessive deflection of any of the components shall be cause for rejection.

C18.1.5.2.8

This test is only for bearings which must resist prescribed horizontal forces. The purpose of the test is to verify that the bearing is stable and that the guide or restraint system has adequate strength under the most severe realistic combination of horizontal and vertical loads. Selection of an impossible load combination may result in unwarranted rejection of the bearing. Bearings which must carry a large ratio of horizontal to vertical force are frequently an indicator of a poorly thought-out bearing system.

18.1.6—Performance Criteria

If one bearing of the sample fails, all the bearings of that lot shall be rejected, unless the Manufacturer elects to test each bearing of the lot at his expense. In lieu of this procedure, the Engineer may require every bearing of the lot to be tested.

18.1.7—Construction and Installation

Bearings shall be installed by qualified personnel to the positions shown in the contract documents. Bearings shall be set at time of installation to the dimensions and offsets prescribed by the Manufacturer, by the Engineer, and as shown in the contract documents and shall be adjusted as necessary to take into account the temperature and future movements of the bridge due to temperature changes, release of falsework, and shortening due to prestressing.

Each bridge bearing shall be located within ±0.125 in. of its correct position in the horizontal plane and oriented to within an angular tolerance of 0.02 rad. Guided bearings and bearings which rotate about only one axis shall be oriented in the direction specified in the contract documents to within a tolerance of 0.005 rad. All bearings except those which are placed in opposing pairs shall be set horizontal to within an angular tolerance of 0.005 rad, and must have full and even contact with load plates, where these exist. The superstructure supported by the bearing shall be set so that, under full dead load, its slope lies within an angular tolerance of 0.005 rad of the design value. Any departure from this tolerance shall be corrected by means of a tapered plate or by other means approved by the Engineer. If shim stacks are needed to level the bearing, they shall be removed after grouting and before the weight of the superstructure acts on the bearing.

C18.1.7

Bridge bearings are precisely engineered products and must be treated as such when they are installed. Frequently, the bearing has a relatively low profile which is gained by providing only the minimum required rotation capacity. Thus, it is crucial that the bearing be installed level and that the girder which will be seated on it also has a level underside. Furthermore, guided bearings must be oriented correctly or else large horizontal forces may be introduced.

Metallic bearing assemblies not embedded in the concrete shall be bedded on the concrete with a filler or fabric material conforming to Article 18.10, "Bedding of Masonry Plates."

Where bearings are seated directly on steel work, the supporting surface shall be machined so as to provide a level and planar surface upon which the bearing is placed.

Bearings or masonry plates which rest on steel supports may be directly installed on the supports, provided the support is flat within a tolerance of 0.002 times the nominal dimension and is sufficiently rigid so as not to deform under specified loads.

18.2—ELASTOMERIC BEARINGS

18.2.1—Scope

Elastomeric bearings as herein defined shall include unreinforced pads (consisting of elastomer only) and reinforced bearings with steel or fabric laminates.

18.2.2—General Requirements

Bearings shall be furnished with the dimensions, material properties, elastomer grade, and type of laminates specified in the contract documents. The design load shall be shown in the contract documents and testing shall be performed accordingly. Unless otherwise specified in the contract documents, bearings shall be Grade 3, 60-durometer elastomer, and steel reinforced, and shall be subjected to the load-testing requirements specified herein.

18.2.3—Materials

The raw elastomer shall be either virgin Neoprene (polychloroprene) or virgin natural rubber (polyisoprene). The elastomer compound shall be classified as being of low-temperature Grade 0, 2, 3, 4, or 5. The grades and other material properties are defined in the *AASHTO LRFD Bridge Design Specifications*, Section 14 and AASHTO M 251. A higher grade of elastomer may be substituted for a lower one.

C18.2.3

At present, only natural rubber (polyisoprene) and Neoprene (polychloroprene) are permitted. This is because both have an extensive history of satisfactory use. In addition, much more field experience exists with these two materials than with any other and almost all of it is satisfactory.

The low-temperature grading system addresses the problem of stiffening of the elastomer at low temperatures. Special compounding and curing are needed to avoid the problem, but they increase cost and, in extreme cases, may adversely affect some other properties. These adverse effects can be minimized by choosing a grade of elastomer appropriate for the conditions prevailing at the site. The grades follow the approach of AASHTO M 251 with some stringent low-temperature test criteria for higher grades.

18.2.4—Fabrication

Bearings shall meet the fabrication requirements of AASHTO M 251.

18.2.5—Testing

Materials for elastomeric bearings and the finished bearings themselves shall be subjected to the tests described in AASHTO M 251.

C18.2.5

Testing requirements fall into two main categories: material quality-control tests and load tests on the finished bearings to detect poor fabrication.

Complete bearings may be tested and this is most easily done using two identical bearings on top of one another with a shear load plate between them. However, in bearings with more than two or three layers, bending and buckling effects may reduce the shear stiffness of the complete bearing below the value GA/h_{rt} given by the simple shear model. It is important to distinguish between unacceptable material and failure to analyze the rather complicated behavior with sufficient accuracy.

18.2.6—Installation

Elastomeric bearings without external load plates may be placed directly on a concrete or steel surface provided that it is flat to within a tolerance of 0.005 of the nominal dimension for steel reinforced bearings and 0.01 of the nominal dimension for others. Bearings shall be placed on surfaces that are horizontal to within 0.01 rad. Any lack of parallelism between the top of the bearing and the underside of the girder that exceeds 0.01 rad shall be corrected by grouting or as otherwise directed by the Engineer.

Exterior plates of the bearing shall not be welded unless at least 1.5 in. of the steel exists between the weld and the elastomer. In no case shall the elastomer or the bond be subjected to temperature higher than 400°F.

C18.2.6

If the bearing seat is not horizontal, gravity loads will cause shear in the elastomer. The underside of the girder and the top surface of the bearing must also be parallel to avoid imposing excessive rotation and the stresses it causes in the bearing.

Welding to load plates should be avoided if possible. If it must be done, proper precautions should be taken to avoid damaging the bond by heat.

18.3—POT AND DISC BEARINGS

18.3.1—General

Pot and disc bearings:

- shall be adequate for the design loads and movements shown in the contract documents or specified and
- shall be tested at the appropriate level.

18.3.2—Materials

18.3.2.1—General

All materials shall be new and unused, with no reclaimed material incorporated in the finished bearing.

18.3.2.2—Steel

All steel except stainless steel components of the pot and disc bearing shall conform to the requirements of Article 11.3, “Materials,” for carbon steel or high-strength, low-alloy structural steel for welding.

18.3.2.3—Stainless Steel

Stainless steel shall conform to the requirements of Article 18.1.2.5.

18.3.2.4—Elastomeric Rotational Element for Pot Bearings

The elastomeric rotational element used in the construction of pot bearings shall contain only virgin, crystallization-resistant polychloroprene (Neoprene), AASHTO M 251 (ASTM D4014) or virgin natural polyisoprene (natural rubber), AASHTO M 251 as the raw polymer. The physical properties of Neoprene and natural rubber shall conform to the Specifications above with modifications as follows:

- The Shore A Durometer hardness shall be 50 ±10 points.
- Samples for compression set tests shall be prepared using a Type 2 die.

18.3.2.5—Sealant for Pot Bearings

The elastomer shall be lubricated between the steel pot and the top steel bearing plate with a silicon grease which does not react chemically with the elastomer and does not alter its properties within the range of environmental conditions expected at the site or as recommended by the Manufacturer.

18.3.2.6—Sealing Rings for Pot Bearings

The sealing rings between the steel piston and the elastomeric rotational element of pot bearings shall be made of brass conforming to ASTM B36/B36M for rings of rectangular cross-section and ASTM B121/B121M for circular sections. The Engineer may approve other sealing ring material on the basis of test evidence conforming to Article 14.7.4.5 of the *AASHTO LRFD Bridge Design Specifications.*

C18.3.2.6

See Article 14.7.4.5, "Sealing Rings," of the *AASHTO LRFD Bridge Design Specifications* for sealing ring requirements.

18.3.2.7—Polytetrafluorethylene (PTFE) Sheet and Strip

PTFE sheet and strip requirements for pot and disc bearings shall conform to the provisions of Article 18.8.1.

18.3.2.8—Polyether Urethane Structural Element for Disc Bearings

The polyether urethane structural element used in the construction of disc bearings shall be molded from a monolithic polyether urethane compound. The physical properties of the polyether urethane shall conform to the minimum requirements listed in Table 18.3.2.8-1.

C18.3.2.8

Polyether urethane is a hard tough plastic material. However, its tensile strength varies significantly depending on the quality-control exercised during processing. The properties required here are intended to ensure a good quality material.

Required minimums for tensile stress at specific elongations, tensile strength, ultimate elongation, and compression set may be interpolated for durometer hardness values between 45 and 55, and 55 and 65.

Table 18.3.2.8-1—Physical Properties of Polyether Urethane

Physical Properties	ASTM Test Method	Requirements		
Hardness, Type D Durometer	D2240	45	55	65
Minimum Tensile Stress, ksi At 100% elongation At 200% elongation	D412	 1.5 2.8	 1.9 3.4	 2.3 4.0
Minimum Tensile Strength, ksi	D412	4.0	5.0	6.0
Minimum Ultimate Elongation, %	D412	350	285	220
Maximum Compression Set, 22 h at 158°F Method B, %	D395	40	40	40

18.3.3—Fabrication Details

18.3.3.1—General 2011 Revision

The Contractor shall provide the Engineer with written notification 30 days prior to the start of bearing fabrication.

The finish of the mold used to produce the elastomeric rotational element for pot bearings or the polyether urethane structural element for disc bearings shall conform to good machine shop practice.

PTFE sheet shall be bonded to a grit-blasted steel substrate using an epoxy resin adhesive under controlled factory conditions in accordance with the instructions of the adhesive Manufacturer. The PTFE sheet shall be recessed into its steel substrate for at least one-half of its thickness. If on a vertical surface, the PTFE sheet may be mechanically fastened to the substrate. The attachment of the PTFE sheet to its substrate shall be done in accordance with the manufacturing requirements of Article 18.8.2, "Materials."

After fabrication, steel surfaces exposed to the atmosphere, except stainless steel surfaces, shall be shop painted or coated to protect against corrosion as specified in the contract documents. Prior to coating, the exposed steel surfaces shall be cleaned in accordance with the recommendations of the coating Manufacturer. Metal surfaces to be field-welded shall be given a coat of clear lacquer or other protective coating approved by the Engineer, if the time of exposure before welding takes place is to exceed three months. The lacquer coating shall be removed at the time of welding. The final painting or coating of these surfaces shall be done after the completion of welding.

Stainless steel sheet shall be attached to a steel substrate with a continuous seal weld.

All welding shall conform to and all welders shall be qualified in accordance with the requirements of the current AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code.*

Except as noted, all bearing surfaces of steel plates shall be finished or machined flat within 0.0008 in./in. Out-of-flatness greater than 0.0008 in./in. on any plate shall be cause for rejection. The bottom surfaces of lower bearing plates (masonry plates) designed to rest on bearing pads shall not exceed an out-of-flatness value of 0.005 in./in. Oxygen-cut surfaces shall not exceed a surface roughness value of 1000 μin., as defined by ANSI B46.1.

C18.3.3.1 2011 Revision

Stainless steel should be attached by welding all around. This not only ensures a uniform transfer of stress from the PTFE to the backing plate when the stainless steel is subjected to shear from sliding forces but it also minimizes the corrosion which can occur behind the stainless steel plate.

Gross bearing dimensions shall have a tolerance in accordance with Table 18.1.4.2-1.

Every bearing shall have the project identification number, lot number, and individual bearing number indelibly marked with ink on a side that will be visible after erection.

18.3.3.2—Fabrication Requirements for Pot Bearings

18.3.3.2.1—The Pot

The pot shall be manufactured by welding or machining from a single piece of plate. In pots made by welding a ring to a base plate, the weld shall be a full-penetration butt weld.

C18.3.3.2.1

The most common way of fabricating a pot is to machine it from a single piece of steel plate. However, for very large pots, this may be uneconomical because it means a large amount of machining. In such cases, casting, forging, or fabrication by welding are possible but they introduce extra difficulties beyond those found in pots machined from a single plate. If the pot is made by welding a ring to a base plate, the weld is critically important. The weld must be made on both the inside and outside of the ring and then the weld on the inside must be machined, if necessary, to give the correct final profile. The welds must be verified by suitable ultrasonic or radiographic examination methods and the flatness of the plates after welding must be ensured.

The piston shall be machined from a single piece of steel. The outside diameter of the piston shall be no more than 0.030 in. less than the inside diameter of the pot at the level of interface between the piston and elastomeric rotational element. The sides of the piston shall be beveled to facilitate rotation.

If guides are used, they shall be attached to the pot by welding or bolting.

18.3.3.2.2—Sealing Rings

The sealing rings shall be recessed into the elastomeric disc and shall fit snugly against the pot wall. Rings of rectangular cross-section shall be installed with their gaps equally spaced round the circumference. The gap between the ring and the wall shall nowhere exceed 0.01 in. The gap between the cut ends of the ring shall not exceed 0.05 in.

C18.3.3.2.2

Sealing rings are presently made from brass in the United States. Attempts were made to use PTFE in the past but these were unsuccessful because the PTFE ring squeezed out through the gap between the pot and the piston and, thereafter, was ineffective as a seal. However, certain proprietary materials have also been used in Europe with success. They would require verification testing before being accepted in the United States.

18.3.3.2.3—Elastomeric Rotational Element

The elastomeric pad shall have the same nominal diameter as the pot. The elastomeric pad shall be individually molded or cut from sheet and shall be made of no more than three separate layers, of which none may have a nominal thickness of less than 0.5 in. The sealing ring recess depth shall be the same as the total ring thickness if rectangular rings are used.

C18.3.3.2.3

The rotational element should be made from a flexible elastomer. The elastomer is fully confined in the pot and, therefore, cannot undergo large deflections so no advantage is gained by using a stiffer elastomer as might be the case in a laminated elastomeric bearing.

18.3.3.3—Fabrication Requirements for Disc Bearings

18.3.3.3.1—Steel Housing

The steel housing shall be manufactured by welding or machining from a single piece of plate. The shear restriction mechanism shall be connected to the bearing plate by mechanically fastening, welding, or other means approved by the Engineer.

18.3.3.3.2—Polyether Urethane Rotational Element

The polyether urethane rotational element for disc bearings shall be molded as a single piece. The finish of the mold shall be free of burrs and shall conform to good shop practice.

18.3.4—Sampling and Testing

18.3.4.1—Lot Size

Sampling, testing, and acceptance consideration shall be made on a lot basis. A lot shall be the smallest number of bearings as determined by the following criteria:

- Shall not exceed a single contract document or project quantity.
- Shall not exceed 25 bearings.
- Shall consist of those bearings of the same type regardless of load capacity. Bearing types may be fixed or expansion types. Guided and nonguided expansion bearings shall be considered to be a single type.

C18.3.4.1

A lot may be further defined as those bearings presented for inspection at a specific time or date.

18.3.4.2—Sampling and Acceptance

The Manufacturer shall sample, at random, a minimum of two (2) bearings and, as permitted or required, a minimum of two (2) individual bearing components from each completed lot of bearings for material certification and performance testing by the Manufacturer. The Manufacturer shall complete the required testing and determine compliance with this specification before submitting the lot(s) for quality-assurance inspection, testing, and acceptance consideration. The results of the Manufacturer's tests shall be furnished to the Engineer.

C18.3.4.2

Individual bearing components sampled for material certification or performance testing should be sampled from the materials used to fabricate a lot of bearings, and not from completed bearings.

18.3.4.3—Quality-Assurance Testing by the Engineer

When quality-assurance testing is specified in the contract documents, the Manufacturer shall furnish to the Engineer the required number of complete bearings and component samples to perform quality-assurance testing in accordance with Table 18.3.4.3-1.

At least one elastomeric element shall be tested per lot of bearings for pot bearings and at least one set of material property tests shall be conducted per lot for disc bearings. All exterior surfaces of sampled production bearings shall be smooth and free from irregularities or protrusions that might interfere with testing procedures.

A minimum of 30 days shall be allowed for inspection, sampling, and quality-assurance testing of production bearings and component materials.

Bearings with tapered sole plates shall conform to the provisions of Article 18.1.5.1.2.

The Engineer may select, at random, the required sample bearing(s) from completed lots of bearings and samples of the elastomeric and PTFE materials for quality-assurance testing.

The Contractor shall assume the cost of transporting all samples from the place of manufacture to the test site and back or, if applicable, to the project site.

Table 18.3.4.3-1—Sampling

Test	Samples Required
Proof Load	One production bearing per lot
Coefficient of Friction	One production bearing per lot
Physical Properties of Elastomeric Rotational Element	One elastomeric element per lot
Physical Properties of PTFE Sheet	One 10.0-in. × 15.0-in. sheet of PTFE material per project
Physical Properties of Polyether Urethane Structural Element (except compression set)	One 10.0-in. × 15.0-in. sheet of polyether urethane material (thickness of 0.0625 in. to 0.125 in. per lot
Compression Set of Polyether Urethane Structural Element	One 4.0-in. × 4.0-in. sheet of polyether urethane per lot, molded or cut to the thickness requirements of ASTM D395, Method B

18.3.4.4—Performance Testing

18.3.4.4.1—Material Certification Testing

Material certification tests shall be conducted in accordance with Article 18.1.5.2.2.

Certification shall be provided for all elastomeric and polyether urethane elements. Their material properties shall satisfy the requirements specified in the contract documents and the tests described in Article 18.3.2.4 for pot bearings and Article 18.3.2.8 for disc bearings. Additional tests may be required by the Engineer.

18.3.4.4.2—Dimension Check and Clearance Test

Dimensions and clearances shall be checked for all bearings according to Articles 18.1.5.2.4 and 18.1.5.2.5.

C18.3.4.4.2

The clearance test between the piston and the pot on a pot bearing and the clearance between the shear-restricting element and disc on a disc bearing are two examples of critical clearances that are verified through the clearance test, described in Article 18.1.5.2.5, to ensure proper functioning of the bearing through its entire design displacement and rotation.

18.3.4.4.3—Long-Term Deterioration Test

The long-term deterioration test shall satisfy the requirements of Article 18.1.5.2.7. When testing is specified by the Owner to be on a per lot basis, the long-term deterioration test shall be performed on one (1) sampled disc bearing of each lot and one (1) sampled pot bearing of each lot.

18.3.4.4.4—Proof Load Test

Sampled bearings shall be short-term load-tested to 150 percent of the specified rated capacity at 0.02 rad. If the size of the bearing prohibits adequate testing with available equipment, the Owner may specify a test on one (1) scaled down bearing with comparable requirements. The load shall be held for 5 minutes, removed, then reapplied for 5 minutes. If the load drops below the required value during either application, the test shall be restarted from the beginning.

At the discretion of the Owner, a long-term proof load test may also be specified or substituted for the short-term proof load test. Except as specified below, the magnitude of the loading, test procedures, and failure criteria for the long-term test shall be identical to that of the short-term test. The first loading shall be held for 5 minutes and the second loading shall be held for 15 hours. If the load drops below 90 percent of its target value during this time, the load shall be increased to the target value and the test duration shall be increased by the period of time for which the load was below the required value.

During the test, the steel bearing plate and steel piston shall maintain continuous and uniform contact for the duration of the test.

The bearing shall be visually examined both during the test and upon disassembly after the test. Any resultant visual defects, such as extruded or deformed elastomer, polyether urethane, or PTFE; damaged seals or limiting rings; evidence of metal-to-metal contact between the pot wall and the top plate; or cracked steel, shall be cause for rejection of the lot.

For disc bearings, continuous and uniform contact shall be maintained between the polyether urethane element and the bearing plates and between the sliding steel top plate and the upper bearing plate for the duration of the test. Any observed lift-off shall be cause for rejection of the lot.

C18.3.4.4.4

For manufactured bearings, the specified rated capacity is typically greater than the service limit state reaction used for design.

Visual defects include, but are not limited to, bond failure, physical destruction, cold flow of PTFE to the point of debonding, or damaged components.

18.3.4.4.5—Sliding Coefficient of Friction

For all guided and nonguided expansion-type bearings, the sliding coefficient of friction shall be measured at the bearing's design capacity for each sampled bearing and bearing component in accordance with Articles 18.1.5.2.3 and 18.1.5.2.6.

The sliding coefficient of friction shall be calculated as the horizontal load required to maintain continuous sliding of one bearing, divided by the bearing's vertical design capacity.

C18.3.4.4.5

At the Owner's discretion, the requirement to test according to Article 18.1.5.2.3 may be waived. Testing according to Article 18.1.5.2.6 verifies that the friction values achieved in the material friction tests conducted according to Article 18.1.5.2.3 are adequate predictors of the friction in the finished bearing.

The test results shall be evaluated as follows:

- Pass/fail criteria outlined in Article 18.1.5.2.3.
- Pass/fail criteria outlined in Article 18.1.5.2.6.
- For bearings tested according to Article 18.1.5.2.6, the bearings will be visually examined both during and after the test. Any resultant visual defects, such as bond failure, physical destruction, cold flow of PTFE to the point of debonding, or damaged components, shall be cause for rejection of the lot.

Visual defects include, but are not limited to, bond failure, physical destruction, cold flow of PTFE to the point of debonding, or damaged components.

Bearings not damaged during the testing of performance characteristics may be used in the work.

18.3.5—Installation

Pot and disc bearings shall be installed in accordance with the contract documents and on the approved working drawings. A technical representative of the bearing Manufacturer shall be readily available to provide guidance to the Contractor during the entire installation process. Upon final installation of the bearings, the Engineer shall inspect the bearing components to assure that they are level and parallel to within 0.03125 in./ft. Any deviations in excess of the allowed tolerances shall be corrected.

C18.3.5

The Owner may require a Manufacturer's technical representative to be on-site during the installation of the bearings. The Manufacturer's representative should be independent of the Contractor's work force or display proof of certification to install materials from the Manufacturer. The Contractor should discuss the work to be done with the Manufacturer's representative to review the methods of installation and the equipment needed prior to beginning the work.

The Manufacturer's representative should advise both the Engineer and the Contractor regarding proper installation procedures to correctly install the bearings.

18.4—ROCKER AND ROLLER BEARINGS

18.4.1—Materials

Steels used in rocker and roller bearings shall be of the types and grades as specified in the contract documents. The steel at the contact surface of a metal bearing may be hardened provided that, after hardening, the steel satisfies the strength and ductility requirements of the contract documents and the material specifications.

18.4.2—Fabrication

18.4.2.1—Steel

Rocker bearings shall be manufactured by casting, forging, or fabricating from plate. Roller bearings more than 9.0 in. in diameter shall be forged and annealed. Smaller roller bearings shall either be forged and annealed or made from cold-finished carbon steel shafting.

Roller bearings with diameters greater than 9.0 in. shall not have a hole less than 2.0 in. in diameter bored full-length along the axis after the forging has cooled to a temperature below the critical range and before annealing. Boring shall be done under conditions which prevent damage by cooling too rapidly.

Fabrication shall be performed in a manner that conforms to the practice in modern commercial shops. Burrs, rough and sharp edges, and other flaws shall be removed.

18.4.2.2—Lubrication

Lubrication shall be applied to all gear mechanisms and to all other components of roller bearings for which it is required. The type of lubricant shall be as specified on the contract documents, and shall be applied in accordance with the Manufacturer's recommendations.

18.4.3—Installation

Setting of rocker and roller bearings shall take into account any variation from mean temperature of the supported span at time of setting and any other anticipated changes in length of the supported span so that at mean temperature, after release of falsework and any shortening due to prestressing force and shrinkage, the rockers and rollers will be vertical. Care shall be taken that full and free movement of the superstructure at movable bearings is not restricted by improper settings or adjustment of bearings.

The Contractor shall coat all contact surfaces thoroughly with oil and graphite prior to placing roller bearings.

Cylindrical bearings shall be positioned so their axes of rotation are in alignment and coincide with the axis of rotation of the superstructure.

18.5—SPHERICAL BEARINGS

Spherical bearings shall be fabricated, tested, and installed as specified in the contract documents.

18.6—BRONZE OR COPPER-ALLOYED PLATES FOR BEARINGS

18.6.1—Bronze Bearing and Expansion Plates

Bronze bearing and expansion plates shall conform to the Specification for Bronze Castings for Bridges and Turntables, AASHTO M 107 (ASTM B22) alloy C91100, C86300, or C90500. Alloy C91100 shall be furnished unless otherwise specified in the contract documents. Components may be cast, rolled, or forged. Castings shall be free of blow-holes larger than 0.125 in. and contact surfaces shall be free of all blow-holes of any size.

Bronze plates shall be cast according to details shown in the contract documents. Sliding surfaces shall be planed parallel to the movement of the spans and polished unless detailed otherwise in the contract documents.

18.6.2—Rolled Copper-Alloy Bearings and Expansion Plates

Rolled copper-alloy bearing and expansion plates shall conform to the Specification for Wrought Copper-Alloy Bearing and Expansion Plates and Sheets for Bridge and Other Structural Use, AASHTO M 108 (ASTM B100). Alloy C51000 or C51100 shall be furnished unless otherwise specified in the contract documents.

Copper-alloy plates shall be furnished according to details shown in the contract documents. Finishing of the rolled plates shall not be required provided they have a plane, true, and smooth surface.

18.6.3—Test Requirements

Material certification tests for bronze or copper-alloy bearings shall be performed to verify the properties of the material.

Bearing friction tests as described in Article 18.1.5.2.6 or material friction tests as described in Article 18.1.5.2.3 may be required by the Engineer.

18.7—MASONRY, SOLE, AND SHIM PLATES FOR BEARINGS

18.7.1—Materials

Metal plates used in masonry, sole, and shim plates, unless otherwise specified in the contract documents, shall conform to AASHTO M 270M/M 270 (ASTM A709/A709M) Grade 36 (Grade 250). Bronze or copper-alloy bearing and expansion plates shall conform to the requirements of Article 18.6.

18.7.2—Fabrication

Holes in bearing plates may be formed by drilling, punching, or accurately controlled oxygen-cutting. All burrs shall be removed by grinding.

18.7.3—Installation

Bearing plates shall be accurately set in level position as shown in the contract documents and shall have a uniform bearing over the whole area. When plates are to be embedded in concrete, provision shall be made to keep the plates in correct position as the concrete is being placed.

18.8—POLYTETRAFLUORETHYLENE (PTFE) SURFACES FOR BEARINGS

18.8.1—General

PTFE sheet and strip shall be manufactured either from pure virgin (not reprocessed), unfilled PTFE resin; from PTFE resin uniformly blended with either 15 percent glass fiber or 25 percent carbon (maximum filler, percent by weight (mass)); or from fabric containing PTFE fibers.

Horizontally installed PTFE sheet shall be bonded to and recessed into its steel substrate. Vertically installed PTFE sheet shall be bonded to and recessed into or bonded to and mechanically fastened to its steel substrate. PTFE sheet shall have a minimum thickness of 0.125 in. and shall be recessed for at least one-half of its thickness into its steel substrate.

Finished PTFE sheet and strip shall be resistant to all acids, alkalis, and petroleum products; stable at temperatures from –360°F to 500°F; nonflammable; and nonabsorbing of water. The epoxy used to bond the PTFE to its steel substrate shall be a heat-cured, high-temperature epoxy capable of withstanding temperatures of –320°F to 500°F.

18.8.2—Materials

18.8.2.1—Polytetrafluorethylene (PTFE) Sheet and Strip

PTFE resin sheets, PTFE fabric, interlocked bronze and PTFE structures, PTFE-perforated metal composite, back-up materials, and all other parts of fixed or expansion bearings containing PTFE materials shall have the friction, mechanical, physical, and weathering properties specified in the contract documents.

18.8.2.2—PTFE Resin

PTFE resin shall be 100 percent pure, new material meeting the requirements of ASTM D4894 or D4895. It shall satisfy the requirements of Table 18.8.2.6-1. No reclaimed material shall be used.

Finished PTFE sheet, strip, and fabric shall be:

- resistant to acids, alkalis, and petroleum products,
- stable at temperatures from –360°F to 500°F ,
- nonflammable, and
- nonabsorbing of water.

C18.8.2.2

Testing of PTFE is intended to determine the purity and crystallinity of the PTFE. The purity influences the coefficient of friction which can be obtained and the crystallinity influences both the resistance to wear and the friction coefficient. The specific gravity of PTFE is indirectly a measure of the crystallinity.

18.8.2.3—Filler Material

Filler material, when used, shall be milled glass fiber, carbon fiber, or other approved filler material. The filler shall not react chemically but shall act compositely with the PTFE.

18.8.2.4—Adhesive Material

Adhesive material used for bonding sheet PTFE shall be an epoxy resin satisfying the requirements of AASHTO M 235M/M 235 (ASTM C881/C881M), FEP film or equal, as approved by the Engineer.

18.8.2.5—Unfilled PTFE Sheet

Finished, unfilled PTFE sheet shall be made from virgin PTFE resin and shall conform to Table 18.8.2.6-1 with the exception that the ASTM test methods for tensile strength and elongation shall conform to ASTM D2256.

18.8.2.6—Filled PTFE Sheet 2015 Revision

Filled PTFE sheet shall be made from virgin PTFE resin uniformly blended with approved inert filler. The maximum filler content shall be 15 percent for fiberglass and 25 percent for carbon fibers. The maximum filler content for other materials shall be determined by the Engineer.

Finished filled PTFE sheets containing glass fiber or carbon shall conform to the following requirements of Table 18.8.2.6-1 with the exception that the ASTM test methods for tensile strength and elongation shall conform to ASTM D638.

Table 18.8.2.6-1—Filled PTFE Sheet

Mechanical	ASTM Method	Sheet Unfilled	Sheet with 15% Glass Fibers	Sheet with 25% Carbon Fibers	Woven Fabric
Tensile Strength min. ksi	D638 or D2256	2.8	2.0	1.3	2.4
Elongation min. %	D638 or D2256	200	150	75	35
Specific Gravity, min.	D792	623 ± 2	2.20 ± 0.03	2.10 ± 0.03	—
Melting Point, °F	D4894, D4895 or D5977	622 ± 3	621 ± 18	621 ± 18	—

Values for intermediate filler contents may be obtained by interpolation.

18.8.2.7—Fabric Containing PTFE Fibers

Woven Fabric PTFE shall be manufactured from oriented multi-filament PTFE fluorocarbon fibers or from a mixture of PTFE fibers made from twisted, slit PTFE tape and other fibers as required by proprietary designs. Typical physical properties of the PTFE fibers shall be taken from Table 18.8.2.6-1 with the exception that the ASTM Test Methods for tensile strength and elongation shall conform to ASTM D2256.

18.8.2.8—Lubricants

Lubricants, if used, shall consist of a combination of solids which does not react chemically or electrolytically with the PTFE and its mating surface and shall remain stable in the environmental conditions expected at the bridge site.

C18.8.2.8

The lubricant most frequently used in Europe with sheet PTFE is based on lithium grease. It has proved to be effective and stable over long periods.

18.8.2.9—Interlocked Bronze and Filled PTFE Structures

Interlocking bronze and filled PTFE structures shall consist of a phosphor bronze plate with a 0.01-in. thick porous bronze surface layer into which is impregnated a lead/PTFE compound. There shall be an overlay of compounded PTFE not less than 1.0 in. thick. The phosphor bronze back plate shall conform to AASHTO M 108 (ASTM B100) and the porous bronze layer shall conform to ASTM B103/B103M.

18.8.2.10—Surface Treatment

Where PTFE sheets are to be epoxy-bonded, one side of the PTFE sheet shall be factory-treated by an approved Manufacturer by the sodium naphthalene or sodium ammonia process.

18.8.2.11—Stainless Steel Mating Surface

Stainless steel shall conform to the requirements of Article 18.1.2.5.

18.8.3—Fabrication Requirements

18.8.3.1—GENERAL

Expansion bearings shall be manufactured to the dimensions and to meet the requirements of the method of fastening to the structure as shown in the contract documents.

18.8.3.2—Attachment of PTFE Material

18.8.3.2.1—General

When mechanically fastened, PTFE sheet shall be fastened as shown in the contract documents with the size, type, and number of fasteners required. The fastener used in the PTFE sheet and back-up material shall be installed to provide full bearing.

18.8.3.2.2—Flat Sheet PTFE

All flat sheet PTFE attached to a metal backing plate shall be attached by recessing into the backing of the plate for one-half of the PTFE thickness and bonding. PTFE attached to other materials, such as elastomers, shall be attached by a method specified in the contract documents or approved by the Engineer.

The PTFE shall be factory-bonded, using an adhesive that is approved by the Engineer, in accordance with the instructions of the adhesive's Manufacturer. Prior to bonding, the surface shall be etched by an approved Manufacturer using sodium naphthalene or sodium ammonia process. When the backing plate is metal, the bonding shall be conducted under a uniform pressure greater than 0.100 ksi.

The peel strength of the bond shall not be less than 20.0 lb/in., tested in accordance with ASTM D429, Method B. The finished surface of the PTFE shall be smooth, free from bubbles, and shall conform to the tolerances shown in Table 18.1.4.2-1. Filled PTFE sheets shall be polished after bonding.

C18.8.3.2.2

PTFE sheets should be both recessed and bonded to obtain the best performance. The recessing inhibits creep or cold flow and the bonding keeps the PTFE in the recess when the mating surface of stainless steel slides over it. Without the bond, there is a risk that conditions such as eccentric loading would cause the PTFE to come out of the recess.

The minimum bonding pressure of 0.100 ksi is intended to ensure that the adhesive under the PTFE is well distributed and that the final PTFE surface will be flat.

Filled PTFE is much rougher and leads to higher friction coefficients than pure PTFE. Polishing is intended to minimize the adverse effects of the filler.

18.8.3.2.3—Curved Sheet

Curved sheet PTFE, such as used in spherical bearings, shall be attached by recessing for one-half the PTFE thickness. The dimensions of the PTFE element shall be selected so that it fits tightly in the recess even when the bearing is subjected to its lowest design temperature.

18.8.3.2.4—Fabric Containing PTFE Fibers

Fabric made from woven PTFE fibers shall be bonded or mechanically attached to a rigid substrate. The fabric shall be capable of carrying unit loads of 10.0 ksi without cold flow. The fabric-substrate bond shall be capable of withstanding, without delamination, a shear force equal to $(0.1 + \mu)P$ at the same time as the normal load P, where μ is the design coefficient of friction between the PTFE and its mating surface and P is the design load acting perpendicularly to the interface.

C18.8.3.2.4

Woven PTFE cannot be kept in place by a recess in the same way that sheet PTFE can so some other means is necessary. It can be attached to its backing substrate, either by bonding or by forming in the metal substrate mechanical indentations into which the PTFE weave is pressed. The effectiveness of such a mechanical connection can be judged by a test in which one piece of woven PTFE is compressed between two indented metal substrates and the PTFE is pulled out from between them.

18.8.3.3—Stainless Steel Mating Surface

Each stainless steel element specified in the contract documents as a single piece shall be so supplied. Each sheet shall be attached to its backing material by seal-welding around the entire perimeter so as to prevent entry of moisture between the stainless steel and the backing material. Welds shall conform to the current AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*. After welding, the stainless steel sheet shall be flat, free from wrinkles, and in continuous contact with its backing plate.

C18.8.3.3 2011 Revision

Stainless steel should be attached by welding all around. This not only ensures a uniform transfer of stress from the PTFE to the backing plate when the stainless steel is subjected to shear from sliding forces but it also minimizes the corrosion which can occur behind the stainless steel plate.

18.8.3.4—Lubrication

Lubrication shall be applied to the entire PTFE surface if specified in the contract documents or by the Engineer. If the PTFE is dimpled, enough lubricant shall be used to fill all the dimples.

18.8.4—Testing and Acceptance

18.8.4.1—General

Inspection of the completed bearings or representative samples of bearings with PTFE surfaces shall be required by the Engineer. Inspectors, if appointed, shall be allowed free access to the necessary part of the Manufacturer's plant and test facility. When testing is performed by the Manufacturer, copies of the test results shall be submitted to the Engineer.

The Manufacturer is required to perform material tests on the materials used in the sliding surface in accordance with Article 18.1.5.2.3. A minimum of one test shall be performed for each lot of bearings.

If requested by the Engineer and available test facilities permit, complete bearings shall be tested for complete bearing friction as defined in Article 18.1.5.2.6. If the test facility does not permit testing complete bearings, at the direction of the Engineer, extra bearings may be manufactured by the Contractor and samples of at least 100-kip capacity at normal working stresses prepared by sectioning the bearings. As soon as all bearings have been manufactured for a given project, notification shall be given to the Engineer who will select the prescribed test bearings at random from the lot. Manufacturer's

C18.8.4.1

The tests described in this Section are intended to determine the purity and crystallinity of the PTFE. Test results shall conform to the requirements of Table 18.8.2.6-1. The purity influences the friction factor which

can be obtained, and the crystallinity influences both the resistance to wear and the friction. The specific gravity test is indirectly a test of the crystallinity. The strength tests are necessary to make sure that the PTFE does not creep excessively and does not fail in direct tension. The test strength required of woven fabric material is extremely high because it is at present conducted on a single strand of PTFE fiber. This is a carry-over from the existing practice but a strength test which measures the strength of the finished fabric in pounds per inch of fabric would probably be better.

certification of the steel, elastomeric pads, preformed fabric pads, PTFE, and other materials used in the construction of the bearings shall be furnished along with notification of fabrication completion.

Bearings represented by test specimens passing the above requirements shall be approved for use in the structure subject to on-site inspection for visible defects.

18.8.5—Installation

Installation shall be performed as specified in Article 18.4.3.

18.9—ANCHOR BOLTS

18.9.1—Materials

Anchor bolts shall meet the requirements of ASTM A307, or as shown in the contract documents. Anchor bolts shall be provided with anchorage details that permit development of the full tensile strength of the bolt. Hooks or end plates are recommended.

18.9.2—Fabrication

Anchor bolts shall be swedged or threaded to secure a satisfactory grip upon the material used to embed them in the holes.

18.9.3—Installation

The Contractor shall drill holes for anchor bolts and set them in portland cement grout, or preset them as shown in the contract documents or as specified or directed by the Engineer.

Location of anchor bolts shall take into account any variation from mean temperature of the superstructure at time of setting and anticipated lengthening of bottom chord or bottom flange due to dead load after setting; the intention being that, as near as practicable, at mean temperature and under dead load, the anchor bolts at expansion bearings will center their slots. Care shall be taken that full and free movement of the superstructure at movable bearings is not restricted by anchor bolts or nuts.

18.10—BEDDING OF MASONRY PLATES

18.10.1—General

Filler or fabric materials shall be placed as bedding material under masonry plates when shown in the contract documents. Such material shall be of the type specified in the contract documents or as ordered or approved by the Engineer and shall be installed to provide full bearing on contact areas.

Immediately before placing the bedding material and installing bearings or masonry plates, the contact surfaces of the concrete and steel shall be thoroughly cleaned.

18.10.2—Materials

Preformed fabric pads used as bedding shall be composed of multiple layers of 8-oz/yd^2 cotton duck impregnated and bonded with high-quality natural rubber or of equivalent and equally suitable materials compressed into resilient pads of uniform thickness. The number of plies shall be such as to produce the specified thickness, after compression and vulcanizing. The finished pads shall withstand compression loads perpendicular to the plane of the laminations of not less than 10.0 ksi without detrimental reduction in thickness or extrusion.

Sheet lead used as bedding shall be common, desilverized lead conforming to ASTM B29. The sheets shall be of uniform thickness and shall be free from cracks, seams, slivers, scale, and other defects. Unless otherwise specified, lead sheets shall be 0.125 in. in thickness with a permissible tolerance of ±0.03 in.

Caulking material used as bedding shall be a non-sag polysulfide or polyurethane material conforming to the provisions of ASTM C920, Type II.

Grout and mortar used for filling under masonry plates shall conform to Article 8.14, “Mortar and Grout.”

18.11—FABRICATION REQUIREMENTS FOR GUIDES

Guide bars shall be attached to the body of the bearing by a method which minimizes distortion and allows the flatness tolerance on all parts of the bearing to be met after attachment. The sliding surfaces of the guide system shall be flat and parallel.

Bolts or threaded fasteners used to attach the guide bars to their supporting plates shall have an embedded thread length adequate to develop their strength.

If low-friction material is used at the contact interface, it shall be attached to its backing piece by two or more of the following methods simultaneously:

- bonding,
- recessing, and
- mechanical attachment with countersunk fasteners.

If the material is bonded, it shall be etched by the method recommended by the Manufacturer of the material or bonding agent. Recessing shall be one-half of the material thickness. Fasteners shall be countersunk to a depth which ensures that they will not touch the mating material after allowing for wear.

C18.11

Guide bars are usually attached by bolting because welding tends to introduce distortions. If the bolts are held in threaded holes in the plate rather than by nuts, the threaded holes should have adequate length to develop the full tension strength of the bolt. The guide bars must be attached with a relatively fine tolerance in order to prevent locking up when the bearing displaces longitudinally.

Very low-friction coefficients are less necessary for guides than for the PTFE slider which supports the gravity load. This is because friction on the guides contributes only a small percentage of the longitudinal resisting force of the bearing. Thus, filled PTFE, which has a better resistance to creep than pure PTFE, is often used for guides. The use of a filler means that it is not necessary to recess the PTFE in a metal backing plate and this, therefore, saves some machining. PTFE filled with fiberglass or carbon fibers, and a PTFE and sintered metal mixture have been used with success.

The low-friction material used for guides must be attached by two of the three given methods to avoid the debonding problems which have occurred in the past.

18.12—LOAD PLATES

Load plates shall be made from a single plate or they may be built up from several steel laminates, each oriented in the plane perpendicular to the direction of the load. Built-up load plates shall be joined by complete seal-welding to prevent ingress of moisture. Such welds shall also provide sufficient shear strength to resist the applied loads. The load plates shall have no sharp corners or edges. Holes may be formed by drilling, punching, or accurately controlled oxygen cutting. All burrs shall be removed by grinding.

18.13—MEASUREMENT

Bearing devices shall be measured either by the pound as determined from scale weight (mass) or by a unit basis for each type of bearing assembly listed in the contract documents. Scale weight (mass) is not required when calculated weight (mass) is shown in the contract documents, in which case the weight (mass) shown in the contract documents shall be used as the basis of payment.

18.14—PAYMENT

Bearing devices shall be paid for at the contract price per pound or per unit. Such payment shall include full compensation for furnishing all labor, materials, tools, equipment, and incidentals and for doing all the work involved in furnishing, testing, and installing said bearing devices, complete in place, as shown in the contract documents, as specified in these Specifications, and as directed by the Engineer.

18.15—REFERENCES

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2009. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 29th Edition, HM-29, American Association of State and Highway Transportation Officials, Washington, DC. Includes AASHTO M, R, and T standards, which are also available individually in downloadable form.

AASHTO and AWS. 2008. AASHTO/AWS D1.5M/D1.5:2008 *Bridge Welding Code*, Fifth Edition, BWC-5, American Welding Society, Miami, FL.

ASME. 2002. *Surface Texture, Surface Roughness, Waviness and Lay*, B46.1, American Society of Mechanical Engineers, Fairfield, NJ.

RMA. 1992. *Rubber Handbook for Molded, Extruded, Lathe Cut and Cellular Products*, 5th Edition, Rubber Manufacturers Association, Inc., Washington, DC.

SAE. 1995. *Society of Automotive Engineers Specification Manual*, Volume 1, Society of Automotive Engineers, Warrendale, PA.

SAE. 2004. "Chemical Composition of SAE Carbon Steels," SAE J403, *SAE Handbook*, Society of Automotive Engineers, Warrendale, PA.

Back to 2010 Edition

SECTION 18: BEARING DEVICES

18.1.4.2—Fabrication Tolerances

Revise the Article as follows:

Plain elastomeric pads and laminated bearings shall be built as specified in the contract documents within the tolerances of AASHTO M 251.

Other fabrication tolerances are given in Table 18.1.4.2-1. The classes of tolerances given in Table 18.1.4.2-1 shall be as follows:

- Class A = 0.001 × nominal dimensions
- Class B = 0.002 × nominal dimensions
- Class C = 0.005 × nominal dimensions

Load plate overall dimensions for flatness tolerance and surface finish shall apply only to surfaces in contact with the bearing.

Insert new commentary to this Article:

C18.1.4.2

Some of the tolerances have been changed to relative values because an absolute tolerance, such as 0.0625 in., may be overly large for a small bearing and unrealistically small for a large bearing. Parallelism of the two faces of a single layer is controlled by the limitation of the thickness at any point.

Each bearing type has one or more tolerances which are particularly important. In bearings which depend on rocking or rolling surfaces, it is most important to ensure that the curvature of the curved surface is constant to within a fine tolerance. This is more important than the actual value of the radius of curvature. In nested roller bearings, it is also important that all the rollers have exactly the same radius of curvature to ensure that the load will be equally shared among them. In flat PTFE sliding surfaces, the surface finish of the mating surface, usually stainless steel, is particularly important. A #8 mirror finish or better is recommended in all cases.

In bearings which depend on the sliding of one curved surface over another, such as curved PTFE sliding bearings, curved bronze sliding bearings, or pins and bushings which allow rotation, the difference in diameter of the two curved surfaces is the most important tolerance. The out-of-round or the variation in curvature of the curved surface is also important and, again, the actual value of the radius of curvature is less important. If two parts of the bearing are made by different Fabricators, machining by fitting the two parts is not possible and it is necessary to machine each part to a specific radius within a very high accuracy. In the past, bearings made of components which are fabricated by different Manufacturers have resulted in problems because of lack of a good fit. In pot bearings, the most important tolerances are those on the clearance between the pot and the piston and on the vertical clearance between the upper and lower parts of the bearing.

2010
Revision

Back to 2010 Edition

SECTION 18: BEARING DEVICES

18.3.3.1—General

Replace paragraphs 5 and 6 with the following:

Stainless steel shall be attached to a steel substrate with a continuous seal weld. The welding shall conform to and all welders shall be qualified in accordance with the requirements of the current AWS D1.6M/D1.6 *Structural Welding Code-Stainless Steel*. Welding for metals other than stainless steel, as identified in Article 11.3, shall conform to and all welders shall be qualified in accordance with the requirements of the current AASHTO/AWS D1.5 M/D1.5 *Bridge Welding Code*.

C18.3.3.1

Revise the Article as follows:

Stainless steel should be attached by welding all around. This not only ensures a uniform transfer of stress from the PTFE to the backing plate ~~when~~ where the stainless steel is subjected to shear from sliding forces but it also minimizes the corrosion which can occur behind the stainless steel plate. AWS D1.6M/D1.6 *Structural Welding Code-Stainless Steel* requires that welds be performed with the addition of an appropriate alloy filler metal to mitigate solidification cracking tendencies.

18.8.3.3—Stainless Steel Mating Surface Back to 2010 Edition

Revise the Article as follows:

Each stainless steel element specified in the contract documents as a single piece shall be so supplied. Each sheet shall be attached to its backing material by seal-welding around the entire perimeter so as to prevent entry of moisture between the stainless steel and the backing material. Welds shall conform to the current ~~AASHTO/~~AWS ~~D1.5M/D1.5~~ D1.6M/D1.6 ~~*Bridge*~~ *Structural Welding Code-Stainless Steel*. After welding, the stainless steel sheet shall be flat, free from wrinkles, and in continuous contact with its backing plate.

2011 Revision

Back to 2010 Edition

SECTION 18: BEARING DEVICES

18.8.2.6—Filled PTFE Sheet

Revise Row 2, Column 6; and Row 4, Column 3 as follows:

Table 18.8.2.6-1—Filled PTFE Sheet

Mechanical	ASTM Method	Sheet Unfilled	Sheet with 15% Glass Fibers	Sheet with 25% Carbon Fibers	Woven Fabric
Tensile Strength min. ksi	D638 or D2256	2.8	2.0	1.3	~~2.4~~ 24
Elongation min. %	D638 or D2256	200	150	75	35
Specific Gravity, min.	D792	~~623 ± 2~~ 2.16 ± 0.03	2.20 ± 0.03	2.10 ± 0.03	—
Melting Point, °F	D4894, D4895 or D5977	622 ± 3	621 ± 18	621 ± 18	—

Values for intermediate filler contents may be obtained by interpolation.

2015 Revision

SECTION 19: BRIDGE DECK JOINT SEALS

TABLE OF CONTENTS

SECTION 19

BRIDGE DECK JOINT SEALS

19.1—GENERAL

This work shall consist of the furnishing and installing of joint sealing systems in bridge decks of the types used where significant movements are expected across the joint.

Joint seals specified in the contract documents as poured joint seals shall conform to the requirement of Article 8.9, "Expansion and Contraction Joints."

The type and dimensions or movement rating for bridge deck joint seals at each location shall be as shown in the contract documents or as ordered by the Engineer.

All joint seals shall prevent the intrusion of material and water through the joint system.

C19.1

Bridge deck joint seals include compression seal joints consisting of preformed elastomeric material compressed and installed in specially prepared joints and joint seal assemblies consisting of assemblies of metal and elastomeric materials installed in recesses in the deck surface.

19.2—WORKING DRAWINGS

19.2.1—General

If not given in the contract documents, calculations showing the joint settings for their installation shall be required before approval to install joints in any bridge deck can be given. The Contractor shall submit working drawings to the Engineer showing the installation procedure and joint assembly for bridge decks using proprietary joint systems. Shop drawings shall be submitted to the Engineer for approval for joints having a total movement of more than 1.75 in.

No work on the deck joint seal shall be performed prior to approval of working drawings by the Engineer. Such approval shall not relieve the Contractor of any responsibility under the contract documents for the successful completion of the work.

19.2.2—Special Contract-Document Requirements for Modular Bridge Joint Systems (MBJS)

The MBJS axis is defined as any axis paralleled to the axes of the edgebeams and seals of the MBJS. The skew angle is the angle between the longitudinal axis of the support bars and a line perpendicular to the MBJS axis. Movements parallel to the longitudinal axis of the support bars will be referred to as longitudinal movements. The longitudinal axis of the support bars is typically coincident with the direction of the net expected thermal movement. Movements perpendicular to the longitudinal axis of the support bars will be referred to as transverse movements.

C19.2.2

Close cooperation is required between the Designer, Contractor, and joint Manufacturer to ensure a quality joint installation. For example, Designers should work with the Manufacturers when detailing blockout reinforcement. By working together, experienced Designers (typically more familiar with reinforcement needs or details of the adjacent structural elements) and Manufacturers (often more familiar with installation problems and factors affecting MBJS durability) will likely develop good details that reduce placement problems during construction. Unfortunately, it is difficult to anticipate reinforcement requirements during design because the joint system and Manufacturer are not known until after contract award.

Many MBJS durability problems are a result of poor detailing. Problems with poor detail design include improper detailing of superstructure elements and reinforcement steel, reflective cracking in the concrete cover directly above support boxes, and lack of access to the underside of the MBJS for inspection and maintenance.

Reflective cracking in the concrete deck directly above support boxes has been noted at almost every MBJS. These cracks permit water intrusion that may eventually cause delamination of the concrete cover over the support-bar boxes. Reflective cracking is most significant in a region of the deck which has transverse negative moment causing transverse tension stress in the top of the deck, i.e., where the deck is continuous over a girder in a direction transverse to the longitudinal axis of the bridge.

Among the factors believed to affect reflective cracking are:

- the discontinuity in the slab thickness caused by the support box,
- the relative flexibility of the thin plates used to construct the top of the support boxes, and
- the slump of the concrete mixture.

In either case, it seems that the solution may be to provide adequate concrete cover and transverse reinforcement over the support boxes to minimize the crack widths.

Blockouts are used for modular joints to facilitate placement and adjustment. The support boxes of MBJS are typically 7.5 in. or more in depth and extend 12.5 in. or more beyond the edge of the gap. Therefore, a blockout that is 15.0 in. deep that extends 27.0 in. beyond the edge of the gap may be required. The blockout must be designed to support the weight of the joint, particularly on deck overhangs. Sometimes the ends of steel girders are notched to accommodate the joint. The possible fatigue problem at these notches should be considered. Careful installation of the joint and placement of blockout concrete are critical to ensure a durable joint.

Problems can occur when the reinforcing bars are too close to the edgebeams or anchorages, preventing the flow of aggregate under the edgebeam. Part of the problem is that the MBJS is a bid item and there are usually several qualified Manufacturers. Therefore, the configuration of the MBJS is not known at the time the reinforcement is designed. However, some aspects of the MBJS configuration may be anticipated and allowances made in the reinforcement. This should be accomplished through appropriate details and/or Contractor submittal of revised reinforcement details.

In steel superstructures, it is acceptable to attach the MBJS directly to the end diaphragm, a bulkhead plate, or the girders, although these attachments and the supporting members must be designed for strength and fatigue to resist the repeated large impact forces imparted to the MBJS. Any possible differential movements of the deck and the attachment points should also be considered. Short-term differential movements of the deck and the attachment points could cause a fatigue failure and long-term differential movement can cause distortion or a strength failure.

The contract documents shall include the following:

- A cross-section of the deck at every unique MBJS configuration shall be shown. The MBJS shall be shown near midrange of its movement capacity. The total gap dimension between a reference vertical plane near the inside surfaces of the edgebeams and the bridge temperature corresponding to this position shall be clearly noted.

- If the support boxes are supported by the deck or abutment, there shall be adequate space, a minimum of 2.0 in., between the bottom surfaces of the MBJS and the deck blockouts to allow easy placement of concrete and allow for adequate consolidation of concrete under and around all parts of the MBJS, especially under any horizontal surfaces such as the bottom of support boxes. If a 2.0-in. space cannot be provided, the support boxes shall be set on a grout pad.

- There shall be at least 6.0 in. of clear space between the support boxes or anchorages on the ends of support boxes and the periphery of the blockout to permit placing of concrete around the MBJS.

- A detail showing how the MBJS is to be configured at the curbs and/or parapets.

The Contractor shall submit details of the MBJS to be used together with installation and waterproofing plans to the Engineer for approval prior to fabrication of the MBJS. The shop plans shall include, but not be limited to the following:

- Plan and section views of the MBJS for each movement rating and roadway width, showing dimensions and tolerances.

- All welded and bolted centerbeam/support bar joints and all shop and field splices shall be shown.

- Complete details of all components and sections showing all material incorporated into the MBJS.

- All ASTM, AASHTO, or other material designations.

- Corrosion protection system.

- Lifting locations and lifting mechanisms shall be shown as part of an integral installation plan.

- Temperature adjustment devices and opening dimensions relative to temperature.

Usually, the abutment shelf or the pier caps provide an adequate work surface for inspection and maintenance.

- A unique configuration of MBJS may represent many essentially similar MBJS at various locations in a project. Minor variations in dimensions and location or of nonstructural details such as the support boxes, curb upturn, or slider plates at barriers are permitted for a given configuration and may be covered by special details and/or notes. Variation in the gap opening not requiring a different size or number of seals may be covered in a special note.

- This provision is only applicable when the wheel-load reactions of the support box are not transferred directly through positive attachment to superstructure elements other than the deck or abutment. It has been suggested that 2.0 in. has been sufficient in the past (especially if aggregate size is limited), however, 3.0 in. is recommended, if possible. Grout pads increase costs and should only be used when adequate space cannot be provided.

The Contractor shall also submit the following test reports and certificates for review and approval:

- Manufacturer's certificate of compliance with the AISC Quality Certification Program, Simple Steel Bridges.

- Certification that welding inspection personnel are qualified and certified as welding inspectors under AWS QC1, Standard for Qualification and Certification of Welding Inspectors. Documentation that any personnel performing nondestructive evaluation (NDE) are certified by ASNT.

- Manufacturer's certificate of compliance for the PTFE sheeting and fabric.

- Certification that MBJS passed the Prequalification Tests required in Article 19.3.2.

- Certification that the bearings, springs, and equidistant devices are the same formulation, Manufacturer, and configuration that were used in the Prequalification Tests required in Article 19.3.2. In each certification, the name and address of the Manufacturer of the springs, bearings, and equidistant devices shall be provided.

- Design calculations sealed by a registered Professional Engineer. The design calculations shall include a fatigue design and a load factor design for all structural elements, connections, and splices.

- Plan for replacement of parts subject to wear may be allowed for in the design. The Contractor shall submit for the Engineer's approval a written maintenance and part replacement plan prepared by the joint Manufacturer. This plan shall include a list of parts and instructions for maintenance inspection, acceptable wear tolerances, methods for determining wear, and procedures for replacing worn parts.

- Method of installation, including, but not limited to: sequence, installation gap setting for various temperatures, support during placement of the concrete, and installation at curbs.

- Recommendations for storage of MBJS and details of temporary support of joint for shipping and handling.

- Welding procedure specifications.

- Any required changes to the blockout reinforcement in order to accommodate the MBJS.

- Temporary bridging plan for any MBJS for which construction traffic is anticipated following installation.

19.3—MATERIALS

19.3.1—Bridge Deck Joint Seal Materials and Joint Seal Assemblies Other than Modular Bridge Joint Systems

Bridge deck joint seal materials and assemblies shall conform to the following specifications:

- Preformed elastomeric joint seals of multiple-web design shall conform to AASHTO M 297, (ASTM D3542).
- Lubricant-adhesive for use with preformed elastomeric seals shall conform to ASTM D4070.
- Deck joint seal assemblies shall be of an approved type for each size required and shall conform to the specifications provided by the Manufacturer at the time of approval.
- Steel and fabricated steel components shall conform to the requirements of Section 23, "Miscellaneous Metal."

19.3.2—Modular Bridge Joint Systems

MBJS shall conform to the following specifications:

- MBJS shall conform to the specifications provided by the Manufacturer at the time of approval.
- MBJS shall be prequalified by satisfying all testing requirements detailed in Appendix A19, which are designed to allow approved MBJS to be used for a limited range of applications.
- MBJS shall be designed in accordance with Article 14.5, "Bridge Joints," of the *AASHTO LRFD Bridge Design Specifications*.
- Preformed elastomeric joint seals of multiple-web design shall conform to AASHTO M 297 (ASTM D3542).
- Preformed elastomeric joint seals of the strip type shall conform to ASTM D5973.
- Seals shall be continuous and splices are not permitted unless specifically approved by the Engineer.
- Lubricant-adhesive for use with preformed elastomeric seals shall conform to ASTM D4070.

C19.3.2

Modular bridge joint systems (MBJS) are sealed joints with two or more elastomeric seals held in place by edgebeams that are anchored to the structural elements (deck, abutment, etc.) and one or more transverse centerbeams that are parallel to the edgebeams.

Large movement ranges can be accommodated by modular bridge joint systems (MBJS). Present designs for MBJS typically use one or more transverse centerbeams to separate two or more seals. Because it must accommodate larger expansion movements, an MBJS must structurally support the wheel-loads across the gap between bridge elements.

There are two basic types of support-bar MBJS: multiple- and single-support-bar systems. Multiple-support-bar (MSB) MBJS have centerbeams that are rigidly connected to support bars. Each support bar supports only one centerbeam. For the MSB system, a support box will hold as many support bars as there are centerbeams.

Single-support-bar (SSB) MBJS have transverse centerbeams that are attached to only one support bar at each support box location using steel yokes and elastomeric springs and bearings. One special type of SSB MBJS is the swivel-joist system, in which the support bar swivels as well as slides in the support boxes.

- Springs, bearings, and equidistant devices (sometimes referred to as control springs) shall be the same material composition and formulation, Manufacturer, fabrication procedure, and configuration as the ones used in the prequalification test.

- Urethane foam shall conform to ASTM D3574.

- Polytetrafluorethylene (PTFE) shall be 100-percent virgin Teflon®, woven PTFE fabric, or dimpled PTFE conforming to the requirements of Section 18.8, "Polytetrafluorethylene (PTFE) Surfaces for Bearings," and shall be provided on every sliding surface.

- Stainless steel sheets conforming to ASTM A240/A240M, Type 304, shall be provided on sliding surfaces. ASTM A240/A240M, Type 316 is recommended for a severe environment.

The MSB and SSB types of MBJS are the most common and, therefore, will be the focus of these Specifications. Certain parts of these Specifications may not be applicable to alternative types of MBJS. These Specifications permit alternative designs that meet the prequalification tests requirements in Article 19.3.2, "Modular Bridge Joint Systems."

In MBJS that use a support bar that slides on bearings, the support bar usually has thin, stainless steel cover plates joined to the top and bottom of the support bar to provide smooth sliding surfaces. The support bars slide between elastomeric bearings and springs that are fixed in the support boxes, usually by a round boss or protrusion that fits into a hole in the steel plate of the support box. The bearings and springs typically have low-friction polytetrafluorethylene (PTFE) pads bonded to the sliding surface of the spring or bearing.

The elastomeric bearings and springs are both precompressed and located atop and below the support bar, with the bearing on the bottom and the spring on top. The springs exert compression to keep the bearing in place. The vertical component of each wheel-load applied to the centerbeam and transmitted through the support bar compresses the bearings and reacts against the support box and the deck. There is a significant upward rebound of each wheel-load cycle that compresses the springs and reacts on the top plate of the support box, imposing an upward load on the deck.

- Steel and fabricated steel components shall conform to the applicable requirements of Section 11, "Steel Structures," for nonfracture-critical members, with the following exceptions:

  - Welding shall conform to ANSI/AWS D1.1/D1.1M *Structural Welding Code—Steel.*

  - Fasteners and their holes need not conform to Section 11, "Steel Structures," provided they were used in the prequalification tests.

The wheel-load may also impart a horizontal force to the centerbeam and an associated rebound. The horizontal load is transmitted through the centerbeam, into the support bar, and into the springs and bearings through friction. Ultimately, the horizontal force is resisted by the small bosses in the springs and bearings into the support box and deck. These small bosses are subjected to millions of cycles of this reversible shearing action. Shear failure of the bosses leads to systemic failure of the MBJS. The movements of bridge elements provide the necessary forces to open and close the MBJS. An equidistant device is typically required to maintain an approximately equal gap between centerbeams and between centerbeam and edgebeam. A common equidistant device used in support-bar systems is comprised of a series of horizontal elastomeric springs sometimes called control springs. In some MBJS, the equidistant devices tend to close the gap, while in other MBJS, the equidistant devices tend to open the gap between centerbeams.

19.4—MANUFACTURE AND FABRICATION

19.4.1—Compression Seal Joints

Preformed elastomeric joint seals shall not be field-spliced, except when specifically permitted by the Engineer.

19.4.2—Joint Seal Assemblies Other than Modular Bridge Joint Systems

Expansion joint assemblies shall be fabricated by the Manufacturer and delivered to the bridge site completely assembled, unless otherwise specified in the contract documents.

19.4.3—Modular Bridge Joint Systems

19.4.3.1—General

The following requirements apply to both shop welds and field welds (if any) unless specifically noted otherwise. The MBJS shall be fabricated in accordance with the dimensions, shapes, details, material specifications, and procedures shown in the approved shop plans Fillet welds shall be welded continuously. Intermittent fillet welds are not permitted.

Field splices should be avoided if at all possible and the entire MBJS shipped and installed as one unit. If field splices cannot be avoided, it is recommended that the splices be located away from potential wheel paths and preferred that splices be located under the median traffic barrier. Only field-splice details that have been fatigue-tested in accordance with the prequalification tests in Article 19.3.2 may be used for MBJS. Typically, the fatigue design will dictate that the span of the centerbeam with the splice must be smaller than the continuous spans; generally, it is best to make this span as small as possible.

A full-penetration field weld can sometimes be made from the deck when there is only one centerbeam and it can be lifted out enough to access the bottom of the centerbeam. Care must be taken to avoid weld metal getting into the seal retainer grooves, which can lead to seal pullout and leaking. Fillet or partial penetration welds are not permitted. Welded splices are not permitted if there is more than one centerbeam. Edgebeam profiles may be field-spliced with fillet welds across only part of the profile.

Lifting devices shall be provided, and devices to maintain the preset opening of the joint shall be provided at a uniform spacing not greater than 15.0 ft along the length of the MBJS. At least three devices shall be used per segment of MBJS.

When the fabrication is completed, the Manufacturer shall perform the preinstallation inspection described in Article 19.5.4.2 to assure that the MBJS will pass this inspection.

C19.4.3.1

Whenever possible, fillet welds shall be on both sides of an attachment. The MBJS shall be shipped and installed in one piece, wherever possible, to avoid field splicing.

If it can be assured that a splice will remain under a median barrier and it can be assured that water cannot get to this area, it may be permitted to butt the ends of the two segments of MBJS together, but not splice them.

19.4.3.2—Edgebeam Profile and Anchorage

The edgebeams shall be fabricated from structural steel. The web of the edgebeam cross-section shall be at least 0.375 in. in thickness. The same cross-section must be used that was used in the prequalification test. Shop splices in the edgebeam profile shall be two-sided, complete-joint-penetration groove welds. The edgebeam shall be continuously fillet welded to the support boxes.

The anchorage shall be designed in accordance with Article 14.5.6.9, "Modular Bridge Joint Systems (MBJS)," of the *AASHTO LRFD Bridge Design Specifications*. If there is a horizontal element in the edgebeam cross-section, the horizontal element shall also be anchored to resist the full value of the wheel-load with impact acting upward (from rebound).

C19.4.3.2

The use of a horizontal element in the edgebeam cross-section is not recommended due to difficulty with consolidating concrete under the horizontal flanges. If a horizontal element is used, it shall have 0.75-in. diameter air holes spaced every 18.0 in. to improve consolidation of the concrete under the horizontal element. If there is no horizontal element, the top of the profile shall be located between 0 and 0.25 in. below the top of the wearing surface of the deck.

Best results have been obtained with solid shapes with machine-cut grooves in the side to retain the seals. One design that satisfies the load requirements and has been designed according to ACI requirements is a 1.5-in. thick edgebeam with no horizontal element and Grade 50 (345), 0.5-in. diameter welded, headed concrete anchor studs 6.0 in. long, spaced at 12.0 in. on center. This design requires at least 3.0 in. of cover above the anchors (measured from the centerline of the anchor to the surface of the concrete). There is no need to bend the studs, unless the anchor stud falls at an overlay/ structural slab interface.

19.4.3.3—Centerbeam and Support Bar

The centerbeams, support bars, and connection details shall be the same type as were used in the prequalification tests. Shop splices in the centerbeam profile shall be two-sided, complete-joint-penetration groove welds. In welded multiple-support-bar MBJS, the weld joint between the centerbeam and support bar shall be a full-penetration groove weld.

After welding, the centerbeam/support bar assembly shall be placed on a flat surface and it shall be verified that the support bars lie in a single plane, with no part of the bottom of any support bars exceeding 0.25 in. off the surface. The subassembly may be straightened. No more than three attempts may be made to heat-straighten the subassembly.

C19.4.3.3

Best results have been obtained with solid-steel bars. For the centerbeam, best results have been obtained with machine-cut grooves in the side to retain the seals. Techniques to avoid loosening of the bolts include using adhesives, welding the outer surface of the nut to the exposed threads, or galling the threads.

19.4.3.4—Seals

Seals shall be installed by the Manufacturer before shipping unless centerbeam field splices are used. If field splices are necessary, continuous seals (without splices) shall be installed in the field after the construction is complete. In either case, the same lubricant-adhesive that was used in the prequalification tests shall be used when installing the seals. The seals shall extend out from the ends of the edgebeams and centerbeams by at least 2.0 in.

C19.4.3.4

Movement joint seals usually have a maximum movement range of 3.0 in. Seals up to 5.0 in. have been used successfully. However, the maximum opening for seals is set by AASHTO requirements. Seals used for in-service MBJS must be the size that was tested in the prequalification tests required in Article 19.3.2.

The ASTM specifications for the seal material appear to be sufficient to assure adequate durability under normal wear and tear and environmental exposure. Some agencies do not allow the seal to be installed in the field because of the potential for detachment. However, if the MBJS is installed in stages, for the rehabilitation of an existing bridge or new installations on wide bridges, a seal field splice will be required if the seal is installed in the shop. Field splices of the seals should be avoided. The performance of spliced seals is not adequate in protecting the bridge superstructure from deck drainage. Therefore, in the case of staged construction, seals should be installed in the field in one continuous piece.

Another common problem is that the seals fill with debris. Traffic passing over the joint can work the seal from its anchorage by pushing on this debris. Manufacturers contend that MBJS systems are self-cleaning because as the joint approaches its full, open position, debris is expelled from the joint. However, many Designers conservatively oversize the MBJS, thus preventing the joint from being self-cleaning. Debris has been observed to be the cause of damage to many MBJS. Debris has been reported in the expansion gap that reduced the effective movement range. When the bridge expands, debris trapped in the seal gaps is compacted and can cause additional stresses and associated damage in both the joint and the structure.

19.4.3.5—Support Boxes

Support boxes shall be made from steel plate or tubes at least 0.375 in. thick continuously welded. If the support boxes are more than 16.0 in. wide, the thickness of the top plate shall increase so that the width-to-thickness ratio does not exceed 45 or stiffening must be used. If the support box is made of nested tubes, the diameter or width-to-thickness ratio of each tube shall not exceed 45.

C19.4.3.5

The top plate is required to support traffic-loading. Excessive flexibility can result in reflective cracking above the support boxes. If 3.0 in. of cover cannot be provided above the top plate, the top plate may need to be thicker or stiffened to adequately support the traffic loads.

19.4.3.6—PTFE Sliding Surface

The PTFE shall be bonded under controlled conditions and in accordance with the instructions of either the PTFE Manufacturer or the adhesive Manufacturer. After completion of the bonding operation, the PTFE surface shall be smooth and free from underlying bubbles.

19.4.3.7—Stainless Steel Sliding Surface for MBJS

The stainless steel shall be polished to an 8.0-μin. (0.20-μm) mirror finish.

19.4.3.8—Corrosion Protection

All steel surfaces, except the surfaces under stainless steel or those to be bonded to PTFE, shall be protected against corrosion.

C19.4.3.8

Corrosion of steel sections that have been damaged or exposed has been observed in a number of MBJS. Metallic components of MBJS such as bolts, stainless steel sliding plates, and anchors have failed due to corrosion. Accumulation of damp debris in the recesses of the MBJS has been the cause of severe corrosion.

19.5—INSTALLATION

19.5.1—General

All joint materials and assemblies stored at the job site shall be protected from damage and assemblies shall be supported so as to maintain their true shape and alignment. Deck joint seals shall be constructed and installed to provide a smooth ride. Bridge deck joints shall be covered over by protective material after installation until final clean-up of the bridge deck.

After installation and prior to final acceptance, deck joint seals shall be tested in the presence of the Engineer for leakage of water through the joint. Any leakage of the joint seal shall be cause for rejection.

19.5.2—Compression Seal Joints

Joints in the roadway area of bridge decks that are to be sealed with compression seals shall be cast to a narrower width than required for the preformed material. Such joints in curbs and sidewalks may be cast to full width. Prior to installation of compression seals in joints whose width is narrower than needed, a groove of proper width and depth to receive the preformed material shall be saw-cut along the top of the joint.

When making saw cuts into the bridge deck, spalling shall be minimized. Both sides of a groove shall be cut simultaneously to the proper depth and alignment as shown in the contract documents. The alignment of the saw shall be controlled at all times by a rigid guide. The width of the groove shall depend on the temperature and age of the concrete and shall be as directed by the Engineer. Lip of saw cut should be bevelled to avoid later breakage. After saw cutting, any spalls, popouts, or cracks shall be repaired prior to installation of the lubricant-sealant. Saw cuts are not required where armor plates are used.

At the time of installation, the joint shall be clean and dry, and free from spalls and irregularities that might impair a proper joint seal. Concrete or metal surfaces shall be clean and free of rust, laitance, oils, dirt, dust, or other deleterious materials. Premolded elastomeric compression joint seals shall be installed without damage to the seal by suitable hand methods or machine tools. The lubricant-adhesive shall be applied to both faces of the joint prior to installation and in accordance with the Manufacturer's instructions. The preformed elastomeric seal shall be compressed to the thickness specified in the contract documents or as approved by the Engineer for the rated opening and ambient temperature at the time of installation. Loose-fitting or open points between the seal and the deck shall not be permitted.

19.5.3—Joint Seal Assemblies Other than Modular Bridge Joint Systems

Expansion joint seal assemblies shall be constructed to provide absolute freedom of movement through a range consistent with that prescribed by the Engineer or as shown in the contract documents. Installation shall be in accordance with the Manufacturer's recommendations. Final settings of the deck joint seal assembly at the time of casting in the anchorages of the unit depend on the relationship of the current temperature of the superstructure to its expected mean temperature and shall be as specified by the Manufacturer or Engineer or as shown in the contract documents.

19.5.4—Modular Bridge Joint Systems

The Contractor shall follow the Manufacturer's written installation guidelines and the following guidelines. The Engineer in the following guidelines may be the authorized representative of the Engineer.

19.5.4.1—Shipping and Handling

The MBJS shall be delivered to the job site and stored in accordance with the Manufacturer's written recommendations as approved by the Engineer. Damage to the corrosion protection system shall be repaired to the satisfaction of the Engineer. Seals shall not be damaged or cut.

19.5.4.2—Preinstallation Inspection

Immediately prior to installation, the MBJS and the blockout shall be inspected by the Engineer for:

- Proper alignment.
- Complete bond between the seals and the steel.
- Proper placement and effectiveness of studs or other anchorage devices.
- Proper placement of elastomeric springs and bearings.
- The proper placement of waterproofing membranes shall be verified, if utilized.
- The clearance specified on the drawings 3.0 in. is recommended) between the bottoms of the support boxes of MBJS and the surface of the blockout should be verified.

Cutting of bridge deck reinforcing steel can compromise the structural integrity of the blockout and requires approval of the Engineer. The Engineer shall verify that reinforcing mesh or bars are at least 2.0 in. from the edgebeam or anchorages and do not prevent the flow of concrete around the MBJS.

No bends or kinks in the MBJS steel shall be allowed (except as required to follow the roadway crown and grades). Any MBJS exhibiting bends or kinks shall be repaired to the Engineer's satisfaction or replaced at the expense of the Contractor.

Seals not fully connected to the steel shall be fully connected at the expense of the Contractor. Headed concrete anchors shall be inspected visually and shall be given a light blow with a hammer. Any headed concrete anchor that does not have a complete end weld or does not emit a ringing sound when struck a light blow with a hammer shall be replaced. Headed concrete anchors located more than 1.0 in. along the length of the edgebeam from the location shown on the shop drawings and headed concrete anchors located more than 0.25 in. too high in elevation

C19.5.4.2

If the bridge deck or abutment reinforcement has not been designed to accommodate the configuration of the MBJS, the reinforcement may have to be altered.

Plastic wrap or foam covers should be placed over the interior opening of the support boxes. These will aid in preventing debris and animals from entering the support box. These should not be added until the MBJS is ready to be installed so that the preinstallation inspection can be performed on the interior components of the support box. The wrap or covers should be easy to remove for future inspections.

(reducing cover) shall be carefully removed and a new anchor welded in the proper location. All anchor replacement shall be at the expense of the Contractor.

19.5.4.3—Installation

Prior to installation of the joint, the blockout and supporting system shall be protected from damage and construction traffic.

19.5.4.3.1—Setting Gap Opening

The MBJS shall be installed at the proper gap opening corresponding to the installation temperature, as shown on the approved shop plans. The opening devices should be removed immediately after the concrete is placed.

19.5.4.3.2—Formwork

The Contractor shall ensure that formwork excludes concrete entry into support boxes or in any way impeding free movement of the MBJS.

19.5.4.3.3—Supporting MBJS during Placing of Concrete

The MBJS shall be fully supported during the placement of the concrete. Welds for temporary attachments to the centerbeams or support bars for erection purposes must be removed and the surface ground smooth. The corrosion protection system shall be repaired to the satisfaction of the Engineer using a method approved by the Engineer. To reduce corrosion of the MBJS, it should be electrically isolated by not connecting the bridge deck reinforcement to the MBJS.

C19.5.4.3.3

MBJS have been supported during installation (prior to placement of the deck concrete) in a number of ways. The preferred method of supporting the MBJS during installation is to suspend it from a series of beams spaced at no more than 10.0 ft spanning the blockout between the deck and abutment or between adjacent decks. These beams allow for more precise setting of the joint height and grade. The deck provides a reference to establish the final MBJS profile.

One common practice has been to weld joint anchorages to the deck reinforcing steel for support. This practice is not recommended because it is desired to keep the MBJS and the deck reinforcement electrically separated from each other in order to reduce corrosion and because of concern for cracking in either the reinforcing steel, the anchorage, or both. If the MBJS is tack welded to deck reinforcement, deflection of the reinforcement must be considered.

In some installations, leveling bolts attached to girder top flanges are used to support the joint. These bolts permit adjustment of the joint height during installation as well as provide support. At least two problems have been reported with the use of leveling bolts. First, some larger MBJS are so heavy that the leveling bolts fail. A second problem with these bolts is that they may carry wheel-loads directly from the edgebeam to the bridge girder, which may not have been considered in the design. The wheel-loads may eventually cause movement of the leveling bolts, which can result in problems.

Temporary connectors between edgebeams (for shipping and handling) should be removed before placing concrete.

19.5.4.3.4—Placing the Concrete

The concrete shall be controlled, mixed, and handled as specified in Section 8, "Concrete Structures," and/or Agency construction specifications. Very-high-slump concrete shall not be used in the blockout. Concrete shall not be deposited in the forms until the Engineer has inspected and approved the placement of the reinforcement, conduits, anchorages, and prestressing steel.

If there is a vertical grade, concrete shall be placed on the downhill side of the blockout first. The concrete shall be vibrated thoroughly so as to adequately consolidate concrete underneath the support boxes and edgebeams. Care should be taken to avoid displacement of the forms and reinforcing steel. The concrete shall not be placed during extremely cold weather or during heavy rain.

C19.5.4.3.4

The bridge deck concrete may be finished to the top of a MBJS that is set at a slightly incorrect elevation when the local change in deck profile could cause increased impact forces on the MBJS.

19.5.4.3.5—Finished MBJS Tolerances

The MBJS shall be inspected after installation and again after at least 1 yr of traffic-loading (or longer if there is a guarantee period) to verify the following:

- The top surfaces of the MBJS shall be recessed from the finished roadway profile 0 to 0.25 in.
- There shall be no more than 0.125-in. difference in elevation among the tops of any of the centerbeams or edgebeams. This variation shall be measured vertically from a straight line connecting the top of the deck profile on each side of the MBJS.
- There shall be no more than 0.5-in. difference among gap widths at either end of a seal or among the multiple gaps of MBJS.

C19.5.4.3.5

A 0.125- to 0.25-in. recess of the MBJS is recommended to minimize potential snowplow collision effects; however, the deck may be finished to the top of the joint for rideability if desired.

19.5.4.3.6—Bridging MBJS After Installation

Construction loads shall not be allowed on the MBJS for at least 72 h after installation is completed. If it is necessary to cross the MBJS, the Contractor shall bridge over the MBJS in a manner approved by the Engineer.

C19.5.4.3.6

Movements of the edgebeam prior to complete concrete curing may cause gaps or openings between the edgebeam or anchorage and the plastic deck concrete. These gaps may result in movement of the edgebeam under traffic-loading and associated rapid deterioration.

19.5.4.3.7—Removal of Forms and Debris

All forms and debris shall be removed after installation.

C19.5.4.3.7

Forms and debris tend to interfere with the free action of the MBJS. They may also interfere with the inspection of the MBJS.

19.5.4.3.8—Watertightness Test

If specified in the contract documents, the watertightness test shall be conducted. After the MBJS has been installed and completed, the MJBS shall be flooded for a minimum of 1 h to a minimum depth of 3.0 in. If leakage is observed, the MJBS shall be repaired to the Engineer's satisfaction and retested at the Contractor's expense. The repair procedure shall be recommended by the Manufacturer and approved by the Engineer.

19.5.4.3.9—Acceptance

An MBJS that fails inspection or testing shall be replaced or repaired to the satisfaction of the Engineer at the Contractor's expense. Any proposed corrective procedure shall be submitted to the Engineer for approval before corrective work is begun.

19.6—MEASUREMENT AND PAYMENT

Deck joint seals shall be measured by the linear foot of acceptable joint seal completely installed by measurements made along the slope of the centerline of the joint seal.

Payment of linear feet of joint seal as measured, for each type of seal for which separate payment is provided, shall include full compensation for the cost of labor, equipment, and materials to furnish and install the deck joint seal.

19.7—REFERENCES

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2009. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 29th Edition, HM-29, American Association of State and Highway Transportation Officials, Washington, DC.

AWS. 2004. AWS D1.1M/D1.1 *Structural Welding Code—Steel*, American Welding Society, Miami, FL.

APPENDIX A19—PROPOSED STANDARD TEST METHOD FOR MODULAR BRIDGE JOINT SYSTEMS

The following is a proposed standard test method for modular bridge joint systems to be included in the AASHTO *Standard Specifications for Transportation Materials and Methods of Sampling, Part 2: Test Methods.*

It is suggested that the proposed standard test methods for modular bridge joint systems be grouped with T 42, Preformed Expansion Joint Filler for Concrete Construction, in the table of contents under a revised heading of "Expansion Joints, Joint Filler, and Asphalt Plank" (currently "Joint Filler and Asphalt Plank").

Present specification is AASHTO T 42.

[Referenced in new Section 19.4.3, "Modular Bridge Joint Systems."]

APPENDIX A19

Standard Test Methods for Modular Bridge Joint Systems

1. SCOPE

1.1 This specification describes three test procedures for modular bridge joint systems (MBJS):

- Opening movement and vibration test………….……………….Section 5.1
- Seal push-out test ……….………………………………..…. Section 5.2
- Fatigue test ………………………………………………...Section 5.3

These test procedures are applicable to all types of MBJS, including (but not limited to) multiple-support-bar systems, welded or bolted single-support-bar systems, scissor joints, and swivel joints. The test procedures are not intended to be applied to deck joints other than MBJS (e.g., finger joints, roller joints, cushion joints, strip seal joints, compression seals, etc.) used in bridge structures. The functional design of MBJS is left to the discretion of the Manufacturer. These Specifications will only test the performance of those designs.

1.2 The values stated in SI units are to be regarded as standard.

1.3 This standard does not purport to address all of the safety concerns associated with its use. It is the responsibility of the user of this standard to consult and establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to its use.

2. REFERENCED DOCUMENTS

2.1 AASHTO Standards
- *AASHTO LRFD Bridge Design Specifications*
- *AASHTO LRFD Bridge Construction Specifications*
- M 297 (ASTM D3542) Preformed Polychloroprene Elastomeric Joint Seals for Bridges

2.2 ASTM Standards
- ASTM D5973 Elastomeric Strip Seals with Steel Locking Edge Rails Used in Expansion Joint Sealing

3. TERMINOLOGY

3.1 Definitions

3.1.1 **Modular Bridge Joint System (MBJS)**—A sealed joint with two or more elastomeric seals held in place by edgebeams that are anchored to the structural elements (deck, abutment, etc.) and one or more transverse centerbeams that are parallel to the edgebeams (see Figures 1–3). Typically used for movement ranges greater than 100 mm (4.0 in.).

3.1.2 **Single-Support-Bar System (SSB)—**An MBJS designed so that only one support bar is connected to all of the centerbeams. The centerbeam/support bar connection typically consists of a yoke through which the support-bar slides (see Figure 2).

3.1.3 **Strip Seal**—A sealed joint with an extruded elastomeric seal retained by edgebeams that are anchored to the structural elements (deck, abutment, etc). Typically used for expected total movement ranges from 40 to 100 mm (1.5 to 4.0 in.), although single seals capable of spanning a 125-mm (5.0-in.) gap are also available.

3.1.4 **Swivel-Joint System—**Sometimes also called Swivel-Joist System; a special type of SSB system designed so that the support bars (joists) also swivel about the centerbeam support-bar yoke connection. The largest known swivel joint has a 1220-mm (48.0-in.) movement range (see Figure 3).

3.1.5 **Welded Multiple-Support-Bar System (WMSB)**—A MBJS designed so that each support bar is welded to only one centerbeam (see Figure 1). Although some larger WMSB systems have been built and are performing well, WMSB systems are typically impractical for more than nine seals or for movement ranges larger than 680 mm (27.0 in.).

3.1.6 The MBJS axis is defined to be parallel to the axes of the edgebeams and centerbeams of the MBJS.

3.1.7 The skew angle is the angle between the longitudinal axis of the support bars and a line perpendicular to the MBJS axis.

3.1.8 Movements parallel to the longitudinal axis of the support bars will be referred to as longitudinal movements.

3.1.9 A unique MBJS configuration is defined by:

- the number of centerbeams,
- the number of support bars in each support box,
- the skew angle of support bars relative to centerbeams,
- centerbeam field-splice detail and location,
- centerbeam shop-splice detail,
- the cross-section and span of the centerbeams and support bars,
- the cross-section and span of the support bars,
- the location of attachments for equidistant devices,
- the cross-section and anchorage of the edgebeams, and
- the type of seals.

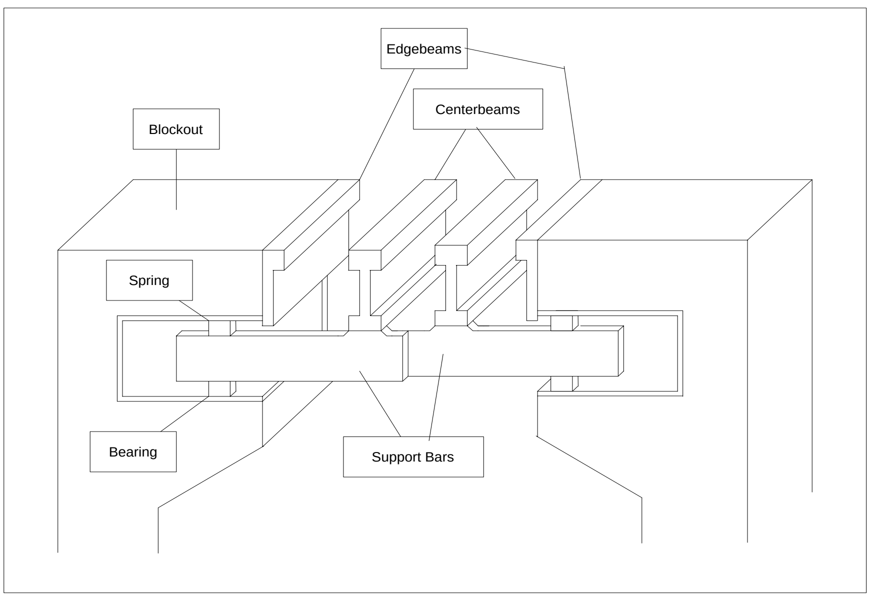


Figure 1—Cut-Away View of Typical Welded-Multiple-Support-Bar (WMSB) Modular Bridge Joint System (MBJS) Showing Support Bars Sliding within Support Boxes (Seals are not shown for clarity).

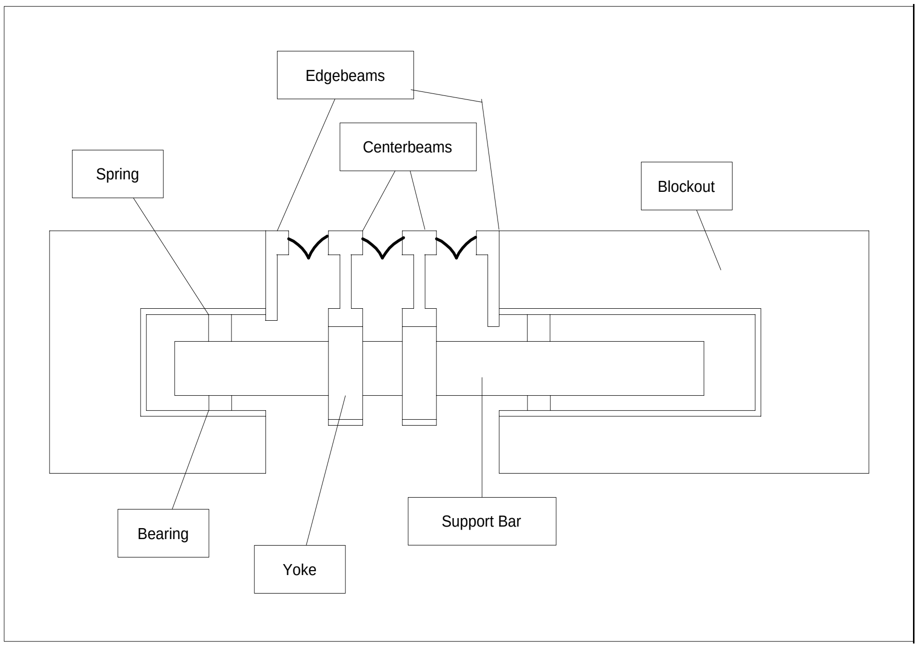


Figure 2—Cross-Section View of Typical Single-Support-Bar (SSB) Modular Bridge Joint System (MBJS) Showing Multiple Centerbeams with Yokes Sliding on a Single Support Bar.

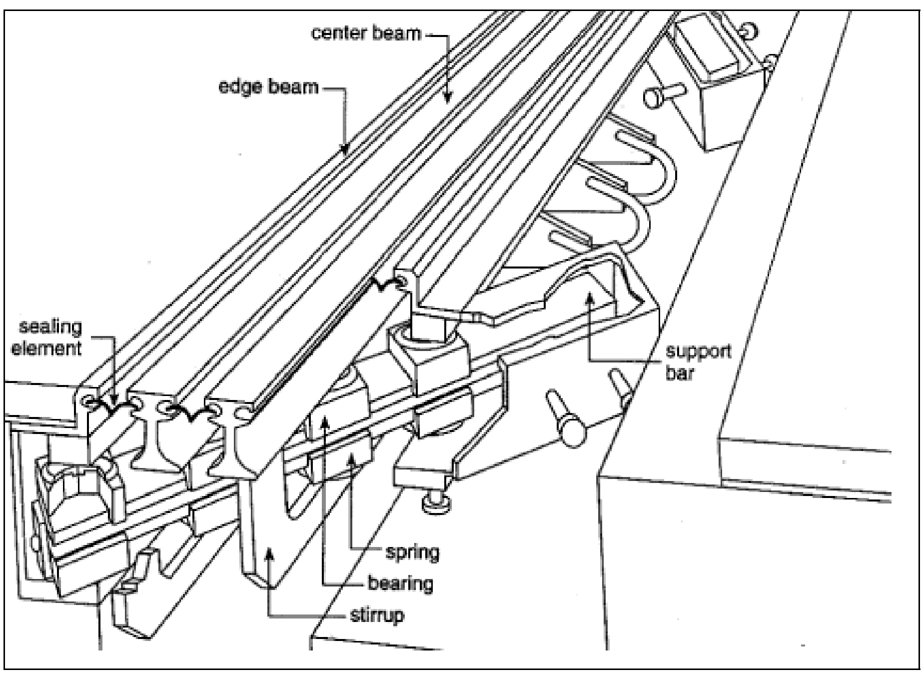


Figure 3—Cut-Away View of a "Swivel Joint," i.e., a Special Type of Single-Support-Bar (SSB) Modular Bridge Joint System (MBJS) with a Swiveling Single Support Bar.

4. SIGNIFICANCE AND USE

4.1 These test procedures may be used to qualify specific types of MBJS produced by a specific supplier. The test procedures are intended to verify that a MBJS meets minimum levels of performance. The fatigue tests described in Section 5.3 establish the appropriate fatigue detail categories for the centerbeam/support bar connection, field splice, shop splice, or any other critical details. Once the fatigue category of a connection or other detail has been established through a suitable number of tests, this category is considered applicable to the full range of configurations with different cross-section sizes, different numbers of centerbeam, different centerbeam and support-bar span, and different skew angles.

The four main types of elastomeric MBJS parts are the seals, bearings, springs, and equidistant devices. Bearings, springs, and other elastomeric components are qualified with the opening movement and vibration test as described in Section 5.1 and therefore need not be explicitly designed for strength or fatigue. The ASTM specifications for the seal material assure adequate durability under normal wear and tear and environmental exposure. The seal push-out test described in Section 5.2 addresses the particular problem of seal detachment from centerbeams.

Once a specific type of design has passed the opening movement and vibration test, the seal push-out test, and the fatigue-testing, it may be accepted for use in a range of configurations as discussed in Sections 5.1 and 5.2. The material and Manufacturer of the structural components, seals, bearings, springs, and equidistant devices must be documented for the samples tested. In order to be considered prequalified, the MBJS must be manufactured with the same components from the same suppliers as provided in the specimen tested. If an alternative centerbeam or support bar not meeting the prequalified variations in Sections 5.1 and 5.2 is to be used, the opening movement and vibration test, the seal push-out test, and the fatigue-testing shall be repeated. If only other new components are used, then only the opening movement and vibration test and the seal push-out test shall be repeated.

In addition to passing these test requirements, each unique MBJS configuration, as defined in Section 3.1.9, shall be explicitly designed for strength and fatigue in accordance with Article 14.5, “Bridge Joints,” of the *AASHTO LRFD Bridge Design Specifications*, with calculations submitted as required in Article 19.2.2, “Special Contract-Document Requirements for Modular Bridge Joint Systems (MBJS),” of these Specifications.

5. TEST METHODS

5.1 Opening Movement and Vibration (OMV) Test

5.1.1 Apparatus

5.1.1.1 Actuator to open and close MBJS with stroke control and ability to measure load

5.1.1.2 Controller to control the frequency and count the cycles applied

5.1.1.3 Test fixture to restrain the specimen. Fixtures shall be capable of adequately supporting and securing the specimen during test. All support boxes shall be supported throughout the test. The fixtures shall be designed so that the specimen is supported at a minimum height of 760 mm (30.0 in.) to allow for the visual inspection of all components of the specimen during testing, as shown in Figure 4. It is recommended that the fixtures provide a 1:10 slope, i.e. 5.7 degrees, to facilitate movement of the springs and bearings. One side of the specimen shall be securely attached (no movement) to the fixtures. The other side shall be free to move in the longitudinal direction (parallel to the support bars of the MBJS). It is recommended that linear bearings be attached to the testing fixtures. These devices will allow for movement only in the longitudinal direction. Verification testing has shown that only one linear bearing is required per specimen. More than two linear bearings is not recommended due to the possibility of nonparallel movement and binding of the bearings. The opening and closing displacement shall be applied at the horizontal center of the specimen. The displacement shall be applied to a spreader beam that is attached to the edgebeam and each support box on the freely moving side of the MBJS. The center of the simulated longitudinal opening movement displacement shall be at a height of 100 mm (4.0 in.) above the top of the support box. The spreader beam shall be capable of withstanding all displacements and loads applied to it and transferring those displacements and loads to the specimen.

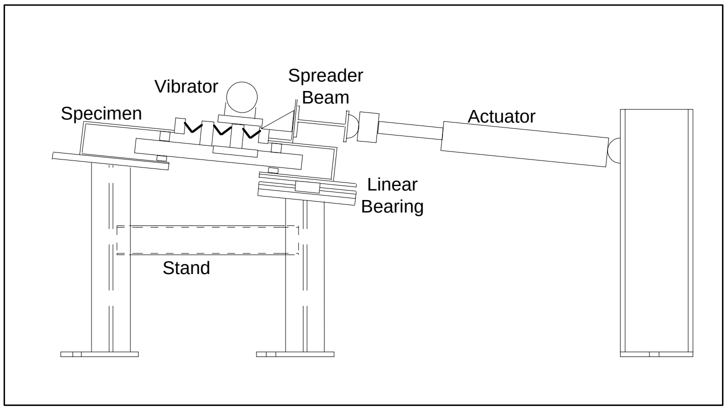


Figure 4—Recommended OMV Test Set-Up

5.1.1.4 Vibrator to apply simulated traffic loads. The vibrator shall produce a force amplitude of at least 22 kN (5.0 kip) at a frequency of between 70 and 100 Hz (total force range of at least 44 kN (10.0 kip). Any suitable vibrator may be used, however a Vibco SVRLS 8000 pneumatic high-frequency vibrator has been found to meet the requirements. The required force amplitude of another type of vibrator shall be verified by either:

- instrumenting the centerbeam to which the vibrator is attached and measuring the strain response or
- attaching the vibrator to any instrumented beam with known boundary conditions and measuring the strain response.

The vibrator typically has its own fixture that should be welded to a plate. A clamping device should be made from the plate with the vibrator welded to it and another similarly sized plate with four 25-mm (1.0-in.) minimum diameter threaded rods. (Rods of any strength or grade are suitable). The vibrator shall be securely fixed to one centerbeam. The seals may be cut for the threaded rods to pass through. The vibrator shall be placed as close as possible to the center centerbeam/support bar connection, with its centerline not to exceed 300 mm (12.0 in.) from center of connection to center of vibration. The clamp fixture shall not interfere with the opening and closing action of the MBJS other than to prevent it from closing all the way.

5.1.2 Sampling

A Manufacturer shall submit for testing one sample of each basic type of MBJS. Successful completion of fatigue tests described in Section 5.3 and passing both this OMV test and the SPO test described in Section 5.2 prequalifies a specific configuration MBJS with specific components. The following limited variations of that configuration are allowed and the MBJS shall still be considered prequalified:

- MBJS with any number of support boxes,
- MBJS with from one to $n + 4$ centerbeams (where n is the number of centerbeams in the configuration that was tested),
- MBJS with a support bar with any span, any depth up to twice as deep as what was tested, and with a width 1.25 times greater than was tested,
- MBJS with any centerbeam span less than 1.25 times the span that was tested,
- MBJS with smaller skew,
- MBJS with a lower angle of upturn or no upturn,
- MBJS with a flatter vertical crown or less of a horizontal kink, or
- MBJS with centerbeams or edgebeams of the same shape and relative proportions with cross-sectional area that is from 75 percent to 125 percent of the shape that was tested.

However, if the MBJS parameters are outside of these limits, then the OMV, Seal-Push-Out and Fatigue tests will need to be performed again. Additionally, any change to the following MBJS characteristics will necessitate complete retesting:

- specification of the material or component supplier changes, other than the steel components, weld metal, or the studs,

- seal-retaining system profile or device changes,
- dimensions of the elastomeric components varies (fatigue test need not be repeated in this case),
- different type of splice is to be used,
- different type of weld joint or weld location is to be used, or
- Fabricator change.

To prequalify MBJS with more than six centerbeams, a specimen with a larger number of centerbeams must be tested and, if successful, this test shall prequalify similar MBJS with from seven to the number of centerbeams in the test specimen.

5.1.3 Preparation of Test Specimen

Specimens shall be full-scale MBJS representative of typical production. Slight modifications are allowed to secure parts of the support boxes that would normally be secured by the surrounding concrete. The specimen shall include at least two centerbeams. Specimens shall be designed for fatigue in accordance with Article 14.5, "Bridge Joints," in the *AASHTO LRFD Bridge Design Specifications*. Each specimen shall contain at least three support boxes. Anchorage devices need not be attached to the specimen. Provisions shall be made so that the condition of the components inside the support boxes can be viewed from the outside of the MBJS. If any of the following are to be used in this type of MBJS, they must be included in the test specimen: skew, shop splices, field splices, vertical upturns for curbs or parapets, vertical crowns, or horizontal kinks.

5.1.4 Procedures

5.1.4.1 General

Prior to testing, specimens shall be visually inspected for any flaws, loose fasteners, etc. that could possibly affect the performance of the specimen. Any observed problem shall also be reported with the data.

Simulated longitudinal opening and closing movement ranges shall be applied through the actuator. The specimen shall be cycled at a frequency not to exceed 0.1 Hz. The specimen shall be cycled with a displacement of ±50.8 mm (±2.0 in.) about the mid-opening. The load requirements of the specimen shall be monitored continuously throughout the test as a function of the MBJS actuator stroke. The displacement shall be applied as described in Section 5.1.1.3 above.

Simulated traffic vibration loads shall be applied by a high-frequency vibrator as described in Section 5.1.1.4 above. The vibrator shall be run continuously while the specimen is undergoing simulated opening and closing movement cycles.

5.1.4.2 Definition of Failure

If the functioning of any component of the specimen during the test reduces the load capacity of the MBJS or inhibits the correct functioning of the MBJS, then the test shall be deemed to be a failure. The following criteria have been observed in preliminary tests and, although not a complete list of possible failure modes, are typical.

Welded connections: Occurrence of fatigue cracks or fractures in any weld shall be considered as failure.

Elastomeric components: The movement out of designed placement of springs or bearings shall be considered as failure. The movement out of designed placement of the equidistant devices shall be considered as failure. The equidistant devices shall be considered failed if the largest gap between any two adjacent edgebeams or centerbeams is greater than twice the smallest gap. The loss of seal bond or integrity against passage of water through the seal (except at the location where the seals have been cut for the vibrator clamping device) shall be considered a failure if it occurs before 9,130 simulated longitudinal movements (equivalent to a 25-y life).

Bolted connections: The loosening, fracture, or movement out of place of bolts used in any connection shall be considered a failure.

5.1.4.3 Termination of Test

The test shall be continued through failures until:

- 27,400 simulated longitudinal cycling movements have been completed (equivalent to a 75-y life),
- the MBJS is not functioning properly,

- it is deemed unsafe to continue testing, or
- failure mode has altered the test specimen so that no further failures will occur.

5.1.4.4 Report

This OMV test approximately simulates a 75-y service life with 27,400 opening and closing movement cycles, which is approximately the number of days in 75 y. At a frequency of about 0.1 Hz, this number of cycles is applied in approximately 76 h of continuous testing.

Data shall be reported in a tabular format and shall contain the following information:

- Cumulative number of simulated longitudinal movement cycles until failure(s) or end of test.
- Relative in-service life. Each simulated cycle shall represent one daily movement cycle.
- Cumulative number of simulated vibration cycles until each failure or until end of test.
- Failure mode(s).

The following additional information shall also be reported:

- MBJS type and Manufacturer.
- Photographs of the specimen, the test set-up with the specimen, and the failure modes.
- Drawings showing shape, size, and dimensions of the specimen along with the connections of the actuator and vibrator and their positions.
- Section properties and detail dimensions of the centerbeam and support bars.
- Manufacturer(s), position, and material properties, including ASTM or other testing agency specifications and values, of elastomeric components, including but not limited to springs, bearings, equidistant devices, and seals.
- Fatigue design calculations submitted as required in Article 14.5.6.9.2, "Contract-Document Requirements," of the *AASHTO LRFD Bridge Design Specifications*.

5.2 Seal Push-Out (SPO) Test

5.2.1 Apparatus

5.2.1.1 Jack or actuator to open and close MBJS. Loads may be applied to the seals by any device capable of a delivering a minimum load of 22.2 kN (5.0 kip) in displacement control at a controlled stroke rate while measuring the applied load. A pressure gage may be used to monitor the load if the pressure can be correlated accurately to force. A load cell is also an acceptable method of measuring the load. Load shall be applied to the seals through a 25.4-mm (1.0-in.) diameter cylindrical steel bar 533.4 mm (21.0 in.) in length. The ends of the cylindrical bar shall be tapered to prevent tearing of the seal.

5.2.1.2 Test fixture to restrain the specimen. A stiff self-reacting test frame capable of resisting the vertical push-out force shall be provided. The test frame may be clamped to the edgebeams of the specimen as shown in Figure 5. The seals shall be opened to 75 ± 10 mm (3.0 ± 0.4 in.) while performing this test.

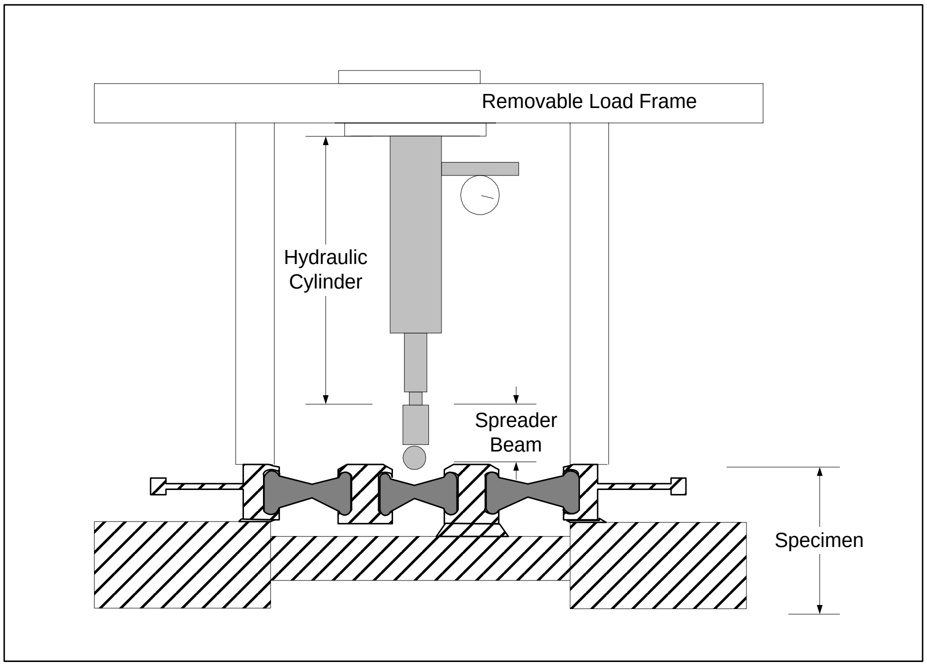


Figure 5—Recommended Seal Push-Out Set-Up

5.2.2 Sampling

A minimum of five consecutive SPO tests shall be performed on each specimen.

Successful completion of fatigue tests described in Section 5.3, and passing both the OMV test described in Section 5.1 and this SPO test prequalifies a specific configuration MBJS with specific components, with the variations in configuration allowed as specified in Section 5.1.2.

5.2.3 Preparation of Test Specimen

This test shall be performed on the same specimen as the OMV test, described in Section 5.1, after it has successfully passed the OMV test, with the intent that the results of the SPO test reflect any damage or loosening to the seal or its retainers that might occur in the full duration of the OMV test.

In the event that an MBJS is successfully prequalified and then the seal Manufacturer or seal profile changes in the future but there are no other changes in the overall MBJS, the SPO test may be conducted on a new specimen without repeating the OMV test. The specimen for this SPO retest shall consist of 1-m (40.0-in.) long sections similar in cross-section to the full-size specimens used for the OMV tests. If an acceptable result is obtained in such an SPO retest, the MBJS shall be considered to continue to be qualified. Note: In cases in which the SPO test following OMV testing fails, both the OMV test and SPO test must be performed on the specimen again.

5.2.4 Procedures

5.2.4.1 General

Prior to testing, specimens shall be visually inspected for any flaws, loose fasteners, etc. that could possibly affect the performance of the specimen. Any observed problem shall also be reported with the data.

5.2.4.2 Application of Load

The displacement shall be applied perpendicular to the plane of the centerbeams. The load shall be applied in displacement control at a stroke rate of 10 mm/s (0.4 in./s).

5.2.4.3 Definition of Failure

If any of the tests for a particular specimen fail to reach a minimum force of 6000 N (1350 lb), it shall be considered a failure. If more than one of the five consecutive tests fails to reach this load, the MBJS shall be considered to have failed the prequalification tests and both the OMV test and the SPO test must be repeated on the improved design. In the event that only one of the five consecutive tests fails to reach the minimum requirements that one test may be discarded and replaced by three new consecutive tests using the original specimen. All three of the replacement tests must reach the minimum force level for the MBJS to be considered to have successfully passed the prequalification tests.

5.2.5 Report

Data shall be reported in tabular format and shall include the following information:

- Manufacturer of MBJS and supplier of seals (if different),
- type of seal (box or strip),
- lubricant-adhesive use, Manufacturer of lubricant-adhesive, and chemical formulation and material properties of lubricant-adhesive,
- load at failure,
- description of failure mode,
- length of failure, and
- a drawing showing the loading rod, the centerbeam, edgebeam, seal cross-sections, and connection method.

5.3 Fatigue Test

This specification describes a fatigue-test procedure for the primary load-carrying metal components of MBJS. This test procedure shall be used to establish the fatigue resistance of the centerbeam/support bar (CB/SB) connection and any field-splice details. The procedure is based on the nominal-stress S-N-curve approach and is consistent with current *AASHTO LRFD Bridge Design Specifications*. Full-scale testing of representative details is required. Data reported include the applied nominal-stress range (S_r) and the number of cycles (N) at a predetermined extent of crack propagation (defined as failure) or test termination. The test results are used to establish the appropriate fatigue category of the CB/SB connection or the field splice for use in fatigue design. The procedure can also be used to establish the fatigue resistance of these details in terms of any other nominal-stress-based S-N curves.

The CB/SB connection and the field splice are the primary focus of this test. The tests to establish the fatigue resistance of the field-splice details may be conducted separately from the tests to establish the fatigue resistance of the CB/SB connection. Some bolted splice details are designed to act as a hinge and, in this case, the static test described in Section 5.3.4.2 shall verify that the splice does not transfer vertical bending moment. Provided the bolted hinge splice does not crack during the course of the test, the splice need not be designed explicitly for fatigue. If a field-splice detail has been previously qualified in another test with a similar centerbeam, it need not be tested for each type of CB/SB connection.

The specimens to establish the fatigue resistance of the CB/SB connection shall include any shop-splice details, attachments, or other details (other than the field splice) that are to be used in this MBJS type in service. If these details, other than the CB/SB connection, do not crack during the course of this test, the fatigue resistance of these details may be inferred from the fatigue design specifications in Article 6.6.1, “Fatigue,” of the *AASHTO LRFD Bridge Design Specifications*.

If there are any details that are more susceptible to fatigue than the CB/SB connection, this test will yield data for the fatigue resistance of these details as well. If there are other details that crack prior to cracking of the CB/SB connection, these details may have to be modified in order to obtain data on the CB/SB connection. Typically, the MBJS is not efficiently designed if details other than the CB/SB connection are governing the fatigue life.

5.3.1 Apparatus

5.3.1.1 Controller to control the frequency and count the cycles applied

5.3.1.2 Actuator to apply axle loads with load control. Vertical loads and horizontal loads equal to 20 percent of the vertical loads shall be applied. This may be accomplished by holding the specimen in the fixture at an angle 11.3 degrees off of the horizontal plane and applying load through actuators oriented in the vertical plane. Load shall be applied through two 250-mm (10.0-in.) long patches, typically steel plates with hard rubber bearing pads, placed in contact with the centerbeam. Each patch shall be located in the center of each outer span and need not be 1.8 m (6.0 ft) apart. Load may be supplied through one actuator and a spreader beam or through two actuators. Figure 6 shows a typical fixture to deliver the vertical and horizontal loads (*P*) to the inclined specimen. The specimen may be tested in the normal configuration with the centerbeams on top of the support bars or upside down, with the centerbeam below the support bars. In either case, the load fixture may be designed so that the applied load vector intersects the centerline of the centerbeam cross-section somewhere within the cross-section; i.e., the vector need not be applied to the centerline at the top surface. In the normal configuration, the loads shall be entirely upward. In the upside-down configuration, the loads shall be entirely in compression. This specified loading creates a tensile force reaction in the centerbeam/support bar connection to avoid long crack propagation lives that result in greater variability.

5.3.1.3 Test fixture to restrain the specimen. Fixtures shall be capable of adequately supporting and securing the specimen during test. To provide vertical and horizontal loading through one actuator, the fixture will typically hold the specimen at an angle 11.3 degrees off of the horizontal plane (with a slope of 20 percent). The fixture shall be fabricated to sufficient tolerance so that additional stresses are not generated in the specimen as a result of fixture misalignment.

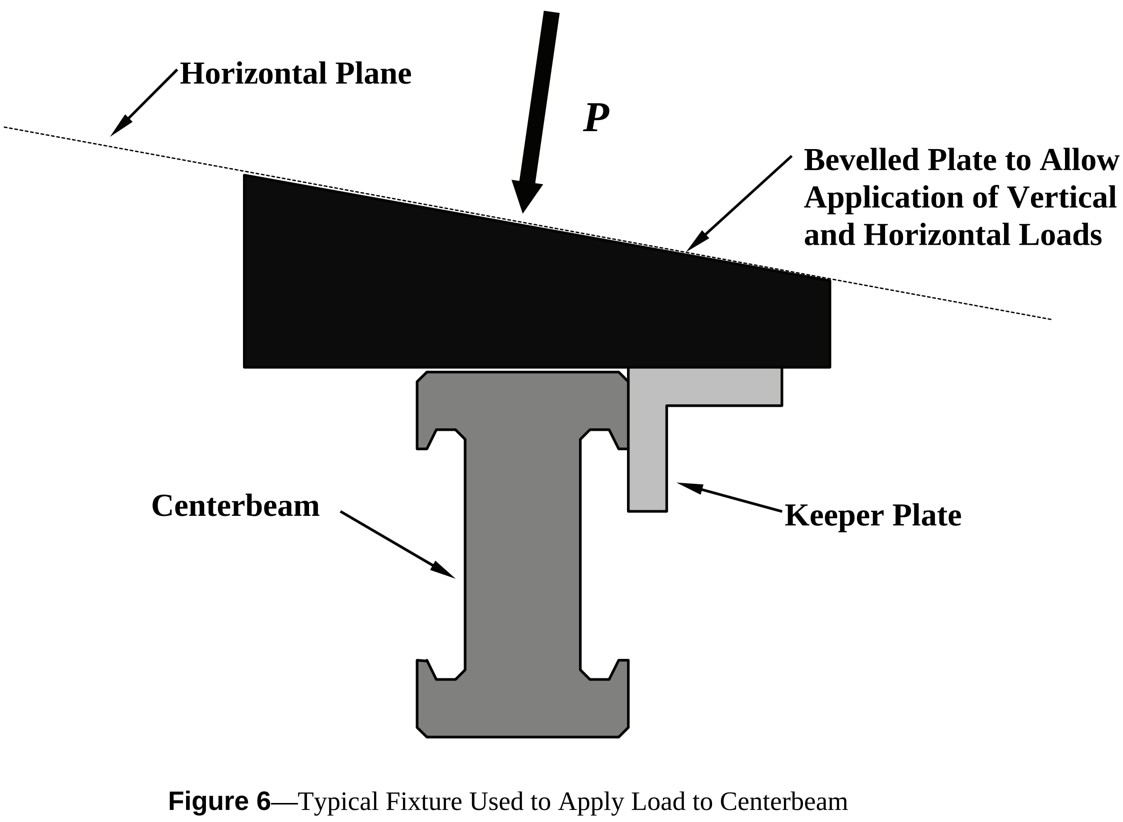


Figure 6—Typical Fixture Used to Apply Load to Centerbeam

5.3.2 Sampling

Ten reportable fatigue cracks (data) in separate connections must be acquired for each detail. This will usually require at least three specimens. Reportable data should be in the very long life range, i.e., as close to the threshold as practical but in no case less than 200,000 cycles. The lower bound S-N curve to the data is used for design. Reportable data will consist of the number of cycles and nominal stress range at the detail either at the occurrence of a crack (criteria for the size of the crack are given later) or, if no crack occurs, after the number of cycles exceeds the S-N curve that is anticipated to be the lower bound to the data by at least ten percent. The data point when no crack occurs is called a run-out.

More than ten data may be obtained, but none of these data may be ignored in the data analysis except as allowed in the following. In the event that all but one of ten or more data points fall above a given lower-bound S-N curve, that one data point may be discarded and replaced by three new data obtained through additional testing. The additional testing shall be conducted at the same stress range at which the discarded data point was obtained. These three additional data shall be plotted along with the remaining nine or more data. None of the additional data acquired shall be discarded.

5.3.3 Preparation of Test Specimen

5.3.3.1 Metal Components—General

Specimens shall be full-scale centerbeam/support bar assemblies representative of those to be used in the field applications. Each specimen shall consist of at least two continuous centerbeam spans over at least three equally spaced support bars. Spans between centerbeams shall be a minimum of 910 mm (36.0 in.) and a maximum of 1370 mm (54.0 in.). Support-bar sections and spans shall be chosen to give a bending stress range in the support bar within ±40 percent of the combined bending stress range in the centerbeam. All miscellaneous welded or bolted attachments, e.g., the welded attachments used to secure the elastomeric seals to the centerbeam, shall also be included on the subassembly specimens. Seals need not be provided with the specimens, however.

Two types of specimens are permitted, multiple-centerbeam assemblies and subassemblies. Multiple-centerbeam assemblies are required for single-support-bar MBJS while multiple-support-bar MBJS may be tested with either type of specimen.

The multiple-centerbeam assemblies shall consist of at least three centerbeams attached to the support bars. Edgebeams and support boxes may be included but are not required. If there are elastomeric springs and bearings in a yoke or stirrup attachment to the support bar as in most single-support-bar MBJS, they may be damaged by the relatively high, unrealistic loads in the fatigue test. Since the performance of these components is tested in the opening movement and vibration test described in Section 5.1, they may be replaced in the fatigue test specimen with steel discs or rectangular blocks of the same dimensions (within 3 mm (0.125 in.)). If support boxes are included and there are springs or bearings in the support boxes, these may also be replaced with steel discs or rectangular blocks of similar dimensions. Equidistant device attachments must be provided but equidistant devices may be replaced with steel blocks of similar dimensions. If required, shims may be placed between the centerbeams at the support-bar locations only to prevent rotation of the centerbeams under load.

Figure 7 illustrates a typical welded multiple-support-bar subassembly specimen. The centerbeam/support bar connection shall be located at midspan of each support bar. Any welded or bolted attachments used to secure equidistant devices to a support bar, centerbeam, or stirrup shall be included on the specimen.

Support bars of subassembly specimens that are components of a single-support-bar swivel-joist type MBJS shall be orientated perpendicular to the longitudinal axis of the centerbeam.

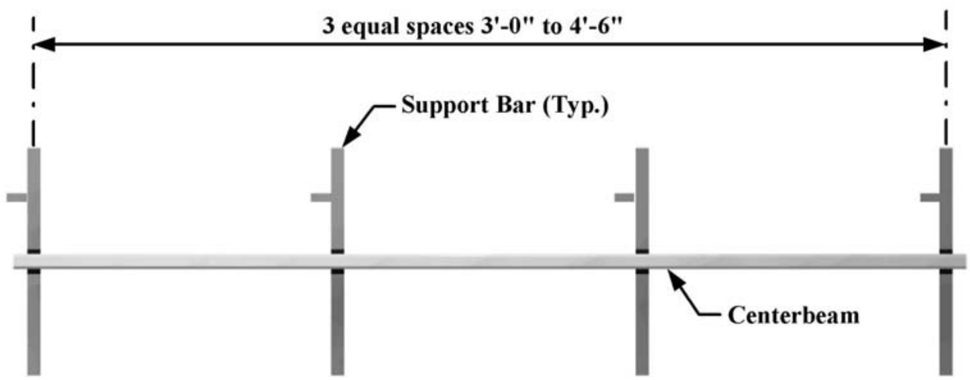


Figure 7—Typical Subassembly Specimen

5.3.3.2 Instrumentation

Each specimen shall be sufficiently instrumented to measure static nominal strain ranges in the specimen for a specific interval of loading. Best results can usually be obtained when the loading interval for the static calibration tests does not pass through zero load. Strain measurements shall be made at locations not influenced by local effects (i.e., away from weld toes or boltholes). As a minimum, specimens shall be instrumented as illustrated in Figures 8a and 8b.

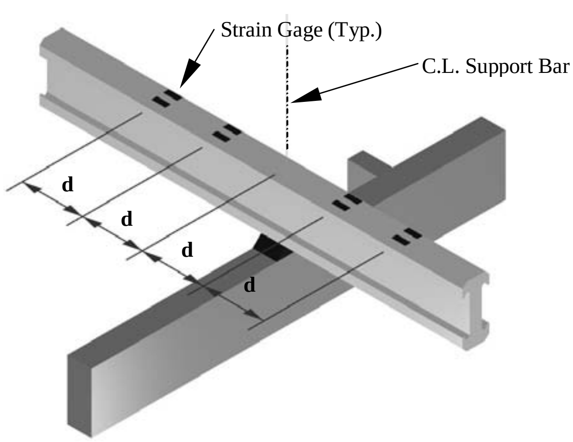


Figure 8a—Strain-Gage Layout at Interior Centerbeam/Support-Bar Connection (Exterior Connection Similar)

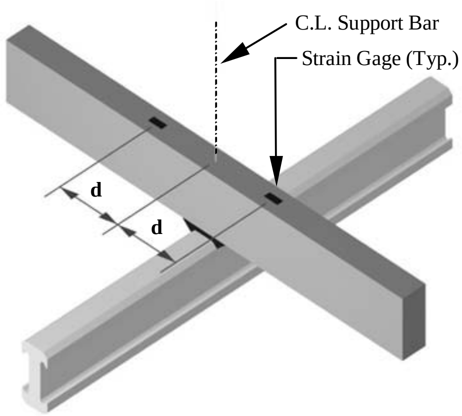


Figure 8b—Typical Support-Bar Strain-Gage Layout (Bottom View)

5.3.4 Procedures

5.3.4.1 Preliminary inspection

Prior to testing, specimens shall be visually inspected for any flaws, defects, loose fasteners, etc. that could possibly affect the fatigue resistance of the detail under consideration. Defects and flaws shall be defined as per the appropriate governing Specification. Data obtained from specimens containing defects shall not be excluded from the data set. Any observed defect shall also be reported with the data.

5.3.4.2 Static Test

A static calibration test shall be conducted in order to obtain strain range measurements required to verify the structural analysis model. Analyzing the centerbeam as a continuous beam over rigid supports has been found to give good agreement with measured strains for loads in the vertical direction. For loads in the horizontal direction, the continuous-beam model is conservative. For the loads in the horizontal direction, more accurate results can be achieved by treating the centerbeams and support bars as a coplanar frame pinned at the ends of the support bars.

In order to assure sufficient seating of the specimen in the fixture, a minimum of 44 kN (10.0 kip) shall be applied at each patch load. (This requirement shall be waived for tests of single-support-bar systems conducted with load reversal.) Once this load is applied, all strain-measuring devices shall be rebalanced to zero strain while the preload is maintained. An additional load, which is approximately equal to the desired fatigue-load range, shall be applied. Strain ranges shall be measured for the load range from 44 kN (10.0 kip) to the peak load. Each static calibration test shall be repeated three times while still maintaining a minimum 44 kN (10.0 kip) load at each load patch throughout each. The measured strain ranges from each repetition should vary by no more than 25 percent from the mean value. In the event the strain ranges are not repeatable, appropriate modifications to the fixture shall be made until the strain ranges are repeatable. Once it is established that the data are repeatable, the measured strain range shall be compared to the strain range calculated using the structural analysis model.

The structural analysis model shall be considered verified when predicted strain ranges are within ±25 percent of the measured strain ranges at every location. Upon verification, the results from the structural analysis model shall be used in preference to the measurements to determine the nominal stress ranges at the details of interest for reporting results.

5.3.4.3 Fatigue Loading

The magnitude of the vertical load range (ΔP_v) shall be maintained throughout the entire test (i.e., "load control"). Load ranges shall be monitored continuously throughout the test to ensure that the desired load range is maintained. Both vertical and horizontal load ranges shall be applied to the test specimen simultaneously in the following proportions:

- Vertical Load Range = ΔP_v
- Horizontal Load Range = $0.2(\Delta P_v)$

The load ranges used in the test shall not be so large as to alter the observed failure mode from that which would be observed under service conditions. Under no circumstance shall applied stresses exceed the yield stress of the material in any portion of the specimen. Testing shall be performed at a minimum of two different load (stress) ranges.

- Multiple-Support-Bar Systems

  The direction of loading shall be either all tension or all compression and shall be applied at a constant amplitude at any desired frequency. The applied load range shall be in a direction such that the reaction force between the centerbeam and support bar is tensile. The load range shall not pass through the zero-load position and a minimum preload shall be maintained throughout the test.

- Single-Support-Bar Systems

  Single-support-bar systems may be loaded using the same procedures as specified above for multiple-support-bar systems. In the event that premature failure of the stirrup occurs, a load range may be used which is 70 percent downward and 30 percent uplift.

5.3.4.4 Definition of Failure

For welded or bolted MBJS specimens, the entire CB/SB connection shall be considered as a single detail. For example, cracks occurring at a welded centerbeam/support bar connection may occur either: 1) at the centerbeam weld toe, 2) at the support-bar weld toe, or 3) through the weld throat. In any case, this cracking defines failure of the CB/SB connection (i.e., detail).

The following criteria shall be used to define failure of a given centerbeam/support bar connection.

- Welded Centerbeam/Support Bar Connections

  Occurrence of fatigue cracks or fractures specified below shall be considered as sufficient reason to declare a connection as failed.

  - Type A Cracking

    Type A cracking originates at or near the centerbeam weld toe, propagates up into the centerbeam at some angle, and grows back over the connection. Typically, Type A cracks grow at an angle of about 45 degrees. A specimen shall be considered as failed due to Type A cracking when a crack has grown on any vertical face a length of $d/2$ from point of origin, where d is the depth of the centerbeam.

  - Type B Cracking

    Type B cracking originates at or near the support-bar weld toe, propagates down into the support bar and grows back under the connection at some angle, typically about an angle of 45 degrees. A specimen shall be considered as failed due to Type B cracking when a crack has reached a length of $d/2$ from point of origin on any vertical support-bar face, where d is the depth of the support bar.

  - Type C Cracking

    Type C cracking originates in the weld throat and typically grows in a plane parallel to the longitudinal axis of the support bar at about mid-depth of the weld throat. A specimen shall be considered as failed due to Type C cracking when a complete fracture of the weld throat has occurred. Type C cracks have been observed to turn down into the support bar, but only after significant growth. In such instances, the criteria for Type B cracking shall be applied.

- Welded Stirrup Connections

The following criteria shall be used to define failure of a given CB/SB connection that utilizes welded stirrups:

1. Fatigue cracks that result in complete fracture of any leg of a stirrup.
2. Fatigue cracks originating at or near a stirrup weld toe that have grown a distance of $d/2$ into the centerbeam on any face, where d is the depth of the centerbeam.

- Bolted Centerbeam/Support Bar Connections

The following criteria shall be used to define failure of a given bolted centerbeam support-bar connection:

1. Fatigue cracks that have grown out of a bolthole resulting in complete fracture of the tension flange in the centerbeam.
2. Fatigue cracks growing out of a bolthole and extending a distance of $\frac{d}{2} - t_f$ into the centerbeam web on any face, where d is the depth of the centerbeam and t_f is the flange thickness.
3. Complete fracture of any portion of a stirrup.
4. Complete fracture of any single bolt.

- Alternate Criteria for Termination of a Fatigue Test—Optional

A test may also be terminated when, for a given stress range, the specimen has survived the number of cycles required to plot the data above a particular fatigue-resistance curve.

5.3.4.5 Continuing Test after Cracking

- Subassemblies

Several data may be obtained from a single subassembly. If, before a crack has reached the defined length at failure, another crack has formed in the other connection of that span, the loading in that span may be continued until the second crack has also reached the defined length at failure. The effect of the first crack influencing the stress range for the second crack need not be taken into account.

If one of the connections in a loaded span has cracked and the crack has grown to failure, but the other connection has not yet cracked, the cracked connection may be weld-repaired and the loading then continued. The weld repair must have minimal effects on the proportion and orientation of stress ranges at other uncracked details. If a detail that has been repaired cracks again, this shall not be included as additional data.

In a two-span specimen, once the center connection cracks and the crack reaches the defined length at failure, either the crack shall be weld-repaired or the subassembly test specimen shall not be further tested.

When a crack has occurred and grown to failure in one span and two actuators are being used to apply the load, the actuator loading the span with the crack may be shut off and the loading can be continued with the second actuator in the uncracked span. In this case, the loading in the remaining span shall be adjusted to keep the stress range in the uncracked connections approximately constant.

- Multiple-Centerbeam Assemblies

Several data may be obtained from each centerbeam of a multiple-centerbeam assembly. The requirements for each centerbeam are the same as those specified above for a subassembly. When one centerbeam cannot be tested any further, the fixture may be shifted and the testing may continue on another centerbeam. However, if one of the support bars cracks and the crack grows to the defined length at failure, either the support bar shall be replaced or the specimen shall not be further tested.

5.3.5 Reporting

5.3.5.1 Nominal Stress Range

The nominal stress ranges (S_r) at all fatigue-critical details shall be obtained from structural analyses of the modular joint system due to the simultaneous application of vertical and horizontal loads. The nominal stress ranges (S_r) shall be calculated as follows for specific types of MBJS:

- Single-Support-Bar Systems
  - Centerbeam: The nominal stress range (S_r) in the centerbeam at a critical section adjacent to a welded or bolted stirrup shall be the sum of the stress ranges in the centerbeam resulting from horizontal and vertical bending at the critical section. The effects of stresses in any load-bearing attachments such as the stirrup or yoke need not be considered when calculating the stress range in the centerbeam. For bolted single-support-bar systems, stress ranges shall be calculated on the net section.
  - Stirrup: The nominal stress range (S_r) in the stirrup or yoke shall consider the force effects of the vertical reaction force range between the centerbeam and support bar. The stress range shall be calculated by assuming a load range in the stirrup that is equal to the tensile part of the applied vertical load range. The calculation of the nominal stress range in the stirrup or yoke need not consider the effects of stresses in the centerbeam. The effects of horizontal loads may be neglected in calculating the nominal stress range in the stirrup.

- Welded Multiple-Support-Bar Systems
  - Centerbeam Weld-Toe Cracking (Type A Cracking): The nominal stress range (S_r) for Type A cracking shall include the concurrent effects of vertical and horizontal bending stress ranges in the centerbeam (S_{rB}) and the vertical stress ranges in the top of the weld (S_{rZ}) as shown in Figure 9. The nominal stress range (S_r) for Type A cracking shall be calculated from the following:

$$S_r = \sqrt{S_{rB}^{\;2} + S_{rZ}^{\;2}} \tag{1}$$

where: S_{rB} = combined bending stress range in the centerbeam:

$$S_{rB} = \frac{M_V}{S_{Xcb}} + \frac{M_H}{S_{Ycb}} \tag{2}$$

where: M_V = vertical bending moment range in the centerbeam on the critical section located at the weld toe due to the vertical force range,

M_H = horizontal bending moment range in the centerbeam on the critical section located at the weld toe due to horizontal force range,

S_{Xcb} = vertical section modulus to the bottom of the centerbeam, and

S_{Ycb} = horizontal section modulus of the centerbeam.

S_{rZ} = vertical stress range in the top of the centerbeam/support bar weld from the concurrent reaction of the support beam:

$$S_{rZ} = \frac{M_{OT}}{S_{Wtop}} + \frac{R_V}{A_{Wtop}} \tag{3}$$

where: R_V = vertical reaction force range in the connection,
M_{OT} = $R_H\, d_{cb}$,
R_H = horizontal reaction force range in the connection,
d_{cb} = distance from the point of intersection of the load vector with the vertical centerline of the centerbeam to the bottom of the centerbeam,
S_{Wtop} = section modulus of the weld at the top for bending in the direction normal to the centerbeam axis, and
A_{Wtop} = area of weld at the top.

- Support-Bar Weld-Toe Cracking (Type B Cracking): The nominal stress range (Δf) for Type B cracking shall include the concurrent effects of vertical bending stress ranges in the support bar (S_{rB}) and the vertical stress ranges in bottom of the weld (S_{rZ}) as shown in Figure 10. The nominal stress range (S_r) for Type B cracking shall be calculated from the following:

$$S_r = \sqrt{S_{rB}^{\;2} + S_{rZ}^{\;2}} \tag{4}$$

where: S_{rB} = bending stress range in the support bar due to maximum moment including moment from vertical reaction and overturning at the connection:

$$S_{rB} = \frac{M_V}{S_{Xsb}} + \frac{1}{2}\,\frac{R_H\left[d_{cb} + h_W + \left(\frac{d_{sb}}{2}\right)\right]}{S_{Xsb}} \tag{5}$$

where: M_V = component of vertical bending moment range in the support bar due to the vertical reaction force range in the connection located on the critical section at the weld toe,
S_{Xsb} = vertical section modulus of the support bar to the top of the support bar, and
h_w = height of the weld,
d_{sb} = depth of support bar,
S_{rZ} = vertical stress range in the bottom of the centerbeam/support bar weld from the vertical and horizontal reaction force ranges in the connection:

$$S_{rZ} = \frac{R_H\,(d_{cb} + h_W)}{S_{Wbot}} + \frac{R_V}{A_{wbot}} \tag{6}$$

where: S_{Wbot} = section modulus of the weld at the bottom for bending in the direction of the support-bar axis and
A_{Wbot} = area of weld at the bottom.

- Cracking through the Throat of the Weld (Type C Cracking): The nominal stress range (S_r) for Type C cracking is the vertical stress range (S_{rZ}) at the most narrow cross-section of the centerbeam/support bar weld from the vertical and horizontal reaction force ranges in the connection, as shown in Figure 11. The nominal stress range (S_r) for Type C cracking shall be calculated from the following:

$$S_r = \frac{R_V}{A_{W_{mid}}} + \frac{R_H\left[d_{cb} + \left(\frac{h_w}{2}\right)\right]}{S_{W_{mid}}} \tag{7}$$

where:

S_{Wmid} = section modulus of the weld at the most narrow cross-section for bending in the direction normal to the centerbeam axis and

A_{Wmid} = minimum cross-sectional area of weld.

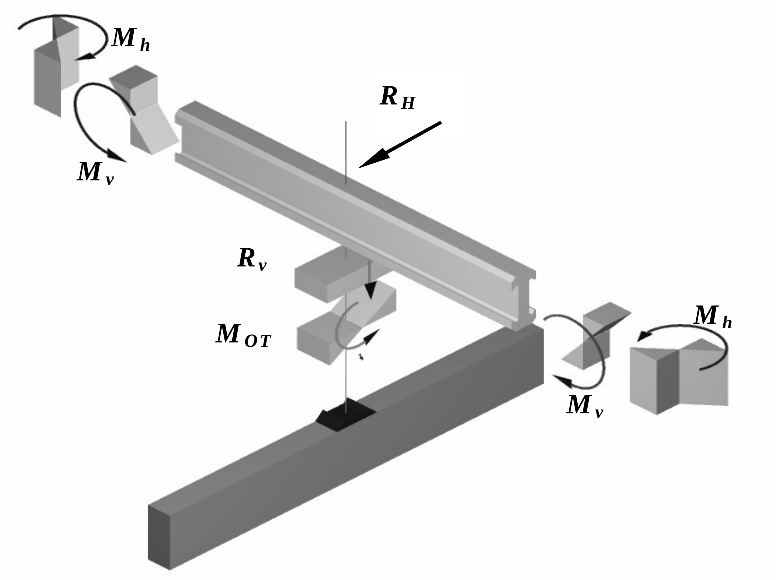


Figure 9—Force Effects Associated with Type A Cracking

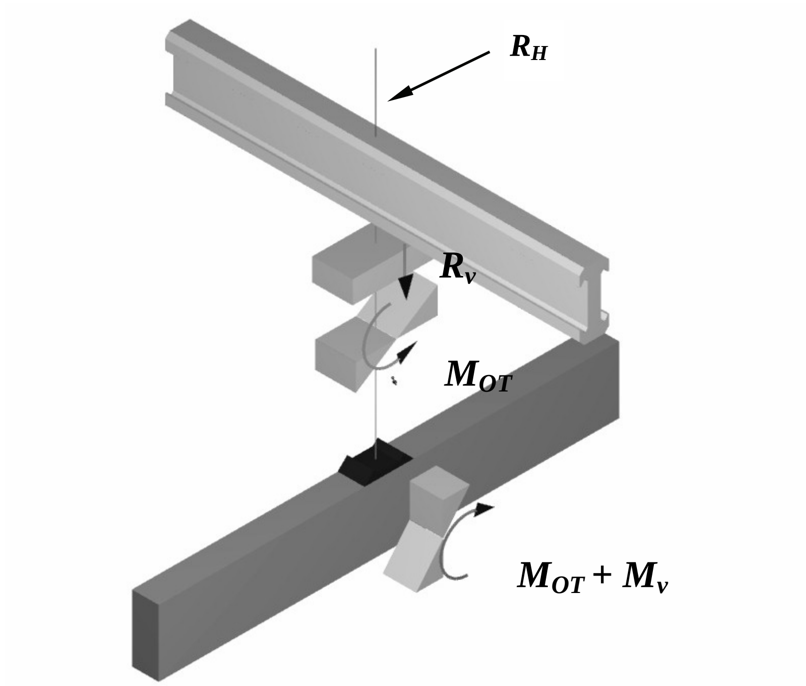


Figure 10—Force Effects Associated with Type B Cracking

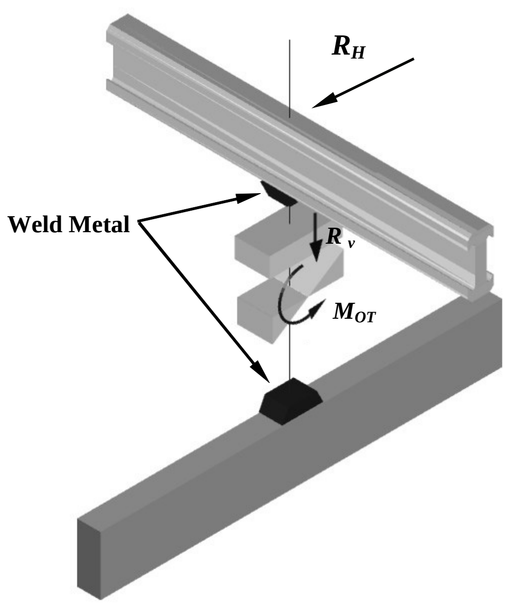


Figure 11—Force Effects Associated with Type C Cracking

5.3.5.2 S-N Curve

A minimum of ten nominal stress range/number of cycles data are required to establish the fatigue resistance for each detail under consideration. All data must be included in the S-N curve analysis. The entire CB/SB connection shall be considered as a single detail, so data from Type A, B, or C may be grouped together for the purposes of defining the fatigue resistance of that detail.

The cycles to failure (N) and the applied constant-amplitude nominal stress range (S_r) data shall be plotted on a log-log scale together with several standard S-N curves. The fatigue category shall be the category associated with the highest S-N curve that represents a lower bound to the ten data. The equation for the AASHTO S-N curves is as follows:

$$N = A /(S_r)^3 \tag{8}$$

where: N = Number of cycles,
S_r = Nominal stress range at the detail under consideration, MPa (ksi), and
A = Detail constant* for a particular detail category, 1×10^{11} $(\text{MPa})^3$ (1×10^8 $(\text{ksi})^3$)

* See Table 6.6.1.2.5-1 Detail Category Constant, A, in the *AASHTO LRFD Bridge Design Specifications.*

Since it is possible with so few fatigue-test data to get an overly optimistic result, maximum fatigue-resistance categories have been established through prior testing. The maximum fatigue resistance of any detail shall not exceed that prescribed in Table 1 below.

Type of Detail	Maximum Permitted Category [1]
Welded Multiple-CB/SB Connections	C
Welded Stirrup Attachments for SSB Systems	B
Bolted Stirrup Attachments for SSB Systems	D
Groove-Welded Centerbeam Splices[2]	C
Miscellaneous Welded Connections[3]	C
Miscellaneous Bolted Connections	D

Notes:

1. The maximum permitted category applies only to the S-N curve at stress ranges above the constant-amplitude fatigue limit (CAFL). A CAFL that is higher than the CAFL associated with these categories may be used if the CAFL is established with a minimum of ten test data.
2. Groove-welded, full-penetration splices may be increased to category B if weld soundness is established by NDT.
3. Miscellaneous connections include attachments for equidistant devices.

Table 1—Maximum Permitted Fatigue Category for MBJS Details

Alternatively, if the data are all run-outs, the fatigue category shall be the category associated with the largest constant-amplitude fatigue threshold that is less than the stress range associated with at least two run-out data. The constant-amplitude fatigue thresholds for the AASHTO fatigue categories are shown in Table 6.6.1.2.5-3 in the *AASHTO LRFD Bridge Design Specifications*. In this case, all the data used to infer the constant-amplitude fatigue threshold must be within 28 MPa (4.0 ksi) of the constant-amplitude fatigue threshold. There is no maximum fatigue-resistance category when the constant-amplitude fatigue threshold has been established.

5.3.5.3 Summary

In addition, the data shall also be reported in a table including the following information:

- Nominal stress range at detail of interest (S_r).

- Applied load range for each patch.
- Number of cycles at first observation of a crack (for report purpose only, not to be used in plotting the S-N data).
- Number of cycles at failure or termination of the test (*N*). The reason for stopping the test shall also be noted (i.e., defined crack length at failure reached, run-out, or termination).
- Type of crack as described in Section 5.3.4.4. If the observed crack does not resemble any of the crack types described in Section 5.3.4.4, then a detailed description of the fatigue crack shall be provided.

The following additional information shall also be reported:

- Joint type and Manufacturer.
- A drawing showing shape, size, and dimensions of the specimen. Drawings shall also present details of the fixture including specimen orientation.
- Photographs of the test specimen in the test fixture and photographs of any cracks. Cracks may be enhanced with dye penetrant.
- Section properties and detail dimensions of the centerbeam and support bars.
- For welded MBJS, the weld procedures specification shall be attached. For bolted MBJS, the bolt size, material specification, location, and method of tightening shall be reported.

6. KEYWORDS

Deck joint, expansion joint, durability, seal, fatigue

TABLE OF CONTENTS

SECTION 20

RAILINGS

20.1—GENERAL

20.1.1—Description

This work shall consist of furnishing all materials and constructing railings on structures. Railings constructed at each location shall conform to the type and details specified in the contract documents for that location. The work shall include the furnishing and placing of mortar or concrete, anchor bolts, reinforcing steel dowels, or other devices used to attach the railing to the structure.

C20.1.1

The types of railings included in this work consist of hand railings; pedestrian railings; traffic railings, which are sometimes called barriers; and railings for other such purposes.

20.1.2—Materials

All materials not otherwise specified in the contract documents shall conform to the requirements of the applicable AASHTO material specifications.

20.1.3—Construction

Unless otherwise permitted by the Engineer, railing shall not be placed until the centering or falsework for the span has been released, rendering the span self-supporting.

20.1.4—Line and Grade

The line and grade of the railing shall be true to that specified in the contract documents and may include an allowance for camber in each span but shall not follow any unevenness in the superstructure. Unless otherwise specified in the contract documents, railings on bridges, whether super-elevated or not, shall be vertical.

20.2—METAL RAILING

20.2.1—Materials and Fabrication

20.2.1.1—Steel Railing

Materials and fabrication of steel railings shall conform to the applicable requirements of Section 11, “Steel Structures,” except that formed sections may be fabricated from mild steel and pipe sections shall be of standard steel pipe. Nuts and bolts not designated as high strength shall conform to the requirements of ASTM A307, and steel tubing shall conform to the requirements of ASTM A500, Grade B.

20.2.1.2—Aluminum Railing

For aluminum railings or portions of railings, cast aluminum posts shall conform to the requirements of AASHTO M 193 and extruded components shall conform to the requirements of ASTM B221.

C20.2.1.2

AASHTO M 193 was discontinued in 1998 but is still a valid specification for cast aluminum alloy railing posts.

20.2.1.3—Metal Beam Railing

Metal beam rail, posts, and hardware shall conform to the requirements in Section 606 of the *AASHTO Guide Specification for Highway Construction,* Section 606.

C20.2.1.3

Metal beam railing requirements refer to Section 606 of the *AASHTO Guide Specification for Highway Construction*.

20.2.1.4—Welding

All exposed welds shall be finished by grinding or filing to give a smooth surface. Welding of aluminum materials shall be done by an inert gas shielded, electric arc welding process using no welding flux. Torch- or flame-cutting of aluminum will not be permitted.

20.2.2—Installation

Metal railings shall be carefully adjusted prior to fixing in place to ensure proper matching at abutting joints, correct alignment, and camber throughout their length. Holes for field connections shall be drilled with the railing in place on the structure at proper grade and alignment.

Where aluminum alloys come in contact with other metals or concrete, the contacting surfaces shall be thoroughly coated with a dielectric aluminum-impregnated caulking compound or a synthetic rubber gasket may be placed between the two surfaces.

20.2.3—Finish

Unless otherwise specified in the contract documents, anchor bolts, nuts, and all steel portions of railings shall be galvanized and aluminum portions shall be unpainted. Galvanizing of rail element shall conform to the requirements of AASHTO M 111M/M 111 (ASTM A123/A123M) and galvanizing of nuts and bolts shall conform to the requirements of AASHTO M 232M/M 232 (ASTM A153/A153M). Minor abrasions to galvanized surfaces shall be repaired with zinc-rich paint. After erection, all sharp protrusions shall be removed and the railing cleaned of discoloring foreign materials.

When painting is specified in the contract documents, the type and coating shall conform to the requirements of Section 13, “Painting,” or the requirements specified in the contract documents.

20.3—CONCRETE RAILING

20.3.1—Materials and Construction

Concrete railings, depending on the design, may be constructed by the cast-in-place, the precast, or, when approved by the Engineer, the slip form method.

All materials and construction shall conform to the requirements in Section 8, “Concrete Structures,” and Section 9, “Reinforcing Steel.” Unless otherwise specified in the contract documents, concrete shall conform to Class AE, except that Class A may be used in areas where freezing seldom occurs. When the minimum thickness of the railing at any point is less than 4.0 in., Class C (AE) or, where freezing seldom occurs, Class C concrete may be used. Forms for cast-in-place railing shall not be removed until adequate measures to protect and cure the concrete are in place and the concrete has sufficient strength to prevent surface or other damage caused by form removal. Finish for railings constructed with fixed forms shall be Class 2—Rubbed Finish. Finish for railings constructed with slip forms and for temporary railings shall be Class 1—Ordinary Finish.

20.4—TIMBER RAILING

Unless otherwise specified in the contract documents, posts, rails, and other timber for wood railings shall be constructed according to the requirements of Section 16, "Timber Structures." When treated wood is called for, the preservative treatment shall conform to the requirements of Section 17, "Preservative Treatment of Wood." The surfaces of all elements of treated wooden railings that are located where contact with people could occur shall be sealed with two coats of an acceptable sealer.

C20.4

Acceptable sealers are urethane, shellac, latex epoxy, enamel, and varnish.

20.5—STONE AND BRICK RAILINGS

Stone and brick railings shall conform to the requirements of Section 14, "Stone Masonry," and Section 15, "Concrete Block and Brick Masonry."

20.6—TEMPORARY RAILING

Temporary railings shall be constructed of materials and to the details specified in the contract documents. Railings shall be properly joined and aligned at the required locations. Temporary precast barriers shall be installed on a solid base. The temporary railing shall be maintained in first-class condition and shall not be removed until all work requiring the railing has been completed. Previously used units may be employed provided they are in a clean and undamaged condition. After removal, temporary railing shall continue to be the property of the Contractor.

20.7—MEASUREMENT AND PAYMENT

20.7.1—Measurement

Railings shall be measured by the linear foot between the ends of the railing or the outside ends of end posts, whichever is greater. Measurements shall be made along the slope of the railing and no deductions shall be made for electrolier or other small openings called for in the contract documents.

20.7.2—Payment

Railings shall be paid for by the contract prices per linear foot for the various types listed in the contract documents. Such payment shall include full compensation for furnishing all labor, materials, equipment, and incidentals and for doing all work involved in constructing the railings or barriers complete in place, including the furnishing and installation of reinforcing steel and steel dowels or anchor bolts which are either placed or drilled and bonded into the structure for attachment of the railing.

20.8—REFERENCES

AASHTO. 2008. *AASHTO Guide Specification for Highway Construction*, Ninth Edition, GSH-9, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2009. S*tandard Specifications for Transportation Materials and Methods of Sampling and Testing*, 29th Edition, HM-29, American Association of State Highway and Transportation Officials, Washington, DC.

SECTION 21: WATERPROOFING

TABLE OF CONTENTS

SECTION 21

WATERPROOFING

21.1—GENERAL

This work shall consist of furnishing and installing materials to waterproof or dampproof concrete or masonry surfaces. The surfaces to be waterproofed or dampproofed and the type of system to be installed shall be as specified in the contract documents.

21.1.1—Waterproofing

Waterproofing shall consist of either a constructed-in-place asphalt membrane system or a preformed membrane system, both of which include appropriate priming materials and, when required, protective coverings. Unless a specific type of waterproofing system is specified in the contract documents, the type of system to be used will be at the option of the Contractor.

21.1.2—Dampproofing

Dampproofing shall consist of a coating of primer and two moppings of waterproofing asphalt.

21.2—MATERIALS

21.2.1—Asphalt Membrane Waterproofing System

21.2.1.1—Asphalt

Waterproofing asphalt shall conform to the Specification for Asphalt for Dampproofing and Waterproofing, ASTM D449. Type I shall be used below ground and Type II used above ground.

21.2.1.2—Primer

Primer for use with waterproofing asphalt shall conform to the Specification for Primer for Use with Asphalt in Dampproofing and Waterproofing, ASTM D41.

21.2.1.3—Fabric

The fabric shall conform to either the Specification for Woven Cotton Fabrics Saturated with Bituminous Substances for Use in Waterproofing, ASTM D173, or the Specifications for Woven Glass Fabric Treated with Asphalt, ASTM D3515.

The fabric shall be stored in a dry, protected place. The rolls shall not be stored on end.

21.2.2—Preformed Membrane Waterproofing Systems

21.2.2.1—Primer

Primer for use with the rubberized asphalt membrane shall be a neoprene-based material, and the primer for use with the modified bitumen membrane shall be a resin- or solvent-based material. Primers shall be of a type recommended by the Manufacturer.

21.2.2.2—Preformed Membrane Sheet

Preformed membrane sheet shall be of either the rubberized asphalt type or the modified bitumen type. The rubberized asphalt type shall consist of a rubberized asphalt sheet reinforced with a polyethylene film or mesh. The modified bitumen sheet type shall consist of a polymer modified bitumen sheet reinforced with a stitch-bonded polyester fabric or a fiberglass mesh. The membrane sheet shall conform to the following requirements in Tables 21.2.2.2-1 and 21.2.2.2-2.

Table 21.2.2.2-1—Preformed Membrane Sheet for Surfaces Other than Bridge Decks

		Value	
Property	Test	Rubberized Asphalt Type	Modified Bitumen Type
Tensile Strength in Machine Direction	ASTM D882	20 lb/in.	20 lb/in.
% Elongation at break in Machine Direction	ASTM D882	150% at 73.4°F ± 3.6°F	25% at 73.4°F ± 3.6°F
Pliability	ASTM D146	No cracks	No cracks
Thickness, minimum		60 mils	60 mils
Softening Point, minimum	ASTM D36	165°F	210°F

Table 21.2.2.2-2—Preformed Membrane Sheet for Bridge Deck Surfaces

		Value	
Property	Test	Rubberized Asphalt Type	Modified Bitumen Type
Tensile Strength in Machine Direction	ASTM D882	50 lb/in.	40 lb/in.
% Elongation at break in Machine Direction	ASTM D882	15% at 73.4°F ± 3.6°F	10% at 73.4°F ± 3.6°F
Pliability	ASTM D146	No cracks	No cracks
Thickness, minimum		65 mils	70 mils
Softening Point, minimum	ASTM D36	165°F	210°F

ASTM D882 shall be based on a Method A, 1.0-in. wide strip with 4.0-in. minimum initial grip separation and 4.0-in. gage length at 2.0 in. per minute. The acceptance criterion shall be based on an average of five samples.

ASTM D146 shall be based on a 180-degree bend over a 4.0-in. mandrel at 10°F.

21.2.2.3—Mastic

The mastic for use with preformed rubberized sheets shall be a rubberized asphalt cold-applied joint sealant. The mastic for use with modified bitumen sheet shall be a blend of bituminous and synthetic resins.

21.2.3—Protective Covers

Materials for protective covers shall conform to the following unless another type is specified in the contract documents.

For surfaces against which backfill will be placed, the protective cover shall consist of 0.125--in. hardboard or other material that will furnish equivalent protection from damage due to sharp coarse backfill material or from construction equipment.

For roadway surfaces of bridge decks, the protective cover shall consist of a layer of special asphalt concrete as specified in the contract documents.

For horizontal surfaces above which reinforced concrete structures are to be constructed, the protective cover shall consist of a 2.0-in. course of concrete mortar conforming to the requirements of Article 8.14, “Mortar and Grout,” except that the proportions shall consist of one part portland cement to three parts of fine aggregate. This mortar course shall be reinforced midway between its top and bottom surfaces with 6 × 6—W1.4 × W1.4 (150 × 150—MW9 × MW9) welded wire fabric, or its equivalent. The top surface shall be finished smooth and true to grade.

21.2.4—Dampproofing

The primer and asphalt used for dampproofing shall conform to that specified in Article 21.2.1, “Asphalt Membrane Waterproofing System.”

21.2.5—Inspection and Delivery

All waterproofing and dampproofing materials shall be tested before shipment. Unless otherwise ordered by the Engineer, they shall be tested at the place of manufacture, and, when so tested, a copy of the test results shall be sent to the Engineer by the chemist or inspection bureau that has been designated to make the tests, and each package shall have affixed to it a label, seal, or other mark of identification, showing that it has been tested and found acceptable, and identifying the package with the laboratory tests.

Factory inspection is preferred but, in lieu thereof, the Engineer may order that representative samples, properly identified, be sent to the Engineer for testing prior to shipment of the materials. After delivery of the materials, representative check samples shall be taken that shall determine the acceptability of the materials.

All materials shall be delivered to the work in original containers, plainly marked with the Manufacturer's brand or label.

21.3—SURFACE PREPARATION

All concrete surfaces that are to be waterproofed or dampproofed shall be reasonably smooth and free of foreign material that would prevent bond and of projections or holes which might cause puncture of the membrane or dampproofing. The surface shall be dry and, immediately before the application of the primer, the surface shall be thoroughly cleaned of dust and loose materials.

No waterproofing or dampproofing shall be done in wet weather, nor when the surface temperature is either below 35°F or below that recommended by the Manufacturer, unless approved by the Engineer in advance of the work. Should the surface of the concrete become temporarily damp, it shall be covered with a 2.0-in. layer of hot sand, which shall be allowed to remain in place from one to two hours, or long enough to produce a warm and surface-dried condition, after which the sand shall be swept back, uncovering sufficient surface for beginning work, and the operation repeated as the work progresses.

21.4—APPLICATION

Waterproofing shall not be applied to any surface until the Contractor is prepared to follow its application with the placing of the protective covering and backfill within a sufficiently short time so that the membrane will not be damaged by workers or equipment, exposure to weathering, or from any other cause. Damaged membrane or protective covering shall be repaired or replaced at the expense of the Contractor.

Care shall be taken to confine all materials to the areas to be waterproofed or dampproofed and to prevent disfigurement of any other parts of the structure by dripping or spreading of the primer or asphalt.

21.4.1—Asphalt Membrane Waterproofing

21.4.1.1—General

Asphalt membrane waterproofing shall consist of a coat of primer applied to the prepared surface and a firmly bonded membrane composed of two layers of saturated fabric and three moppings of waterproofing asphalt and, when required, a protective cover.

21.4.1.2—Installation

Asphalt shall be heated to a temperature between 300°F and 350°F. The heating kettles shall be equipped with thermometers.

In all cases, the waterproofing shall begin at the low point of the surface to be waterproofed, so that water will run over and not against or along the laps.

The first strip of fabric shall be of half-width; the second shall be full-width, lapped the full-width of the first sheet; and the third and each succeeding strip shall be full-width and lapped so that there will be two layers of fabric at all points with laps not less than 2.0 in. wide. All end laps shall be at least 12.0 in.

Beginning at the low point of the surface to be waterproofed, a coating of primer shall be applied and allowed to dry before the first coat of asphalt is applied. The waterproofing shall then be applied as follows.

Beginning at the low point of the surface to be waterproofed, a section about 20.0 in. wide and the full-length of the surface shall be mopped with the hot asphalt, and there shall be rolled into it, immediately following the mopping, the first strip of fabric, of half-width, which shall be carefully pressed into place so as to eliminate all air bubbles and obtain close conformity with the surface. This strip and an adjacent section of the surface of a width equal to slightly more than half of the width of the fabric being used shall then be mopped with hot asphalt, and a full width of the fabric shall be rolled into this, completely covering the first strip, and pressed into place as before. This second strip and an adjacent section of the concrete surface shall then be mopped with hot asphalt and the third strip of fabric “shingled” on so as to lap the first strip not less than 2.0 in. This process shall be continued with each strip of fabric lapping at least 2.0 in. over the second previous strip so that the entire surface is covered with at least two layers of fabric. The entire surface shall then be given a final mopping of hot asphalt.

The completed waterproofing shall be a firmly bonded membrane composed of two layers of fabric and three moppings of asphalt, together with a coating of primer. Under no circumstances shall one layer of fabric touch another layer at any point or touch the surface, as there must be at least three complete moppings of asphalt.

In all cases, the mopping on concrete shall cover the surface so that no gray spots appear, and on cloth it shall be sufficiently heavy to completely conceal the weave. On horizontal surfaces not less than 12 gal/100 ft^2 of asphalt shall be used for finished work, and on vertical surfaces, not less than 15 gal/100 ft^2 shall be used. The work shall be so regulated that, at the close of a day's work, all cloth that is laid shall have received the final mopping of asphalt. Special care shall be taken at all laps to see that they are thoroughly sealed.

21.4.1.3—Special Details

At the edges of the membrane and at any points where it is punctured by such appurtenances as drains or pipes, suitable provisions shall be made to prevent water from getting between the waterproofing and the waterproofed surface.

All flashing at curbs and against girders, spandrel walls, etc., shall be done with separate sheets lapping the main membrane not less than 12.0 in. Flashing shall be closely sealed either with a metal counter-flashing or by embedding the upper edges of the flashing in a groove poured full of joint filler.

Joints that are essentially open joints, but that are not designed to provide for expansion, shall first be caulked with oakum or other material approved by the Engineer, and then filled with hot joint filler.

Expansion joints, both horizontal and vertical, shall be provided with sheet copper or lead in "U" or "V" form in accordance with the contract documents. After the membrane has been placed, the joint shall be filled with hot joint filler. The membrane shall be carried continuously across all expansion joints.

At the ends of the structure, the membrane shall be carried well down on the abutments and suitable provision made for all movement.

21.4.1.4—Damage Patching

Care shall be taken to prevent injury to the finished membrane by the passage over it of workers or equipment, or by throwing any material on it. Any damage which may occur shall be repaired by patching. Patches shall extend at least 12.0 in. beyond the outermost damaged portion, and the second ply shall extend at least 3.0 in. beyond the first.

21.4.2—Preformed Membrane Waterproofing Systems

21.4.2.1—General

Preformed membrane waterproofing systems shall consist of a primer applied to the prepared surface, a single layer of adhering preformed membrane sheet and, when required, a protective cover.

21.4.2.2—Installation on Bridge Decks

Prior to applying the primer, an oil-resistant construction paper mask shall be taped or held with an adhesive to any deck areas that will later be covered by expansion dams or headers.

The membrane seal and asphalt concrete shall be placed continuously across such paper masks; however, the mask and the preformed sheet shall be cut at or near the expansion joint when ordered by the Engineer.

The neoprene-based primer shall be applied in one coat at a rate of approximately 300.0 ft^2/gal. The resin- or solvent-based primer shall be applied in one coat at a rate of approximately 120.0 ft^2/gal. Primer shall be applied by spray or squeegee methods to the entire area to be sealed.

All primers shall be thoroughly mixed and continuously agitated during application.

Primers shall be allowed to dry to a tack-free condition before placing membrane sheets.

Should membrane sheets not be placed over solvent-based primed surfaces within 24 h, or neoprene-based primed surfaces within 36 h, or resin-based primed surfaces within 8 h, the surfaces shall be reprimed.

The preformed membrane sheets shall be applied to the primed surfaces either by hand methods or by mechanical applicators. The membrane sheet shall be placed in such a manner that a shingling effect is achieved in the direction that water will drain. First, a 12.0-in. minimum width membrane strip shall be placed along the juncture of deck and base of barrier railing or curb face at the low side of the deck with the sheet extending up the face 3.0 in. Next, starting at the gutter line, sheets shall be laid longitudinally and side lapped with adjacent sheets by not less than 2.5 in. and end lapped by not less than 6.0 in. A 12.0-in. minimum width strip shall then be placed at the juncture of deck and base of curb or railing at the high side of the deck extending up the face 3.0 in. After being laid, the membrane sheets shall be rolled with hand rollers or other apparatus as necessary to develop a firm and uniform bond with the primed concrete surfaces. Procedures shall be used that minimize wrinkles and air bubbles. Any tears, cuts, or narrow overlaps shall be patched, using a satisfactory adhesive and by placing sections of membrane sheet over the defective area in such a manner that the patch extends at least 6.0 in. beyond the defect. On modified bitumen sheets with a permanent polyester film, a propane torch shall be used to melt the polyester film on the section to be patched. The patch shall then be placed over the heated surface. All patches shall be rolled or pressed firmly onto the surface.

At all open joints, at deck bleeder pipes, and at other locations when ordered by the Engineer, the membrane sheet shall be cut and turned into the joint or bleeder as the membrane sheet is laid.

For rubberized asphalt sheets and modified bitumen sheets, mastic shall be applied as a bead along the exposed edge of the membrane sheet that extends up the barrier railing or curb face, and that terminates in the high-side gutter after the sheets have been installed.

21.4.2.3—Installation on Other Surfaces

Installation of preformed membranes on surfaces other than bridge decks shall conform to the applicable requirements for bridge decks and to the following:

- Preformed membrane material shall be placed vertically with each successive sheet lapped to the preceding by a minimum of 3.0 in. Horizontal splices shall be lapped by a minimum of 6.0 in.

- Exposed edges of membrane sheets shall have a troweled bead of Manufacturer's recommended mastic or sealing tape applied after the membrane is placed.

- All projecting pipe, conduits, sleeves, or other facilities passing through the preformed membrane waterproofing shall be flashed with prefabricated or field-fabricated boots, fitted coverings, or other devices as necessary to provide watertight construction.

21.4.3—Protective Cover

Protective covers shall be installed sufficiently soon after the application of waterproofing to prevent any damage to the waterproofing from exposure to sunlight or the weather or damage from traffic or subsequent construction operations.

Hardboard protective covering shall be placed on a coating of adhesive of a type recommended by the waterproofing Manufacturer. The adhesive shall be applied at a rate sufficient to hold the protective covering in position until the backfill is placed.

21.4.4—Dampproofing

Concrete, brick, or other surfaces that are to be protected by dampproofing shall be thoroughly cleaned before the primer is applied. The surface to be dampproofed shall be primed and then thoroughly mopped with waterproofing asphalt. When the first mopping of asphalt has set sufficiently, the entire surface shall be mopped with the second coating of hot asphalt. Special care shall be taken to see that there are no skips in the coatings and that all surfaces are thoroughly covered.

21.5—MEASUREMENT AND PAYMENT

Waterproofing and dampproofing will be measured by the square yard complete in place and accepted.

Payment will be made on the basis of the number of square yards of waterproofing or dampproofing measured.

Payment for waterproofing includes full compensation for the cost of furnishing all equipment, materials, and labor necessary for the satisfactory completion of the waterproofing membrane and the protection cover.

Payment for dampproofing includes full compensation for the cost of furnishing all equipment, materials, and labor necessary for the satisfactory completion of the dampproofing.

TABLE OF CONTENTS

SECTION 22

SLOPE PROTECTION

22.1—GENERAL

This work shall consist of the construction of bank and slope protection courses in accordance with these specifications and in reasonably close conformity with the lines, grades, and thicknesses shown in the contract documents or established by the Engineer. These provisions shall apply to riprap, concrete slope paving, and precast concrete slope paving.

C22.1

Types of slope protection are designated as follows:

- Riprap
  - Hand-Placed Riprap—Hand-placed stones on earth or gravel bedding
  - Machine-Placed Riprap—Machine-placed stones on earth or gravel bedding
  - Wire-Enclosed Riprap (Gabions)—Stones placed in wire fabric enclosures
  - Grouted Riprap—Hand-placed riprap as described above with voids filled with sand-cement grout
  - Sacked Concrete Riprap—Hand-placed sacked concrete
- Concrete Slope Paving
- Cast-in-Place Slope Paving—Portland cement concrete, pneumatically applied mortar, or, when permitted, fabric forms filled with structural concrete grout
- Precast Concrete Slope Paving—Portland cement concrete slabs, blocks, or shapes precast prior to placement

22.2—WORKING DRAWINGS

Whenever specified in the contract documents or requested by the Engineer, the Contractor shall provide working drawings with design calculations and supporting data in sufficient detail to permit a structural review of the proposed design of a slope protection system. When concrete is involved, such data shall include the sequence and rate of placement. Sufficient copies shall be furnished to meet the needs of the Engineer and other entities with review authority. The working drawings shall be submitted sufficiently in advance of proposed use to allow for their review; revision, if needed; and approval without delay to the work.

The Contractor shall not start the construction of any slope protection system for which working drawings are required until the drawings have been approved by the Engineer. Such approval will not relieve the Contractor of responsibility for results obtained by use of these drawings or any other responsibilities under the contract documents.

22.3—MATERIALS

22.3.1—Aggregate

Aggregate for riprap shall conform to the requirements of the *AASHTO Guide Specifications for Highway Construction*, Subsection 703.15, "Aggregate for Riprap."

Aggregate for under drains and filter blankets shall conform to Section 704, "Aggregate for Drainage," and Subsection 705, "Stone Blanket Protection, Filter Blanket, and Fabric," respectively, of the *AASHTO Guide Specifications for Highway Construction*.

C22.3.1

Aggregate for riprap and for underdrains and filter blankets refers to the *AASHTO Guide Specifications for Highway Construction*.

22.3.2—Wire-Enclosed Riprap (Gabions)

Gabions shall be constructed of wire mesh. The wire mesh shall be made of galvanized steel wire having a minimum size of 0.120-in. diameter (U.S. Wire Gage No. 11). The tensile strength of the wire shall be in the range of 60.0 to 85.0 ksi, determined in accordance with ASTM A392. The minimum zinc coating of the wire shall be 0.80 oz/ft^2 of uncoated wire surface, as determined in accordance with AASHTO T 65M/T 65 (ASTM A90/A90M).

Selvage, tie, and connection wire shall meet the same strength and coating requirements specified above for wire used in the wire mesh.

C22.3.2

Wire steel gages are in accordance with the "Wire and Sheet Metal Gages" table in the AISC *LRFD Manual of Steel Construction*.

22.3.3—Filter Fabric

Filter fabric shall meet the requirements of the *AASHTO Guide Specifications for Highway Construction*, Subsection 705.03.

C22.3.3

Filter fabric requirements refer to the *AASHTO Guide Specifications for Highway Construction*, 1998.

22.3.4—Grout

Grout shall consist of one part portland cement and three parts of sand, thoroughly mixed with water to produce a workable mix.

22.3.5—Sacked Concrete Riprap

Concrete for sacked concrete riprap shall consist of a mixture of clean pit run or washed sand and gravel, cement, and water. The mixture shall contain not less than 376 lb/yd^3 of portland cement and sufficient water to obtain a slump of 3.0 to 5.0 in. Sacks for sacked concrete riprap shall be made of 10-oz burlap or other fabric having equal or greater strength. Sacks shall be approximately 19.5 × 36.0 in. measured inside the seams when the sack is laid flat, with an approximate capacity of 1.25 ft^3. Sound, reclaimed sacks may be used.

22.3.6—Portland Cement Concrete

Portland cement concrete for cast-in-place slope paving shall conform to the provisions in Section 8, "Concrete Structures," for Class B or Class B (AE) concrete using the 1.0-in. maximum combined grading.

Expansion joint filler shall conform to the provisions in Article 8.9.2.1, "Premolded Expansion Joint Fillers."

22.3.7—Pneumatically Applied Mortar

Materials for pneumatically applied mortar shall conform to the requirements of Section 24, "Pneumatically Applied Mortar."

22.3.8—Precast Portland Cement Concrete Blocks and Shapes

Precast portland cement concrete blocks and shapes shall meet the requirements of ASTM C129, C139, or C90 grade as specified in the contract documents. Materials for precast portland cement concrete slabs shall conform to the requirements in Article 8.13, "Precast Concrete Members."

22.3.9—Reinforcing Steel

Reinforcement shall conform to the provisions in Section 9, "Reinforcing Steel."

22.3.10—Geocomposite Drain

Geocomposite drain shall consist of a manufactured core with one or both sides covered with a layer of filter fabric.

The manufactured core shall be a preformed grid of embossed plastic, a mat of random shapes of plastic fibers, a drainage net consisting of a uniform pattern of polymeric strands forming two sets of continuous flow channels, a system of plastic pillars and interconnections forming a semi-rigid mat, or other system approved by the Engineer that will conduct the flow of water designated in the contract documents.

Filter fabric shall conform to the requirements of Article 22.3.3, "Filter Fabric," and shall be integrally bonded to the core material.

The Contractor shall furnish to the Engineer a signed certification from the Manufacturer stating that the geocomposite drain proposed for use is capable of withstanding design loadings at all planned locations without appreciably decreasing the carrying capacity of the designed drainage voids for the entire height or length of the drain.

22.4—CONSTRUCTION

22.4.1—Preparation of Slopes

Where required, slopes shall be shaped to allow the full thickness of the specified slope protection and any bedding or filter gravel. Slopes shall not be steeper than the natural angle of repose of the slope specified in the contract documents. Where the slopes cannot be excavated to undisturbed material, the underlying material shall be compacted to 95 percent standard density as specified in AASHTO T 99.

22.4.2—Bedding

When called for in the contract documents, a layer of filter gravel or filter fabric shall be placed on the slope immediately prior to placement of the riprap or slope paving. The layer of filter gravel shall be shaped to provide the minimum thickness specified in the contract documents.

22.4.3—Filter Fabric

When specified in the contract documents, filter fabric shall be spread uniformly over the prepared slope or surface. The fabric shall be unrolled directly on the surface to the lines and dimensions shown in the contract documents. The filter fabric shall be lapped a minimum of 12.0 in. in each direction and shall be anchored in position with approved anchoring devices. The Contractor shall place the riprap in a manner that will not tear, puncture, or shift the fabric. Tracked or wheeled equipment shall not be permitted on the fabric covered slopes.

22.4.4—Geocomposite Drain

Geocomposite drains shall be installed at locations shown in the contract documents and where directed by the Engineer. Collection and discharge systems shall be installed as shown in the contract documents or as directed by the Engineer.

Core material manufactured from impermeable plastic sheeting having connecting corrugations shall be placed with the corrugations approximately perpendicular to the drainage collection system.

When only one side of the geocomposite drain is covered with filter fabric, the drain shall be installed with the filter fabric side facing the embankment. The fabric facing the embankment side shall overlap a minimum of 3.0 in. at all joints and wrap around the exterior edges a minimum of 3.0 in. beyond the exterior edge. If additional fabric is needed to provide overlap at joints and wrap-around at edges, the added fabric shall overlap the fabric on the geocomposite drain at least 6.0 in. and be attached thereto.

Should the fabric on the geocomposite drain be torn or punctured, the damaged section shall be replaced completely or repaired by placing a piece of fabric that is large enough to cover the damaged area and provide a 6.0-in. overlap all around the damaged area.

22.4.5—Hand-Placing Stones

Where hand-placing of stones is specified in the contract documents, the larger stones shall be placed first with close joints in the footing trench. Stones shall be placed with their longitudinal axis normal to the embankment face and arranged so that each stone above the foundation course has a three-point bearing on the underlying stones. Bearing on smaller stones that may be used for chinking voids shall not be acceptable. Placing of stones by dumping shall not be permitted. Interstices shall be filled with smaller stones and spalls.

C22.4.5

The foundation course is the course placed on the slope in contact with the ground surface.

22.4.6—Machine-Placed Stones

22.4.6.1—Dry Placement

Machine-placed stones shall be so placed so as to provide a minimum of voids, and the larger stones shall be placed in the toe course and on the outside surface of the slope protection. The stone may be placed by dumping and may be spread in layers by bulldozers or other suitable equipment. At the completion of slope protection work, the footing trench shall be filled with excavated material, and compaction will not be required.

22.4.6.2—Underwater Placement

When placed under water, free dumping shall not be permitted without written permission of the Engineer. Placement shall be by controlled methods using bottom dump buckets or wire rope baskets lowered through the water to the point of placement.

22.4.7—Wire-Enclosed Riprap (Gabions)

22.4.7.1—Fabrication

The wire mesh shall be twisted to form hexagonal openings of uniform size. The maximum linear dimension of the mesh opening shall not exceed 4.5 in. and the area of the mesh opening shall not exceed 8.0 in.^{2}. The mesh shall be fabricated in such a manner as to be nonravelling.

Gabions shall be fabricated so the sides, ends, lid, and diaphragms can be assembled at the construction site into rectangular baskets of the specified size. Gabions shall be of single-unit construction, i.e., base, lid, ends, and sides shall be either woven into a single unit, or one edge of these members connected to the base section of the gabion in a manner such that strength and flexibility at the point of connection is at least equal to that of the mesh.

Where the length of the gabion exceeds its horizontal width, the gabion shall be equally divided by diaphragms of the same mesh and gage as the body of the gabions, into cells the length of which does not exceed the horizontal width. The gabion shall be furnished with the necessary diaphragms secured in proper position on the base in a manner that no additional tying at this junction will be necessary.

C22.4.7.1

Nonravelling is defined as the ability to resist pulling apart at any of the twists or connections forming the mesh when a single wire strand in a section is cut.

All perimeter edges of the mesh forming the gabion shall be securely clip-bound or selvaged so that the joints formed by tying the selvages have at least the same strength as the body of the mesh.

Selvage wire used through all the edges (perimeter wire) shall not be less than 0.148-in. diameter (U.S. Wire Gage No. 9) and shall meet the same strength and coating specifications as the wire mesh.

Tie and connection wire shall be supplied in sufficient quantity to securely fasten all edges of the gabion and diaphragms and to provide for at least four cross-connecting wires in each cell whose height is equal to the width and at least two cross-connecting wires in each cell whose height is one-half the width of the gabion. Cross-connecting wires shall not be required when the height of the cell is one-third the width of the gabion. Tie and connection wire shall meet the same strength and coating specifications as the wire used in the mesh, except that it may be as much as two gages smaller.

In lieu of tie wire, two-gage (6.668-mm) galvanized hog rings may be used to connect adjacent baskets and to secure basket lids. Spacing of the hog rings shall not exceed 6.0 in.

Vertical joints in the completed work shall be staggered at approximately one-third or one-half the length of the full baskets.

22.4.7.2—Installation

The gabions shall be placed on a smooth foundation. Final line and grade shall be approved by the Engineer.

Each gabion unit shall be assembled by binding together all vertical edges with wire ties on approximately 6.0-in. spacing or by a continuous piece of connecting wire stitched around the vertical edges with a coil about every 4.0 in. Empty gabion units shall be set to line and grade as shown in the contract documents or as directed by the Engineer. Wire ties, hog rings, or connecting wire shall be used to join the units together in the same manner as described above for assembling. Internal tie wires shall be uniformly spaced and securely fastened in each cell of the structure.

A standard fence stretcher, chain fall, or iron rod may be used to stretch the wire baskets and hold alignment.

The gabions shall be filled with stone carefully placed by hand or machine to ensure alignment and avoid bulges with a minimum of voids. Alternate placing of rock and connection wires shall be performed until the gabion is filled. After a gabion has been filled, the lid shall be bent over until it meets the sides and edges. The lid shall then be secured to the sides, ends, and diaphragms with the wire ties or connecting wire in the manner described above for assembling.

Wire steel gages are in accordance with the "Wire and Steel Gages" table in the AISC *LRFD Manual of Steel Construction*.

22.4.8—Grouted Riprap

Stones shall be placed on the slope as specified in Article 22.4.5, "Hand-Placing Stones," and shall be thoroughly moistened with water after placement. Grout shall be applied while the stone is moist and shall be worked into the interstices to completely fill the voids.

Where the depth is in excess of 12.0 in., the stone shall be placed in 12.0-in. lifts and each lift grouted prior to placement of the next lift. Succeeding lifts shall be constructed and grouted before grout in the previous lift has set.

Grout shall be placed only when the weather is suitable and shall be protected from freezing for at least four days. The surface shall be cured by covering with moist earth, wet rugs, or curing blankets for at least three days after grout placement.

Weep holes shall be provided through the riprap as shown in the contract documents or as directed by the Engineer.

22.4.9—Sacked Concrete Riprap

Sacks shall be filled with approximately 1.0 ft^3 of concrete, leaving room at the top to fold the sacks and retain the concrete during placement. Immediately after being filled, the sacks shall be placed and lightly trampled to conform with the earth face and with adjacent sacks.

The first two courses shall provide a foundation of double thickness. The first foundation course shall consist of a double row of stretchers with the long dimension of the sack parallel to contour of slope laid level and adjacent to each other in a neatly trimmed trench. The trench shall be located as shown in the contract documents or, as directed by the Engineer, cut to the proper depth and width to accommodate placement of the first two foundation courses, and cut back into the slope a sufficient distance to enable proper subsequent placement of the riprap. The second foundation course shall consist of a row of headers with the long dimension at right angles to the stretchers and placed directly above the double row of stretchers. The remaining courses shall consist of stretchers and shall be placed with staggered joints.

Dirt and debris shall be removed from the top of the sacks before the next course is placed. Stretchers shall be placed so that the folded ends are not adjacent. Headers shall be placed with the folds toward the earth face. Not more than four vertical courses of sacks shall be placed in any tier until initial set has taken place in the first course.

When there will not be proper bearing or bond for the concrete because of delays in placing succeeding layers of sacks, a small trench shall be excavated back of the row of sacks and filled with fresh concrete before the next layer of sacks is laid. Header courses may be required at any level to provide additional stability.

Sacked concrete riprap shall be cured with a blanket of wet earth or by sprinkling with a fine spray of water every two hours during the daytime for four days.

Weep holes shall be provided through the riprap as shown in the contract documents or as directed by the Engineer.

22.4.10—Concrete Slope Paving

22.4.10.1—General

This work shall consist of constructing cast-in-place and precast portland cement concrete slope paving. At the option of the Contractor, the cast-in-place slope paving shall be constructed of either portland cement concrete or pneumatically applied mortar. Where specified in the contract documents or permitted by the Engineer, this work shall also include woven fabric forms filled with fine aggregate portland cement concrete grout.

22.4.10.2—Cast-in-Place Slope Paving

Concrete shall be mixed and placed in conformance with the provisions in Section 8, "Concrete Structures," and shall be spread and tamped until it is thoroughly compacted and mortar flushes to the surface. If the slope is too steep to permit the use of concrete sufficiently wet to flush with tamping, the concrete shall be tamped until consolidated and a mortar surface 0.25 in. thick, troweled on immediately. The mortar shall consist of one part portland cement and three parts of fine aggregate. The mortar surface shall be considered as a part of the concrete and no separate payment will be made therefore.

After striking off to grade, the concrete shall be hand floated with wooden floats. Edges and joints shall be edged with a 0.25-in. radius edger prior to the brooming. The entire surface shall be broomed with a fine- texture hair push broom to produce a uniform surface with the broom marks parallel to the edges of the panel.

Pneumatically applied mortar shall be placed and finished in accordance with the provisions in Section 24, "Pneumatically Applied Mortar."

Expansion joints shall be installed transversely at intervals of 20.0 ft. Longitudinal expansion joints shall be installed at the locations shown in the contract documents. Expansion joints shall be filled with expansion joint filler 0.5 in. thick.

Cast-in-place concrete and pneumatically applied mortar shall be cured as provided in Sections 8, "Concrete Structures," and 24, "Pneumatically Applied Mortar," respectively.

Weep holes shall be provided through the slope paving as shown in the contract documents or as directed by the Engineer.

When permitted or specified in the contract documents, the Contractor may use woven fabric forms filled with pumpable fine aggregate portland cement concrete grout as the slope protection system. The request by the Contractor to use a particular system shall be in writing accompanied by working drawings and complete information as to the materials, construction, and performance characteristics of the proposed system.

Pervious backfill material, if required by the contract documents, shall be placed as shown. 2.0 ft^3 of pervious backfill material wrapped in filter fabric shall be placed at each weep hole and drain hole.

At the completion of the work, footing trenches shall be filled with excavated material and compaction shall not be required.

22.4.10.3—Precast Slope Paving

Precast slabs, blocks, and shapes shall be laid on a 3.0-in. bed of cushion sand in the pattern shown in the contract documents. Blocks and shapes shall be thoroughly rammed in place to provide a uniformly even surface and solid bedding under each block or shape.

In the areas where grouting is specified in the contract documents or required by the Engineer, the blocks shall be laid in running bond with the length parallel to the slope and with 0.25-in. joints. Following the laying of the blocks, in the area to be grouted, sufficient mortar sand shall be spread over the surface and swept into the joints to fill the latter to 4.0 in. from the surface. The blocks shall be wetted to the satisfaction of the Engineer before any grout is placed. The joints shall be filled with grout flush with the top of the block.

After grouting has been completed and the grout has sufficiently hardened, the blocks shall be wetted, covered, and cured with curing blankets or covers for the first seven days after grouting. Grout shall not be poured during freezing weather.

22.5—MEASUREMENT AND PAYMENT

22.5.1—Method of Measurement

22.5.1.1—Stone Riprap and Filter Blanket

Hand-placed riprap, machine-placed riprap, grouted riprap, and filter blanket aggregate shall be measured by the square yard, cubic yard, or ton, as specified in the contract documents. The area shall be that actually placed to the limiting dimensions shown in the contract documents or the dimensions as may have been revised by the Engineer, measured along the upper surface. If measured by the cubic yard, the volume shall be computed on the basis of the measured area and the thickness specified in the contract documents. If measured by the ton, the quantity shall be the number of tons, loose measure, incorporated into the work.

22.5.1.2—Sacked Concrete Riprap

Sacked concrete riprap shall be measured by the cubic yard of concrete placed. Measurement shall be based on mixer volumes.

22.5.1.3—Wire-Enclosed Riprap (Gabions)

Wire-enclosed riprap (gabions) shall be measured as the number of square yards of surface area.

22.5.1.4—Cast-in-Place Concrete Slope Paving

Cast-in-place concrete or pneumatically applied mortar slope paving shall be measured in square yards or cubic yards. The area will be that actually placed to the limiting dimensions shown in the contract documents, or the dimensions as may have been revised by the Engineer, measured along the upper sloped surface. If measured in cubic yards, the volume shall be computed on the basis of the measured area and the thickness shown in the contract documents. No additional compensation shall be allowed for additional concrete or pneumatically applied mortar placed by reason of low foundation.

22.5.1.5—Precast Concrete Slope Paving

Precast concrete slabs, blocks, or shapes placed as slope paving shall be measured in square yards computed from the payment lines shown in the contract documents, or as directed by the Engineer.

22.5.1.6—Filter Fabric

Filter fabric shall be measured in square yards on the ground surface, excluding overlaps, complete in place.

22.5.2—Payment

22.5.2.1—General

Payment for slope protection of the various classes at the unit prices bid shall include full compensation for all labor, materials, equipment, or other incidentals in connection with the preparation of subgrade (except for the furnishing and placement of filter blanket material and filter fabric); excavating and backfilling toe trenches where required; furnishing and placing the stones, slabs, blocks, shapes, grout, mortar, portland cement concrete, pneumatically applied mortar, reinforcing steel, expansion joint filler, if required; and all other work and incidental material required to complete the work as specified in the contract documents.

22.5.2.2—Stone Riprap

Hand-placed riprap, machine-placed riprap, and grouted riprap measured in accordance with Article 22.5.1.1, “Stone Riprap and Filter Blanket,” will be paid for at the price bid per square yard, per cubic yard, or per ton, as specified in the contract documents.

22.5.2.3—Sacked Concrete Riprap

Sacked concrete riprap measured in accordance with Article 22.5.1.2, “Sacked Concrete Riprap,” shall be paid for at the price bid per cubic yard.

22.5.2.4—Wire-Enclosed Riprap (Gabions)

Wire-enclosed riprap (gabions) measured in accordance with Article 22.5.1.3, “Wire-Enclosed Riprap (Gabions),” shall be paid for at the price bid per square yard. Such price shall include wire baskets, connection hardware, anchors, aggregate filling, and any other materials, labor, and equipment necessary to complete the work as specified in the contract documents.

22.5.2.5—Cast-in-Place Concrete Slope Paving

Cast-in-place concrete or pneumatically applied mortar slope paving measured in accordance with Article 22.5.1.4, “Cast-in-Place Concrete Slope Paving,” shall be paid for at the price bid per square yard or per cubic yard as specified in the contract documents.

22.5.2.6—Precast Concrete Slope Paving

Precast concrete slope paving measured in accordance with Article 22.5.1.5, “Precast Concrete Slope Paving,” shall be paid for at the price bid per square yard. Such price shall include cushion sand and shall include portland cement grout or mortar, if specified in the contract documents.

22.5.2.7—Filter Blanket

Filter blanket or filter gravel measured in accordance with Article 22.5.1.1, “Stone Riprap and Filter Blanket,” shall be paid for at the price bid per square yard, per cubic yard, or per ton as specified in the contract documents.

22.5.2.8—Filter Fabric

Filter fabric measured in accordance with Article 22.5.1.6, “Filter Fabric,” shall be paid for at the price bid per square yard.

22.5.2.9—Geocomposite Drain System

Geocomposite drain system shall be paid for on the basis of a contract lump-sum price. Such lump-sum price shall include full compensation for furnishing all labor, materials, tools, equipment, and incidentals, and for doing all the work involved in constructing geocomposite drain systems complete in place including geocomposite drain, collection, and discharge systems as shown in the contract documents and as directed by the Engineer.

22.6—REFERENCES

AASHTO. 1998. AASHTO *Guide Specifications for Highway Construction*, GSH-8 American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2009. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 29th Edition, HM-29, American Association of State Highway and Transportation Officials, Washington, DC.

AISC. 2003. *LRFD Manual of Steel Construction*, 3rd Edition, American Institute of Steel Construction, Chicago, IL, Table 17-10, “Wire and Sheet Metal Gages.”

TABLE OF CONTENTS

SECTION 23

MISCELLANEOUS METAL

23.1—DESCRIPTION

This work shall consist of furnishing and installing metal items in structures which are not otherwise provided for.

C23.1

Miscellaneous metalwork includes but is not limited to the following items:

- Expansion joint armor in bridge decks, and sliding plate and finger-type expansion joints.
- Manhole frames and covers, drainage pipes, frames and grates, ladders or ladder rungs, access opening covers, and access door assemblies.
- Other items specifically identified as miscellaneous metal in the contract documents.

23.2—MATERIALS 2014 Revision

Miscellaneous metal items shall be constructed of materials conforming to the following AASHTO (or ASTM) material specifications:

Material	*Specification*
Steel bars, plates, and shapes	ASTM A36/A36M, Grade 36 (Grade 250)
Bolts and nuts	ASTM A307
High-strength bolts, nuts, and washers	AASHTO M 164 (ASTM A325)
Steel castings	As per Article 11.3.5, "Steel Castings"
Grey iron castings	AASHTO M 105 or ASTM A48/A48M, Class 30
Sheet metal	Commercial quality

23.3—FABRICATION

Fabrication of miscellaneous metal items shall be performed in a manner in conformance with the practice in modern commercial shops. Burrs, rough and sharp edges, and other flaws shall be removed. Warped pieces shall be straightened after fabrication and galvanizing.

23.4—GALVANIZING

Unless otherwise specified in the contract documents, all steel items which are not embedded at least 2.0 in. in concrete and all cast iron sidewalk frames and covers shall be galvanized in accordance with Article 11.3.2.4, “Galvanized High-Strength Fasteners” and Article 11.3.7, “Galvanizing.” Assemblies shall be galvanized after fabrication.

23.5—MEASUREMENT

Measurement of miscellaneous metal shall be by the scale weight (mass). When requested by the Engineer, each delivery shall be accompanied with a certified weighmaster's ticket. Scale weight (mass) is not required when calculated weight (mass) is shown in the contract documents, in which case this weight (mass) shall be used as the basis of payment.

23.6—PAYMENT

Miscellaneous metal shall be paid for by the contract unit price per pound. Such payment shall include full compensation for furnishing all labor, materials, tools, equipment, and incidentals, and for doing all the work involved in furnishing and installing miscellaneous metal, complete in place, as specified in the contract documents, specified in these Specifications, and as directed by the Engineer.

23.7—REFERENCE

AASHTO. 2009. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 29th Edition, HM-29, American Association of State Highway and Transportation Officials, Washington, DC.

Back to 2010 Edition

SECTION 23: MISCELLANEOUS METAL

23.2—MATERIALS

Delete AASHTO M 164 citation as follows:

Miscellaneous metal items shall be constructed of materials conforming to the following AASHTO (or ASTM) material specifications:

Material	***Specification***
Steel bars, plates, and shapes	ASTM A36/A36M, Grade (Grade 250)
Bolts and nuts	ASTM A307
High-strength bolts, nuts, and washers	~~AASHTO M 164 (~~ASTM A235~~)~~
Steel Castings	As per Article 11.3.5, “Steel Castings”
Grey iron castings	AASHTO M 105 or ASTM A48/A48M, Class 30
Sheet metal	Commercial quality

2014 Revision

SECTION 24: PNEUMATICALLY APPLIED MORTAR

TABLE OF CONTENTS

SECTION 24

PNEUMATICALLY APPLIED MORTAR

24.1—DESCRIPTION

This work shall consist of the furnishing and placing of pneumatically applied mortar for the construction of portions of structures, repairing concrete structures, texturing concrete surfaces, encasement of structural steel members, lining ditches and channels, paving slopes, and for other miscellaneous work, all as specified in the contract documents.

This work shall also include the preparation of surfaces to receive the mortar and the furnishing and placing of any reinforcing steel and anchors for reinforcement.

Pneumatically applied mortar shall consist of either dry mixed fine aggregate and portland cement pneumatically applied by a suitable mechanism, to which mixture the water is added immediately previous to its expulsion from the nozzle, or mortar premixed by mechanical methods and pneumatically applied through a nozzle onto the prepared surface.

C24.1

The *AASHTO LRFD Bridge Design Specifications*, 2007, do not cover pneumatically applied mortar. The provisions of the 17th Edition of the *AASHTO Standard Specifications for Highway Bridges* shall be used.

24.2—MATERIALS

24.2.1—Cement, Aggregate, Water, and Admixtures

Cement, aggregate, water, and admixtures, when used, shall conform to the requirements of Section 8, “Concrete Structures.” Aggregate shall be fine aggregate, except that up to 30 percent coarse aggregate, conforming to AASHTO M 43 (ASTM D448) for size 0.375 in. to No. 8 or No. 16, may be substituted for fine aggregate.

Recovered rebound which is clean and free of foreign material may be reused as fine aggregate in quantities not to exceed 20 percent of the total fine aggregate requirements.

24.2.2—Reinforcing Steel

Reinforcing steel shall conform to the requirements of Section 9, “Reinforcing Steel.”

24.2.3—Anchor Bolts or Studs

Anchor studs used to support reinforcing wire fabric or bars when placing mortar against existing concrete or rock shall consist of 0.25-in. minimum diameter expansion hook bolts placed in drilled holes. Each bolt shall have sufficient engagement in sound masonry to resist a pullout force of 0.150 kips.

When permitted by the Engineer, driven steel studs of not less than 0.125-in. diameter and a minimum length of 2.0 in. may be used. The equipment used for driving such studs shall be of the type which uses an explosive for the driving force, and shall be capable of inserting the stud or pin to the required depth without damage to the surrounding concrete.

24.3—PROPORTIONING AND MIXING

24.3.1—Proportioning

The Contractor shall submit the proposed mix design to the Engineer for approval prior to start of the work.

Unless otherwise specified in the contract documents, the mix design shall provide a cement to aggregate ratio, based on dry loose volumes, of not less than 1:3.5 for the construction and repair of concrete structures and for encasing steel members, or not less than 1:5 for lining ditches and channels and for paving slopes.

The water content shall be as low as practical and shall be adjusted so that the mix is sufficiently wet to adhere properly and sufficiently dry so that it will not sag or fall from vertical or inclined surfaces or separate in horizontal work.

24.3.2—Mixing

Mixing shall be done either by the dry mix or wet mix process. Before being charged into the placing equipment, the materials shall be thoroughly and uniformly mixed using either a paddle-type or drum-type mixer designed for use with pneumatic application. Transit mix equipment and methods may be used for the wet process.

24.4—SURFACE PREPARATION

24.4.1—Earth

When pneumatically applied mortar is to be placed against earth, the area shall be accurately graded to the elevation and dimensions specified in the contract documents and shall be thoroughly compacted, with sufficient moisture to provide a firm foundation and to prevent absorption of water from the mortar, but shall not contain free surface water.

When shown in the contract documents, joints, side forms, headers, and shooting strips shall be provided for backing or paneling. Ground or gaging wires shall be used where necessary to establish thicknesses, surface planes, and finish lines.

24.4.2—Forms

When mortar is to be placed against forms, the forms shall conform to the requirement of Section 3, "Temporary Works."

24.4.3—Concrete or Rock

When mortar is to be placed against concrete or rock, all deteriorated or loose material shall be removed by chipping with pneumatic or hand tools. Square or slightly undercut shoulders shall be cut approximately 1.0 in. deep along the perimeter of repair areas. The surface shall be sandblasted as necessary to clean all rust from exposed steel and to produce a clean, rough-textured surface on the concrete or rock. The surface against which mortar is to be placed shall be kept wet for at least 1 h and then allowed to dry to a surface dry condition just prior to application of the mortar.

24.5—INSTALLATION

24.5.1—Placement of Reinforcing

Reinforcing steel, when required, shall be installed in conformance with the requirements of Section 9, "Reinforcing Steel."

Reinforcement in new construction shall be placed as specified in the contract documents and secured to ensure that no displacement results from impact of the pneumatically placed mortar during application.

For repair work, the reinforcing steel shall be supported by anchor studs installed in the existing masonry, except where existing reinforcing steel in the repair area is considered by the Engineer to be satisfactory for this purpose. Anchors shall be spaced no more than 12.0 in., center-to-center, on overhead surfaces; 18.0 in., center-to-center, on vertical surfaces; and 36.0 in., center-to-center, on top horizontal surfaces. At least three anchors shall be used in each individual patch area.

The Engineer shall be notified in advance of the date when installation of anchor studs is to begin. The locations of the studs shall be such that damage will not occur to prestressing tendons or conduits embedded in the concrete.

Unless otherwise specified in the contract documents, for repair work, all areas where the thickness of the mortar exceeds 1.5 in. shall be reinforced with a single layer of either 2 × 2—W1.2 × W1.2 or 3 × 3—W1.5 × W1.5 (50 × 50—MW8 × MW8 or 75 × 75—MW10 × MW10) welded wire fabric. For areas where the thickness of the mortar exceeds 4.0 in., a single layer of wire fabric shall be used to reinforce each 4.0-in. thickness of patch or fractional part thereof. All fabric shall be placed parallel to the proposed finished surface. Before the succeeding layer of fabric is installed, each layer of fabric shall be completely encased in mortar that has taken its initial set. Fabric supported adjacent to the prepared masonry surface shall be no closer than 0.5 in. to said surface. Fabric shall be carefully prebent before installation to fit around corners and into re-entrant angles, and shall in no case be sprung into place.

All steel items, including anchors, reinforcing bars, and wire fabric, shall be no closer than 1.0 in. to the finished surface of the mortar.

C24.5.1

Please refer to the Wire Reinforcement Institute's *Structural Welded Wire Fabric Detailing Manual, Part I.*

24.5.2—Placement of Mortar

Only experienced personnel shall be employed, and satisfactory evidence of such experience shall be furnished when requested by the Engineer.

The mortar shall be applied by pneumatic equipment that sprays the mix onto the prepared surface at as high velocity as needed to produce a compacted, dense, homogeneous mass. The air compressor and delivery hose lines shall be of adequate capacity and size to provide a minimum pressure of 35 psi at the nozzle for 1.0-in. nozzles and proportionally greater for larger nozzles. The velocity of the material as it leaves the nozzle must be maintained uniform at a rate determined for the given job conditions to produce minimum rebound.

Water which is added at the nozzle shall be supplied at a uniform pressure of not less than 15 psi greater than the air pressure at the nozzle.

The mortar shall be applied as dry as practicable to prevent shrinkage cracking. Shooting strips shall be employed to ensure square corners, straight lines, and a plane surface of mortar, except as otherwise specified in the contract documents or approved by the Engineer. They shall be so placed as to keep the trapping of rebound at a minimum. At the end of each day's work, or similar stopping periods requiring construction joints, the mortar shall be sloped off to a thin edge. Before placing an adjacent section, construction joints shall be thoroughly cleaned and wetted as required under Article 24.4, "Surface Preparation." In shooting all surfaces, the stream of flowing material from the nozzle shall impinge as nearly as possible at right angles to the surface being covered, and the nozzle shall be held 2.0 to 4.0 ft from the working surface.

A sufficient number of mortar coats shall be applied to obtain the required thickness. On vertical and overhead surfaces, the thickness of each coat shall be not greater than 1.0 in., except as approved by the Engineer, and shall be so placed that it will neither sag nor decrease the bond of the preceding coat. The time interval between successive layers in sloping, vertical, or overhanging work shall be sufficient to allow initial but not final set to develop. At the time the initial set is developing, the surface shall be cleaned to remove the thin film of laitance in order to provide for a bond with succeeding applications.

Rebound or accumulated loose sand shall be removed from the surface to be covered prior to placing of the original or succeeding layers of mortar and shall not be embedded in the work.

Materials that have been mixed for more than 45 min and have not been incorporated in the work shall not be used, unless otherwise permitted by the Engineer.

After curing and before final acceptance, all repaired areas shall be sounded. All unsound and cracked areas shall be removed and replaced.

24.5.2.1—Weather Limitations

Pneumatically placed mortar shall not be placed on a frozen surface nor when the ambient temperature is less than 40°F; nor shall it be placed when it is anticipated that the temperature during the following 24 h will drop below 32°F.

The application of pneumatically placed mortar shall be suspended if high winds prevent proper application, or rain occurs which would wash out the pneumatically placed mortar.

24.5.2.2—Protection of Adjacent Work

During the progress of the work, where appearance is important, adjacent facilities that may be permanently discolored, stained, or otherwise damaged by overspray, dust, or rebound shall be adequately protected and, if contacted, shall be cleaned by early scraping, brushing, or washing, as the surroundings permit.

24.5.3—Finishing

After mortar has been placed to the desired thickness, all high spots shall be cut off with a sharp trowel, or screeded to a true plane as determined by shooting strips or by the original masonry surface, or as directed. Cutting screeds, where used, shall be lightly applied to all surfaces so as not to disturb the mortar for an appreciable depth, and they shall be worked in an upward direction when applied on vertical surfaces. Unless otherwise specified in the contract documents, the finished mortar surface shall be given a final flash coat of about 0.125 in. of mortar. Special care shall be taken to obtain a uniform appearance on all exposed surfaces.

24.5.4—Curing and Protecting

Pneumatically placed mortar shall be water cured in conformance with the requirements of Article 8.11.3.2, "Water Method." The minimum water curing duration shall be 96 h. The mortar shall be protected from freezing during the curing period.

24.6—MEASUREMENT AND PAYMENT

The quantity of pneumatically applied mortar shall be measured either by the square yard or by the cubic yard as specified in the contract documents.

Square-yard measurements shall be based on measurements of the surface area of acceptable mortar placed in the work made along the plane or curve of each surface. Cubic-yard measurement shall be based on the dimensions of such work shown in the contract documents or ordered by the Engineer.

Pneumatically applied mortar shall be paid for by the unit contract price specified. Such payment shall be considered to be full compensation for the cost of furnishing all labor, materials, equipment, and incidentals, and for doing all work involved in preparing the surface and installing the mortar, reinforcing steel, anchor studs, headers, joint fillers, and other items as specified in the contract documents.

24.7—REFERENCES

AASHTO. 2002. *Standard Specifications for Highway Bridges*, 17th Edition, HB-17, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2009. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 29th Edition, HM-29, American Association of State Highway and Transportation Officials, Washington, DC.

WRI. 2001. *Structural Welded Wire Fabric Detailing Manual, Part I*, WWR-500, Wire Reinforcement Institute, Hartford, CT.

SECTION 25: STEEL AND CONCRETE TUNNEL LINERS

TABLE OF CONTENTS

SECTION 25

STEEL AND CONCRETE TUNNEL LINERS

25.1—SCOPE

These Specifications are intended to cover the installation of tunnel liner plates in tunnels constructed by conventional tunnel methods.

C25.1

For the purposes of these Specifications, tunnels excavated by full face, heading and bench, or multiple drift procedures are considered conventional methods.

Liner plates used with any construction procedure utilizing a full or partial shield, a tunneling machine, or other piece of equipment that will exert a force on the liner plates for the purpose of propelling, steering, or stabilizing the equipment are considered special cases and are not covered by these Specifications.

25.2—DESCRIPTION

This work shall consist of furnishing cold-formed steel tunnel liner plates or precast concrete plates, conforming to these Specifications and of the sizes and dimensions required in the contract documents, and installing such plates at the locations specified in the contract documents or by the Engineer and in conformity with the lines and grades established or by the Engineer. The completed liner shall consist of a series of liner plates assembled with staggered longitudinal joints.

Steel tunnel liner plates should be of a type that is commercially available. Precast concrete tunnel liner plates shall be such that their size and shape suits the method and equipment being used to install them.

25.3—MATERIALS AND FABRICATION

25.3.1—General

Steel liner plates herein described must meet the sectional properties of thickness, area, and moment of inertia shown in the contract documents. If not shown in the contract documents, the properties shall be as specified in the *AASHTO LRFD Bridge Design Specifications*, Article 12.13.3.

All steel plates shall be connected by bolts on both longitudinal and circumferential seams or joints and shall be so fabricated as to permit complete erection from the inside of the tunnel. Bolt sizes and properties shall be in accordance with the Manufacturer's standard, but not less than those specified in the *AASHTO LRFD Bridge Design Specifications*, Article 12.13.3.1.

Grout holes 2.0 in. or larger in diameter shall be provided as shown in the contract documents to permit grouting as the erection of tunnel liner plates progresses.

C25.3.1

Liner plates shall be fabricated to fit the cross-section of the tunnel.

Steel liner plate requirements refer to Article 12.13.3, “Safety Against Structural Failure,” of the *AASHTO LRFD Bridge Design Specifications*, 2007.

Bolt size and property standards refer to Article 12.13.3.1, “Section Properties,” of the *AASHTO LRFD Bridge Design Specifications*, 2007.

Precast concrete tunnel liner plates shall conform to the details shown in the contract documents and the requirements of Section 8, "Concrete Structures." If such details are not provided and the contract documents allow the Contractor to propose the use of concrete liner plates, the Contractor shall submit working drawings to the Engineer for approval. Such drawings shall describe materials to be used, plate dimensions, reinforcement details, connecting details, and erection procedures. The fabrication of Contractor-proposed types of concrete tunnel liner plates shall not begin until the working drawings have been approved. Such approval shall not relieve the Contractor of any responsibility under the contract documents for the successful completion of the work.

25.3.2—Forming and Punching of Steel Liner Plates

All plates shall be formed to provide circumferential flanged joints. Longitudinal joints may be flanged or of the offset lap seam type. All plates shall be punched for bolting on both longitudinal and circumferential seams or joints. Bolt spacing in circumferential flanges shall be in accordance with the Manufacturer's standard spacing and shall be a multiple of the plate length so that plates having the same curvature shall be interchangeable and will permit staggering of the longitudinal seams. Bolt spacing at flanged longitudinal seams shall be in accordance with the Manufacturer's standard spacing. For lapped longitudinal seams, bolt size and spacing shall be in accordance with the Manufacturer's standard but not less than that necessary to meet the longitudinal seam strength requirements specified in the *AASHTO LRFD Bridge Design Specifications*, Article 12.13.3.1.

C25.3.2

Article 12.13.3.1, Bolt Size and Property Standards refers to "Section Properties" of the *AASHTO LRFD Bridge Design Specifications,* 2007.

25.4—INSTALLATION

25.4.1—Steel Liner Plates

All steel liner plates for the full length of a specified tunnel shall be of one type only, either the flanged or the lapped seam type of construction.

Liner plates shall be assembled in accordance with the Manufacturer's instructions.

Coated steel plates shall be handled in such a manner as to prevent bruising, scaling, or breaking of the coating. Any plates that are damaged during handling or placing shall be replaced at the Contractor's expense, except that small areas with minor damage may be repaired by the Contractor as directed by the Engineer.

25.4.2—Precast Concrete Liner Plates

Installation of precast concrete tunnel liner plates shall not start prior to receipt of approval of working drawings submitted as required by Article 25.3.1, "General." Installation shall conform to the specified or approved erection procedures.

25.4.3—Grouting

When directed by the Engineer, voids occurring between the liner plate and the tunnel wall shall be force-grouted. The grout shall be forced through the grouting holes in the plates with such pressure that all voids will be completely filled. Full compensation for back packing or grouting shall be considered as included in the contract price paid for tunnel and no separate payment will be made therefore.

25.5—MEASUREMENT

Tunnel liner shall be measured by the linear foot along the liner plate invert.

25.6—PAYMENT

Payment for the length of each size of tunnel liner as determined under measurement shall be at the contract unit prices per linear foot bid for the various sizes, which payment shall include full compensation for furnishing all labor, materials, tools, equipment, and incidentals to complete this item, including the force-grouting of voids.

25.7—REFERENCES

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

SECTION 26: METAL CULVERTS

TABLE OF CONTENTS

SECTION 26

METAL CULVERTS

26.1—GENERAL

26.1.1—Description

2012 Revision

This work shall consist of furnishing, fabricating, installing, and inspecting metal pipe, structural plate metal pipe, arches, pipe arches, box structures, and deep corrugated structures in conformance with these Specifications, and the details shown in the contract documents. As used in this specification, long-span structures are metal plate horizontal elliptic, inverted pear and multiple radius arch shapes, as well as special shape culverts as defined in Section 12 of the *AASHTO LRFD Bridge Design Specifications.*

C26.1.1

The terms "metal pipe" and "metal structural plate pipe" includes circular and pipe arch, underpass and elliptical shapes. "Metal structural plate arches" consist of a metal plate arch supported on reinforced concrete footings with or without a paved invert slab. "Pipe arches" are constructed to form a pipe having an arch-shaped crown and a relatively flat invert. "Structural plate metal box structures" are conduits, rectangular in cross-section, constructed of metal plates.

The metal culvert description refers to Section 12, "Buried Structures and Tunnel Liners," of the *AASHTO LRFD Bridge Design Specifications*.

26.2—WORKING DRAWINGS

Where specified or requested by the Engineer, the Contractor shall provide Manufacturer's installation instructions or working drawings and substantiating calculations in sufficient detail to permit a structural review. The working drawings shall be submitted in advance of construction to allow for their review, revision, and approval without delay to the work.

The Contractor shall not start the construction of any metal culvert for which working drawings are required until the drawings have been approved by the Engineer. Such approval will not relieve the Contractor of responsibility for results obtained by use of these drawings or any other contractual responsibilities.

26.3—MATERIALS

26.3.1—Corrugated Metal Pipe

Steel pipe shall conform to the requirements of AASHTO M 36 (ASTM A760/A760M).

Aluminum pipe shall conform to the requirements of AASHTO M 196 (ASTM B745/B745M).

26.3.2—Structural Plate

Steel structural plate shall conform to the requirements of AASHTO M 167M/M 167 (ASTM A761/A761M).

Aluminum alloy structural plate shall conform to the requirements of AASHTO M 219 (ASTM B746/B746M).

2010 Edition

26.3.3—Nuts and Bolts

Nuts and bolts for steel structural plate pipe, arches, pipe arches, and box structures shall conform to the requirements of AASHTO M 167M/M 167 (ASTM A761/A761M). Nuts and bolts for aluminum structural plate shall be aluminum conforming to the requirements of ASTM F468 or standard strength steel conforming to ASTM A307.

26.3.4—Mixing of Materials

Aluminum and steel materials shall not be mixed in any installation unless the materials are adequately separated or protected to avoid galvanic reactions. Hot dip galvanized steel and stainless steel bolts and nuts are acceptable for connection of aluminum structural plate.

26.3.5—Fabrication

Plates at longitudinal and circumferential seams shall be connected by bolts with the seams staggered so that not more than three plates come together at any one point.

26.3.6—Welding

If required, welding of steel shall conform to the current AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*. All welding of steel plates, other than fittings, shall be performed prior to galvanizing.

If required, welding of aluminum shall conform to the ANSI/AWS D1.2/D1.2M *Structural Welding Code—Aluminum*.

C26.3.6

Welding references AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code* and ANSI/AWS D1.2/D1.2M *Structural Welding Code—Aluminum.*

26.3.7—Protective Coatings

When required in the contract documents, metal pipes and structural metal plate culverts shall be protected with bituminous coating or have the invert paved with bituminous material. Bituminous coatings shall be applied as provided in AASHTO M 190, Type A, unless otherwise specified in the contract documents. If required, bituminous pavings shall be applied over the bituminous coatings to the inside bottom portion of pipe as provided in AASHTO M 190, Type C, unless otherwise specified in the contract documents. The portion of all nuts and bolts used for assembly of coated structural plate pipe, arches, pipe arches, and box culverts, projecting outside the pipe shall be coated after installation. The portions of the nuts and bolts projecting inside the pipe need not be coated.

When required in contract documents, polymeric coatings shall conform to the requirements of AASHTO M 246 (ASTM A742/A742M). The polymeric coating shall be applied to the galvanized sheet prior to corrugating and, unless otherwise specified in the contract documents, the thickness shall be not less than 0.010 in. Any pinholes, blisters, cracks, or lack of bond shall be cause for rejection. Polymeric coatings are not permitted on structural plate.

26.3.8—Bedding and Backfill Materials

26.3.8.1—General

2012 Revision

Bedding shall be loose native or granular material with a maximum particle size less than one-half the corrugation depth. Backfill for metal culverts shall be granular material, as specified in the contract documents and specifications, and shall be free of organic material, rock fragments larger than 3.0 in. in the greatest dimension, and frozen lumps, and shall have a moisture content within the limits required for compaction. As a minimum, backfill materials shall meet the requirements of AASHTO M 145 for A-1, A-2, or A-3.

26.3.8.2—Long-Span Structures

Bedding and backfill materials shall meet the general requirements of Article 26.3.8.1. As a minimum, backfill materials for structures with less than 12.0 ft of cover shall meet the requirements of AASHTO M 145 for A-1, A-2-4, A-2-5, or A-3. Minimum backfill requirements for structures with 12.0 ft or more cover shall meet AASHTO M 145 requirements for A-1 or A-3.

26.3.8.3—Box Culverts

Bedding and backfill materials shall meet the general requirements of Article 26.3.8.1. As a minimum, backfill shall meet the requirements of AASHTO M 145 for A-1, A-2-4, A-2-5, or A-3.

26.3.8.4—Deep Corrugated Structures

For deep corrugated structures, the select backfill within the structural backfill zone shall meet the requirements of AASHTO M 145 A-1, A-2-4, A-2-5, or A-3 (ASTM D2487 classifications GW, GP, SW, SP, GM, SM, SC, GC) and the manufacturer's requirements.

2012 Revision

26.4—ASSEMBLY

26.4.1—General

2012 Revision

Corrugated metal pipe and structural plate pipe shall be assembled in accordance with the Manufacturer's instructions. All pipe shall be unloaded and handled with reasonable care. Pipe or plates shall not be rolled or dragged over gravel or rock, and shall be prevented from striking rock or other hard objects during placement in the trench or on the bedding.

Corrugated metal pipe shall be placed in the bed starting at the downstream end. Pipes with circumferential seams shall be installed with their inside circumferential sheet laps pointing downstream.

C26.4.1

Bituminous coated pipe, polymer coated pipe, and paved invert pipe shall be installed in a similar manner to corrugated metal pipe with special care in handling to avoid damage to coatings. Paved invert pipe shall be installed with the invert pavement placed and centered on the bottom.

Structural metal plate culverts and pipes shall be assembled and installed as specified in the contract documents and detailed erection instructions. Copies of the Manufacturer's assembly instructions shall be furnished as specified in Article 26.2. Bolted longitudinal seams shall be well fitted with the lapping plates parallel to each other. The applied bolt torque for 0.75-in. diameter (M20) high strength steel bolts (ASTM A449) for the assembly of steel structural plate shall be a minimum of 100 ft · lb and a maximum of 300 ft · lb. Aluminum structural plate shall be assembled using 0.75-in. diameter (M20) aluminum bolts (ASTM F468) or standard strength steel bolts (ASTM A307) which shall be torqued to a minimum of 100 ft · lb and a maximum of 150 ft · lb.

Longitudinal seams in deep corrugated structures shall be staggered.

There is no structural requirement for residual torque; the important factor is the seam fit-up.

When seam sealant tape or a shop-applied asphalt coating is used, bolts should be retightened no more than once, and generally within 24 h after initial tightening.

26.4.2—Joints

2012 Revision

Joints for corrugated metal culvert and drainage pipe shall meet the following performance requirements.

26.4.2.1—Field Joints

Transverse field joints shall be of such design that the successive connection of pipe sections form a continuous line free from appreciable irregularities in the flow line. In addition, the joints shall meet the general performance requirements described in Articles 26.4.2.2 through 26.4.2.4.

C26.4.2.1

Suitable transverse field joints, which satisfy the requirements for one or more of the subsequently defined joint performance categories, can be obtained with the following types of connecting bands furnished with the suitable band-end fastening devices:

- Corrugated bands
- Bands with projections
- Flat bands
- Bands of special design that engage factory reformed ends of corrugated pipe
- Other equally effective types of field joints may be used with the approval of the Engineer.

26.4.2.2—Joint Types

The contract document should specify either "Standard" or "Special" joints as appropriate for the requirements at hand.

C26.4.2.2

Standard joints are for pipe not subject to large soil movements or disjointing forces. These joints are satisfactory for ordinary installations where simple slip-type joints are typically used. Special joints are for more severe requirements such as the need to withstand soil movements or resist disjointing forces. Examples of conditions leading to more severe requirements include poor foundation conditions or conditions producing longitudinal hydraulic forces requiring downdrain joints such as pipes on steep slopes or sharp curves.

26.4.2.3—Soil Conditions

Special joints should be specified when poor soil conditions are encountered.

C26.4.2.3

An example of poor soil conditions is when the backfill or foundation material is characterized by large soft spots or voids. If construction in such soil is unavoidable, this condition can only be tolerated for relatively low fill heights, because the pipe must span the soft spots and support imposed loads.

26.4.2.4—Joint Properties

The requirements for joint properties shall be taken as specified in Table 26.4.2.4-1. The values for various types of pipe may be determined by a rational analysis or a suitable test.

The following design issues shall be considered in the design of, or selection of, pipe joints:

- Joint Overlap—Standard joints which do not meet the moment strength alternatively shall have a minimum sleeve width overlapping the abutting pipes. The minimum total sleeve width shall be as given in Table 26.4.2.4-1. Any joint meeting the requirements for a special joint may be used in lieu of a standard joint.

- Soil Tightness—No opening may exceed 1.0 in. In addition, for all categories, if the size of the opening exceeds 0.125 in., the length of the channel shall be at least four times the size of the opening. For nonerodible or erodible soils, the ratio of D_{85} soil size to size of opening must be greater than 0.3 for medium to fine sand or 0.2 for uniform sand; these ratios need not be met for cohesive backfills where the plasticity index exceeds 12 percent. Alternatively, a joint which withstands 2 psi hydrostatic test without leakage shall be considered soil tight. Joints that do not meet these requirements may be made soil tight by wrapping with a suitable geotextile.

C26.4.2.4

The joint resistances in shear and flexure are given in Table 26.4.2.4-1 as a percentage of the respective resistance of a transverse cross-section remote of the joint.

Tensile strength is required in a joint when the possibility exists that a longitudinal load could develop which would tend to separate adjacent pipe sections.

Soil tightness refers to openings in the joint through which soil may infiltrate. Soil tightness is influenced by the size of the opening (maximum dimension normal to the direction that the soil may infiltrate) and the length of the channel (length of the path along which the soil may infiltrate).

As a general guideline, a backfill material containing a high percentage of fine-grained soils requires investigation for the specific type of joint to be used to guard against soil infiltration.

Backfill which is not subject to piping action is classified as "nonerodible." Such backfill typically includes granular soil (with grain sizes equivalent to coarses and, small gravel, or larger) and cohesive soils.

Backfill that is subject to piping action, and would tend to either infiltrate the pipe or to be easily washed by exfiltration of water from the pipe, is classified as "erodible." Such backfill typically includes fine sands and silts.

- Watertightness—The adjoining pipe ends in any joint shall not vary more than 0.5 in. in diameter or more than 1.5 in. in circumference for watertight joints.

Watertightness may be specified for joints of any category where needed to satisfy other criteria. The leakage rate shall be measured with the pipe in place or at an approved test facility. The tolerances indicated may be attained by proper production controls or by match-marking pipe ends.

2012 Revision

Table 26.4.2.4-1—Categories of Pipe Joints

	Soil Condition				
	Nonerodible		Erodible		
	Joint Type		Joint Type		
Joint Property	Standard	Special	Standard	Special	Downdrain
Shear Resistance	2%	5%	2%	5%	2%
Moment Resistance	5%	15%	5%	15%	15%
Tensile Resistance 0–42.0 in. dia.	0	5.0 kips	—	5.0 kips	5.0 kips
Tensile Resistance (48.0 in.–84.0 in.), dia.	—	10.0 kips	—	10.0 kips	10.0 kips
Joint Overlap, minimum	10.5 in.	NA	10.5 in.	NA	NA
Soil tightness	NA	NA	0.3 or 0.2	0.3 or 0.2	0.3 or 0.2
Watertightness	See Article C26.4.2.4				

26.4.3—Assembly of Long-Span Structures

Unless held in shape by cables, struts, or backfill, longitudinal seams should be tightened when the plates are hung. Care shall be taken to align plates to ensure properly fitted seams prior to bolt tightening. The variation in structure dimensions before backfill shall comply with the following provisions:

- For horizontal elliptic shapes having a ratio of top to side radii of three or less, the span and rise shall not deviate from the specified dimensions by more than two percent.

- For arch shapes having a ratio of top to side radii of three or more, the rise shall not deviate from the specified dimensions by more than one percent of the span.

- For all other long-span structures, the span and rise shall not deviate from the specified dimensions by more than two percent, nor more than 5.0 in., whichever is less.

When required by structural design, reinforcing ribs shall be attached to the structural plate corrugation crown prior to backfilling, using a bolt spacing of not more than 12.0 in. Legible identifying letters or numbers shall be placed on each rib to designate its proper position in the finished structure.

C26.4.3 2011 Revision

Long-span structures may require deviation from the normal practice of loose bolt assembly.

The process of erection specified herein may require temporary shoring.

When required for control of structure shape during installation, reinforcing ribs shall be spaced and attached to the corrugated plates at the discretion of the manufacturer with the approval of the Engineer.

26.5—INSTALLATION

26.5.1—General 2012 Revision

For trench conditions, the trench shall be excavated to the width, depth, and grade shown in the contract documents.

Proper preparation of foundation, placement of foundation material where required, and placement of bedding material shall precede the installation of all culvert pipe. This work shall include necessary leveling of the native trench bottom or the top of the foundation material as well as placement and compaction of required bedding material to a uniform grade so that the entire length of pipe will be supported on a uniform base. The backfill material shall be placed and compacted around the pipe in a manner to meet the requirements specified.

Materials used for foundation improvements, bedding and structure backfill must have gradations compatible with adjacent soils to avoid migration. Where material gradations cannot be properly controlled, adjacent materials must be separated with a suitable geotextile.

All pipes shall be protected by sufficient cover before permitting heavy construction equipment to pass over them during construction.

26.5.2—Foundation

The foundation under the pipe and structure backfill shall be investigated for its adequacy to support the loads. A foundation shall be provided, such that the structure backfill does not settle more than the pipe to avoid downdrag loads on the pipe.

The foundation must provide uniform support for the pipe invert. Boulders, rock or soft spots in the foundation shall be excavated to a suitable depth and backfilled with material compacted sufficiently to provide uniform bearing as shown in Figure 26.5.2-1.

C26.5.2

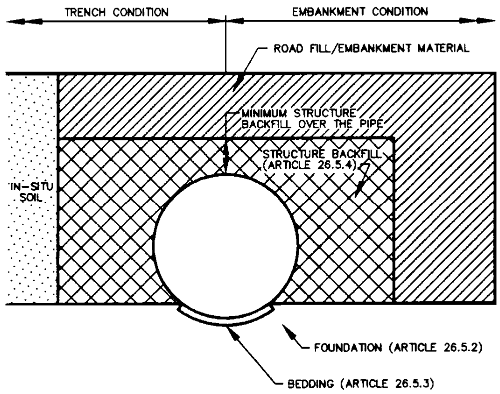


Figure 26.5.2-1—Pipe Installation Nomenclature

Where the natural foundation is judged inadequate by the Engineer to support the pipe or structure backfill, it shall be excavated to a suitable depth and backfilled with material compacted sufficiently to control settlements as shown in Figures 26.5.2-2 and 26.5.2-3.

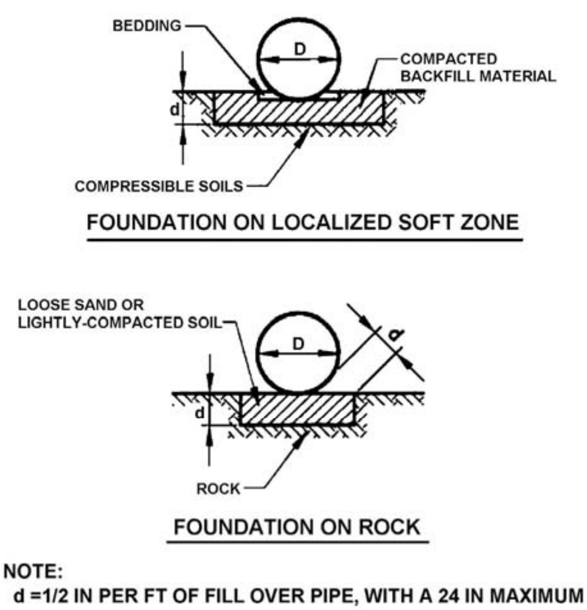


Figure 26.5.2-2—Foundation Treatment for Localized Soft Spots or Rock

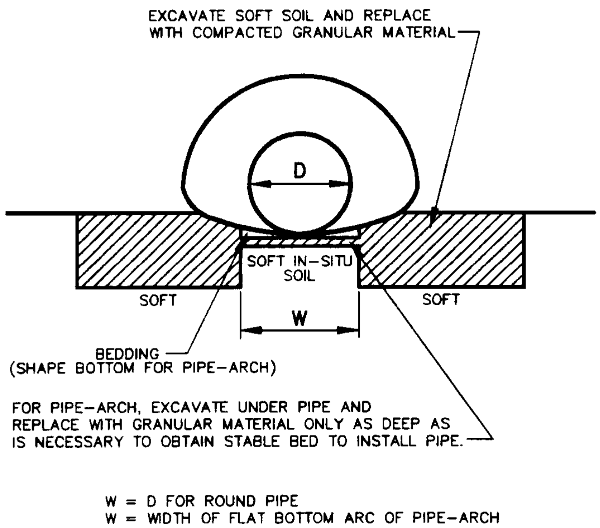


Figure 26.5.2-3—Foundation Treatment for Settlement Control

Where relatively large-radius inverts adjoin small radius corners or sides for sections such as pipe arches, elliptic pipe or underpasses, the foundation shall be designed to support the radial pressures exerted by the smaller radius portions of the pipe. The principal foundation support shall be provided in the area extending radially outward from the smaller radius areas.

These pressures may be two to five times the loading pressures on top of the pipe, depending on the pipe shape.

When corrective measures are necessary, providing less support under the invert allows the pipe to maintain its shape as minor settlements occur as shown in Figure C26.5.2-1.

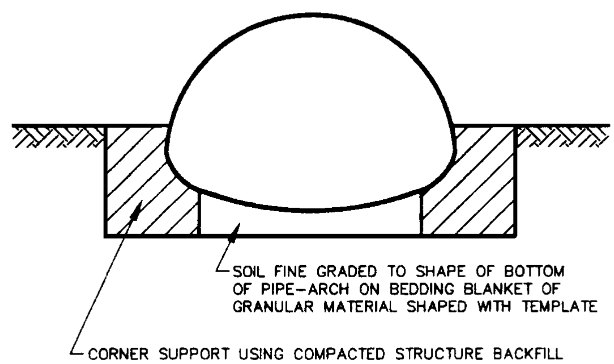


Figure C26.5.2-1—Foundation Treatment for Support of Corner or Side Plate Pressures

Where settlement of the pipe is expected to be so large that the required grade under high fills will not be maintained, pipe may be cambered to prevent excessive sag. The amount of camber shall be determined based on consideration of the flow line, gradient, fill height, the compressive characteristics of the foundation material, and the depth to incompressible strata.

The use of camber under a high fill is shown in Figure C26.5.2-2.

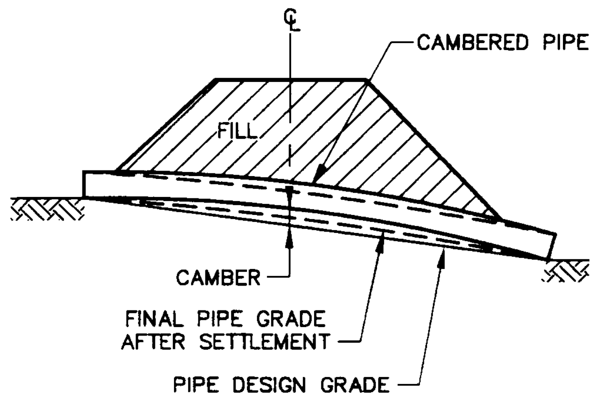


Figure C26.5.2-2—Pipe Camber for Settlement Control under High Fills

26.5.3—Bedding

When, in the opinion of the Engineer, the natural soil does not provide a suitable bed, a bedding blanket shall be provided with a minimum thickness of twice the corrugation depth.

Pipe arch, horizontal elliptic and underpass shapes with spans exceeding 12.0 ft should be placed on a shaped bed. The shaped area should be centered beneath the pipe and should have a minimum width of one-half the span for pipe arch and underpass shapes, and one-third the span for horizontal elliptic shapes. Preshaping may consist of a simple "V" graded into the soil as shown in Figure 26.5.3-1.

C26.5.3

The pipe bedding is a relatively thin layer of loosely-placed material that cushions the pipe invert and allows the corrugation to nest or seat into it, thus supporting the corrugation.

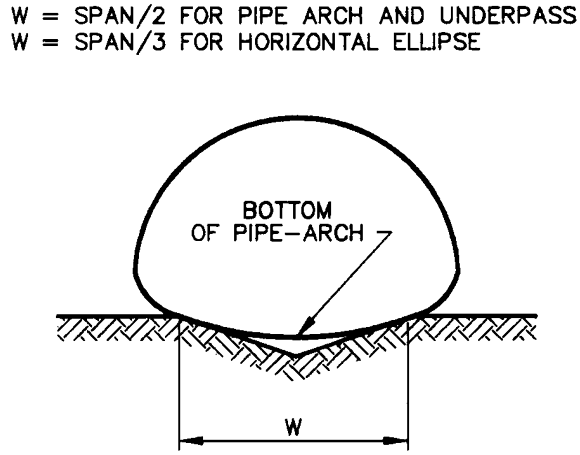


Figure 26.5.3-1—Shaped Bedding for Large Pipe-Arch, Horizontal Ellipse and Underpass Structures

26.5.4—Structure Backfill

26.5.4.1—General

Sufficient inspection and testing should be undertaken to be certain that the quality of the soil and the compactive effort obtained is as specified.

Backfill material shall meet the requirements of Article 26.3.8 and shall be placed in layers not exceeding 8.0-in. loose lift thickness to a minimum 90-percent standard density per AASHTO T 99. Equipment used to compact backfill within 3.0 ft from sides of pipe or from edge of footing for arches and box culverts shall be approved by the Engineer prior to use. Except as provided below for long-span structures, the equipment used for compacting backfill beyond these limits may be the same as used for compacting embankment.

The backfill shall be placed and compacted with care under the haunches of the pipe and shall be raised evenly on both sides of the pipe by working backfill operations from side to side. The side to side backfill differential shall not exceed 24.0 in. or one-third of the rise of the structure, whichever is less. Backfill shall continue to not less than 1.0 ft above the top for the full length of the pipe. Fill above this level shall be embankment fill or other materials as specified to support the pavement. The trench shall be kept to the minimum width required for placing pipe, placing adequate bedding and sidefill, and safe working conditions. Ponding or jetting of backfill shall not be permitted except upon written permission by the Engineer.

Where single or multiple structures are installed at a skew to the embankment, proper support for the pipe shall be provided. Support may be achieved with a rigid, reinforced concrete headwall or by warping the embankment fill to provide the necessary balanced side support. Figure 26.5.4.1-1 provides guidelines for warping the embankment.

C26.5.4.1

Quality control is of extreme importance because the structural integrity of the corrugated metal structure is vitally affected by the quality of construction in the field.

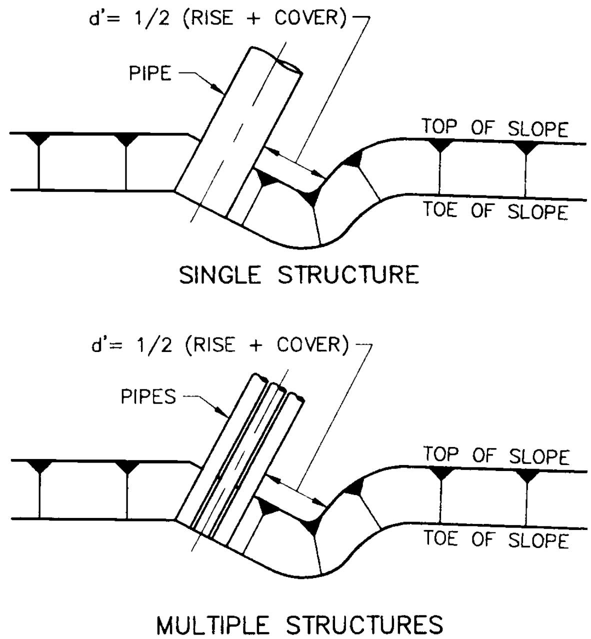


Figure 26.5.4.1-1—End Treatment of Skewed Flexible Culvert

26.5.4.2—Arches

Arches may require special shape control during the placement and compaction of structure backfill.

Prior to construction, the Manufacturer shall attend a preconstruction conference to advise the Contractor(s) and Engineer of the more critical functions to be performed during backfilling and to present the intended quality control steps to be used to control loads, shape and movements.

C26.5.4.2

Pin connections at the footing restrict uniform shape change. Arches may peak excessively or experience curvature flattening in their upper quadrants during backfilling. Using lighter compaction equipment, more easily compacted structure backfill or top loading by placing a small load of structure backfill on the crown will aid installation.

26.5.4.3—Long-Span Structures

Prior to construction, the Manufacturer shall attend a preconstruction conference to advise the Contractor(s) and Engineer of the more critical functions to be performed during backfilling and to present the intended quality control steps to be used to control loads, shape and movements.

C26.5.4.3

Backfill requirements for long-span structural-plate structures are similar to those for smaller structures. Their size and flexibility require special control of backfill and continuous monitoring of structure shape.

Equipment and construction procedures used to backfill long-span structural plate structures shall be such that excessive structure distortion will not occur. Structure shape shall be checked regularly during backfilling to verify acceptability of the construction methods used. Magnitude of allowable shape changes will be specified by the Manufacturer (Fabricator of long-span structures). The Manufacturer shall provide a qualified shape-control Inspector to aid the Engineer during the placement of all structure backfill to the minimum cover level over the structure. The shape-control Inspector shall advise the Construction Engineer on the acceptability of all backfill material and methods and the proper monitoring of the shape. Structure backfill material shall be placed in horizontal uniform layers not exceeding an 8.0-in. loose lift thickness and shall be brought up uniformly on both sides of the structure. Each layer shall be compacted to a density not less than 90 percent modified density per AASHTO T 180. The structure backfill shall be constructed to the minimum lines and grades shown in the contract documents, keeping it at or below the level of adjacent soil or embankment. The following exceptions to the required structure backfill density shall be permitted:

- the area under the invert,
- the 12.0-in. to 18.0-in. width of soil immediately adjacent to the large radius side plates of high-profile arches and inverted-pear shapes, and
- the lower portion of the first horizontal lift of overfill carried ahead of and under the small, tracked vehicle initially crossing the structure.

26.5.4.4—Box Culverts

A preconstruction conference on backfilling shall be required only when specified in the contract document or required by the Engineer. Shape control considerations should be similar to those needed for a metal culvert.

Structure backfill material shall be placed in uniform, horizontal layers not exceeding an 8-in. maximum loose lift thickness and compacted to a density not less than 90 percent modified density per AASHTO T 180. The structure backfill shall be constructed to the minimum lines and grades shown in the contract documents, keeping it at or below the level of the adjacent soil or embankment.

C26.5.4.4

Metal box culverts are not long-span structures because they are relatively stiff, semi-rigid frames.

2011 Revision

26.5.5—Bracing

When required, temporary bracing shall be installed and shall remain in place as long as necessary to protect workers and to maintain structure shape during erection.

For long-span structures which require temporary bracing or cabling to maintain the structure in shape, the supports shall not be removed until the structure backfill is placed to an elevation to provide the necessary support. In no case shall internal braces be left in place when backfilling reaches the top quadrant of the pipe or the top radius arc portion of a long-span structure.

26.5.6—Arch Substructures and Headwalls

Substructures and headwalls shall be designed in accordance with the applicable requirements of *AASHTO LRFD Bridge Design Specifications*.

The ends of the corrugated metal arch shall rest in a keyway formed into continuous concrete footings, or shall rest on a metal bearing surface, usually an angle or channel shape, which is securely anchored to or embedded in the concrete footing.

When specified, the metal bearing may be a hot-rolled or cold-formed galvanized steel angle or channel, or an extruded aluminum angle or channel. These shapes shall be not less than 0.1875 in. in thickness and shall be securely anchored to the footing at a maximum spacing of 24.0 in. When the metal bearing member is not completely embedded in a keyway in the footing, one vertical leg shall be punched to allow the end of the corrugated plates to be bolted to this leg of the bearing member.

Where an invert slab is provided which is not integral with the arch footing, the invert slab shall be continuously reinforced.

26.5.7—Inspection Requirements for CMP 2012 Revision

26.5.7.1—Visual Inspection 2012 Revision

CMP shall be inspected after placement in the trench, as required during backfilling, and after completion of installation to ensure that final installation conditions allow the pipe to perform as designed. Installation of bedding and backfill materials, as well as their placement and compaction, shall be determined to meet the requirements of this Section.

During the initial phase of the installation process, inspection shall concentrate on detecting improper practice and poor workmanship. Errors in line and grade, as well as any improper assembly or backfill techniques, shall be corrected prior to placing significant backfill or trench fill. Coupling bands shall be properly indexed with the corrugation and tightened, and bell/spigot joints shall be properly seated to prevent the infiltration of soil fines. Where gaskets are used, they shall not bulge or hang into the pipe and, if visible, should appear uniformly oriented around the pipe.

C26.5.7.1

See Article 14.2, "Inspection," of *Highway Drainage Guidelines*.

Inspections at the appropriate times during installation allow corrections to be made in assembly and backfill practices. The timing and number of visual inspections depend on the significance of the structure and its cover depth. Construction inspection during early stages of the project will allow the contractor to evaluate and, if necessary, modify construction and quality control practices.

Deeply buried structures perform closer to their full, allowable strength level. Where the depth of cover will be significant, it is especially important to detect any problems before the pipe is buried to a depth where repair will be difficult or expensive.

Soil consolidation continues with time after installation of the pipe. While 30 days will not encompass the time frame for complete consolidation of the soil surrounding the pipe, it is intended to give sufficient time to

Racking or denting of the pipe shall be taken to indicate improper backfill placement. At the contractor's expense, pipe sections damaged during installation shall be evaluated by a Professional Engineer and when directed, that section of the pipe shall be repaired or replaced.

Coated pipes shall be inspected to ensure the coating has no cracks, scratches, or locations of peeling. Coatings shall be repaired in accordance with material specification requirements.

Final internal inspections shall be conducted on all buried CMP installations to evaluate issues that may affect long-term performance. Final inspections shall be conducted no sooner than 30 days after completion of installation and final fill.

The inspection will verify that bedding, backfill, and compaction requirements are followed during installation. The pipe shall be checked for alignment, joint separation, cracking at bolt holes, localized distortions, bulging, flattening, or racking. Shallow cover installations shall be checked to ensure the minimum cover level is provided and inspected prior to and immediately after vehicular load is applied.

observe some of the effects that this consolidation will have. However, occasionally pavement is placed over the pipe sooner than 30 days. While the 30-day time limit should be maintained, a brief inspection of the pipe prior to paving over it, particularly for the first few joints, may be prudent to ensure that good construction practices are being applied.

It is recommended that inspection personnel not enter culverts less than 24.0 in. in diameter. Internal inspection of culverts in this size range is best conducted using video cameras. Culverts should only be entered by inspection personnel trained in working within confined spaces and using procedures in full compliance with applicable Local, State, and Federal OSHA regulations.

Racking, or loss of symmetry, is structurally important in larger pipes because a flattened area is formed on one side of the crown as the top centerline is racked to the opposite side. Differential shape changes at the joint, or joint separation may allow exfiltration or infiltration resulting in erosion of the backfill material.

Slight peaking of the cross-sectional shape should be taken as indicative of achieving or exceeding minimum compaction requirements.

26.5.7.2—Installation Deflection 2012 Revision

The pipe shall be evaluated to determine whether the internal diameter of the barrel has been reduced more than the limits set forth in this Article when measured not less than 30 days following completion of installation.

Because of their broad diameter tolerances, metal pipes 24 in. in diameter and smaller typically are not deflection tested. A visual inspection should be performed to check for denting or other damage using a video camera or other means. If deflection testing is required by the owner or the visual inspection indicates excessive deflection, a device approved by the Engineer that can physically verify the dimensions of the pipe and is not limited by poor lighting, waterflow, pipe length, or other limiting conditions of the installed environment shall be used. If deflection testing is performed, deflection for metal pipes 24 in. in diameter and smaller shall not exceed 7.5 percent of the nominal diameter of the pipe plus a manufacturing tolerance as determined to be appropriate by the owner.

Pipes larger than 24 in. may be entered and deflection levels measured directly. In lieu of direct measurements, a calibrated video camera or any other device approved by the Engineer that can physically verify the dimensions of the pipe and is not limited by poor lighting, waterflow, pipe length, or other limiting conditions of the installed environment may be used.

In all installations of pipes larger than 24 in. in diameter, at least ten percent of the total number of pipe runs representing at least ten percent of the total pipe footage on the project shall be randomly selected by the Engineer and inspected for deflection. Also, as determined by the 100 percent visual inspection in Article 26.5.7.1, all areas in which deflection can be visually detected shall be inspected for deflection.

C26.5.7.2

Ten percent of each pipe installation shall be defined as ten percent of the number of pipe runs, and not less than ten percent of the total length of installed pipe on the project. The requirement of deflection testing ten percent of each pipe installation is intended to serve as a minimum and does not limit owners from more stringent requirements.

There are many appropriate methods for measuring deflection, including video inspection equipment and direct measurement. Whichever method is used for deflection measurement, a minimum of ten percent of the total length of installed pipe shall be tested, in addition to any areas that were identified in the visual inspection as having deflection.

The deflection limits provided are similar to the deflection criteria for other flexible pipes in these Specifications. These limits do not necessarily reflect the capability of the pipe, but were chosen as limits at which the installation indicates poor workmanship that needs to be corrected to prevent future maintenance problems. To prevent owners from having to measure every single pipe to establish base dimensions, deflection measurements shall be based on nominal pipe dimensions. Manufacturing tolerances per AASHTO M 36 for individual products were added to a base deflection limit of 7.5 percent to arrive at the limits as defined in this Article.

Where direct measurements are made, a measurement shall be taken once every 10 ft. for the length of the pipe, and a minimum of four measurements per pipe installation is required.

Pipes larger than 24 in. in diameter should be evaluated by direct measurement. Deflection shall be determined by comparing span and rise measurements with the nominal pipe diameter. Vertical deflection, as a percentage, shall be expressed as: 100((rise/diameter) – 1.0). Similarly, horizontal deflection shall be expressed as: 100((span/diameter) – 1).

For all round pipes larger than 24 in. in diameter, including round and single radius arch structural plate, deflections exceeding 7.5 percent the nominal diameter of the pipe plus the manufacturing tolerance of either one percent of the nominal diameter or 0.5 in., whichever is greater, shall be considered as indicative of poor backfill materials, poor workmanship or both. These pipes shall require remediation or replacement. Passing deflection criterion shall not eliminate the need to evaluate associated denting, racking or other shape damage.

For pipe arches, deflections resulting in a decrease in rise or increase in span exceeding 7.5 percent shall be considered indicative of poor backfill materials, poor workmanship, or both. These pipes shall require remediation or replacement. Passing deflection criterion shall not eliminate the need to evaluate associated denting, racking or other shape damage.

Structural plate structures should be inspected by direct measurement. They shall be assembled in accordance with the shape tolerances of Article 26.4.3. Immediately after backfilling, the structure shall be measured to check for any immediate deflections that occurred during the backfilling operation. After 30 days, the structure shall be measured again to check for any additional deflection. All deflection measurements shall be based on design dimensions. For multiple radius structures such as ellipses, pipe-arches, and low profile and high profile arches, the crown (top) radius shall not increase by more than ten percent of the design radius as calculated from the measured middle ordinate off a suitable length straight edge. If the top radius exceeds the design value by more than ten percent or if the structure is racked or unsymmetrical by more than two percent, it shall require remediation. The degree of racking or loss of symmetry shall be determined by dropping a plumb line from the actual top centerline of the installed structure and measuring the half spans that exist on each side of the plumb line to the maximum span line. For a symmetrical structure, these measurements at each individual cross-section should be equal. The degree of racking or loss of symmetry shall be expressed as a percentage: 100 ((half span A – half span B)/span) < 2.

Due to the broad diameter tolerances on small diameter metal pipes, it is difficult to perform deflection testing as the pass/fail criterion are often inaccurate. However they do need to be checked for denting and other damage. A thorough visual inspection for these pipes is recommended rather than deflection testing. If owners choose to deflection test small diameter pipes, they should keep in mind that the tolerances for round metal pipe are ±1 percent or ±0.5 in., whichever is greater. Manufacturing processes use these tolerances and the diameter often varies within the pipe. Especially in smaller pipes, this is significant to any perceived deflection. A –0.5 in. tolerance in a 12-in. pipe itself amounts to 4.2 percent of the diameter. This manufacturing tolerance, if it is not taken into account, can result in the acceptance of poorly installed pipe or the rejection of well installed pipe. Alternatively, owners who choose to deflection test pipes 24.0 in. and less in diameter can require a manufacturer's certification of the mean diameter, and deflection test based on that data.

According to the tolerance limits established in AASHTO M 36 and AASHTO M 196, the tolerance for the rise in a pipe arch can vary greatly to the positive, but is zero for negative tolerance. Additionally, the tolerance for span in a pipe arch is zero to the positive and can vary greatly to the negative. As such, the threshold criterion for deflection has been set at a 7.5 percent decrease for rise and at a 7.5 percent increase for span. This eliminates the large tolerances for pipe arches as a factor in checking for deflection.

For structural plate structures, a ten percent increase in crown radius does not indicate a ten percent change in rise. Depending on the shape, related rise deflections are more typically five percent or less. Since there are nearly an infinite number of possible design shapes for structural plate, the dimension change limits are compared to base dimensions shown on the working drawings for that particular structure. Measurements should be taken immediately after installation and backfilling, as well as after 30 days, so that corrective measures can be taken if necessary before additional construction over the structure is completed.

2012 Revision

26.6—CONSTRUCTION PRECAUTIONS

The structures covered by this section shall be investigated for all critical stages in their installation and in the final intended purpose. For construction loads, additional cover may be required beyond that required in the final condition to which the design loads apply. In the absence of more specific information, the cover depths in Table 26.6-1 may be considered for the smaller structures indicated. The minimum covers indicated should be increased when site conditions so indicate. The Engineer or the Manufacturer shall provide guidance for structure spans or axle loads not listed.

C26.6

These structures can support the design loads once the backfill is placed and compacted to the minimum cover level over the pipe, as defined in Section 12, "Buried Structures and Tunnel Liners," *AASHTO LRFD Bridge Design Specifications*.

Table 26.6-1—Minimum Cover for Construction Loads on Circular, Pipe-Arch, Elliptic, and Underpass Shapes

	Minimum Cover, ft, for Indicated Axle Loads, kips			
Pipe Span, in.	18.0–50.0	50.0–75.0	75.0–110.0	110.0–150.0
12.0–42.0	2.0	2.5	3.0	3.0
48.0–72.0	3.0	3.0	3.5	4.0
78.0–120.0	3.0	3.5	4.0	4.0
126.0–144.0	3.5	4.0	4.5	4.5

The Contractor shall provide any additional cover required to avoid damage to the pipe. Minimum cover shall be measured from the top of the pipe to the top of the maintained construction roadway surface.

The surface shall be maintained to provide adequate cover until paving is completed, or until the project is accepted by the Owner if paving is not required.

26.7—MEASUREMENT

Corrugated metal and structural plate pipe, pipe-arches, arches, and box culverts shall be measured in linear feet installed in place, completed and accepted. The number of linear feet shall be the average of the top and bottom centerline lengths for pipe, the bottom centerline length for pipe arches and box culverts, and the average of springline lengths for arches.

26.8—PAYMENT

Separate pay items or provision for including excavation, backfill, and concrete for arches shall be provided for in the contract documents.

The lengths as measured above shall be paid for at the contract prices per linear foot bid for corrugated metal and structural plate pipe, pipe-arch, arch, or box culvert of the sizes specified in the contract documents. Such price and payment shall constitute full compensation for furnishing, handling, erecting, inspecting shape control, and installing the pipe, pipe-arches, arches, or box culverts, and for all materials, labor, equipment, tools, and incidentals necessary to complete this item. Such price and payment shall also include the cost of excavation, bedding material, backfill, concrete headwalls, endwalls, and foundations for pipe, pipe-arches, and box culverts. Separate payment will be made for excavation, backfill, concrete or masonry headwalls, and foundations for arches.

26.9—REFERENCES

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2007. *Highway Drainage Guidelines*, Fourth Edition, HDG-4, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2009. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 29th Edition, HM-29, American Association of State Highway and Transportation Officials, Washington, DC. Includes AASHTO M, R, and T standards, which are also available individually in downloadable form.

AASHTO and AWS. 2008. AASHTO/AWS D1.5M/D1.5:2008 *Bridge Welding Code*, Fifth Edition, BWC-5, American Welding Society, Miami, FL.

AWS. 2003. ANSI/AWS D1.2/D1.2M *Structural Welding Code—Aluminum*, American Welding Society, Miami, FL.

Back to 2010 Edition

SECTION 26: METAL CULVERTS

26.4.3—Assembly of Long-Span Structures

Revise paragraph 2 of this Article as follows:

When required by structural design, reinforcing ribs shall be attached to the structural plate corrugation ~~crown~~ crest at the necessary locations around the circumference of the structure prior to backfilling, using a bolt spacing of not more than 12.0 in. or 16.0 in. for deep corrugated structural plate structures. Legible identifying letters or numbers shall be placed on each rib to designate its proper position in the finished structure.

C26.4.3

In this Article, add the following paragraph vertically aligned with paragraph 2 of 26.4.3:

It is acceptable to measure bolt spacing either at the centroid or crest of the structural plate corrugation.

In this Article, add the following paragraph vertically aligned with the last paragraph of 26.4.3:

Where reinforcing ribs are only used as an installation tool during backfill, they may be clamped or bolted as necessary to provide shape control.

Insert new Article 26.5.4.5

26.5.4.5—Deep Corrugated Structural Plate Structures Back to 2010 Edition

Prior to construction, the Manufacturer shall conduct a preconstruction conference to advise the Contractor(s) and Engineer of the more critical functions to be performed during backfilling and to present the intended quality control steps to be used to control loads, shape, and movements.

Equipment and construction procedures used to backfill deep corrugated structural plate structures shall be such that excessive structure distortion will not occur. A Manufacturer's representative shall be on site during initial sidefill placement and compaction, and shall review data on the shape when backfill reaches the minimum cover level over the top of the structure as set forth in the *AASHTO LRFD Bridge Design Specifications*. Structure backfill material shall be placed in horizontal uniform layers not exceeding an 8.0 in. loose lift thickness and shall be brought up uniformly on both sides of the structure. Each layer shall be compacted per the contract documents, but not less than 90 percent maximum density per AASHTO T 180 (modified Proctor test). The structure backfill shall be constructed to the lines and grades shown on the contract documents, keeping it at or below the level of adjacent soil.

SECTION 26: METAL CULVERTS

26.1.1—Description

Revise sentence 1 of this Article as follows:

This work shall consist of furnishing, fabricating, installing, and inspecting metal pipe, structural plate metal pipe, arches, pipe arches, box structures, ~~and~~ deep corrugated structures, and steel-reinforced thermoplastic pipe in conformance with these Specifications and the details shown in the contract documents.

Page left intentionally blank.

26.3.8.1—General

Revise sentence 2 of this Article as follows:

Backfill for metal and steel-reinforced thermoplastic culverts shall be granular material, as specified in the contract documents and specifications, and shall be free of organic material, rock fragments larger than 3.0 in. (1.5 in. for steel-reinforced thermoplastic pipe) in the greatest dimension, and frozen lumps, and shall have a moisture content within the limits required for compaction.

Page left intentionally blank.

Insert new Article 26.3.9:

26.3.9—Steel-Reinforced Thermoplastic Culverts

Steel-reinforced thermoplastic culverts shall conform to the requirements of AASHTO MP 20.

26.4.1—General

Revise sentence 1 in paragraph 1 of this Article as follows:

Corrugated metal pipe, ~~and~~ structural plate, and steel-reinforced thermoplastic culvert shall be assembled in accordance with the Manufacturer's instructions.

26.4.2—Joints Back to 2010 Edition

Revise the Article as follows:

Joints for corrugated metal culvert and drainage pipe shall meet the ~~following~~ performance requirements specified in Articles 26.4.2.1 through 26.4.2.4.

Joints for steel-reinforced thermoplastic culverts shall meet the requirements of Article 26.4.2.5.

2012 Revision

Page left intentionally blank.

Insert new Article 26.4.2.5:

26.4.2.5—Steel-Reinforced Thermoplastic Culvert Joints

Joints for steel-reinforced thermoplastic pipe shall comply with the details shown in the contract drawings and on the approved working drawings. Each joint shall be sealed to prevent infiltration of soil (soiltight), fines (silttight), or water (watertight) as required by the contract documents. Field tests may be required by the Engineer whenever there is a question regarding compliance with the contract requirements.

Joints shall be installed so that the connection of pipe sections will form a continuous surface free from irregularities in the flow line.

Insert new commentary to this Article:

C26.4.2.5

Joint types include bands, bell-and-spigot pipe ends, double-bell couplings, and fusion-welded joints. Joints may or may not require gaskets. Other joint types may be used provided that documentation is submitted to demonstrate that the joint meets the project requirements.

Joints are often provided as soiltight or watertight. Definitions of soiltight and silttight are vague. Examples can be found elsewhere in this Section. Watertight joints are normally specified to meet ASTM D3212. Pressure capability of joints shall be based on project requirements. Commonly available pressure capabilities are 2, 5, and 10 psi.

2012
Revision

Page left intentionally blank.

26.5.1—General

Add the following paragraph at the end of this Article:

All pipe laying, joining, and backfilling shall be in accordance with the strictest of the following requirements: the Manufacturer's instructions, the contract documents, or these Specifications.

Page left intentionally blank.

Back to 2010 Edition

Revise the title of Article 26.5.7 as follows:

26.5.7—Inspection Requirements ~~for CMP~~

26.5.7.1—Visual Inspection

Revise paragraph 1, sentence 1 of this Article as follows:

~~CMP~~All culverts shall be inspected after placement in the trench, as required during backfilling, and after completion of installation to ensure that final installation conditions allow the pipe to perform as designed.

Revise paragraph 5, sentence 1 of this Article as follows:

Final internal inspections shall be conducted on all buried ~~CMP~~ installations to evaluate issues that may affect long-term performance.

Add the following paragraph to the end of this Article:

Steel reinforced thermoplastic pipe shall be inspected prior to backfilling to ensure that the external reinforcing ribs do not lean an average of more than 15 degrees off of vertical within any 24.0-in. long continuous section of the pipe. If this condition exists, that portion of the pipe shall be remediated or replaced.

26.5.7.2—Installation Deflection

Back to 2010 Edition

Add the following heading over all of the current text in this Article:

26.5.7.2.1—Installation Deflection for CMP and Structural Plate Structures

Back to 2010 Edition

Revise the commentary heading as follows:

C26.5.7.2.1

Insert new Article 26.5.7.2.2:

26.5.7.2.2—Installation Deflection for Steel-Reinforced Thermoplastic Culverts

The pipe shall be evaluated to determine whether the internal diameter of the barrel has been reduced more than five percent when measured not less than 30 days following completion of installation.

Pipes shall be checked for deflection using a mandrel or any other device approved by the Engineer that can physically verify the dimensions of the pipe and that is not limited by poor lighting, waterflow, pipe length, or other limiting conditions of the installed environment. Pipes larger than 24.0 in. may be entered and deflection levels measured directly.

In all pipe installations, at least ten percent of the total pipe footage on the project shall be randomly selected by the Engineer and inspected for deflection. Also, as determined by the 100 percent visual inspection in Article 26.5.7.1, all areas in which deflection can be visually detected shall be inspected for deflection.

Where direct measurements are made, a measurement shall be taken once every 10.0 ft for the length of the pipe and a minimum of four measurements per pipe installation are required.

If a mandrel is used for the deflection test, it shall be a nine- (or greater odd number) arm mandrel and shall be sized and inspected by the Engineer prior to testing. A properly sized proving ring shall be used to check or test the mandrel for accuracy. The mandrel shall be pulled through the pipe by hand with a rope or cable. Where applicable, pulleys may be incorporated in the system to change the direction of pull so that inspection personnel need not physically enter the pipe or manhole.

For locations where pipe deflection exceeds five percent of the inside diameter, an evaluation shall be conducted by the Contractor utilizing a Professional Engineer and submitted to the Engineer for review and approval, taking into consideration the severity of the deflection, structural integrity, environmental conditions, and the design service life of the pipe. Pipe remediation or replacement shall be required for locations where the evaluation finds that the deflection could be problematic. For locations where pipe deflection exceeds 7.5 percent of the inside diameter, remediation or replacement of the pipe is required.

Insert new commentary to this Article:

C26.5.7.2.2

Ten percent of each pipe installation shall be defined as ten percent of the number of pipe runs, and not less than ten percent of the total length of installed pipe on the project. The ten percent deflection testing requirement for each pipe installation is intended to serve as a minimum and it does not limit owners from more stringent requirements.

The pipe inside diameters should be provided by the pipe manufacturer for every size and type of pipe delivered. If the pipe inside diameter is not provided or is not available, pipe inside diameter can be developed by averaging the diameters measured at eight equally spaced locations around a section of unloaded pipe for every given size and manufacturer.

There are many appropriate methods suitable for measuring deflection including video inspection equipment, mandrels, and other direct measurement devices. For pipes tested by a mandrel, the mandrel shall be pulled through the entire pipe. Whichever method is used for deflection measurement, a minimum of ten percent of the total length of installed pipe shall be tested, in addition to any areas that were identified in the visual inspection as having deflection.

Installed pipe deflections that exceed five percent of the initial inside diameter may indicate that the installation was substandard. Appropriate remediation, if any, will depend on the severity of the deflection, the condition of the pipe, and evaluation of the factor of safety using Section 12, "Buried Structures and Tunnel Liners," of the *AASHTO LRFD Bridge Design Specifications*. Installed pipe deflections that exceed 7.5 percent of the initial inside diameter will require remediation or replacement of the pipe.

2012
Revision

SECTION 27: CONCRETE CULVERTS

TABLE OF CONTENTS

SECTION 27

CONCRETE CULVERTS

27.1—GENERAL

This work shall consist of fabricating, furnishing, installing, and inspecting buried precast concrete culverts conforming to these Specifications, Section 12 of the *AASHTO LRFD Bridge Design Specifications*, and the details shown in the contract documents. Precast reinforced concrete pipe shall be circular, arch, or elliptical, as specified in the contract documents. Precast reinforced concrete box sections shall be of the dimensions specified in the contract documents.

C27.1

The concrete culvert criteria refers to Section 12, "Buried Structures and Tunnel Liners," of the *AASHTO LRFD Bridge Design Specifications*, 2007.

27.2—WORKING DRAWINGS

When complete details are not provided in the contract documents or the standard details/specifications, the Contractor shall submit working drawings and substantiating calculations of the proposed structure or installation system. Fabrication or installation of the structure shall not begin until the Engineer has approved the drawings. The working drawings shall show complete details of the structure, the materials, equipment, and installation methods proposed.

Working drawings shall be submitted in advance of the start of the work to allow for their review, revision, and approval without delay to the work. Approval by the Engineer shall not relieve the Contractor of any contractual responsibility.

27.3—MATERIALS

27.3.1—Reinforced Concrete Culverts

The materials for reinforced concrete culverts shall meet the requirements of the classes and sizes specified in Table 27.3.1-1.

Table 27.3.1-1—Reinforced Concrete Culvert Specification Requirements

Culvert Type	Specification
Circular Pipe	AASHTO M 170 (ASTM C76) or AASHTO M 242 (ASTM C655)
Arch Pipe	AASHTO M 206M/M 206 (ASTM C506)
Elliptical Pipe	AASHTO M 207M/M 207 (ASTM C507)
Box Sections	AASHTO M 259 and AASHTO M 273 or ASTM C1433

27.3.2—Surface Finish

Defects that indicate proportioning, mixing, and molding not in compliance with the specification, or surface defects indicating honeycombed or open texture that would adversely affect the function of the pipe, shall be repaired or the pipe replaced.

27.3.3—Joint Sealants

27.3.3.1—General

The Contractor shall furnish to the Engineer a certificate of compliance stating that the material being furnished conforms to the joint property requirements.

27.3.3.2—Cement Mortar

Mortar shall be composed of one part Portland Cement and two parts sand by volume. Sand shall be well graded and of such size that all will pass a No. 8 (2.36-mm) sieve. The materials shall be mixed to a consistency suitable for the purpose intended and used within 30 min after the mixing water has been added. Admixtures, if any, shall be approved by the Engineer prior to use.

27.3.3.3—Flexible Watertight Gaskets

Flexible watertight gasketed joints shall conform to the requirements of AASHTO M 198 (ASTM C990) or AASHTO M 315 (ASTM C433) and shall be flexible and capable of withstanding expansion, contraction, and settlement of the pipeline.

All rubber gaskets shall be stored in as cool a place as practicable, preferably at 70°F or less.

Rubber gaskets, of the type requiring lubrication, shall be lubricated with the lubricant recommended and supplied by the Manufacturer of the pipe.

27.3.3.4—Other Joint Sealant Materials

Other joint sealant materials shall be submitted for testing in advance of their use and shall not be used prior to receiving approval from the Engineer.

27.3.4—Bedding Material and Backfill

The provisions of Article 27.5 shall apply.

27.4—ASSEMBLY

27.4.1—General

Precast concrete units or elements shall be assembled in accordance with the Manufacturer's instructions. All units or elements shall be handled with reasonable care and shall not be rolled or dragged over gravel or rock. Care shall be taken to prevent the units from striking rock or other hard objects during placement.

27.4.2—Joints

Joints for reinforced concrete pipe and precast reinforced concrete box sections shall comply with the details shown in the contract documents and on the approved working drawings. Each joint shall be sealed to prevent infiltration of soil fines or water as required by the contract documents. Field tests may be required by the Engineer whenever there is a question regarding compliance with contract requirements.

27.5—INSTALLATION

27.5.1—General

For trench conditions, the trench shall be excavated to the dimensions and grade specified in the contract documents or ordered by the Engineer. The Contractor shall make such provisions as required to insure adequate drainage of the trench to protect the bedding during construction operations.

Proper preparation of foundation, placement of foundation material where required, and placement of bedding material shall precede the installation of the culvert. This work shall include necessary leveling of the native trench bottom or the top of foundation materials as well as placement and compaction of required bedding material to a uniform grade so that the entire length of pipe shall be supported on a uniformly slightly yield bedding. The backfill material shall be placed and compacted around the culvert in a manner to meet the requirements specified.

Materials used for foundation improvements, bedding, and structure backfill shall have gradations compatible with adjacent soils to avoid migration. Where material gradations cannot be properly controlled, adjacent materials shall be separated with a suitable geotextile.

C27.5.1

See ASTM C1479, Standard Practice for Installation of Precast Concrete Sewer, Storm Drain, and Culvert Pipe Using Standard Installations, for additional information.

27.5.2—Foundation Bedding and Backfill

27.5.2.1—General

If rock strata or boulders are encountered under the culvert within the limits of the required bedding, the rock or boulders shall be removed and replaced with bedding material. The foundation shall be comprised of stiff to hard in-situ soil, stabilized soil, or compacted fill material. If the

C27.5.2.1

If the foundation is rock or other unyielding soil, the minimum bedding of 6.0 in. may not be adequate for deep fills. A minimum of 2 percent of the fill height should be considered.

foundation is rock or other unyielding soil, then bedding shall be 6.0 in. as a minimum. Where, in the opinion of the Engineer, the natural foundation soil is such as to require stabilization, such material shall be replaced by a layer of bedding material. Where an unstable and/or unsuitable material (e.g., peat or muck) is encountered at or below invert elevation during excavation, the necessary subsurface exploration and analysis shall be made and corrective treatment shall be as directed by the Engineer.

Where rock or other unyielding foundations exist and blasting is anticipated, the Engineer may require that suitable steps be taken to protect the pipe.

27.5.2.2—Precast Reinforced Concrete Circular Arch and Elliptical Pipe

A bedding of a class, specified in the contract documents or on the working drawings, shall be provided. The type of installation specified shall conform to:

- one of the types of installation given in Tables 27.5.2.2-1 and 27.5.2.2-2, and
- the details shown in Figures 27.5.2.2-1 through 27.5.2.2-4 which defines soil areas and critical dimensions for circular, arch, and elliptical pipes.

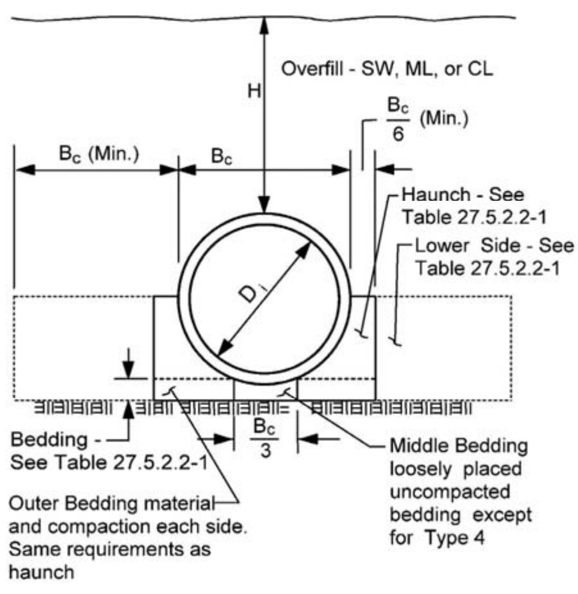


Figure 27.5.2.2-1—Standard Embankment Installation—Round Pipe

C27.5.2.2

Tables 27.5.2.2-1 and 27.5.2.2-2 list general soil types and minimum compaction requirements, and minimum bedding thicknesses for the four standard installation types for embankment and trench installations, respectively.

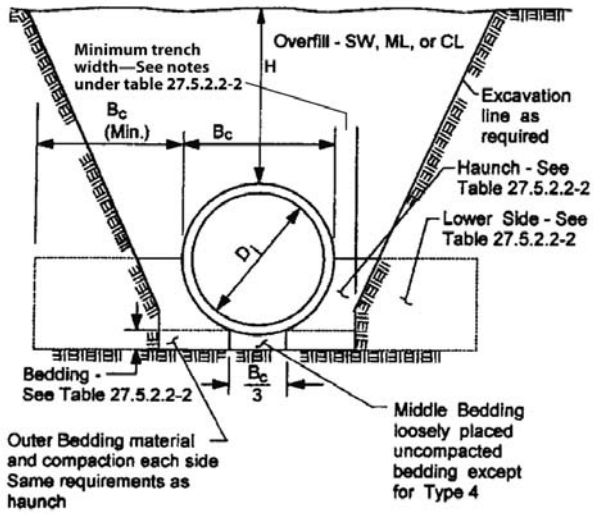


Figure 27.5.2.2-2—Standard Trench Installation—Round Pipe

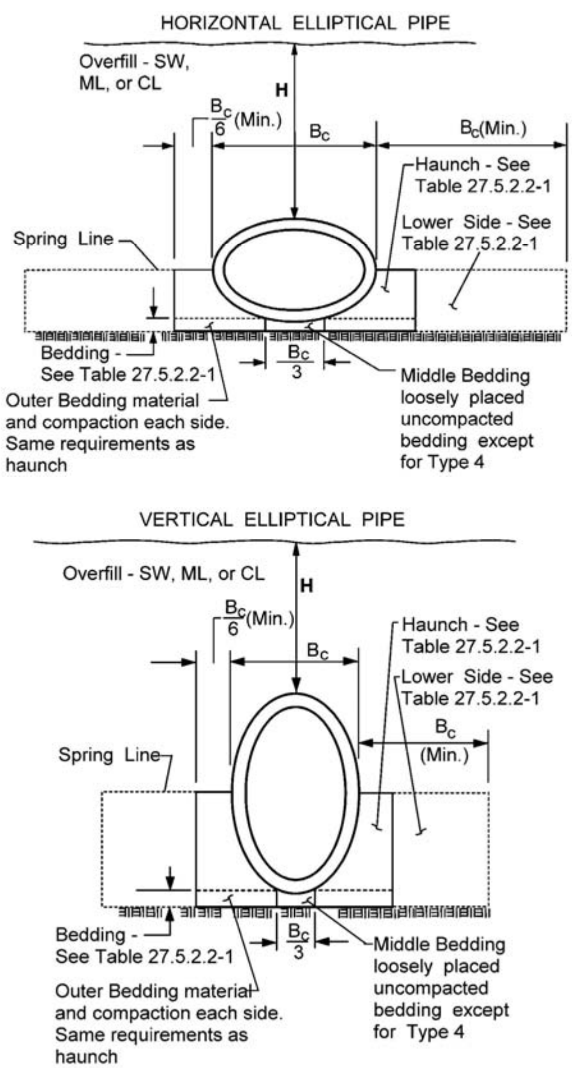


Figure 27.5.2.2-3—Embankment Beddings—Miscellaneous Shapes

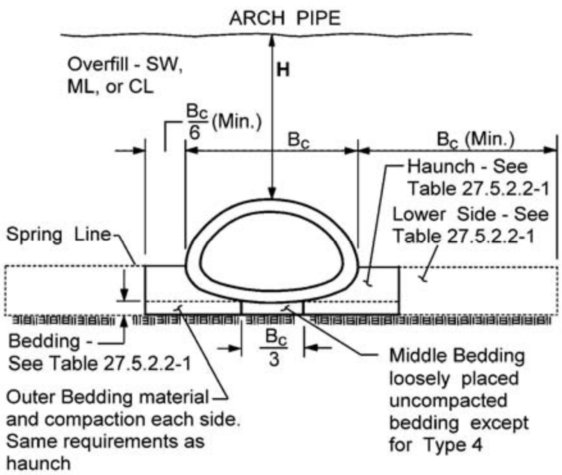
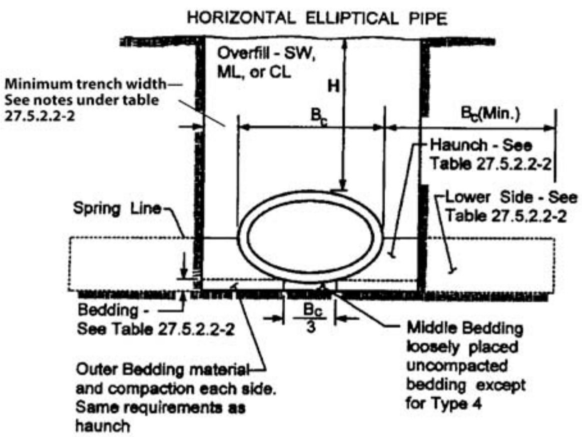


Figure 27.5.2.2-3 (continued)—Embankment Beddings—Miscellaneous Shapes



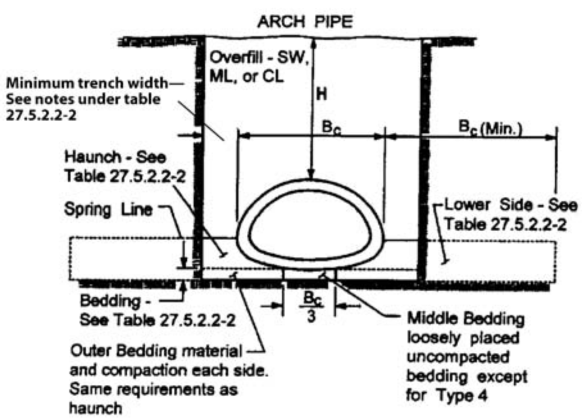


Figure 27.5.2.2-4—Trench Beddings—Miscellaneous Shapes

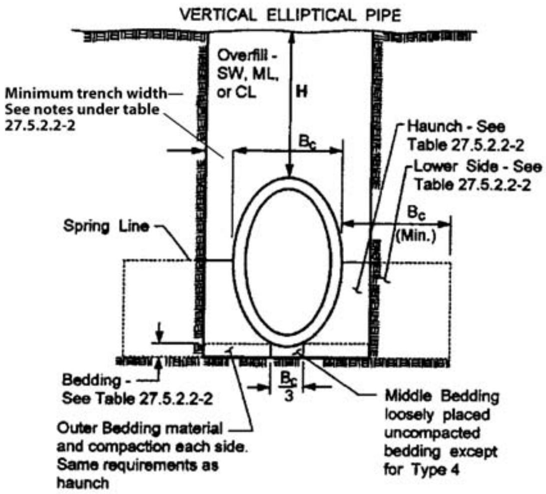


Figure 27.5.2.2-4 (continued)—Trench Beddings—Miscellaneous Shapes

Table 27.5.2.2-1—Standard Embankment Installation Soils and Minimum Compaction Requirements

Installation Type	Bedding Thickness	Haunch and Outer Bedding	Lower Side
Type 1	For soil foundation, B_c/24.0 in. minimum, not less than 3.0 in. For rock foundation, use B_c/12.0 in. minimum, not less than 6.0 in.	95% SW	90% SW, 95% ML or 100% CL
Type 2—Installations are available for horizontal elliptical, vertical elliptical and arch pipe	For soil foundation, B_c/24.0 in. minimum, not less than 3.0 in. For rock foundation, use B_c/12.0 in. minimum, not less than 6.0 in.	90% SW or 95% ML	85% SW, 90% ML or 95% CL
Type 3—Installations are available for horizontal elliptical, vertical elliptical and arch pipe	For soil foundation, B_c/24.0 in. minimum., not less than 3.0 in. For rock foundation, use B_c/12.0 in. minimum, not less than 6.0 in.	85% SW, 90% ML or 95% CL	85% SW, 90% ML or 95% CL
Type 4	For soil foundation, no bedding required. For rock foundation, use B_c/12.0 in. minimum, not less than 6.0 in.	No compaction required, except if CL, use 85% CL	No compaction required, except if CL, use 85% CL

The following interpretations apply to Table 27.5.2.2-1:

- Compaction and soil symbols, i.e., “95 percent SW,” shall be taken to refer to SW soil material with minimum standard proctor compaction of 95 percent. Equivalent modified proctor values shall be as given in Table 27.5.2.2-3.

- Soil in the outer bedding, haunch and lower side zones, except within B_c/3 from the pipe springline, shall be compacted to at least the same compaction as the majority of soil in the overfill zone.

- The minimum width of a subtrench for Type 1 through Type 3 installations shall be B_c + 24.0 in. or 1.33 B_c, whichever is greater, or wider if required for adequate space to attain the specified compaction in the haunch and bedding zones.

- For subtrenches with walls of natural soil, any portion of the lower side zone in the subtrench wall shall be at least as firm as an equivalent soil placed to the compaction requirements specified for the lower side zone and as firm as the majority of soil in the overfill zone, or shall be removed and replaced with soil compacted to the specified level.

- Only Type 2 and 3 installations are available for horizontal elliptical, vertical elliptical, and arch pipe.

- The required bedding thickness is the thickness of the bedding after placement of the pipe on the bedding and is prior to placement of the backfill.

A subtrench is defined as a trench in the natural material under an embankment used to retain bedding material with its top below finished grade by more than ten percent of the depth of soil cover on the top of the culvert or pipe, or, for roadways, its top is at an elevation lower than 1.0 ft below the bottom of the pavement base material.

Table 27.5.2.2-2—Standard Trench Installation Soils and Minimum Compaction Requirements

Installation Type	Bedding Thickness	Haunch and Outer Bedding	Lower Side
Type 1	For soil foundation, B_c/24.0 in. minimum, not less than 3.0 in. For rock foundation, use B_c/12.0 in. minimum., not less than 6.0 in.	95% SW	90% SW, 95% ML or 100% CL, or natural soils of equal firmness
Type 2—Installations are available for horizontal elliptical, vertical elliptical and arch pipe	For soil foundation, B_c/24.0 in. minimum, not less than 3.0 in. For rock foundation, use B_c/12.0 in. minimum, not less than 6.0 in.	90% SW or 95% ML	85% SW, 90% ML, 95% CL, or natural soils of equal firmness
Type 3—Installations are available for horizontal elliptical, vertical elliptical and arch pipe	For soil foundation, B_c/24.0 in. minimum, not less than 3.0 in. For rock foundation, use B_c/12.0 in. minimum, not less than 6.0 in.	85% SW, 90% ML or 95% CL	85% SW, 90% ML or 95% CL, or natural soils of equal firmness
Type 4	For soil foundation, no bedding required. For rock foundation, use B_c/12.0 in. minimum, not less than 6.0 in.	No compaction required, except if CL, use 85% CL	85% SW, 90% ML, 95% CL, or natural soils of equal firmness

The following interpretations apply to Table 27.5.2.2-2:

- Compaction and soil symbols, i.e., “95 percent SW,” shall be taken to refer to SW soil material with minimum standard proctor compaction of 95 percent. Equivalent modified proctor values shall be as given in Table 27.5.2.2-3.

- The trench top elevation shall be no lower than $0.1H$ below finish grade, or for roadways, its top shall be no lower than an elevation of 1.0 ft below the bottom of the pavement base material.

- Soil in bedding and haunch zones shall be compacted to at least the same compaction as specified for the majority of soil in the backfill zone.

- The minimum trench width for Type 1 through Type 3 installations shall be B_c + 24.0 in. or 1.33 B_c, whichever is greater, or wider if required for adequate space to attain the specified compaction in the haunch and bedding zones.

- For trench walls that are within ten degrees of vertical, the compaction or firmness of the soil in the trench walls and lower side zone need not be considered.

- For trench walls with greater than ten-degree slopes that consist of embankment, the lower side shall be compacted to at least the same compaction as specified for the soil in the backfill zone.

- Only Type 2 and 3 installations are available for horizontal elliptical, vertical elliptical, and arch pipe.

- The required bedding thickness is the thickness of the bedding after placement of the pipe on the bedding and prior to placement of the backfill.

The AASHTO M 145, ASTM D2487, and USCS soil classifications equivalent to the generic soil types in the standard installations may be taken from Table 27.5.2.2-3.

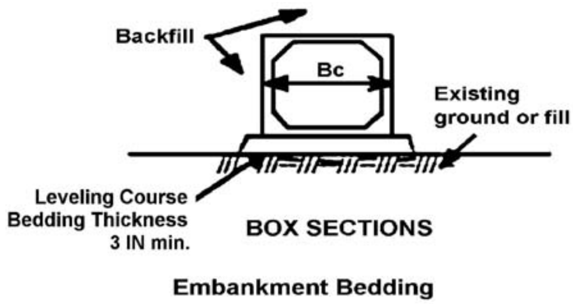
Table 27.5.2.2-3—Equivalent USCS and AASHTO Soil Classifications for SIDD Soil Designations

SIDD Soil	Representative Soil Types: USCS	Representative Soil Types: AASHTO	Percent Compaction: Standard Proctor	Percent Compaction: Modified Proctor
Gravelly Sand (SW)	SW, SP, GW, GP	A1, A3	100 95 90 85 80 61	95 90 85 80 75 59
Sandy Silt (ML)	GM, SM, ML Also GC, SC with less than 20% passing a No. 200 sieve	A2, A4	100 95 90 85 80 49	95 90 85 80 75 46
Silty Clay (CL)	GL, MH, GC, SC	A5, A6	100 95 90 85 80 45	90 85 80 75 70 40
	CH Not allowed for haunch or bedding	A7	100 95 90 45	90 85 80 40

27.5.2.3—Precast Reinforced Concrete Box Sections

Unless otherwise permitted herein, a bedding shall be provided for the type of installation specified conforming to Figure 27.5.2.3-1, unless in the opinion of the Engineer, the natural soil provides a suitable bedding.

Bedding material may be granular material, all of which passes a 0.375-in. sieve and not more than ten percent of which passes a No. 200 (75-µm) sieve. Backfill shall be select material and shall be free of organic material, rock fragments larger than 3.0 in. in the greatest dimension, frozen lumps, and shall have a moisture content within the units required for compaction.



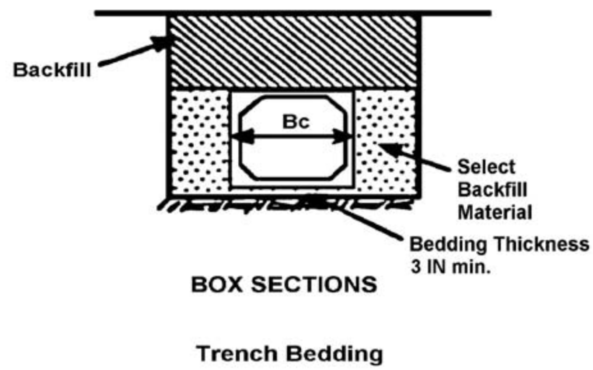


Figure 27.5.2.3-1—Bedding and Backfill Requirements

27.5.3—Placing Culvert Sections

Unless otherwise authorized by the Engineer, the laying of culvert sections on the prepared bedding shall be started at the outlet and with the bell end pointing upstream and the spigot or tongue end pointing downstream and shall proceed toward the inlet end with the abutting sections properly matched, true to the established lines and grades. Where pipe with bells is installed, bell holes shall be excavated in the bedding to such dimensions that the entire length of the barrel of the pipe will be supported by the bedding when properly installed as shown in Figure 27.5.3-1. Proper facilities shall be provided for hoisting and lowering the sections of culvert into the trench without disturbing the prepared bedding and the sides of the trench. The ends of the section shall be carefully cleaned before the section is jointed. The section shall be fitted and matched so that when laid in the bed it shall form a smooth, uniform conduit. When elliptical pipe with circular reinforcing or circular pipe with elliptical reinforcing is used, the pipe shall be laid in the trench in such position that the markings "Top" or "Bottom," shall not be more than five degrees from the vertical plane through the longitudinal axis of the pipe. Adjustments in grade by exerting force on the culvert with excavating equipment or by lifting and dropping the culvert shall be prohibited. If the installed culvert section is not on grade after joining, the section shall be completely unjoined, the grade corrected and the section rejoined.

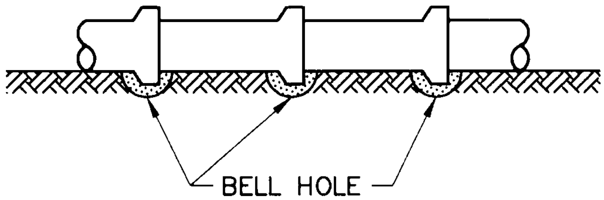


Figure 27.5.3-1—Excavation of Bell Holes for Uniform Support

Multiple installations of reinforced concrete culverts shall be laid with the center lines of individual barrels parallel at the spacing shown in the contract documents. Pipe and box sections used in parallel installations require positive lateral bearing between the sides of adjacent pipe or box sections. Compacted earth fill, granular backfill, or grouting between the units are considered means of providing positive bearing.

27.5.4—Haunch, Lower Side, and Backfill or Overfill

27.5.4.1—Precast Reinforced Concrete Circular Arch and Elliptical Pipe

Haunch material, low side material, and overfill material shall be installed to the limits shown on Figures 27.5.2.2-1 through 27.5.2.2-4.

27.5.4.2—Precast Reinforced Concrete Box Sections

Backfill material shall be installed to the limits shown in Figure 27.5.2.3-1 for the embankment or trench condition.

C27.5.4.2

Although usually constructed with vertical walls, installation of precast box culverts in trenches with sloping sidewalls has not been a problem.

27.5.4.3—Placing of Haunch, Lower Side, and Backfill or Overfill

Fill material shall be placed in layers with a maximum loose thickness of 8.0 in. and compacted to obtain the required density. The fill material shall be placed and compacted with care under the haunches of the culvert and shall be raised evenly and simultaneously on both sides of the culvert. The width of trench shall be kept to the minimum required for installation of the culvert. Ponding or jetting will be only by the permission of the Engineer.

C27.5.4.3

Generally, compaction of fill material to the required density is dependent on the thickness of the layer of fill being compacted, soil type, soil moisture content, type of compaction equipment, and amount of compactive force and length of time the force is applied.

27.5.4.4—Cover over Culvert During Construction

If the passage of construction equipment over an installed culvert is necessary during project construction, compacted overfill in the form of a ramp shall be constructed to a minimum elevation of 3.0 ft over the top of the culvert or to a height such that the equipment loads on the culvert do not exceed the culvert design strength. In an embankment installation, the overfill shall extend a minimum of one culvert diameter width or 3.0 ft whichever is greater, beyond each side of the culvert to prevent possible lateral displacement of the culvert. If a large volume of construction traffic must cross an installed culvert, the point of crossing shall be changed occasionally to minimize the possibility of lateral displacement.

27.6—FIELD INSPECTION

27.6.1—General

Internal inspections shall be conducted on all buried rigid pipe installations to evaluate issues that may affect long-term performance, such as cracks, joint quality, and alignment. Inspections shall be conducted no sooner than 30 days after completion of installation and final fill.

C27.6.1

See Article 14.2, “Inspection,” of *Highway Drainage Guidelines*.

In order to evaluate the structural integrity of a culvert, it is necessary to perform an internal inspection of the conduit. Soil consolidation continues with time after installation of the pipe. While 30 days will not encompass the time frame for complete consolidation of the soil surrounding the pipe, it is intended to give sufficient time to observe some of the effects that this consolidation will have. However, occasionally pavement is placed over the pipe sooner than 30 days. While the 30-day time limit should be maintained, a brief inspection of the pipe prior to paving over it may be prudent to ensure that good construction practices are being applied.

It is recommended that inspection personnel not enter culverts less than 2.0 ft in diameter. Internal inspection of rigid culverts in this size range is best conducted using video cameras. Culverts should only be entered by inspection personnel trained in working within confined spaces and using procedures in full compliance with applicable State, Local, and Federal OSHA regulations.

27.6.2—Misalignment

Misalignment may be taken to indicate the presence of problems in the supporting soil or contractor grade control. The vertical and horizontal alignment of the culvert barrel shall be checked by sighting along the crown, invert, and sides of the culvert, and by checking for differential movement or settlement at joints between pipe sections. Vertical alignment shall be checked for sagging, faulting, and invert heaving. The inspector shall take into account pipes laid with camber or a grade change. Horizontal alignment shall be checked for straightness or smooth curvature.

C27.6.2

Rigid culverts such as precast concrete pipe do not deflect appreciably before cracking or fracturing. As a result, shape inspections, while important in flexible structures, are of little value in inspecting precast concrete culverts.

Sags, which trap water, may aggravate settlement problems by saturating the supporting soil, or lead to sediment buildup in the pipe reducing conduit efficiency. Improper installation, undermining, or uneven settlement of fill may cause alignment problems.

27.6.3—Joint Defects

Leaking joints may be detected during low flows by visual observation of the joints, by checking around the ends of the culvert for evidence of piping, and if required in the contract, by special testing methods typically employed for sanitary sewers.

Differential movement, cracks, spalling, improper gasket placement, movement or settlement of pipe sections, and leakage shall be noted in the inspection report. Severe joint cracks are similar in significance to separated joints.

Joint separations greater than pipe manufacturer's maximum limit shall be considered significant because they accelerate damage caused by exfiltration and infiltration resulting in the erosion of the backfill material.

Evidence of any soil migration through the joint warrants further investigation to determine the source and if repair or replacement is required.

C27.6.3

Joint defects can range from minor problems to problems that are serious in nature. Typical joint defects include leakage (exfiltration and infiltration), cracks, and joint separation. Exfiltration occurs when leaking joints allow water flowing through the pipe to leak into the supporting material. Many culverts are constructed with joints that are not watertight or with mortar joints that may crack with minor deflection, movement, or settlement of the pipe. Minor leakage may not be a significant problem unless soils are quite erosive. However, if leaking joints contribute to or cause piping, serious misalignment of the culvert or even failure may result.

Joint design criteria should be reviewed for allowance of open joints to perform as subdrains.

Infiltration is the reverse of exfiltration. Many culverts are essentially empty except during peak flows. When the water table is higher than the culvert invert, water may seep into the culvert. This infiltration of water, if it carries fine-grained soil particles from the surrounding backfill, can cause settlement and misalignment problems.

Infiltration may be difficult to detect visually in its early stages although it may be indicated by open joints, staining at the joints on the sides and top of the culvert, deposits of soil in the culvert, or by depressions over the culvert.

Improper handling during installation, improper gasket placement, and movement or settlement of pipe sections may cause cracks in the joint area.

If no other problems are evident, such as differential movement between pipe sections, and the cracks are not wider than 0.1 in., spalling, or sheared, they may be considered a minor problem. Cracked joints are more than likely soil tight, but may not be watertight even if gaskets were used.

Separated joints are often found when severe misalignment is found. In fact, either problem may cause or aggravate the other. Movement of the soil in the general direction of the culvert's centerline may cause sections to gradually pull apart. Embankment slippage may also cause separations to occur.

27.6.4—Longitudinal Cracks

Hairline longitudinal cracks in the crown or invert indicate that the steel has accepted part of the load. Cracks equal to or less than 0.01 in. in width are considered minor and only need to be noted in the inspection report.

Other signs of distress, such as differential movement, efflorescence, spalling, or rust stains shall also be noted. When cracks are wider than 0.01 in., measurements shall be taken of the width, length, and locations of the cracks and diameter of the pipe, both horizontally and vertically. An evaluation shall be conducted by the Contractor and shall be submitted to the Engineer for review and approval considering the structural integrity, environmental conditions, and the design service life of the culvert.

C27.6.4

Generally, in noncorrosive environments (pH > 5.5), cracks 0.10 in. or less in width are considered acceptable. Longitudinal cracking in excess of 0.10 in. in width may indicate overloading or poor bedding. If pipe is placed on hard material and backfill is not adequately compacted around the pipe or under the haunches of the pipe, loads will be concentrated along the bottom of the pipe and may result in flexure or shear cracking. Use ofhydraulic compaction equipment without adequate soil cover may result in longitudinal cracks in the crown of the pipe.

Cracking may also be caused by the improper act of using construction equipment to push on the pipe to obtain proper grade.

Cracks having widths equal to or greater than 0.01 in. and determined to be detrimental shall be sealed by a method approved by the Engineer. Pipes with cracks having widths greater than 0.1 in. and determined by the Engineer to be beyond satisfactory structural repair shall be remediated or replaced. Pipes having displacement across the crack shall be repaired or replaced.

Inspection records for pipes with crack widths exceeding 0.01 in., shall be kept on file for monitoring conditions during subsequent inspections. Crack measurements and photographs shall be taken for monitoring conditions during subsequent inspections.

Displacement across the crack is shown by a differential movement or unevenness of the crack and is indicative of a high shear condition. While flexural cracks appear at the invert and crown locations of the pipe, cracks resulting from shear or spalling are more likely found in the haunch area of the pipe.

27.6.5—Transverse Cracks

Where transverse cracks are observed, they shall be monitored as described in Article 27.6.4.

C27.6.5

Poor bedding may also cause transverse or circumferential cracks. Cracks can occur across the bottom of the pipe (broken belly) when the pipe is only supported at the ends of each section.

This is generally the result of poor installation practices such as not providing indentations (bell holes) in hard foundation material for the end of bell and spigot-type pipe or not providing a sufficient depth of suitable bedding material.

Cracks may occur across the top of pipe (broken back) when settlement occurs and rocks or other areas of hard foundation material near the midpoint of a pipe section are not adequately covered with suitable bedding material.

27.6.6—Spalls

Spalling may be detected by visual examination of the concrete along the edges of cracks. Tapping with a hammer shall be performed along the cracks to check for areas that have fractured but are not visibly separated. Such areas will produce a hollow sound when tapped. These areas may be referred to as delaminations or incipient spalls. Pipe experiencing this type of problem shall be repaired or replaced.

C27.6.6

Spalling is a fracture of the concrete parallel or inclined to the surface of the concrete. Minor or isolated spalls are not a problem and are easily repaired to prevent corrosion of the steel. In precast concrete pipe, spalls often occur along the edges of either longitudinal or transverse cracks when the crack is due to overloading or poor support rather than simple tension cracking.

27.6.7—Slabbing

Any pipe experiencing slabbing shall be repaired or replaced.

C27.6.7

Slabbing is a serious problem that can occur under high fills.

The terms slabbing, shear slabbing, or slab shear refer to a radial failure of the concrete which occurs from straightening of the reinforcement cage due to excessive tension.

Slabbing is characterized by large slabs of concrete "peeling" away from the sides of the pipe and a straightening of the reinforcing steel.

27.6.8—End Section Drop-Off

End drop-offs are caused by erosion of the material supporting the pipe sections on the outlet end of the culvert barrel. The end section shall be reset.

C27.6.8

This type of distress is usually due to outlet soil erosion.

27.6.9—Follow Up

If any repairs or remedial action is performed on the pipe, a follow-up inspection following the same guidelines as outlined in Section 27.6 shall be performed between one and two years' time.

27.7—MEASUREMENT

Culverts shall be measured in linear feet installed in place, completed and accepted. The number of feet shall be the average of the top and bottom centerline lengths for pipe and box sections.

27.8—PAYMENT

The length determined as herein given shall be paid for at the contract unit prices per linear foot bid for culverts of the several sizes and shapes, as the case may be, which prices and payments shall constitute full compensation for furnishing, handling, and installing the culvert and for all materials, labor, equipment, tools, and incidentals necessary to complete this item. Such price and payment shall also include excavation, bedding material, backfill, reinforced concrete headwalls and endwalls, and any required foundations.

C27.8

Reinforced concrete headwalls and endwalls are normally included in the payment. However, in some cases they are paid for as separate bid items, per each unit, and are not included in the per lineal unit price. Normally rock excavation is not made incidental to earth excavation and is paid for under a separate pay item.

27.9—REFERENCES

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2007. *Highway Drainage Guidelines*, Fourth Edition, HDG-4, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2009. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 29th Edition, HM-29, American Association of State Highway and Transportation Officials, Washington, DC.

ASTM. 2001. *Standard Practice for Installation of Precast Concrete Sewer, Storm Drain, and Culvert Pipe Using Standard Installations,* ASTM C1479, American Society for Testing and Materials, West Conshohocken, PA.

SECTION 28: WEARING SURFACES

TABLE OF CONTENTS

SECTION 28

WEARING SURFACES

28.1—DESCRIPTION

This work shall consist of placing a wearing surface of durable and impervious material on the roadway surface of bridge decks, including the preparation of the surfaces of either existing or new decks to receive such an overlay of surfacing material.

The type and thickness of the wearing surface shall be as designated in the contract documents. The materials and installation requirements for wearing surfaces of types other than latex-modified concrete shall be as specified in the contract documents. Latex-modified concrete wearing surfaces shall be furnished and installed in accordance with these Specifications.

28.2—LATEX-MODIFIED CONCRETE TYPE WEARING SURFACE

28.2.1—General

All equipment used to prepare the surface and to proportion, mix, place, and finish the latex concrete shall be subject to approval by the Engineer prior to use. This approval will be contingent on satisfactory performance and will be rescinded in the event such performance is not being achieved. Equipment shall be on hand sufficiently ahead of the start of construction operations to be examined and approved. Any equipment leaking oil or any other containment onto the deck shall be immediately removed from the job site until repaired.

A technician who is well experienced in the proportioning, mixing, placing, and finishing of latex-modified concrete shall be employed by the Contractor and shall be present and in technical control of the work whenever these operations are underway. The qualifications of this technician, including a list of projects on which the technician was employed and the technician's level of responsibility on each, shall be submitted to and approved by the Engineer prior to the start of these operations.

Approval by the Engineer of equipment or technicians shall not relieve the Contractor of any responsibility for the successful completion of the work.

If not otherwise specified in the contract documents, the minimum thickness of latex-modified concrete wearing surfaces shall be 1.25 in.

28.2.2—Materials

28.2.2.1—Portland Cement

Portland cement shall conform to the requirement of Article 8.3.1, "Cements," except that only Types I or II shall be used.

28.2.2.2—Aggregate

Aggregate shall conform to the requirements of AASHTO M 6 for fine aggregate and to AASHTO M 80 for coarse aggregate. Coarse aggregate shall be graded 0.5 in. to No. 4 per AASHTO M 43 (ASTM D448).

28.2.2.3—Water

Water for mixing concrete shall conform to the requirements of Article 8.3.2, "Water."

28.2.2.4—Latex Emulsion

Formulated latex emulsion admixture shall be a nonhazardous, film forming, polymeric emulsion in water, to which all stabilizers have been added at the point of manufacture, and shall be homogeneous and uniform in composition.

Physical Properties—The latex modifier shall conform to the following requirements:

Polymer Type Stabilizers Styrene Butadiene

- Latex — Nonionic Surfactants
- Portland Cement Composition — Polydimethyl Siloxane
- Percent Solids — 46.0–49.0
- Weight per gallon, lb at 77°F — 8.4
- Color — White

A Certificate of Compliance signed by the Manufacturer of the latex emulsion, certifying that the material conforms to the above Specifications, shall be furnished for each shipment used in the work.

Latex admixture to be stored shall be kept in suitable enclosures that will protect it from freezing and from prolonged exposure to temperatures in excess of 85°F. Containers of latex admixture may be stored at the bridge site for a period not to exceed ten days. Such stored containers shall be covered completely with suitable insulating blanket material to avoid excessive temperatures.

28.2.2.5—Latex-Modified Concrete

The latex-modified concrete for use on the project shall be a workable mixture and meet the following requirements specified in Table 28.2.2.5-1 and the following:

- Following sampling of the discharged, normally mixed material, the commencement of the slump test shall be delayed from 4 to 5 min.

- Water may be added to obtain slump within the prescribed limits.

- The dry weight (mass) ratios are approximate and should produce good workability but, due to gradation changes, may be adjusted within limits by the Engineer. The parts by weight (mass) of sand may be increased by as much as 0.2 if the coarse aggregate is reduced by an equivalent volume.

Table 28.2.2.5-1—Latex-Modified Concrete Mixture

Material or Property	Concrete
Cement, Parts by Weight	1.0
Fine Aggregate, Parts by Weight	2.5
Coarse Aggregate, Parts by Weight	2.0
Latex Emulsion Admixture, gal/bag of cement	3.5
Air Content of Plastic Mix, % (AASHTO T 152) (ASTM C231)	3.0–6.0
Slump, in.	3.0–6.0

28.2.3—Surface Preparation

28.2.3.1—New Decks

The surfaces of new decks upon which a wearing surface overlay is to be placed shall be finished to a rough texture by coarse brooming or other approved methods.

After curing of the deck concrete is complete and before placing the overlay, the entire area of the deck surface and the vertical faces of curbs, concrete parapets, barrier walls, etc., up to a height of 1.0 in. above the top elevation of the overlay shall be blast-cleaned to a bright, clean appearance that is free from laitance, curing compound, dust, dirt, oil, grease, bituminous material, paint, and all foreign matter. The blast cleaning of an area of the deck shall normally be performed within the 24-h period preceding placement of the overlay on the area. The blast cleaning may be performed by either wet sandblasting, high-pressure water blasting, blasting grits, shrouded dry sandblasting with dust collectors, or another method approved by the Engineer. Water blasting equipment shall operate with a minimum pressure of 3.5 ksi. The method used shall be performed so as to conform to applicable air and water pollution regulations and to applicable safety and health regulations. All debris, including dirty water, resulting from the blast cleaning operations shall be immediately and thoroughly cleaned from the blast-cleaned surfaces and from other areas where debris may have accumulated. The blast-cleaned areas shall be protected as necessary against contamination prior to placement of the overlay. Contaminated areas and areas exposed more than 36 h after cleaning shall be blast-cleaned again as directed by the Engineer and at the Contractor's expense.

Just prior to placement of the overlay, all dust and other debris shall be removed by flushing with water or blowing with compressed air. The prepared surface shall then be soaked with clean water for not less than 1 h prior to the placement of the latex overlay. Before the overlay is applied, all free water shall be blown out and off, and this procedure shall continue until the surface appears dry or barely damp.

The air supply system for blast cleaning and blowing shall be equipped with an oil trap in the air line, and provisions shall be made to prevent oil or grease contamination of the surface by any equipment prior to placement of the overlay.

28.2.3.2—Existing Decks

The surface of existing decks that have become contaminated by traffic usage or by deicing salts shall be scarified to the depth specified in the contract documents. If no depth is shown or specified, a minimum of 0.25 in. of material shall be removed by scarification.

Prior to beginning scarification and until operations are completed, all deck drains, expansion joints, and other openings where the Engineer determines that damage could result shall be temporarily covered or plugged to prevent entry of debris.

Scarifying shall be done with power operated mechanical scarifiers, or other approved devices, capable of uniformly removing the existing surface to the depths required without damaging the underlying concrete. Machine scarifiers shall not be operated so as to damage hardware such as drain grates and expansion joint armor. In areas where machine scarifying cannot reach and in areas of spalling and where steel reinforcement is exposed, scarifying and the removal of deteriorated or unsound concrete shall be accomplished with hand tools. Pneumatic hammers heavier than nominal 45 lb shall not be used.

No scarifying or chipping will be allowed within 6.0 ft of a new overlay until 48 h after its placement.

In areas where the Engineer determines that deteriorated or unsound concrete has been encountered, the concrete shall be removed to a depth of 0.75 in. below the top mat of reinforcing steel. A minimum of 0.75 in. clearance shall be required around the reinforcing steel, except where lower bar mats make this impractical. Care shall be exercised to prevent damaging the exposed reinforcing steel. All reinforcing steel shall be blast-cleaned. The repair areas are to be filled during the overlay operation.

After scarification and removal of unsound concrete has been completed, the deck surface shall be blast-cleaned and prepared as specified in Article 28.2.3.1, "New Decks."

28.2.4—Proportioning and Mixing

The Contractor shall submit to the Engineer for approval, 14 calendar days prior to date of placement, the proposed mix design in writing and samples of all mix materials in sufficient quantity to produce a minimum of 3.0 ft^3 of concrete for laboratory mix design testing.

Proportioning and mixing equipment shall be a self-contained, mobile, continuous-mixing, volumetric proportioning-type mixer.

Continuous-type mixers shall be equipped so that the proportions of the cement, natural sand, and coarse aggregate can be fixed by calibration of the mixer and cannot be changed without destroying a seal or other indicating device affixed to the mixer. In addition to being equipped with a flow meter for calibrating the water supply portion of the mixer, the mixer shall also be equipped with a cumulative-type water meter that can be read to the nearest 0.1 gal. The water meters shall be readily accessible, accurate to within one percent, and easy to read. Both water meters shall be subject to checking by the Engineer each time the mixer is calibrated. Approved methods for adding the admixture shall be provided. The admixtures shall be added so as to be kept separated as far as is practicable. The continuous-type mixer shall be calibrated to the satisfaction of the Engineer prior to starting the work. Yield checks normally will be made for each 50.0 yd^3 of mix. Recalibration will be necessary when indicated by the yield checks and at any other times the Engineer deems necessary to ensure proper proportioning of the ingredients. Continuous-type mixers that entrap unacceptable volumes of air in the mixture shall not be used.

The mixer shall be kept clean and free of partially dried or hardened materials at all times. It shall consistently produce a uniform, thoroughly blended mixture within the specified air content and slump limits. Malfunctioning mixers shall be immediately repaired or replaced with acceptable units.

Aggregate stockpiles being used should be of uniform moisture content.

Mixing capability shall be such that finishing operations can proceed at a steady pace, with final finishing completed before the formation of the plastic surface film.

28.2.5—Installation

28.2.5.1—Weather Restrictions

The placement of latex-modified concrete shall not be started when the temperature is or is expected to fall below 45°F or rise above 80°F, or when high winds, rain, or low humidity conditions are expected prior to final set of the concrete. If any of these conditions occur during placement, the placement shall be terminated and a straight construction joint formed. Placement at night may be necessary when daytime conditions are not favorable. If placement is performed at night, adequate lighting shall be provided by the Contractor.

28.2.5.2—Equipment

Placing and finishing equipment shall include hand tools for placing and brushing in freshly mixed latex-modified concrete and for distributing it to approximately the correct level for striking off with the screed. Hand-operated vibrators, screeds, and floats shall be used for consolidating and finishing small areas.

An approved finishing machine complying with the following requirements shall be used for finishing all large areas of work:

- The finishing machine shall be self-propelled and capable of forward and reverse movement under positive control. The length of the screed shall be sufficient to extend at least 6.0 in. beyond the edge of both ends of the section being placed. The finishing machine shall also be capable of consolidating the concrete by vibration and of raising all screeds to clear the concrete for traveling in reverse. The machine shall be either a rotating roller-type or an oscillating screed-type.

- Rotating roller-type machines shall have one or more rollers, augers, and 25-to 40-Hz vibratory pans.

- Oscillating screed-type machines shall have vibrators on the screeds whose frequency of vibration can be varied between 50 and 250 Hz. The bottom face of the screeds shall be not less than 4.0 in. wide and shall be metal.

Rails shall be required for the finishing machine to travel on. Rails shall be sufficiently rigid to support the weight (mass) of the machine without appreciable deflection and shall be placed outside of the overlay area. Rail anchorages shall provide horizontal and vertical stability and shall not be ballistically shot into concrete that will not be overlaid.

A suitable portable lightweight or wheeled work bridge shall be furnished for use behind the finishing operation.

28.2.5.3—Placing and Finishing

28.2.5.3.1—Construction Joints

Planned construction joints shall be formed by bulkheads set to grade. Before placing concrete against previously placed overlay material, the construction joint shall be sawed to a straight vertical edge. Sawing of joints may be omitted if the bulkhead produces a straight, smooth, vertical surface. The face of the joint shall be sand or water blasted to remove loose material.

Longitudinal construction joints will be permitted only at the centerline of roadway or at lane lines unless otherwise shown in the contract documents or permitted by the Engineer.

In case of delay in the placement operation exceeding 1 h in duration, an approved construction joint shall be formed by removing all material not up to finish grade and sawing the edge in a straight line. During minor delays of 1 h or less, the end of the placement may be protected from drying with several layers of clean, wet burlap.

28.2.5.3.2—Placing

The finishing machine shall be test-run over the entire area to be overlayed each day before placement is started to ensure that the required overlay thickness will be achieved.

Immediately ahead of placing the overlay mixture, a thin coating of the polymer-modified concrete mixture to be used for the overlay shall be thoroughly brushed and scrubbed onto the surface as a grout-bond coat for the overlay. More coarse particles of the mixture which cannot be scrubbed into contact with the surface shall be removed and disposed of in a manner approved by the Engineer. Care shall be taken to ensure that all vertical as well as horizontal surfaces receive a thorough, even coating and that the rate of progress is limited so that the material brushed on does not become dry before it is covered with the full depth of latex-modified concrete.

The latex-modified concrete shall be placed on the prepared and grout-coated surface immediately after being mixed. The mixture shall be placed and struck off approximately 0.25 in. above final grade, then consolidated by vibration and finished to final grade with the approved finishing machine. Spud vibrators may be used in deep pockets, along edges, and adjacent to joint bulkheads. Supplemental vibration shall be provided along the meet lines where adjacent pours come together and along curb lines. Hand finishing with a float may be required along the edge of the pour or on small areas of repair.

Screed rails and construction bulkheads shall be separated from the newly placed material by passing a pointing trowel along their inside face. Expansion dams shall not be separated from the overlay. Care shall be exercised to ensure that this trowel cut is made for the entire depth and length of rails after the mixture has stiffened sufficiently.

28.2.5.3.3—Finishing

The finishing equipment shall be operated so as to produce a uniform, smooth, and even-textured surface. The final surface shall not vary more than 0.125 in. from a 10.0-ft straightedge placed longitudinally thereon. Before the plastic film forms, the surface shall be textured by tining in accordance with the requirements of Article 8.10.2.3, “Texturing.”

28.2.6—Curing

The surface shall be promptly covered with a single layer of clean, wet burlap as soon as the surface will support it without deformation.

Within 1 h of covering with wet burlap, the burlap shall be rewet if necessary and a layer of 4-mil polyethylene film, or wet burlap-polyethylene sheets, shall be placed on the wet burlap, and the surface cured for 24 h. The curing material shall then be removed for an additional 72 h of air cure. If the temperature falls below 45°F during curing, the duration of the wet cure shall be extended as directed by the Engineer.

The overlay shall be protected from freezing during the cure period.

Traffic will not be permitted on the overlay while it is curing.

28.2.7—Acceptance Testing

After curing is completed, the overlay will be visually inspected for cracking or other damage, and inspected for delaminations and bond failures by the use of a chain drag or other suitable device.

Surface cracks not exceeding 0.375 in. in depth shall be sealed with an epoxy-penetrating sealer followed by an application of approved sand.

Any cracks exceeding 0.375 in. in depth shall be repaired by methods approved by the Engineer or the affected portions of the wearing surface shall be removed and replaced. Any delaminated or unbonded portions of the wearing surface or portions damaged by rain or freezing shall be removed and replaced.

After completion of the wet cure, the surface shall be tested for flatness and corrected, if necessary, as provided in Article 8.10.2.4, "Surface Testing and Correction."

All corrective work shall be at the Contractor's expense.

28.2.8—Measurement and Payment

Wearing surfaces and areas requiring scarification will be measured by the square foot based on dimensions of the completed work.

Wearing surfaces will be paid for at the contract price per square foot. Except as otherwise provided, the payment per square foot for wearing surfaces shall be considered to be full compensation for the cost of furnishing all labor, materials, equipment, incidentals, and for doing all work involved in preparing the surface and constructing the wearing surface as shown in the contract documents.

When a separate item is included in the contract documents for scarifying bridge decks, scarifying shall be paid for by the contract price per square foot. Such payment shall be considered to be full compensation for all costs involved with the scarifying work, including removal and disposal of debris.

The removal of unsound concrete which is encountered below the depth specified for scarifying shall be paid for as extra work.

28.3—REFERENCE

AASHTO. 2009. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 29th Edition, HM-29, American Association of State Highway and Transportation Officials, Washington, DC.

SECTION 29: EMBEDMENT ANCHORS

TABLE OF CONTENTS

SECTION 29

EMBEDMENT ANCHORS

29.1—DESCRIPTION

This work shall cover installation and field testing of cast-in-place, grouted, adhesive-bonded, expansion, and undercut steel anchors.

C29.1

The use of embedment anchors is prevalent but standardized installation and field testing is not. Therefore, a new section was created.

29.2—PREQUALIFICATION

Concrete anchors, including cast-in-place; all bonded anchor systems, including grout, chemical compound and adhesives; and undercut steel anchors shall be prequalified by universal test standards designed to allow approved anchor systems to be employed for any construction attachment use.

Tests for adhesive-bonded and other bonding compounds shall be conducted in accordance with ASTM E1512, Standard Test Methods for Testing Bond Performance of Adhesive-Bonded Anchors.

Expansion anchors shall be tested in accordance with ASTM E488, Standard Test Methods for Strength of Anchors in Concrete and Masonry Elements.

Embedment anchor details shall comply with ACI 349, *Code Requirements for Nuclear Safety Related Concrete Structures*, “Appendix B, Steel Embedments.”

For anchor systems other than mechanical expansion anchors, the Contractor shall provide the Engineer with certified test reports prepared by an independent laboratory documenting that the system is capable of achieving the minimum tensile strength of the embedment steel.

29.3—MATERIALS

Mill test reports shall be provided to the Engineer to certify physical properties, chemistry, and strengths used to manufacture the anchors.

Either an epoxy, vinylester, or polyester chemical compound shall be acceptable for adhesive anchors. Moisture-insensitive, high-modulus, low shrinkage, and high-strength adhesives shall be used.

The use of additives to grout and bonding materials that are corrosive to steel or zinc/cadmium coatings shall be prohibited.

29.4—CONSTRUCTION METHODS

Adequate edge distance, embedment depth, and spacing to develop the required strength of the embedment anchors shall be provided. The correct drill-hole diameter shall be used as specified by the Manufacturer. Rotary impact drilling shall be used unless diamond core drilling has been specified or tested. If a reinforcing bar is encountered during drilling, the hole shall be moved to a different location or the reinforcing steel shall be drilled through using a diamond core bit as directed by the Engineer. Abandoned holes shall be patched with an approved bonding material. Holes shall be thoroughly cleaned as recommended by the Manufacturer.

The Contractor shall remove all loose dust and concrete particles from the hole and shall prepare bonding material and install anchors according to the Manufacturer's instructions or as approved by the Engineer.

Improperly installed embedded anchors or anchors not having the required strength shall be removed and replaced to the satisfaction of the Engineer at the Contractor's expense.

29.5—INSPECTION AND TESTING

Where specified, sacrificial tests of the anchor system shall be done at the job site to ultimate loads to document the capability of the system to achieve pullout loads equaling the full minimum tensile value of the anchor employed. Anchor testing shall be done on fully cured concrete samples. At least three anchors shall be tested by ASTM E488 methods, unless otherwise specified. The Contractor may use any prequalified anchor systems meeting the above requirements.

Provision shall be made for use of an alternative system that will reach the designated pullout requirement, without delay in progress, if the job site proof loading proves incapable of achieving minimum tensile values, or the load required by the Engineer if too little concrete exists in which to develop full ductile loads.

After installation and cure of the bonding material, each anchor system shall be torqued to specified values using approved torque methods only. If torque values are not specified, the Manufacturer's recommendation or values provided by the Engineer shall be used.

29.6—MEASUREMENT

Measurement of embedment anchors incorporated into the project shall be the number of each anchor size and orientation shown in the contract documents or authorized for use on the project. Each embedment anchor type satisfactorily installed shall be counted and summarized in the contract documents according to anchor system; orientation, i.e., vertical, horizontal, and diagonal; and size taken as the diameter.

29.7—PAYMENT

Payment shall be based upon the quantity of embedment anchors determined under measurement for each embedment anchor type and shall include full compensation for furnishing all labor, materials, tools, equipment, testing, and incidentals necessary to place each anchor type.

29.8—REFERENCE

ACI. 2001. *Code Requirements for Nuclear Safety Related Concrete Structures*, ACI 349-01, American Concrete Institute, Farmington Hills, MI, Appendix B: Steel Embedments.

SECTION 30: THERMOPLASTIC PIPE

TABLE OF CONTENTS

SECTION 30

THERMOPLASTIC PIPE

30.1—GENERAL 2010 Revision

30.1.1—Description 2012 Revision

This work shall consist of furnishing, installing, and inspecting thermoplastic pipe in conformance with these Specifications and the contract documents.

C30.1.1

As used in this Specification, thermoplastic pipe is defined in the *AASHTO LRFD Bridge Design Specifications*, Section 12, "Buried Structures and Tunnel Liners."

See *AASHTO LRFD Bridge Design Specifications*, Section 12, "Buried Structures and Tunnel Liners," and ASTM D2321, Standard Practice for Underground Installation of Thermoplastic Pipe for Sewers and Other Gravity-Flow Applications, for additional guidance.

30.1.2—Workmanship and Inspection 2010 Revision

All thermoplastic pipe materials shall conform to the workmanship and inspection requirements of AASHTO M 278, M 294, or M 304; or ASTM F679, F714, F794, or F894, as applicable.

30.2—WORKING DRAWINGS 2010 Revision

Where specified or requested by the Engineer, the Contractor shall provide Manufacturer's installation instructions or working drawings and substantiating calculations in sufficient detail to permit a structural review. Sufficient copies shall be furnished to meet the needs of the Engineer and other entities with review authority. The working drawings shall be submitted sufficiently in advance of proposed installation and use to allow for their review, revision, if needed, and approval without delay of the work. The Contractor shall not start construction of any thermoplastic pipe installations for which working drawings are required until the drawings have been approved by the Engineer. Such approval will not relieve the Contractor of responsibility for results obtained by use of these drawings or any of the other responsibilities under the contract.

30.3—MATERIALS

30.3.1—Thermoplastic Pipe 2010 Revision 2012 Revision

Polyethylene pipe shall conform to the requirements of AASHTO M 294, or ASTM F714 or F894.

Poly, i.e., Vinyl Chloride or PVC, pipe shall conform to the requirements of AASHTO M 278, AASHTO M 304, ASTM F679, or ASTM F794.

2010 Edition

30.3.2—Bedding Material and Structural Backfill

2010 Revision

Bedding and structural backfill shall meet the requirements of AASHTO M 145, A-1, A-2-4, A-2-5, or A-3. Bedding material shall have a maximum particle size of 1.25 in. Backfill for thermoplastic pipe shall be free of organic material, stones larger than 1.5 in. in greatest dimension, or frozen lumps. Moisture content shall be in the range of optimum content, typically –3 percent to +2 percent, so as to permit thorough compaction. Consideration should be given to the potential for migration of fines from adjacent materials into open-graded backfill and bedding materials.

For pipe types that are not smooth on the outside, i.e., pipes with corrugated or profile walls, backfill gradations should be selected that will permit the filling of the corrugation or profile valleys.

Flowable fills, such as controlled low strength mortar, CLSM, or controlled density fill, CDF, may be used for backfill and bedding provided adequate flotation resistance can be achieved by restraints, weighing, or placement technique. With CLSM backfill, trench width can be reduced to a minimum of the outside diameter plus 12.0 in. When CLSM is used, all joints shall have gaskets.

30.4—ASSEMBLY

2010 Revision

30.4.1—General

Thermoplastic pipe shall be assembled in accordance with the Manufacturer's instructions. All pipe shall be unloaded and handled with reasonable care. Pipe shall not be rolled or dragged over gravel or rock and shall be prevented from striking rock or other hard objects during placement in trench or on bedding.

Thermoplastic pipe shall be placed in the bed starting at the downstream end.

30.4.2—Joints

2010 Revision

30.4.2.1—General

Joints for thermoplastic pipe shall meet the performance requirements for soiltightness unless watertightness is specified.

C30.4.2.1

Watertightness shall be based on project requirements. Available watertightness pressure levels are 2, 5, and 10 psi.

30.4.2.2—Field Joints

2010 Revision

Joints shall be so installed that the connection of pipe sections will form a continuous line free from irregularities in the flow line.

C30.4.2.2

Suitable field joints can be obtained with the following types of connections:

- Corrugated bands (with or without gaskets)

- Bell and spigot pipe ends (with or without gaskets)
- Double bell couplings (with or without gaskets)

30.5—INSTALLATION

2010 Revision

30.5.1—General Installation Requirements

Trenches shall be excavated in such a manner as to ensure that the sides will be stable under all working conditions. Trench walls shall be sloped or supported in conformance with all standards of safety. Only as much trench as can be safely maintained shall be opened. All trenches shall be backfilled as soon as practicable, but not later than the end of each working day.

Trench details, including foundation, bedding, haunching, initial backfill, final backfill, pipe zone, and trench width shall be taken as shown in Figure 30.5.1-1.

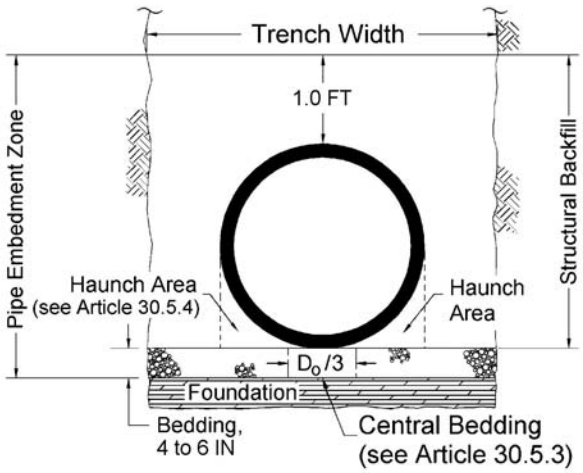


Figure 30.5.1-1—Trench Details

30.5.2—Trench Widths

2010 Revision

Trench width shall be sufficient to ensure working room to properly and safely place and compact haunching and other backfill materials. The space between the pipe and trench wall should be wider than the compaction equipment used in the pipe zone. Minimum trench width shall not be less than 1.5 times the pipe outside diameter plus 12.0 in. Trenches shall be kept to the specified width, as any increase in trench width will increase the load on the pipe. Structural backfill shall be comprised of well-graded, granular materials, such as AASHTO A-1, A-2-4, and A-2-5 soils, to facilitate better consolidation around the pipe and to minimize the possibility of soil migration and piping of the in situ soils.

If the trench walls do not stand without support, the trench width shall be increased in accordance with the design criteria in the *AASHTO LRFD Bridge Design Specifications*, Article 12.12.3.4, "Thrust."

C30.5.2

Flexible pipe require soil support at the sides, and unstable trench walls are an indication that a wider trench width is required. This criterion does not refer to trenches for which trench supports are required only to comply with OSHA.

Determination of trench width in unsupported, unstable soils shall include consideration of the size of the pipe, the stiffness of the backfill and in situ soil, the depth of cover and other site-specific conditions as applicable. The trench shall be excavated to the width, depth, and grade as indicated on the plans and/or given by the Engineer.

30.5.3—Foundations and Bedding

2010 Revision

Foundation and bedding shall meet the requirements of Article 30.3.2 and shall be installed as required by the Engineer according to conditions in the trench bottom. A stable and uniform bedding shall be provided for the pipe and any protruding features of its joint and/or fittings. The middle of the bedding equal to one-third the pipe outside diameter (OD) should be loosely placed, while the remainder shall be compacted to a minimum 90 percent of maximum density per AASHTO T 99. A minimum of 4.0 in. of bedding shall be provided prior to placement of the pipe unless otherwise specified.

When rock or unyielding material is present in the trench bottom, a cushion of bedding of 6.0 in. minimum thickness shall be provided below the bottom of the pipe.

When the trench bottom is unstable, material shall be excavated to the depth required by the Engineer, and replaced with a suitable foundation. A suitably graded material shall be used where conditions may cause migration of fines and loss of pipe support.

30.5.4—Structural Backfill

2010 Revision

Structural backfill shall meet the requirements of Article 30.3.2. Structural backfill shall be placed and compacted in layers not exceeding an 8.0-in. loose lift thickness and brought up evenly and simultaneously on both sides of the pipe to an elevation not less than 1.0 ft above the top of the pipe. Structural backfill shall be worked into the haunch area and compacted by hand.

A minimum compaction level of 90 percent standard density per AASHTO T 99 shall be achieved. Special compaction means may be necessary in the haunch area as shown in Figure 30.5.1-1. All compaction equipment used within 3.0 ft of the pipe shall be approved by the Engineer. Ponding or jetting the structural backfill to achieve compaction shall not be permitted without written permission from the Engineer.

Backfill materials placed in the zone extending more than 1.0 ft above the pipe to final grade shall be selected, placed, and compacted to satisfy the loading, pavement, and other requirements above the pipe.

30.5.5—Minimum Cover

2010 Revision

A minimum depth of cover above the pipe should be maintained before allowing vehicles or heavy construction equipment to traverse the pipe trench. The minimum depth of cover should be established by the Engineer based on an evaluation of specific project conditions. For embedment materials installed to the minimum density given in Article 30.5.4, "Structural Backfill," cover of at least 2.0 ft shall be provided before allowing vehicles or construction equipment to cross the trench surface. Minimum cover for construction loads shall be as shown in Table 30.5.5-1. Hydrohammer type compactors shall not be used over the pipe.

C30.5.5

Diameters greater than 4.0 ft in Table 30.5.5-1 are for information only. See Tables A12-11, A12-12, and A12-13 of the *AASHTO LRFD Bridge Design Specifications* for nominal sizes.

Table 30.5.5-1—Minimum Cover for Construction Loads

	Minimum Cover, in., for Indicated Axle Loads, kips			
Nominal Pipe Diameter, ft	18.0–50.0	50.0–75.0	75.0–110.0	110.0–150.0
2.0–3.0	24.0	30.0	36.0	36.0
3.5–4.0	36.0	36.0	42.0	48.0
4.5–5.0	36.0	36.0	42.0	48.0

Minimum cover shall be measured from the top of the pipe to the top of the maintained construction roadway surface. If unpaved, the surface shall be maintained.

30.5.6—Inspection Requirements

2010 Revision

30.5.6.1—Visual Inspection

All pipes shall undergo inspection during and after installation to ensure proper performance. Installation of bedding and backfill materials, as well as their placement and compaction, shall be determined to meet the requirements of this section.

During the initial phases of the installation process, inspection shall concentrate on detecting improper practice and poor workmanship. Errors in line and grade, as well as any improper assembly or backfill techniques, shall be corrected prior to placing significant backfill or trench fill. Coupling bands shall be properly indexed with the corrugation and tightened, and bell/spigot joints shall be properly assembled to prevent the infiltration of soil fines. Where gaskets are used, they shall be properly seated to prevent groundwater infiltration and should appear uniformly oriented around the pipe. In areas where cracking or joint separation is found, a remediation or replacement plan shall be submitted for approval.

Final internal inspections shall be conducted on all buried thermoplastic pipe installations to evaluate issues that may affect long-term performance. Final inspections shall be conducted no sooner than 30 days after completion of installation and final fill.

Shallow cover installations shall be checked to ensure the minimum cover level is provided.

C30.5.6.1

Inspection at the appropriate times during installation will detect and allow correction of line and grade, jointing, and shape change problems. The timing and number of inspections required will vary with the significance and depth of the installation. The contractor is advised to provide initial inspections himself to avoid problems later on. Racking or flattening of the pipe's curvature indicates improper backfill placement methods that must be corrected. Slight peaking of the cross-sectional shape should be taken as indicative of achieving proper compaction requirements.

Soil consolidation continues with time after installation of the pipe. While 30 days will not encompass the time frame for complete consolidation of the soil surrounding the pipe, it is intended to give sufficient time to observe some of the effects that this consolidation will have. However, occasionally pavement is placed over the pipe sooner than 30 days. While the 30-day time limit should be maintained, a brief inspection of the pipe prior to paving over it, particularly for the first few joints, may be prudent to ensure that good construction practices are being applied.

It is recommended that inspection personnel not enter culverts less than 24.0 in. in diameter. Internal inspection of culverts in this size range is best conducted using video cameras. Culverts should be entered only by inspection personnel trained in working within confined spaces and using procedures in full compliance with applicable State, Local, and Federal OSHA regulations.

30.5.6.2—Installation Deflection

2010 Revision

The pipe shall be evaluated to determine whether the internal diameter of the barrel has been reduced more than five percent when measured not less than 30 days following completion of installation.

Pipes shall be checked for deflection using a mandrel or any other device approved by the Engineer that can physically verify the dimensions of the pipe and is not limited by poor lighting, waterflow, pipe length, or other limiting conditions of the installed environment. Pipes larger than 24.0 in. may be entered and deflection levels measured directly.

In all pipe installations, at least ten percent of the total number of pipe runs representing at least ten percent of the total pipe footage on the project shall be randomly selected by the Engineer and inspected for deflection. Also, as determined by the 100 percent visual inspection in Article 30.5.6.1, all areas in which deflection can be visually detected shall be inspected for deflection.

Where direct measurements are made, a measurement shall be taken once every 10.0 ft for the length of the pipe, and a minimum of four measurements per pipe installation is required.

If a mandrel is used for the deflection test, it shall be a nine (or greater odd number) arm mandrel, and shall be sized and inspected by the Engineer prior to testing. A properly sized proving ring shall be used to check or test the mandrel for accuracy. The mandrel shall be pulled through the pipe by hand with a rope or cable. Where applicable, pulleys may be incorporated into the system to change the direction of pull so that inspection personnel need not physically enter the pipe or manhole.

For locations where pipe deflection exceeds five percent of the inside diameter, an evaluation shall be conducted by the Contractor utilizing a Professional Engineer and submitted to the Engineer for review and approval considering the severity of the deflection, structural integrity, environmental conditions, and the design service life of the pipe. Pipe remediation or replacement shall be required for locations where the evaluation finds that the deflection could be problematic. For locations where pipe deflection exceeds 7.5 percent of the inside diameter, remediation or replacement of the pipe is required.

C30.5.6.2

Inspection criteria are newly added to the specification, as there was minimal guidance in the previous specification. Ten percent of each pipe installation shall be defined as ten percent of the number of pipe runs, and not less than ten percent of the total length of installed pipe on the project. The requirement of deflection testing ten percent of each pipe installation is intended to serve as a minimum and does not limit owners from more stringent requirements.

The pipe inside diameters should be provided by the pipe manufacturer for every size and type of pipe delivered. If the pipe inside diameter is not provided, or is not available, pipe inside diameter can be developed by averaging the diameters measured at eight equally spaced locations around a section of unloaded pipe for every given size and manufacturer.

There are many appropriate methods suitable for measuring deflection, including video inspection equipment, mandrels, and other direct measurement devices. For pipes tested by a mandrel, the mandrel shall be pulled through the entire pipe. Whichever method is used for deflection measurement, a minimum of ten percent of the total length of installed pipe shall be tested, in addition to any areas that were identified in the visual inspection as having deflection.

Installed pipe deflections that exceed five percent of the initial inside diameter may indicate that the installation was substandard. Appropriate remediation, if any, will depend upon the severity of the deflection, the condition of the pipe, and evaluation of the factor of safety using Section 12, "Buried Structures and Tunnel Liners," of the *AASHTO LRFD Bridge Design Specifications*. Installed pipe deflections that exceed 7.5 percent of the initial inside diameter will require remediation or replacement of the pipe.

30.6—MEASUREMENT

2010 Revision 2012 Revision

Pipe installations shall be measured in linear feet installed in place, completed and accepted. The number of feet shall be the centerline lengths of the pipe.

30.7—PAYMENT 2010 Revision 2012 Revision

The length as measured above will be paid for at the contract prices per linear foot bid for thermoplastic pipe of the sizes specified. Such price and payment shall constitute full compensation for furnishing, handling, and installing the pipe and for all materials, labor, equipment, tools, and incidentals necessary to complete this item. Such price and payment shall also include excavation, bedding material, backfill, headwalls, endwalls, and foundations for pipe.

30.8—REFERENCES 2010 Revision

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2009. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 29th Edition, HM-29, American Association of State Highway and Transportation Officials, Washington, DC. Includes AASHTO M, R, and T standards, which are also available individually in downloadable form.

Back to 2010 Edition

SECTION 30: THERMOPLASTIC PIPE

Revise Section 30 as shown in the following attachment.

2010 Revision

Page left intentionally blank.

Back to 2010 Edition

SECTION 30: THERMOPLASTIC ~~PIPE~~CULVERTS

30.1—GENERAL

30.1.1—Description

This work shall consist of furnishing, installing, and inspecting buried thermoplastic ~~pipe~~culverts in conformance with these Specifications and the contract documents.

C30.1.1

Plastic pipe is manufactured by a variety of methods with a variety of wall profiles. Common profiles are solid cylindrical wall with or without a standing rib, corrugated, and corrugated with a smooth liner.

As used in this Specification, thermoplastic pipe is defined in the *AASHTO LRFD Bridge Design Specifications*, Section 12, "Buried Structures and Tunnel Liners."

~~See *AASHTO LRFD Bridge Design Specifications*, Section 12, "Buried Structures and Tunnel Liners," and ASTM D2321, Standard Practice for Underground Installation of Thermoplastic Pipe for Sewers and Other Gravity-Flow Applications, for additional guidance.~~

30.1.2~~—Workmanship and Inspection~~ Importance of Construction Procedures

~~All thermoplastic pipe materials shall conform to the workmanship and inspection requirements of AASHTO M 278, M 294, or M 304; or ASTM F679, F714, F794, or F894, as applicable.~~ Contract documents shall conform to the requirements of Article 12.12.3.5 of the *AASHTO LRFD Bridge Design Specifications* and Section 30 of this Specification.

C30.1.2

In general, as the quality of backfill (represented primarily by the particle size and the portion of the backfill passing the No. 200 sieve) decreases, higher compaction levels (e.g., percentage of maximum density per AASHTO T 99 or T 190 (ASTM D2844)) are required to achieve equivalent culvert performance.

Satisfactory performance of culverts requires proper control of construction procedures at all times. The embedment material placed around a culvert provides a significant support that is relied upon in the culvert structural design. Together, the culvert and embedment form an integral soil-structure system. Therefore, selection of suitable quality backfill materials, which are then properly placed and compacted, is essential.

30.1.3—Terminology

Terminology used in this Section is illustrated in Figure 30.1.3-1 and 30.1.3-2 and is defined below:

Bedding—Material on which the structure is seated. It may be in situ soil, if such soil meets all necessary requirements, or imported backfill material. The bedding may be specified as a different material than the structural backfill.

Culvert Bottom—Lowest point on the outside of the culvert for closed shapes.

Bedding—Material on which the structure is seated. It may be in situ soil, if such soil meets all necessary requirements, or imported backfill material. The bedding may be specified as a different material than the structural backfill.

Culvert Bottom—Lowest point on the outside of the culvert for closed shapes.

Culvert Crown—Highest point on the inside of the culvert.

Culvert Invert—Lowest point on the inside of the culvert for closed shapes.

Culvert Top—Highest point on the outside of the culvert.

Embankment—Soil already placed and compacted in layers at the sides of and above the embedment zone.
Embedment Zone—Zone of structural backfill around the culvert. It consists of bedding, haunch material, sidefill, and initial topfill.

Foundation Soil—Soil that supports the bedding, culvert, and structural backfill. It must provide a firm, stable surface and may be undisturbed, existing (in situ) soil; replaced and compacted in situ soil; or an imported material.

Haunch—Portion of the culvert between the culvert bottom and the springline.

Haunch Zone—Region of the backfill between the bedding or foundation soil and the culvert surface from the culvert bottom to near the springline. It is a region where hand placement and compaction methods are normally required for the backfill. Backfill in the haunch zone is to be the same material as the structural backfill.

In Situ Soil—Native, undisturbed soil existing at the site of the culvert installation.

Shoulder—Portion of the culvert between the culvert top and the springline.

Sidefill—Embedment zone between the haunch and the shoulders of the culvert supporting the sides of the culvert.

Springline—Line along the side of the culvert where the tangent to the culvert wall is vertical. It occurs at the widest point in the culvert.

Structural Backfill—All material placed and compacted around the culvert to help support the culvert.

Topfill—Embedment zone over the top of the culvert beginning at the shoulders and extending upward to the limit of the structural backfill zone. The topfill is to be the same material as the structural backfill.

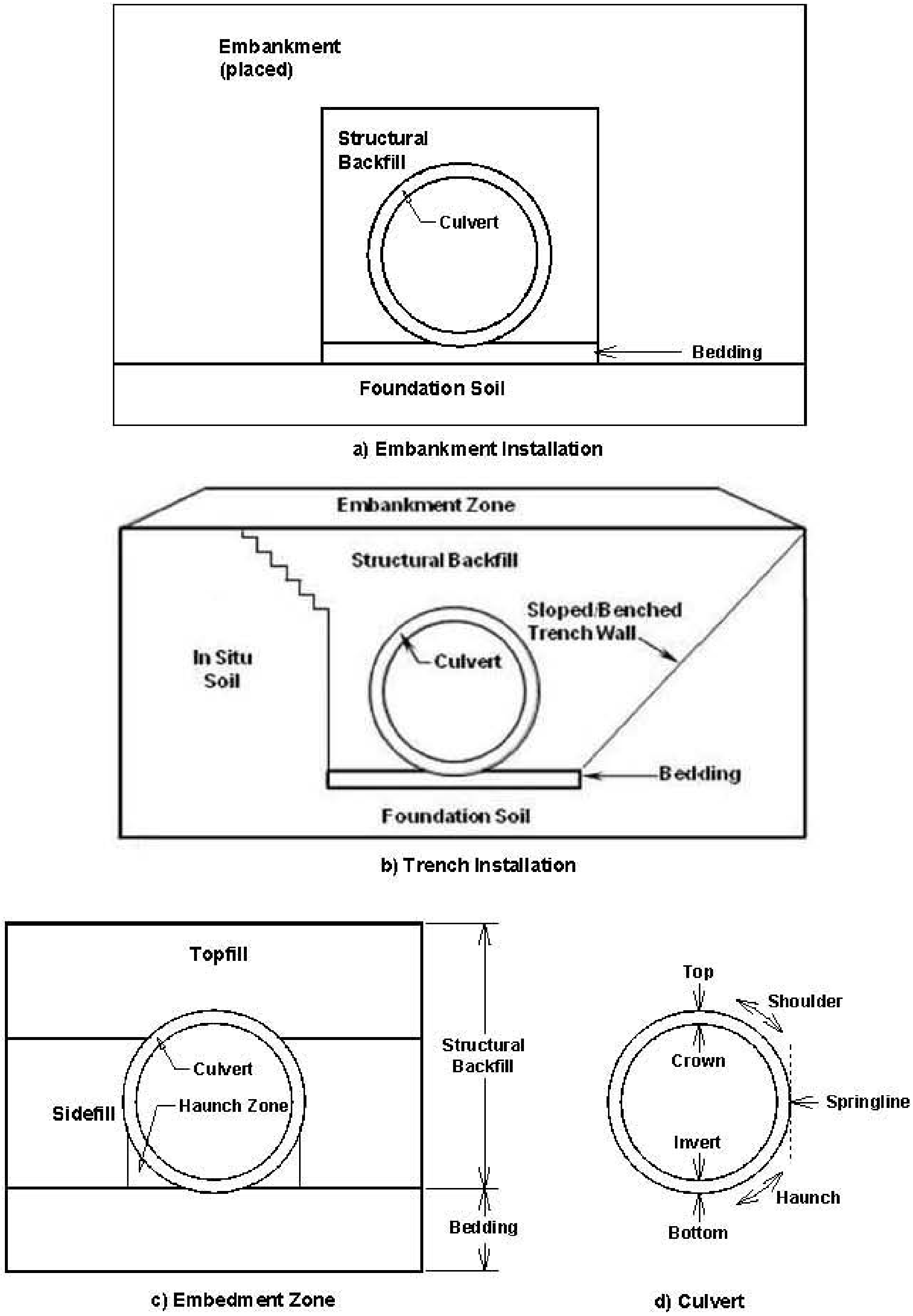


Figure 30.1.3-1—Terminology for Culvert Installation

2010
Revision

Back to 2010 Edition

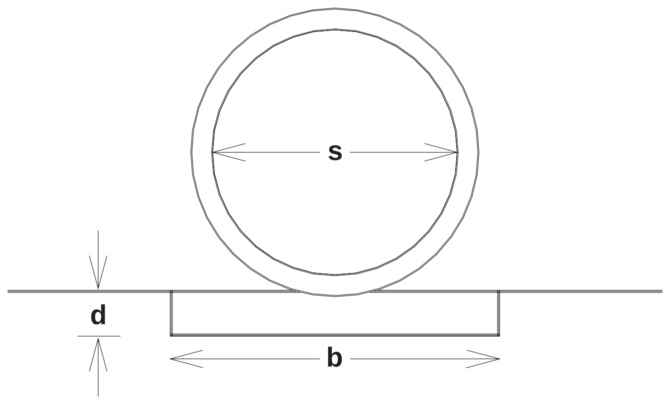


Figure 30.1.3-2—Foundation Treatment with Placed Bedding

where:

s = diameter

b = bedding width

d = bedding depth

30.2—WORKING DRAWINGS

When drawings and specifications are not provided in the contract documents, the Contractor shall provide to the Engineer ~~Where specified or requested by the Engineer, the Contractor shall provide~~ Manufacturer's installation instructions or working drawings and substantiating calculations in sufficient detail to permit a structural review. Sufficient copies shall be furnished to meet the needs of the Engineer and other entities with review authority. The working drawings shall be submitted sufficiently in advance of proposed installation and use to allow for their review, revision, if needed, and approval without delay of the work. The Contractor shall not start construction of any thermoplastic pipe installations for which working drawings are required until the drawings have been approved by the Engineer. Such approval will not relieve the Contractor of responsibility for results obtained by use of these drawings or any of the other responsibilities under the contract.

C30.2

Complete drawings and specifications include a detailed trench cross-section with all applicable dimensions, materials, and compaction levels identified at all locations and requirements for dewatering during construction.

30.3—MATERIALS

30.3.1—Thermoplastic ~~Pipe~~Culverts

2012 Revision

Polyethylene pipe (PE) shall conform to the material workmanship and inspection requirements of AASHTO M294, or ASTM F714 or F894.

Polyvinylchloride (PVC) ~~Poly, i.e., Vinyl Chloride or PVC,~~ pipe shall conform to the material workmanship and inspection requirements of AASHTO M278, or AASHTO M304 ~~ASTM F679, or ASTM F794~~.

Back to 2010 Edition

30.3.2—Bedding Materials and ~~Structural~~ Backfill

~~Bedding and structural backfill shall meet the requirements of AASHTO M 145, A-1, A-2-4, A-2-5, or A-3. Bedding material shall have a maximum particle size of 1.25 in. (30 mm). Backfill for thermoplastic pipe shall be free of organic material, stones larger than 1.5 in. (40 mm) in greatest dimension, or frozen lumps. Moisture content shall be in the range of optimum content, typically −3 percent to +2 percent, so as to permit thorough compaction. Consideration should be given to the potential for migration of fines from adjacent materials into open-graded backfill and bedding materials.~~

~~For pipe types that are not smooth on the outside, i.e., pipes with corrugated or profile walls, backfill gradations should be selected that will permit the filling of the corrugation or profile valleys.~~

~~Flowable fills, such as controlled low strength mortar, CLSM, or controlled density fill, CDF, may be used for backfill and bedding provided adequate flotation resistance can be achieved by restraints, weighing, or placement technique. With CLSM backfill, trench width can be reduced to a minimum of the outside diameter plus 12.0 in. (300 mm). When CLSM is used, all joints shall have gaskets.~~

30.3.2.1—General

Bedding shall be granular material with a maximum particle size of 1.0 in. Backfill shall be granular materials as specified in the contract documents; shall be free of organic material, rock fragments larger than 1.5 in. in the greatest dimension, and frozen lumps; and shall have a moisture content within the limits required for compaction.

Bedding and backfill materials shall meet the requirements of AASHTO M 145 for A-1, A-2-4, A-2-5, or A-3 soils. A maximum of 50 percent of the particle sizes may pass the No. 100 sieve and a maximum of 20 percent may pass the No. 200 sieve.

C30.3.2.1

Granular backfill has 35 percent or less material by weight finer than the No. 200 sieve as defined in AASHTO M 145.

While it is economical to use in situ material for bedding pipes, the Engineer must verify that the in situ material meets the requirements stated in Article 30.3.2.1. This is often difficult, since in situ soils are highly variable. If use of in situ material is allowed, the Engineer should include provisions for assessing it during construction and importing new bedding material if necessary.

Construction of pipes during the winter months may pose potential problems when frozen soils are included in the backfill zone or when frost-susceptible soils are used as backfill material. Frozen soil will not compact effectively and may result in points of concentrated loads when frozen and regions of inadequate support upon thawing.

Frost-susceptible soils should not be used in the embedment zone within the frost penetration depth. This will exclude the use of silty sand or silty gravel where freezing temperatures occur.

The restriction on materials passing the No. 100 sieve and the No. 200 sieve is intended to eliminate soils composed of significant amounts of fine sands and silts. Such materials are difficult to work with, are sensitive to moisture content, and do not provide support comparable to coarser or more-broadly-graded materials at the same percentage of maximum density. Restricted materials include some A-1-b, A-3, A-2-4, and A-2-5 soils. All A-2-6 and A-2-7 soils display similar characteristics and are eliminated from use as backfill materials. The Engineer may permit exceptions to these restrictions in special cases. If so, a suitable plan should be submitted for control of moisture content and compaction procedures.

These silty and clayey materials should never be used in a wet site or if significant live loads will be imposed on the pipe. Increased inspection levels should be considered if such a plan is approved.

30.3.2.2—Control of Particle Migration

The gradation of bedding and backfill materials shall be selected to prevent particle migration between adjacent materials. Gradations of in situ bedding, backfill, and embankment materials shall be evaluated for compliance with this requirement.

Alternatively, a suitable geotextile meeting the requirements of AASHTO M 288 for separation may be used to maintain separation of incompatible materials

C30.3.2.2

Control of migration is based on the relative gradations of adjacent materials. Acceptable materials should meet both the following criteria:

- $D_{15}/d_{85} < 5$ where D_{15} is the sieve-opening size passing 15 percent by weight of the coarser material and d_{85} is the sieve-opening size passing 85 percent by weight of the finer material, and

- $D_{50}/d_{50} < 25$ where D_{50} is the sieve-opening size passing 50 percent by weight of the coarser material and d_{50} is the sieve-opening size passing 50 percent by weight of the finer material. This criterion need not apply if the coarser material is well graded as defined in ASTM D2487.

30.3.2.3—Controlled Low-Strength Material

Controlled low-strength material (CLSM), also known as flowable fill, may be used as structural backfill. If not specified in the contract documents, a mix design and complete construction details must be submitted.Minimum construction details include methods for control of flotation forces and waiting time between placing CLSM and backfilling over the structure.

C30.3.2.3

McGrath et al. (1998) indicates that CLSM can be an effective backfill material for thermoplastic pipes. Other research has been conducted on this subject e.g. (Folliard et al., 2008).

30.4—ASSEMBLY

30.4.1—General

Thermoplastic pipe units shall be assembled in accordance with the Manufacturer's instructions and as specified in the contract documents. Copies of the Manufacturer's assembly instructions shall be furnished to the installation crew.

All pipes shall be unloaded and handled with reasonable care. Pipe and fittings shall not be rolled or dragged over gravel or rock ~~and shall be prevented from striking rock or other hard objects during placement in trench or on bedding~~. Care shall be taken to prevent the units from striking rock or other hard objects during placement.

2010 Revision

Back to 2010 Edition

~~Thermoplastic pipe shall be placed in the bed starting at the downstream end.~~

Damaged pipe or fittings shall not be incorporated into the project.

30.4.2—Joints

Joints for thermoplastic pipe shall comply with the details shown in the contract documents and on the approved working drawings. Each joint shall be sealed to prevent infiltration of soil (soiltight), fines (silttight), or water (watertight) as required by the contact documents. Field tests may be required by the Engineer whenever there is a question regarding compliance with the contract requirements.

Joints shall be installed so that the connection of pipe sections will form a continuous surface free from irregularities in the flow line.

C30.4.2

Joint types include corrugated bands, bell-and-spigot pipe ends, and double-bell couplings. All of these joint types can be supplied with or without gaskets. Other joint types may be used provided that documentation is provided to demonstrate that the joint meets the project requirements.

Joints are often provided as soiltight or watertight. Definitions of soiltight and silttight joints are vague. Examples can be found in these Specifications, Section 26, "Metal Pipes." Watertight joints are normally specified to meet ASTM D3212. Pressure capability of joint shall be based on project requirements. Commonly available pressure capabilities are 2, 5, and 10 psi. The AASHTO materials engineers are currently working on joint definition issues.

~~30.4.2.1—General~~

~~Joints for thermoplastic pipe shall meet the performance requirements for soiltightness unless watertightness is specified.~~

~~C30.4.2.1~~

~~Watertightness shall be based on project requirements. Available watertightness pressure levels are 2, 5, and 10 psi (0.015, 0.035, and 0.070 MPa).~~

~~30.4.2.2—Field Joints~~

~~Joints shall be so installed that the connection of pipe sections will form a continuous line free from irregularities in the flow line.~~

~~C30.4.2.2~~

~~Suitable field joints can be obtained with the following types of connections:~~

- ~~Corrugated bands (with or without gaskets)~~
- ~~Bell and spigot pipe ends (with or without gaskets)~~
- ~~Double bell couplings (with or without gaskets)~~

30.5—INSTALLATION

Back to 2010 Edition

30.5.1—General ~~Installation Requirements~~

Space shall be provided at the site for storage of the culvert pipes unless they are installed as delivered.

Unanticipated ground conditions shall be reported to the Engineer. Water conditions shall be controlled so that pipes are laid in dry conditions.

All pipe laying, joining, and backfilling shall be in accordance with the stricter of the Manufacturer's instructions, contract documents, or these Specifications.

~~Trenches shall be excavated in such a manner as to ensure that the sides will be stable under all working conditions. Trench walls shall be sloped or supported in conformance with all standards of safety. Only as much trench as can be safely maintained shall be opened. All trenches shall be backfilled as soon as practicable, but not later than the end of each working day.~~

C30.5.1

Controlling groundwater without violating the assumptions of the pipe design is important. It is often necessary to consult with a geotechnical engineer to address drainage issues.

2010 Revision

Back to 2010 Edition

~~Trench details, including foundation, bedding, haunching, initial backfill, final backfill, pipe zone, and trench width shall be taken as shown in Figure 30.5-1.~~

~~**FIGURE HERE**~~

~~**Figure 30.5.1-1—Trench Details**~~

30.5.2—Excavation and Groundwater Control

Excavation shall be to the width, depth, and grade shown in the contract documents. Trenches shall be excavated in such a manner as to ensure that the sides will be stable under all working conditions. All construction shall be in conformance to all applicable safety standards.

Only open as much trench as can be safely maintained each working day. Backfill trenches as soon as practicable but no later than the end of the working day.

Trench walls shall be sloped, benched, braced, or otherwise supported to ensure their stability throughout construction in conformance to all applicable safety standards. Large stones, rocks, and any debris falling into the trench shall be removed.

Sloped walls may be benched to facilitate compaction of backfill against them. If horizontal trench bracing is used, it shall be removed as backfill progresses upward. Unless otherwise directed by the Engineer, sheeting driven below the top of the pipe elevation shall be left in place and cut off not less than 1.5 ft above the top of the pipe after backfill has been installed to this elevation.

Where seepage is present, sheeting with soiltight joints shall be used to prevent washing out of soil behind the sheeting. Necessary action shall be taken to prevent surface runoff from entering the trench.

A movable trench shield may be used as an alternative to sheeting and bracing to provide a safe working condition in the bottom of a trench. The trench shield shall be used in a manner that will not leave voids in the backfill or disrupt compacted backfill as the trench shield is advanced.

For installations where the top of culvert extends above or within the rise of existing ground and the existing ground will be covered with an embankment, all vegetation, organic or frozen material, and any soft materials that do not meet the stiffness requirements of the structural backfill shall be removed for a distance at least equal to the culvert diameter each side of the culvert spring line and shall be replaced with embankment material.

C30.5.2

Since trench width has an impact on the performance of the pipe, the *AASHTO LRFD Bridge Design Specifications* also provides guidance on trench width.

The Engineer may allow trenches to be left open overnight provided that the excavation is secured in accordance with all applicable safety standards and prevented from accumulating water from rain.

If possible, the trench walls below the top of the pipe should be vertical.

Trench walls should be undisturbed in situ soil at least up to the top of the culvert at the time of backfilling.

In instances where the depth of fill over the culvert will be significant, the replacement material above the existing ground may need to be structural backfill to provide a suitable embedment zone. The *AASHTO LRFD Bridge Design Specifications*, which considers the stiffness of the material at the site of the pipe, provides guidance.

30.5.3—Groundwater Control

Water conditions shall be controlled. Water seeping into the trench from the sides or top shall be removed. If continuous seepage occurs, a sump pump shall be used to remove the water. When the water table is above the bottom of the trench, quick conditions or instability of the trench bottom can occur in which case the groundwater level shall

Back to 2010 Edition

be lowered to below the trench bottom. This water level shall be maintained until the pipe and sufficient backfill are placed to compensate for the uplift forces.

Sites requiring excavation below the groundwater table shall be dewatered to at least 12.0 in. below the deepest portion of the excavation, or when the culvert is installed in a stream or river bed, the water shall be diverted or separated by cofferdams. Advance approval of the Engineer shall be obtained if construction must continue in water. Under these conditions, free-draining gravels shall be used as foundation and bedding.

30.5.4 ~~30.5.2~~ Trench Widths

Trench width shall be sufficient to ensure working room to properly and safely place and compact haunching and other backfill materials. If not specified in the contract documents, the space between the pipe and trench wall shall should be wider than the compaction equipment used in the pipe zone~~.~~ but not ~~Minimum trench width shall not be~~ less than 1.5 times the pipe outside diameter plus 12.0 in. Trenches shall be kept to the specified width as any increase in trench width will increase the load on the pipe. Structural backfill shall ~~be comprised of well-graded, granular materials, such as AASHTO A-1, A-2-4 and A-2-5 soils to facilitate better consolidation around the pipe and to minimize the possibility of soil migration and piping of the in-situ soils~~ meet the requirements of Article 30.3.2.1.

If the trench walls do not stand without support, the trench width shall be increased in accordance with the design criteria in the *AASHTO LRFD Bridge Design Specifications,* Article ~~12.12.3.4 "Thrust"~~ 12.12.3.5 "Factored and Service Load".

Determination of trench width in soils that are unsupported~~,~~ and unstable ~~soils~~ shall include consideration of the size of the pipe, the stiffness of the backfill and in situ soil, the depth of cover and other site-specific conditions as applicable. ~~The trench shall be excavated to the width, depth, and grade as indicated on the plans and/or given by the Engineer.~~

C30.5.4

Narrower trenches may be allowed if culverts are backfilled with CLSM. CLSM readily flows under and around the culvert to provide good haunch and sidefill support, thus reducing the needed working room at the side of the culvert.

Flexible pipe requires soil support at the sides, and unstable trench walls are an indication that a wider trench width is required. This criterion does not refer to trenches for which trench supports are required only to comply with OSHA.

The *AASHTO LRFD Bridge Design Specifications* provide additional guidance on the width of trench backfill in wet native soil conditions. Generally, stable trench walls, during the excavation process, are predictive of adequate vertical and lateral stiffness for this application.

30.5.5 ~~30.5.3~~ Foundations ~~and Bedding~~

The foundation under the culvert shall be investigated for its adequacy to support the loads. The foundation soil shall be investigated for the full width of the trench, or for wide trench or embankment installations, a width of 12.0 in. or one-half the span of the culvert, whichever is larger, on each side of the culvert springline. The remedies for soft or inadequate foundation soils noted below shall apply to the same widths as investigated (see Figure 30.1.3-2). The foundation depths specified shall be taken to include the combined foundation and bedding depth.

- For rock and boulders, use b = culvert diameter 1D, and use a minimum d = 6.0 in., but not less than twice

C30.5.5

Back to 2010 Edition

A foundation should be provided such that the structure backfill does not settle more than the pipe to avoid downdrag loads on the culvert and to maintain specified pipe invert elevations. If the foundation is firm under the pipe but soft at the sides, compression of the soft material can cause increased load on the pipe due to downdrag. Thus, the foundation quality must be evaluated for a width greater than the pipe.

2010 Revision

the corrugation.

- For soft spots, use $b = 2D$ or the trench width, whichever is smaller, and use a minimum $d = 4.0$ in. but not less than the corrugation depth or less than a depth sufficient to reduce the stress on the soft soil to its allowable bearing value.

If the in situ soil is suitable to support the culvert, it may be considered adequate to serve as the foundation. The bottom of the excavation shall be undisturbed in situ material. If boulders, rock, vegetation, organic or frozen material, or any soft materials that do not meet the stiffness requirements of the structural backfill are present, they shall be removed for a width of at least one-half diameter on either side of the culvert to a depth specified by the Engineer and replaced with specified bedding material. If the foundation is loose, it shall be compacted as specified, but to not less than 90 percent of maximum density per AASHTO T 99 for A-1 or A-3 soils or 95 percent of maximum density for A-2 soils, before placing the culvert. After compaction, the foundation shall be uniformly firm and level to support the culvert along its length.

When the natural foundation soil is judged by the Engineer to be unsatisfactory to support the pipe or structural backfill, the soil shall be excavated to the depth *d* and width *b* prescribed in the contract documents.The excavation shall be backfilled with bedding material compacted as specified but to not less than 90 percent of maximum dry density per AASHTO T 99 for A-1 or A-3 soils or 95 percent of maximum dry density for A-2 soils.

~~Foundation and bedding shall meet the requirements of Article 30.3.2 and shall be installed as required by the Engineer according to conditions in the trench bottom. A stable and uniform bedding shall be provided for the pipe and any protruding features of its joint and/or fittings. The middle of the bedding equal to one-third the pipe outside diameter (OD) should be loosely placed, while the remainder shall be compacted to a minimum 90 percent of maximum density per AASHTO T 99. A minimum of 4.0 in. (100 mm) of bedding shall be provided prior to placement of the pipe unless otherwise specified.~~

~~When rock or unyielding material is present in the trench bottom, a cushion of bedding of 6.0 in. (150 mm) minimum thickness shall be provided below the bottom of the pipe.~~

~~When the trench bottom is unstable, material shall be excavated to the depth required by the Engineer, and replaced with a suitable foundation. A suitably graded material shall be used where conditions may cause migration of fines and loss of pipe support.~~

30.5.6—Bedding

~~A bedding layer shall be provided to the thickness specified. Unless indicated otherwise in the contract documents, the bedding shall be compacted to a minimum~~

C30.5.6

The maximum desired loose bedding layer thickness for compaction is 6.0 in.

Leaving the center third of the bedding uncompacted

2010 Revision

Back to 2010 Edition

~~density equal to 90 percent of the maximum dry density per AASHTO T 99, except that the portion of the bedding layer under the center third of the culvert diameter shall be left uncompacted.~~

The bedding surface shall conform to the specified elevation, grade, and alignment and shall be straight and flat over the length of the pipe section so that unacceptable longitudinal bending does not occur, and the pipe drains as designed.

provides a soft cushion for the pipe, minimizing hard support on the invert. In addition, the compacted bedding at the sides of the culvert provides a path for soil to arch over the top of the culvert. Many states require 95 percent of the maximum dry density per AASHTO T 99 for the bedding material. These high standards may not be achievable without overexcavation and replacement of the foundation soils or geosynthetic reinforcement if the foundation material is not sufficiently stiff.

In soft native soils it may be necessary to over excavate and replace the foundation material to achieve specified compaction of the outer bedding. Proper dewatering of the in situ soils is very important.

The tolerance of the culvert to longitudinal bending depends on the culvert material and geometric properties.

30.5.7—Placing Culvert Sections

The culvert shall be placed after the foundation soil and bedding are prepared. Pipes shall be placed on the bedding starting at the downstream end. If less than a full length of pipe is needed to meet the plan specified length, the partial piece shall not be the terminal piece.

C30.5.7

While it is preferable to lay pipe starting at the downstream end, the Engineer may grant permission to begin at other locations.

30.5.8 ~~30.5.4~~—Structural Backfill

~~Structural backfill shall meet the requirements of Article 30.3.2. Structural backfill shall be placed and compacted in layers not exceeding an 8.0 in. (200 mm) loose lift thickness and brought up evenly and simultaneously on both sides of the pipe to an elevation not less than 1.0 ft (300 mm) above the top of the pipe. Structural backfill shall be worked into the haunch area and compacted by hand.~~

~~A minimum compaction level of 90 percent standard density per AASHTO T 99 shall be achieved. Special compaction means may be necessary in the haunch area as shown in Figure 30.5.4-1. All compaction equipment used within 3.0 ft (900 mm) of the pipe shall be approved by the Engineer. Ponding or jetting the structural backfill to achieve compaction shall not be permitted without written permission from the Engineer.~~

~~Backfill materials placed in the zone extending more than 1.0 ft (300 mm) above the pipe to final grade shall be selected, placed, and compacted to satisfy the loading, pavement, and other requirements above the pipe.~~

30.5.8.1—General

Equipment and construction procedures used to backfill culverts shall be selected such that requirements for backfill density and control of culvert deflection and shape will be met. Sufficient inspection and testing should be undertaken to verify that the quality of the soil and the compactive effort are as specified.

Placing and compacting backfill to the top of the culvert shall be completed in such a manner that the culvert shape is not distorted.

C30.5.8.1

The *AASHTO LRFD Bridge Design Specifications* provides guidance on the suitability of in situ soils for use in the structural backfill zone.

Once a backfilling procedure is established, the primary inspection effort should be to ensure that the established procedure is followed. Only occasional checks

2010 Revision

Back to 2010 Edition

When trench wall supports are used, they shall be left in place below the top of the culvert or removed in a manner that avoids disturbing compacted backfill.

All foreign material falling into the trench during placement and compacting of the backfill shall be removed.

For each of the three basic stages of construction (backfilling) i.e. haunch, sidefill, and topfill stages, procedures shall be established that will achieve the specified degree of compaction without damaging or excessively distorting the culvert.

of soil density may then be required, as long as the material and procedures are unchanged. A good construction control plan will improve efficiency of installation effort and help ensure proper performance without having to rely on time-consuming testing, particularly in the haunch area, which is difficult to access.

Sidefill compaction can impose significant lateral forces on the pipe wall. This will increase the vertical diameter of the pipes and, if excessive, can result in culvert wall distress. Upward deflection of the crown should not exceed three percent during sidefill compaction.

30.5.8.2—Backfilling under the Haunch

Material shall be carefully placed in the haunches using mechanical tampers, manual tampers, or other means that fill all voids and meet the specified compaction levels. Adjacent sidefill zones shall be placed along with the haunch zones to provide lateral support for the haunch material.

Installation of haunch fill shall be carried out on both sides simultaneously to avoid rolling the culvert, and the compaction force shall be controlled so that the culvert is not lifted off grade and the bottom of the culvert is not damaged.

If the culvert is to be backfilled with CLSM, all requirements of the project specifications or the submitted detailed work plan shall be followed. Provide means to prevent culvert flotation shall be provided.

Water jetting to densify the backfill shall not be allowed unless approved in advance by the Engineer.

C30.5.8.2

It is important that the selected tamping procedures will meet the design assumptions. In general, a minimum compaction level exceeding 85 percent AASHTO T 99 is needed to prevent a collapsing soil structure upon saturation. The effort required to achieve a particular degree of compaction varies with the backfill material type. Investigation of various means of achieving compaction in the haunch zone and the effect of haunch support on buried pipe performance is reported in the FHWA Report, *Pipe Interaction with the Backfill Envelope*, FHWA-RD-98-191, by McGrath et al. (1998).

These studies showed that large void spaces result underneath culverts without good compaction in the haunch area. Loose layers should generally not exceed 6.0 in. in thickness to permit the backfill material to be worked into the haunch zone. Shovel slicing was shown to be effective in providing haunch support. Different sized tampers were shown to be effective for different backfill soils. A large faced tamper (3 × 6 in.) was effective for silty sand, while a small faced tamper (1 × 3 in.) was effective for crushed stone backfill. Haunching is best accomplished by placing part of the first layer of backfill, working it into the haunches, and then placing the remainder of the lift. Thick layers prevent material from being worked into the haunches.

Water-jetting has been found to be an effective procedure for compacting backfill and developing uniform support with clean coarse-grained backfills and good drainage; however, problems have been encountered in achieving consistent results and verification is difficult.

30.5.8.3—Sidefill

Structural backfill material in the sidefill zone shall be placed in horizontal, uniform layers not exceeding 8.0 in. loose thickness unless a larger thickness is specified. The layers shall be compacted with appropriate equipment to not less than 90 percent of maximum density per AASHTO T 99 for A-1 and A-3 soils and 95 percent A-2-4 and A2-5 soils.

A maximum of 50 percent of the particle sizes may

C30.5.8.3

Design soil stiffness is very sensitive to the level of compaction (FHWA Report FHWA-RD-98-191 *Pipe Interaction with Backfill Envelope*, McGrath et al. 1998). Many states require 95 percent maximum dry density per AASHTO T99 for all sidefill soils. Achieving these high standards is highly dependent upon local practice and available materials.

Generally, compaction of fill material to the required

2010 Revision

pass the No. 100 sieve and a maximum of 20 percent may pass the No. 200 sieve.

The maximum difference in the sidefill surface elevations between the two sides of the culvert at any time shall not exceed one-quarter of the diameter, or 24.0 in. whichever is smaller. For pipe less than 24.0 in. in diameter, this difference need not be less than one-half the diameter.

The sidefill surface elevation shall be kept at or below the level of adjacent soil or embankment. Placement and compaction of the sidefill layers adjacent to the haunch zone shall be carried out concurrently with backfilling under the haunch.

Sidefill material shall be placed, spread, and compacted working parallel to the culvert to avoid creating areas of unequal support.

Equipment used to compact sidefill within 3.0 ft from each side of the culvert shall not impose excessive force on the culvert that results in distorting the culvert shape. Thermoplastic culverts are flexible, thus sidefill material shall be placed and compacted to avoid excessive and unsymmetrical deformations. The shape shall be monitored to ensure satisfactory results.

density is dependent on the thickness of the layer of fill being compacted, soil type, amount of compactive force, and length of time the force is applied. Experience with compaction indicates that 8.0 in. thick loose layers using two coverages with a given compactor will give better uniformity and higher average level of compaction than one 12.0 in. thick loose layer with four coverages of the same compactor. Alternatively, a 12.0 in. loose layer will require larger compactors to produce the same average compaction as achieved by a smaller compactor with a 6.0 in. thick layer. Larger compactors must be evaluated for possible induced structural distortions.

Unequal support may result when compacting perpendicular to the culvert long axis.

30.5.8.4—Topfill

Placement of structural topfill shall be taken to begin when the sidefill elevation reaches the shoulders. Procedures, as approved by the Engineer, shall be used for placing and compacting topfill. Topfill need not extend above the top of the pipe more than 6.0 in. unless otherwise specified.

Additional material over the topfill shall be provided to protect all culverts before permitting heavy construction equipment to pass over them. Construction loads may require additional cover beyond that required for the final condition to which the design loads apply.

C30.5.8.4

See design specifications for guidance on minimum cover depths.

Specifying a depth of 6.0 in. above the top of the pipe provides protection for the pipe as less controlled materials are placed and compacted to complete the trench backfill. This dimension is unrelated to the depth of fill required to prevent damage from vehicles passing over the pipe.

30.6~~30.5.5~~—MINIMUM COVER

2012 Revision

A minimum depth of cover above the ~~pipe~~culvert should be maintained before allowing vehicles or heavy construction equipment to traverse the pipe trench. The minimum depth of cover should be established by the Engineer based on an evaluation of specific project conditions. For embedment materials installed to the minimum density given in Article 30.5.8, "Structural Backfill," cover of at least 2.0 ft ~~(600 mm)~~ shall be provided before allowing vehicles or construction equipment to cross the trench surface. Minimum cover for construction loads shall be as shown in Table 30.~~5.5~~6-1. Hydrohammer type compactors shall not be used over the pipe.

~~C30.5.5~~ C30.6

Diameters greater than 4.0 ft ~~(1200 mm)~~ in Table ~~30.5.5-1~~ 30.6-1 are for information only. See Tables A12-11, A12-12, and A12-13 of the *AASHTO LRFD Bridge Design Specifications* for nominal sizes.

2010 Revision

Back to 2010 Edition

Table 30.6-1 ~~Table 30.5.5-1~~—Minimum Cover for Construction Loads

	Minimum Cover, in., for Indicated Axle Loads, kips			
Nominal Pipe Diameter, ft	18.0–50.0	50.0–75.0	75.0–110.0	110.0–150.0
2.0–3.0	24.0	30.0	36.0	36.0
3.5–4.0	36.0	36.0	42.0	48.0
4.5–5.0	36.0	36.0	42.0	48.0

Minimum cover shall be measured from the top of the pipe to the top of the maintained construction roadway surface. If unpaved, the surface shall be maintained.

30.7—INSPECTION REQUIREMENTS ~~30.5.6 Inspection Requirements~~

30.7.1~~30.5.6.1~~—Visual Inspection

All pipes shall undergo inspection during and after installation to ensure proper performance. Installation of bedding and backfill materials, as well as their placement and compaction, shall be determined to meet the requirements of this section.

During the initial phases of the installation process, inspection shall concentrate on detecting improper practice and poor workmanship. Errors in line and grade, as well as any improper assembly or backfill techniques, shall be corrected prior to placing significant backfill or trench fill. Coupling bands shall be properly indexed with the corrugation and tightened, and bell/spigot joints shall be properly assembled to prevent the infiltration of soil fines. Where gaskets are used, they shall be properly seated to prevent groundwater infiltration and should appear uniformly oriented around the pipe. In areas where cracking or joint separation is found, a remediation or replacement plan shall be submitted for approval.

Final internal inspections shall be conducted on all buried thermoplastic pipe installations to evaluate issues that may affect long-term performance. Final inspections shall be conducted no sooner than 30 days after completion of installation and final fill.

Shallow cover installations shall be checked to ensure the minimum cover level is provided.

C30.7.1 ~~C30.5.6.1~~

Inspection at the appropriate times during installation will detect and allow correction of line and grade, jointing, and shape change problems. The timing and number of inspections required will vary with the significance and depth of the installation. The contractor is advised to provide initial inspections himself to avoid problems later on. Racking or flattening of the pipe's curvature indicates improper backfill placement methods that must be corrected. Slight peaking of the cross-sectional shape should be taken as indicative of achieving proper compaction requirements.

Soil consolidation continues with time after installation of the pipe. While 30 days will not encompass the time frame for complete consolidation of the soil surrounding the pipe, it is intended to give sufficient time to observe some of the effects that this consolidation will have. However, occasionally pavement is placed over the pipe sooner than 30 days. While the 30-day time limit should be maintained, a brief inspection of the pipe prior to paving over it, particularly for the first few joints, may be prudent to ensure that good construction practices are being applied.

It is recommended that inspection personnel not enter culverts less than 24.0 in. ~~(600 mm)~~ in diameter. Internal inspection of culverts in this size range is best conducted using video cameras. Culverts should be entered only by inspection personnel trained in working within confined spaces and using procedures in full compliance with applicable State, Local, and Federal OSHA regulations.

30.7.2~~30.5.6.2~~—Installation Deflection

The pipe shall be evaluated to determine whether the internal diameter of the barrel has been reduced more than five percent when measured not less than 30 days following completion of installation.

Pipes shall be checked for deflection using a mandrel or any other device approved by the Engineer that can physically verify the dimensions of the pipe and is not

C30.7.2 ~~C30.5.6.2~~ Back to 2010 Edition

~~Inspection criteria are newly added to the specification, as there was minimal guidance in the previous specification.~~Ten percent of each pipe installation shall be defined as ten percent of the number of pipe runs, and not less than ten percent of the total length of installed pipe on the project. The requirement of deflection testing ten percent of each pipe installation is intended to serve as

Back to 2010 Edition

limited by poor lighting, waterflow, pipe length, or other limiting conditions of the installed environment. Pipes larger than 24.0 in. ~~(600 mm)~~ may be entered and deflection levels measured directly.

In all pipe installations, at least ten percent of the total number of pipe runs representing at least ten percent of the total pipe footage on the project shall be randomly selected by the Engineer and inspected for deflection. Also, as determined by the 100 percent visual inspection in ~~Section~~ Article 30.~~5.6.1~~7.1, all areas in which deflection can be visually detected shall be inspected for deflection.

Where direct measurements are made, a measurement shall be taken once every 10.0 ft ~~(3000 mm)~~ for the length of the pipe, and a minimum of four measurements per pipe installation is required.

If a mandrel is used for the deflection test, it shall be a nine (or greater odd number) arm mandrel, and shall be sized and inspected by the Engineer prior to testing. A properly sized proving ring shall be used to check or test the mandrel for accuracy. The mandrel shall be pulled through the pipe by hand with a rope or cable. Where applicable, pulleys may be incorporated into the system to change the direction of pull so that inspection personnel need not physically enter the pipe or manhole.

For locations where pipe deflection exceeds five percent of the inside diameter, an evaluation shall be conducted by the Contractor utilizing a Professional Engineer and submitted to the Engineer for review and approval considering the severity of the deflection, structural integrity, environmental conditions, and the design service life of the pipe. Pipe remediation or replacement shall be required for locations where the evaluation finds that the deflection could be problematic. For locations where pipe deflection exceeds 7.5 percent of the inside diameter, remediation or replacement of the pipe is required.

a minimum and does not limit owners from more stringent requirements.

The pipe inside diameters should be provided by the pipe manufacturer for every size and type of pipe delivered. If the pipe inside diameter is not provided, or is not available, pipe inside diameter can be developed by averaging the diameters measured at eight equally spaced locations around a section of unloaded pipe for every given size and manufacturer.

There are many appropriate methods suitable for measuring deflection, including video inspection equipment, mandrels, and other direct measurement devices. For pipes tested by a mandrel, the mandrel shall be pulled through the entire pipe. Whichever method is used for deflection measurement, a minimum of ten percent of the total length of installed pipe shall be tested, in addition to any areas that were identified in the visual inspection as having deflection.

Installed pipe deflections that exceed five percent of the initial inside diameter may indicate that the installation was substandard. Appropriate remediation, if any, will depend upon the severity of the deflection, the condition of the pipe, and evaluation of the factor of safety using Section 12, "Buried Structures and Tunnel Liners," of the *AASHTO LRFD Bridge Design Specifications*. Installed pipe deflections that exceed 7.5 percent of the initial inside diameter will require remediation or replacement of the pipe.

30.7.3—Compaction Control

Field compaction shall be evaluated based on compacted density and moisture content obtained from acceptable methods, such as the cone replacement (AASHTO T 191, ASTM D1556) or the nuclear gage (ASTM D2922 and D3017). A reference density test shall be performed on a representative sample to obtain a value of maximum dry density (MDD) and optimum moisture content (OMC). This test shall be repeated for each new soil type encountered and for composition variations within the same soil type. Thus, samples should be taken periodically during construction to provide an appropriate series of reference tests.

The contract documents shall determine the number and location of field tests to ensure that the quality of the soil and the compaction obtained is as specified and shall stipulate acceptance criteria for the compacted soil.

C30.7.3

The best approach to compaction control is to conduct frequent tests early in the project to establish the critical control parameters that achieve the specified compaction level, such as type of equipment, number of passes, and moisture content. Once the parameters are established, the test frequency can be reduced as long as the identified parameters are monitored.

2010 Revision

Back to 2010 Edition

30.7.4—Use of Reduced Installation Factor

If the culvert design was completed with an installation factor less than 1.5, additional deflection checks shall be required during construction. Before the beginning of construction, the contractor shall submit and obtain approval for a detailed construction plan that meets the performance measures specified by the Designer.

C30.7.4

2012 Revision

If a reduced installation factor is used, specific installation controls are required in the contract documents to actively monitor backfill gradation and compaction levels at the side of the pipes throughout the construction process. Change in the vertical pipe diameter are to be measured when the backfill reaches the top of the pipe. At this point in the construction process, the vertical pipe diameter is to be greater than the vertical diameter prior to backfilling, but not more than three percent greater than the vertical diameter prior to backfilling.

30.8~~30.6~~—MEASUREMENT

Culvert pipes shall be measured in linear feet installed in place, completed, and accepted. The number of feet shall be the average of the top and bottom centerline lengths of pipe.

~~Pipe installations shall be measured in linear feet (meters) installed in place, completed and accepted. The number of feet (meters) shall be the centerline lengths of the pipe.~~

30.9~~30.7~~—PAYMENT

Back to 2010 Edition

The length as measured above will be paid for at the contract prices per linear foot ~~(meter)~~ bid for thermoplastic pipe of the sizes specified. Such price and payment shall constitute full compensation for furnishing, handling, and installing the pipe and for all materials, labor, equipment, tools, and incidentals necessary to complete this item. Such price and payment shall also include excavation, bedding material, backfill, headwalls, endwalls, and foundations for pipe.

30.10—REFERENCES

AASHTO. 2010. *AASHTO LRFD Bridge Design Specifications*, Fifth Edition, LRFDUS-5 or LRFDUS-5-UL. American Association of State and Highway Transportation Officials, Washington, DC. Available in hard copy with CD-ROM or downloadable form.

AASHTO. ~~2006~~ 2009. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, ~~26th~~ 29th Edition, ~~HM-26~~ HM-29. American Association of State Highway and Transportation Officials, Washington, DC. Includes AASHTO M, R, and T standards, which are also available individually in downloadable form.

McGrath, T. J., E. T. Selig, M. C. Webb, and G. V. Zoladz. 1998. *Pipe Interaction with the Backfill Envelope*, FHWA-RD-98-191. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

NCHRP. 2008. *Development of a Recommended Practice for Use of Controlled Low-Strength Material in Highway Construction*, Report 597. National Cooperative Highway Research Program, Transportation Research Board, National Research Council, Washington, DC. Prepared by K. J. Folliard, D. Lianxiang, D. Trejo, C. Halmen, S Sabol, and D. Leshchinsky.

NCHRP. 2009. *Updated Test and Design Methods for Thermoplastic Drainage Pipe*, Report 631. National Cooperative Highway Research Program, Transportation Research Board, National Research Council, Washington, DC. Prepared by T .J. McGrath, I. D. Moore, and G. Y. Hsuan.

2010
Revision

Back to 2010 Edition

SECTION 30: THERMOPLASTIC PIPE

30.1.1—Description

Revise this Article as follows:

This work shall consist of furnishing, installing, and inspecting buried thermoplastic and fiberglass culverts in conformance with these Specifications and the contract documents. Unless otherwise noted, provisions for thermoplastic culverts shall be applicable to fiberglass culverts.

C30.1.1

Revise paragraph 1 of this Article as follows:

As used in this Specification, thermoplastic and fiberglass pipe ~~is~~ are defined in the *AASHTO LRFD Bridge Design Specifications*, Section 12, "Buried Structures and Tunnel Liners." [The fiberglass pipe definition will appear in the *2013 Interim to the AASHTO LRFD Bridge Design Specifications*; publication pending as of this writing. —ed.]

30.3.1—Thermoplastic Culverts 2010 Revision

Add the following paragraph to the end of this Article:

Fiberglass pipe shall conform to the material workmanship and inspection requirements of ASTM D3262.

Page left intentionally blank.

30.6—MINIMUM COVER 2010 Revision

Revise the title of Table 30.6-1 and add Note 1 below the table as follows:

Table 30.6-1–Minimum Cover for Construction Loads[1]

	Minimum Cover, in., for Indicated Axle Loads, kips			
Nominal Pipe Diameter, ft	18.0–50.0	50.0–75.0	75.0–110.0	110.0–150.0
2.0–3.0	24.0	30.0	36.0	36.0
3.5–4.0	36.0	36.0	42.0	48.0
4.5–5.0	36.0	36.0	42.0	48.0

[1] For fiberglass pipe greater than 5.0 ft in diameter, the minimum cover requirement for 5.0-ft diameter pipe applies.

C30.6

Revise this Article as follows:

Diameters ~~greater than 4.0 ft~~ in Table 30.6-1 are for information only. See Tables A12-11, A12-12, and A12-13 of the *AASHTO LRFD Bridge Design Specifications* for nominal sizes of thermoplastic pipe. See ASTM D3262 for nominal sizes of fiberglass pipe.

2012 Revision

Page left intentionally blank.

Revise Article 30.7.4 as follows:

30.7.4~~—Use of Reduced Installation Factor~~Inspection of Special Installations

2010 Revision

30.7.4.1—Use of Reduced Installation Factor

If the thermoplastic culvert design was completed with an installation factor less than 1.5, additional deflection checks shall be required during construction. Before the beginning of construction, the contractor shall submit and obtain approval for a detailed construction plan that meets the performance measures specified by the Designer. This provision is not applicable to fiberglass pipes, which do not have an optional reduced installation factor for design.

30.7.4.2—Allowable Deflection for Fiberglass Culverts

Where fiberglass culverts are designed with an allowable deflection less than 5.0 percent, the allowable deflection and the upper deflection limit where remediation or replacement is required shall be prominently displayed in the working drawings; and the contractor shall submit an inspection plan to the Engineer for approval, acknowledging the allowable deflection used in design and how the contractor will be evaluating that deflection in the field. The 5.0 percent maximum allowable deflection requirement in Article 30.7.2, "Installation Deflection," shall be replaced by the allowable deflection used in the design.

Insert new Commentary to Article 30.7.4.2:

C30.7.4.2

The minimum stiffness of fiberglass pipe is limited by the *AASHTO LRFD Bridge Design Specifications* and ASTM D3262, which should preclude excessive deflections for properly installed pipe. However, higher stiffness fiberglass pipe can be easily produced by increasing the quantity of glass fiber reinforcement or the wall thickness, or both. The allowable deflection for higher stiffness pipes may need to be reduced because pipes with high values of pipe stiffness will reach the pipe's long-term ring-bending strain limit at a deflection less than 5.0 percent. Therefore, the Engineer must communicate the allowable deflection for such pipes to the contractor and inspector, and the contractor must submit an inspection plan to the Engineer for approval.

2012 Revision

Page left intentionally blank.

30.9—PAYMENT

Revise paragraph 1, sentence 1 of this Article as follows:

The length as measured above will be paid for at the contract prices per linear foot bid for thermoplastic or fiberglass pipe of the sizes specified.

30.10—REFERENCES

Add the following reference:

AWWA. 2005. "Fiberglass Pipe Design." *AWWA Manual of Water Supply Practice M45*, Second Edition. American Water Works Association, Denver, CO.

2012 Revision

SECTION 31: ALUMINUM STRUCTURES

TABLE OF CONTENTS

SECTION 31

ALUMINUM STRUCTURES

31.1—GENERAL

The *AASHTO LRFD Bridge Design Specifications*, Section 7, "Aluminum Structures," and the *LRFD Bridge Construction Specifications*, Section 31, "Aluminum Structures," shall supersede the AASHTO *Guide Specifications for Aluminum Highway Bridges*.

31.1.1—Description

This work shall consist of furnishing, fabricating, and erecting aluminum structures and structural aluminum portions of other structures in accordance with these Specifications, and the contract documents.

Unless otherwise specified, the structural aluminum fabricating plant shall be certified under the AISC Quality Certification Program, Category I. Fracture-critical members shall be fabricated in plants certified as Category III.

Details of design which are permitted to be selected by the Contractor shall conform to the *AASHTO LRFD Bridge Design Specifications,* 2007, and subsequent interim specifications.

Falsework used in the erection of structural aluminum shall conform to the provisions of Section 3, "Temporary Works."

Structural components designated in the contract documents as "fracture-critical" shall conform to the provisions of the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*, Section 12, "Fracture Control Plan (FCP) for Non-Redundant Members."

Welding and weld qualification tests shall conform to the provisions of the current ANSI/AWS D1.2/D1.2M *Structural Welding Code—Aluminum*.

C31.1.1

Additional information on aluminum structural design is available from the Aluminum Association, 900 19th Street, NW, Washington, DC 20006. *Aluminum Standards and Data 2003 (Metric SI)*, published by the Aluminum Association, gives information on aluminum alloys.

31.1.2—Notice of Beginning of Work

The Contractor shall give the Engineer ample notice of the beginning of work at the mill or in the shop, so that inspection may be provided. No material shall be manufactured or fabricated before the Engineer has been so notified.

31.1.3—Inspection

Structural aluminum shall be inspected at the fabrication site.

Except as specified herein, the Contractor shall furnish to the Engineer a copy of all mill orders and certified mill test reports. Mill test reports shall show the chemical analysis and physical test results for each lot of aluminum used in the work. With the approval of the Engineer, certificates of compliance shall be furnished in lieu of mill test reports for material that normally is not supplied with

C31.1.3

Information on certification documentation is contained in *Aluminum Standards and Data 2003 (Metric SI),* Section 4.

mill test reports, and for items such as fills, minor gusset plates, and similar material when quantities are small and the material is taken from stock.

Material to be used shall be made available to the Engineer so that each piece can be examined. The Engineer shall have free access at all times to any portion of the fabrication site where the material is stored or where work on the material is being performed.

31.1.4—Inspector's Authority

The Inspector shall have the authority to reject materials or work which does not fulfill the requirements of these Specifications. In cases of dispute, the Contractor may appeal to the Engineer, whose decision shall be final.

It is expressly understood that inspection at the mill and shop shall not relieve the Contractor of any responsibility in regard to defective material or work and the necessity for replacing the same at the Contractor's cost.

The acceptance of any material or finished members by the Inspector shall not be a bar to their subsequent rejection if found defective. Rejected materials and work shall be replaced or corrected as soon as practical by the Contractor.

C31.1.4

Inspection at the mill or shop is intended as a means of facilitating the work and avoiding errors.

31.2—WORKING DRAWINGS

The Contractor shall expressly understand that the Engineer's approval of the working drawings submitted by the Contractor covers the requirements for “strength and detail,” and that the Engineer assumes no responsibility for errors in dimensions.

Working drawings shall be approved by the Engineer prior to performance of the work involved and such approval shall not relieve the Contractor of any responsibility under the contract for the successful completion of the work.

31.2.1—Shop Drawings

The Contractor shall submit copies of the detailed shop drawings to the Engineer for approval. Shop drawings shall be submitted sufficiently in advance of the start of the affected work to allow time for review by the Engineer and corrections by the Contractor, if any, without delaying the work.

Shop drawings for aluminum structures shall give full detailed dimensions and sizes of component parts of the structure and details of all miscellaneous parts, such as pins, nuts, bolts, drains, etc.

31.2.2—Erection Drawings

The Contractor shall submit drawings fully illustrating the proposed method of erection. The drawings shall show details of all falsework bents, bracing, guys, dead-men, lifting devices, and attachments to the bridge members; and show sequence of erection, location of cranes and barges, crane capacities, location of lifting points on the bridge members, and weights of the members. The drawings shall be complete in detail for all anticipated phases and conditions during erection. Calculations may be required to demonstrate that factored resistances are not exceeded and that member capacities and final geometry will be correct.

31.2.3—Camber Diagram

A camber diagram shall be furnished to the Engineer by the Fabricator showing the camber at each panel point in the cases of trusses or arch ribs, and at the location of field splices and fractions of span length not less than quarter points in the cases of continuous beams and girders or rigid frames. The camber diagram shall show calculated cambers to be used in preassembly of the structure in accordance with Article 31.5.3, "Preassembly of Field Connections."

31.3—MATERIALS

31.3.1—Structural Aluminum

31.3.1.1—General

Aluminum shall be furnished according to the specifications herein. The aluminum alloys furnished shall be as specified in the contract documents.

31.3.1.2—Aluminum Plate

Unless otherwise specified in the contract documents, aluminum plate shall conform to ASTM B209.

31.3.1.3—Aluminum Extrusions

Unless otherwise specified in the contract documents, aluminum extrusions shall conform to ASTM B221, ASTM B308/B308M, or ASTM B429.

31.3.2—Fasteners

31.3.2.1—High-Strength Steel Fasteners

31.3.2.1.1—Material 2014 Revision

High-strength bolts for structural aluminum joints shall conform to AASHTO M 164 (ASTM A325), Type 1.

The supplier shall provide a lot number appearing on the shipping package and a certification noting when and where all testing was done, including rotational capacity tests and galvanizing thickness.

The maximum hardness for AASHTO M 164 (ASTM A325) bolts shall be 33 HRC.

Proof load tests complying with ASTM F606, Method 1, shall be required for the bolts. Wedge tests of full-size bolts are required in accordance with Section 8.3 of AASHTO M 164 (ASTM A325). Galvanized bolts shall be wedge tested after galvanizing. Proof load tests of AASHTO M 291 (ASTM A563) shall be required for the nuts. The proof load tests for nuts shall be performed after galvanizing, overtapping, and lubricating.

C31.3.2.1.1

AASHTO M 253 (ASTM A490), high-strength steel bolts are not used in aluminum structures because they may become embrittled by galvanizing. Galvanizing is typically required to prevent galvanic corrosion of aluminum in contact with steel.

Type 2 bolts have been withdrawn from AASHTO M 164 (ASTM A325) and AASHTO M 253 (ASTM A490) and, therefore, are no longer manufactured. However, Type 2 bolts manufactured before this discontinuation may still be in inventory and are considered acceptable.

Nuts for AASHTO M 164 (ASTM A325) bolts shall conform to AASHTO M 291 (ASTM A253), Grade DH (AASHTO M 291M (ASTM A563M), Property Class 10S).

All nuts shall be lubricated with lubricant containing a visible dye.

Hardened steel washers shall be used with high-strength steel bolts and shall conform to the requirements of AASHTO M 293 (ASTM F436).

31.3.2.1.2—Identifying Marks 2014 Revision

Bolts shall be identified by marks on their heads in accordance with AASHTO M 164 (ASTM A325). Nuts shall be identified by marks on one face in accordance with the specifications referenced in AASHTO M 164 (ASTM A325). Bolt head markings shall identify the grade by the symbol "A325" and the Manufacturer, if Type 3. Nut markings must identify the property class and the Manufacturer. Markings on washers shall identify the Manufacturer.

31.3.2.1.3—Dimensions

Bolt and nut dimensions shall conform to the requirements for Heavy Hexagon Structural Bolts and Heavy Semi-Finished Hexagon Nuts (Metric Heavy Hexagon Structural Bolts and for Metric Heavy Semi-Finished Hexagon Nuts) given in ANSI Standards B18.2.1 and B18.2.2 (B18.2.3.7M and B18.2.4.6M), respectively.

31.3.2.1.4—Galvanizing

High-strength steel fasteners used in aluminum structures shall either be hot-dip galvanized in accordance with AASHTO M 232M/M 232 (ASTM A153/A153M), Class C, or mechanically galvanized in accordance with AASHTO M 298 (ASTM B695), Class 50 (Class 345). Galvanized bolts shall be tension tested after galvanizing. Washers, nuts, and bolts of an assembly shall be galvanized by the same process. The nuts shall be overtapped to the minimum amount required for the fastener assembly, and shall be lubricated with a lubricant containing a visible dye so a visual check can be made for the lubricant at the time of installation.

C31.3.2.1.4 2014 Revision

AASHTO M 164 (ASTM A325) Type 2 bolts shall be mechanically galvanized only.

31.3.2.1.5—Alternative Fasteners 2014 Revision

Subject to the approval of the Engineer, other fastener or fastener assemblies may be used provided they:

- meet the materials, manufacturing, and chemical composition requirements of AASHTO M 164 (ASTM A325),
- meet the mechanical property requirements of AASHTO M 164 (ASTM A325) in full-size tests, and

- have body diameter and bearing areas under the head and nut, or their equivalent, not less than those provided by a bolt and nut of the same nominal dimensions prescribed in Article 31.3.2.1.3, "Dimensions."

Such alternate fasteners may differ in other dimensions from those of the specified bolts and nuts.

31.3.2.2—Aluminum Rivets

Aluminum rivets shall conform to ASTM B316/B316M.

31.3.2.3—Stainless Steel Bolts

Stainless steel bolts shall conform to ASTM F593 Alloy Groups 1, 2, or 3. Stainless steel nuts shall conform to ASTM F594.

C31.3.2.3

ASTM F593 Alloy Groups 1, 2, and 3 are 300 series stainless steel alloys. Fastener of Stainless Steel Alloys 304 and 316 are often used uncoated in aluminum structures.

31.3.2.4—Lock-Pin and Collar Fasteners

Aluminum, stainless steel, and steel lock-pin and collar fasteners shall conform to MIL-P-23469D. The shank and head of lock-pin and collar fasteners shall meet the requirements of Article 31.3.2.1.3, "Dimensions." Each fastener shall:

- provide a solid shank body of sufficient diameter to provide tensile and shear strength equivalent to or greater than that of the bolt specified in the contract documents,
- have a cold forged head of type and dimensions approved by the Engineer on one end,
- have a shank length suitable for material thickness fastened, and
- have locking grooves, breakneck groove, and pull grooves, all annular, on the opposite end.

Each fastener shall include a locking collar of proper size for the shank diameter used, which, by means of suitable installation tools, is cold swaged into the locking grooves forming head for the grooved end of the fastener after the pull groove section has been removed. The locking collar shall be a standard product of an established Manufacturer of lock-pin and collar fasteners and approved by the Engineer.

C31.3.2.4

Lock-pin and collar fasteners, also called lockbolts, are available in aluminum, stainless steel, and steel. Steel lock-pin and collar fasteners are usually zinc-plated when used in aluminum parts to discourage galvanic corrosion.

31.3.3—Welded Stud Shear Connectors

Welded stud shear connectors shall conform to ANSI/AWS D1.2/D1.2M *Structural Welding Code—Aluminum*.

C31.3.3

ANSI/AWS D1.2/D1.2M, Section 6, contains requirements for aluminum stud welding and contains material requirements for aluminum stud welding in Section 6.10. The largest aluminum stud diameter manufactured is 0.5 in. Shear connectors made from shapes that are bolted or welded to girders may also be used.

31.3.4—Aluminum Forgings

Aluminum forgings shall conform to ASTM B247.

31.3.5—Aluminum Castings

Aluminum-alloy sand castings shall conform to ASTM B26/B26M. Aluminum-alloy permanent mold castings shall conform to ASTM B108.

31.4—FABRICATION

31.4.1—Identification of Aluminum Alloys during Fabrication

The Contractor shall issue cutting instructions and mark individual pieces so as to be able to identify the material used for each piece. Metal stamping marks, scribe lines, and center punch marks shall not be used where they will remain on fabricated material.

The Contractor may furnish material that can be identified by lot and mill test report from stock.

During fabrication prior to assembly, each piece shall clearly show its material specification. Writing the material specification number on the piece or by using the identification color codes shown in Table 31.4.1-1 shall be taken as compliance with this provision.

Table 31.4.1-1—Identification Color Codes

Alloy	Color
5083	Red and Gray
5086	Red and Orange
6061	Blue
6063	Yellow and Green

Aluminum alloys not listed in Table 31.4.1-1 shall be marked with colors listed in *Aluminum Standards and Data*.

Any piece which will be subject to fabrication that might obscure its identification prior to assembly shall have a substantial tag affixed showing the material specification number.

Upon request by the Engineer, the Contractor shall furnish an affidavit certifying that the identification of pieces has been maintained in accordance with this specification.

C31.4.1

Aluminum Standards and Data gives color codes for additional alloys and other information on identification marking used by aluminum producers in Section 4.

31.4.2—Storage of Materials

Material shall be stored out of contact with the ground, free from dirt, grease, and foreign matter and out of contact with dissimilar materials such as uncoated steel.

31.4.3—Plates

31.4.3.1—Direction of Rolling

Unless otherwise specified in the contract documents, plates for main members and splice plates for flanges and main tension members, i.e., not secondary members, shall be cut and fabricated so that the primary direction of rolling is parallel to the direction of the main tensile and/or compressive stresses.

31.4.3.2—Plate Edges

Plates more than 0.5 in. thick carrying calculated stress shall not be sheared. All edges that have been cut by the arc process shall be planed to remove edge cracks. Oxygen cutting shall not be used. Re-entrant corners shall be filleted to a radius of 0.75 in. or more.

31.4.3.3—Bent Plates

31.4.3.3.1—General

Bend lines in unwelded, load-carrying, rolled aluminum plates shall be perpendicular to the direction of rolling.

Before bending, the corners of the plates shall be rounded to a radius of 0.0625 in. throughout the portion of the plate over which the bending is to occur.

31.4.3.3.2—Cold Bending 2010 Revision

Cold bending shall not produce cracking. For 90-degree bends, bend radii measured to the concave face of the metal shall not be less than those listed in Table 31.4.3.3.2-1.

Table 31.4.3.3.2-1—Minimum Bend Radii (in.) for 90-Degree Bends

Alloy	Plate Thickness, in.			
	0.1875	0.25	0.375	0.5
5083-H321	0.28	0.38	0.75	1.25
5086-H116	0.28	0.50	0.94	1.50
5456-H116	0.28	0.63	1.13	1.75
6061-T6	0.56	0.88	1.69	2.50

C31.4.3.3.2

Recommended bend radii for 90-degree cold bends for other alloys may be found in Table 7.6 of *Aluminum Standards and Data, 2003 (Metric SI).*

31.4.4—Fit of Stiffeners

End bearing stiffeners for girders and stiffeners intended as supports for concentrated loads shall bear fully on the flanges to which they transmit load or from which they receive load. Intermediate stiffeners not intended to support concentrated loads shall have a tight fit against the compression flange, unless specified otherwise.

C31.4.4

Full bearing may be obtained by milling, grinding, or in the case of compression regions of flanges, by welding.

31.4.5—Abutting Joints

Abutting ends of compression members of trusses and columns shall be milled or saw-cut to give a square joint and uniform bearing. At other joints, the distance between adjacent members shall not exceed 0.375 in.

31.4.6—Facing of Bearing Surfaces

The surface finish of bearing, base plates, and other bearing surfaces that come in contact with each other or with concrete shall meet the ANSI surface roughness requirements as defined in ANSI B46.1, Surface Roughness, Waviness, and Lay, Part 1:

Milled ends of compression members, milled or ground ends of stiffeners and fillers	ANSI 500 µin. (12.5 µm) (RMS)
Bridge rollers and rockers fillers	ANSI 250 µin. (6.3 µm) (RMS)
Pins and pin holes fillers	ANSI 125 µin. (3.2 µm) (RMS)
Sliding bearings fillers	ANSI 125 µin. (3.2 µm) (RMS)

31.4.7—Straightening Material

The straightening of plates, angles, other shapes, and built-up members, when permitted by the Engineer, shall be done by methods that will not produce fracture or other damage to the metal. Distorted members shall be straightened by mechanical means or by heat straightening. Heat straightening of non heat-treatable alloys and of heat-treatable alloys after heat treatment shall be done only under controlled procedures and with the approval of the Engineer. Heat straightening shall conform to ANSI/AWS D1.2/D1.2M *Structural Welding Code—Aluminum*.

C31.4.7

Aluminum may be heated for short periods of time to temperatures up to 400°F without significant loss of strength. Temperature and duration limits are given in ANSI/AWS D1.2/D1.2M Table 3.2. Heating aluminum alloys with magnesium contents greater than three percent, which includes 5083, 5086, and 5456, to temperatures between 150°F and 450°F will also result in decreased resistance to exfoliation corrosion.

31.4.8—Holes

31.4.8.1—General

Holes shall be:

- drilled to the nominal hole diameter, or
- subpunched to a diameter smaller than the nominal hole diameter and then reamed to the nominal hole diameter, or
- subdrilled to a diameter smaller than the nominal hole diameter and then reamed to the nominal hole diameter.

C31.4.8.1

Punching holes to the nominal diameter is not preferred for aluminum parts subject to fatigue.

The difference between a subpunched hole diameter and the nominal hole diameter shall be at least one-fourth the thickness of the part and in no case less than 0.03125 in.

31.4.8.2—Reamed or Drilled Holes

Reamed or drilled holes shall be cylindrical, perpendicular to the member. Burrs shall be removed. The diameter of holes produced by drilling or reaming shall not be more than 0.03125 in. greater than the nominal diameter of the hole.

31.4.8.3—Accuracy of Hole Groups

31.4.8.3.1—Accuracy before Reaming

After assembling, but before any reaming, the holes in any contiguous group shall allow a cylindrical pin that is:

- 0.125 in. smaller in diameter than the nominal diameter of the hole to enter at least 75 percent of the holes perpendicular to the face of the member without drifting, and
- 0.1875 in. smaller in diameter than the nominal diameter of the hole to enter every hole perpendicular to the face of the member without drifting.

31.4.8.3.2—Accuracy after Reaming or Drilling

After reaming or drilling, 85 percent of the holes in any contiguous group shall show no offset greater than 0.03125 in. between parts.

31.4.8.4—Locating Holes

Holes shall be:

- subpunched or subdrilled in unassembled parts and reamed when the parts are assembled, or
- drilled to the nominal diameter using a template or numerically controlled drilling, or
- drilled to the nominal diameter while the parts are assembled.

Parts may be assembled in the shop or in the field for the fabrication of holes.

31.4.9—Pins and Rollers

31.4.9.1—General

Pins and rollers shall be accurately formed to the dimensions shown on the contract documents, and shall be straight, smooth, and free from flaws.

31.4.9.2—Pin Holes

The diameter of pin holes shall not exceed the nominal diameter of the pin by more than 0.015625 in.

31.4.10—Annealing and Stress Relieving

Holes shall be fabricated after all heat treatment has been completed. Aluminum structural members shall not receive heat treatment after welding.

31.4.11—Aluminum Bridge Decks

31.4.11.1—General

Dimensional tolerances specified below for aluminum bridge deck panels shall be applied to each completed, but unloaded panel. The deviation from detailed flatness, straightness, or curvature at any point shall be the perpendicular distance from that point to a template edge which has the detailed straightness or curvature and which is in contact with the panel at two other points. The template edge may have any length not exceeding the lesser of the greatest dimension of the panel and 1.5 times the least dimension of the panel; it may be placed anywhere on the panel. The distance between adjacent points of contact of the template edge with the panel shall be used in the formulas to establish the tolerances for the panel whenever this distance is less than the applicable dimension of the panel specified for the formula.

31.4.11.2—Flatness of Panels

The deviation, δ, from detailed flatness or curvature of a panel shall not exceed:

$$\delta \leq \frac{D}{144\sqrt{T}} \leq 0.1875 \text{ in.} \qquad (31.4.11.2\text{-}1)$$

where:

D = the least dimension along the boundary of the panel, in.

T = the minimum thickness of the top flange of the panel, in.

31.4.11.3—Straightness of Longitudinal Stiffeners Subject to Calculated Compressive Stress

The deviation, δ, from detailed straightness or curvature in any direction perpendicular to the length of a longitudinal stiffener subject to calculated compressive stress shall not exceed:

$$\delta \leq \frac{L}{480} \qquad (31.4.11.3\text{-}1)$$

where:

L = the length of the stiffener over which the deviation in detailed straightness or curvature is measured, in.

31.4.11.4—Straightness of Transverse Web Stiffeners and Stiffeners Not Subject to Calculated Compressive Stress

The deviation, δ, from detailed straightness or curvature in any direction perpendicular to the length of a transverse stiffener or a stiffener not subject to calculated compressive stress shall not exceed:

$$\delta \leq \frac{L}{240} \qquad (31.4.11.4\text{-}1)$$

where:

L = the length of the stiffener over which the deviation in detailed straightness or curvature is measured, in.

31.4.12—Full-Size Tests

When full-size tests of fabricated structural members are required in the contract documents, the Contractor shall provide suitable facilities, material, supervision, and labor necessary for making and recording the required tests. The members tested shall be paid for in accordance with Article 31.7.2, “Basis of Payment.”

31.4.13—Marking and Shipping

Each member shall be painted or marked with an erection mark for identification and an erection diagram showing these marks shall be furnished to the Engineer. Metal stamping shall not be used to mark aluminum parts.

The Contractor shall furnish to the Engineer as many copies of material orders, shipping statements, and erection diagrams as the Engineer may direct. The weight (mass) of the individual members shall be shown on the statements. Members having a weight (mass) of more than 6.5 kips shall have the weight (mass) marked on them. Structural members shall be loaded on trucks or cars in such a manner that they may be transported and unloaded at their destination without being damaged.

Bolts, nuts, and washers from each rotational-capacity lot shall be shipped in the same container. If there is only one production lot number for each size of nut and washer, the nuts and washers may be shipped in separate containers. The gross weight (mass) of any container shall not exceed 0.3 kips. A list showing the quantity and description of materials shall be plainly marked on the outside of each container.

31.4.14—Protection from Corrosion

31.4.14.1—General

Corrosion protection measures should not be required for aluminum, except as specified herein.

Measures to provide protection from corrosion for the alloys listed in Article 31.3.1, "Structural Aluminum," shall be specified where:

- aluminum is in contact with wood or other porous material that may absorb water, the aluminum shall be painted;
- aluminum is in contact with concrete or masonry where moisture is present, the aluminum shall be painted;
- aluminum is in contact with steel and moisture is present, the steel shall be painted.

Galvanized steel, plated steel, and 300 series stainless steel in contact with aluminum need not be painted.

31.4.14.2—Painting

Immediately prior to painting, aluminum surfaces to be painted shall be cleaned by a method that will remove all dirt, oil, grease, chips, and other foreign substances. Metal more than 0.125 in. thick may be abrasion blasted.

Aluminum to be painted shall be given one coat of suitable quality paint such as zinc chromate primer or a heavy coat of alkali-resistant bituminous paint.

31.5—ASSEMBLY

31.5.1—Bolted and Riveted Connections

Parts shall be assembled, well pinned, and firmly drawn together before drilling, reaming, riveting, or bolting. All joint surfaces, including surfaces to be under bolt heads, rivet heads, or nuts, shall be free of dirt or other foreign material. Assemblies shall be taken apart, if necessary, for the removal of burrs and shavings.

Drifting done during assembling shall be limited to that which brings the parts into position and shall not enlarge holes or distort the metal.

31.5.1.1—Bolts

Bolts shall have single self-locking nuts or double nuts, unless otherwise specified in the contract documents. Beveled washers shall be used where faying surfaces have a slope of more than 1:20 with respect to a plane normal to the bolt axis. A washer shall be provided under the nut.

Bolts shall be installed snug tight.

31.5.1.2—Lock-Pin and Collar Fasteners

The installation of lock-pin and collar fasteners shall be by methods and procedures approved by the Engineer.

31.5.1.3—Rivets

Rivets shall be power driven and conform to MIL-R-1150F. Rivets shall fill the holes completely. Defective rivets shall be removed by drilling.

Rivet heads shall be concentric with the rivet holes and in full contact with the surface of the parts. The driven head of rivets shall be of the flat or the cone-point types. Flat heads shall have a diameter not less than 1.4 times the nominal rivet diameter and a height not less than 0.4 times the nominal rivet diameter. Cone-point heads shall have a diameter not less than 1.4 times the nominal rivet diameter and a height to apex of the cone not less than 0.65 times the nominal rivet diameter. The included angle at the apex of the cone shall be approximately 127 degrees.

31.5.2—Welded Connections

Surfaces and edges to be welded shall be smooth, uniform, clean, and free of defects which would adversely affect the quality of the weld.

31.5.3—Preassembly of Field Connections

31.5.3.1—General

Field connections of main members of trusses, arches, continuous beams, plate girders, bents, towers, and rigid frames shall be preassembled prior to erection as necessary to verify the geometry of the completed structure or unit and to verify or prepare field splices. The Contractor shall be responsible for attaining accurate geometry, and the Contractor shall propose an appropriate method of preassembly for approval by the Engineer. The method and details of preassembly shall be consistent with the erection procedure shown on the erection plans and camber diagrams prepared by the Contractor and approved by the Engineer. As a minimum, the preassembly procedure shall consist of assembling three contiguous panels accurately adjusted for line and camber. Successive assemblies shall consist of at least one section or panel of the previous assembly, repositioned if necessary and adequately pinned to assure accurate alignment, plus two or more sections or panels added at the advancing end. In the case of structures longer than 150 ft, each assembly shall be not less than 150 ft long, regardless of the length of individual continuous panels or sections. At the option of the Fabricator, sequence of assembly may start from any location in the structure and proceed in one or both directions so long as the preceding requirements are satisfied.

31.5.3.2—Field Bolted Connections

Major compression members with milled ends shall be assembled in full bearing and then shall have their subsized holes reamed to the specified size while the members are assembled.

31.5.3.3—Check Assemblies for Numerically-Controlled Fabrication

Unless otherwise stated in the contract documents, when the Contractor elects to use numerically controlled hole fabrication, a check assembly shall be provided for each major structural type of each project. Except as noted herein, the check assembly shall consist of at least three contiguous shop sections. In a truss, the check assembly shall consist of all members in at least three contiguous panels, but not less than the number of panels associated with three contiguous chord lengths, i.e., length between field splices.

Check assemblies shall be assembled in accordance with the sequence shown on the erection drawings. If the check assembly fails to demonstrate that the required accuracy is being obtained, further check assemblies may be required by the Engineer at no additional cost to the Owner.

Each check assembly and its camber, alignment, accuracy of holes, and fit of milled joints shall be approved by the Engineer before reaming is commenced or before the check assembly is dismantled.

31.5.3.4—Field-Welded Connections

For field-welded connections, the fit of members, including the proper space between abutting members, shall be prepared or verified with the segment preassembled in accordance with Article 31.5.3.1.

31.5.4—Match-Marking

Connecting parts preassembled in the shop to assure proper fit in the field shall be match-marked, and a diagram showing such marks shall be furnished to the Engineer.

31.5.5—Welding

Brackets, clips, shipping devices, or other material not required by the contract documents shall not be welded or tacked to any member unless specified in the contract documents and approved by the Engineer.

31.6—ERECTION

31.6.1—General

The Contractor shall provide all tools, machinery, and equipment necessary to erect the structure.

31.6.2—Handling and Storing Materials

Material to be stored at the job site shall be placed on skids above the ground and kept clean and well drained. Girders and beams shall be placed upright and shored. If the Contractor's scope of work is for erection only, the Contractor shall check the material received against the shipping lists and report promptly in writing any shortage or damage. After material is received by the Contractor, the Contractor shall be responsible for any damage to or loss of material.

C31.6.2

Where moisture is trapped between adjacent surfaces of closely packed aluminum, white or gray stains, referred to as water stains, may result. Alloys having a high magnesium content are affected to a greater degree, but all aluminum alloys can be affected. Water staining is a superficial condition and does not affect the strength of the material, nor will it progress once the conditions that caused it are removed. It can be avoided by keeping the material dry.

31.6.3—Bearings and Anchorages

Bridge bearings shall be furnished and installed in conformance with Section 18, "Bearing Devices."

If the aluminum superstructure is to be placed on a substructure that was built under a separate contract, the Contractor shall verify that the substructure has been constructed in the right location and to the correct lines and elevations before ordering materials.

31.6.4—Erection Procedure

31.6.4.1—Conformance to Erection Drawings

The erection procedure shall conform to the erection drawings submitted in accordance with Article 31.2.2, "Erection Drawings." Any modifications to or deviations from this erection procedure shall require revised drawings and verification of stresses and geometry.

31.6.4.2—Erection Stresses

Any erection stresses induced in the structure as a result of erection which differs from the contract documents shall be accounted for by the Contractor. Erection design calculations for such changed methods shall be prepared at the Contractor's expense and submitted to the Engineer. The calculations shall indicate any change in stresses or change in behavior for the temporary and final structures. Additional material required to keep both the temporary and final force effects within the limits used in design shall be provided at the Contractor's expense.

The Contractor shall be responsible for providing temporary bracing or stiffening devices to limit stresses in individual members or segments of the structure during erection.

31.6.4.3—Maintaining Alignment and Camber

During erection, the Contractor shall be responsible for supporting segments of the structure in a manner that will produce the proper alignment and camber in the completed structure. Bracing shall be provided and installed by the Contractor as necessary during erection to provide stability and assure correct geometry.

31.6.5—Field Assembly

The parts shall be accurately assembled as specified in the contract documents or erection drawings, and any match-marks shall be followed. The material shall be carefully handled so that no parts will be bent, broken, or otherwise damaged. Hammering that will injure or distort the members is prohibited. Bearing surfaces and surfaces to be in permanent contact shall be cleaned before the members are assembled. Splices and field connections shall have one-quarter of the holes filled with bolts and one-quarter filled with cylindrical erection pins before installing and tensioning bolts in the unfilled holes. Splices and connections carrying traffic during erection shall have three-eighths of the holes filled with bolts and three-eighths of the holes filled with cylindrical erection pins before installing and tensioning bolts in the unfilled holes.

Bolts used as fit-up bolts may be reused for the final installation. If other fit-up bolts are used, they shall be of the same nominal diameter as the final bolts, and cylindrical erection pins shall be 0.03 in. larger.

31.6.6—Pin Connections

Pilot and driving nuts shall be used in driving pins. They shall be furnished by the Contractor at the Contractor's expense. Pins shall be so driven that the members will take full bearing on them. Pin nuts shall be screwed up tight and the threads burred at the face of the nut with a pointed tool.

31.6.7—Misfits

The correction of minor misfits involving minor amounts of reaming, cutting, grinding, and chipping shall be included in the Contractor's scope of work and shall be at the Contractor's expense. However, any error in the shop fabrication or deformation resulting from handling and transporting may be cause for rejection.

The Contractor shall be responsible for all misfits, errors, and damage, and shall make the necessary corrections and replacements.

31.7—MEASUREMENT AND PAYMENT

31.7.1—Method of Measurement

Pay quantities for each aluminum alloy will be measured by the pound, computed from dimensions shown in the contract documents using the following rules and densities in Table 31.7.1-1.

C31.7.1

Table 31.7.1-1—Unit Weights (Mass Densities) of Aluminum Alloys

Alloy	Unit Weight, kip/ft^3
5083	0.166
5086	0.166
5456	0.166
6061	0.169
6063	0.168

The weight (mass) of extruded shapes shall be computed on the basis of their nominal weight per foot, as specified in the contract documents or listed in handbooks.

The weight (mass) of plates shall be computed on the basis of the nominal weight (mass) for their width, length, and thickness as specified in the contract documents.

The weight (mass) of temporary erection bolts, shop and field paint, boxes, crates, and other containers used for shipping, and materials used for supporting members during transportation and erection, shall not be included.

Weight (mass) computations shall take into account deductions for copes, cuts, clips, and all open holes, but not holes for fasteners.

31.7.2—Basis of Payment

The contract price for fabrication and erection of aluminum shall be considered to be full compensation for the cost of all labor, equipment, materials, transportation, and shop and field painting, if not otherwise provided for, necessary for the proper completion of the work in accordance with the contract documents. The contract price for fabrication without erection shall be considered to be full compensation for the cost of all labor, equipment, and materials necessary for the proper completion of the work, other than erection and field assembly, in accordance with the contract documents.

Payment may be made on a pound-unit-price or a lump-sum basis as required by the terms of the contract documents. If the method of payment is not specified in the contract documents, payment shall be taken to be on a pound-unit-price basis.

When tests of full-sized members are required by the contract documents, payment for tested members shall be made at the same rate as for comparable members for the structure. The cost of testing, including equipment, labor, and incidentals, shall be included in the contract price for structural aluminum. The cost of members which fail to meet the contract document requirements, and members rejected as a result of tests, shall be borne by the Contractor.

Densities for other aluminum alloys are listed in Table 2.4 of *Aluminum Standards and Data 2003 (Metric SI)*.

31.8—REFERENCES

AASHTO. 1991. *Guide Specifications for Aluminum Highway Bridges*, GSAHB-1, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO and AWS. 2008. AASHTO/AWS D1.5M/D1.5:2008 *Bridge Welding Code*, Fifth Edition, BWC-5, American Welding Society, Miami, FL.

AISC Quality Certification Program, American Institute of Steel Construction, Chicago, IL, Category I: Structural Steel and Category III: Fracture-Critical. See http://www.aisc.org.

Aluminum Association. 2003. *Aluminum Standards and Data 2003 (Metric SI)*, Aluminum Association, Washington, DC.

ASME. 1979. *Metric Heavy Hex Nuts*, B18.2.4.6M, American Society of Mechanical Engineers, Fairfield, NJ. Reaffirmed 1998.

ASME. 1979. *Metric Heavy Hex Structural Bolts*, B18.2.3.7M, American Society of Mechanical Engineers, Fairfield, NJ. Reaffirmed 1995.

ASME. 1987. *Square and Hex Nuts*, B18.2.2, American Society of Mechanical Engineers, Fairfield, NJ. Inch series. Reaffirmed 1999.

ASME. 1996. *Square and Hex Bolts and Screws, Inch Series*, B18.2.1, American Society of Mechanical Engineers, Fairfield, NJ.

ASME. 2002. *Surface Texture, Surface Roughness, Waviness and Lay*, B46.1, American Society of Mechanical Engineers, Fairfield, NJ.

AWS. 2003. ANSI/AWS D1.2/D1.2M *Structural Welding Code—Aluminum*, American Welding Society, Miami, FL.

DOD. U.S. Military Specification MIL-P-23469D for aluminum, stainless steel, and steel lock-pin and collar fasteners, U.S. Department of Defense, Washington, DC. See https://assist.daps.dla.mil/quicksearch/.

DOD. U.S. Military Specification MIL-R-1150F for rivets, U.S. Department of Defense, Washington, DC. See https://assist.daps.dla.mil/quicksearch/.

Back to 2010 Edition

SECTION 31: ALUMINUM STRUCTURES

31.4.3.3.2—Cold Bending

Revise Table 31.4.3.3.2-1 as shown below:

Table 31.4.3.3.2-1—Minimum Bend Radii (in.) for 90-Degree Bends

Alloy	Plate Thickness, in.			
	0.1875	0.25	0.375	0.5
5083-H321	0.28	0.38	0.75	1.25
5086-H116	0.28	0.50	0.94	1.50
5456-H116	0.38 ~~0.28~~	0.63	1.13	1.75
6061-T6	0.56	0.88	1.69	2.50

2010 Revision

Back to 2010 Edition

SECTION 31: ALUMINUM STRUCTURES

31.3.2.1.1—Material

Delete AASHTO M 164, M 253, and M 291 citations in the Article and Commentary as follows:

High-strength bolts for structural aluminum joints shall conform to ~~AASHTO M 164 (~~ASTM A325~~)~~, Type 1.

The supplier shall provide a lot number appearing on the shipping package and a certification noting when and where all testing was done, including rotational capacity tests and galvanizing thickness.

The maximum hardness for ~~AASHTO M 164 (~~ASTM A325~~)~~ bolts shall be 33 HRC.

Proof load tests complying with ASTM F606, Method 1, shall be required for the bolts. Wedge tests of full-size bolts are required in accordance with Section 8.3 of ~~AASHTO M 164 (~~ASTM A325~~)~~. Galvanized bolts shall be wedge tested after galvanizing. Proof load tests of ~~AASHTO M 291 (~~ASTM A563~~)~~ shall be required for the nuts. The proof load tests for nuts shall be performed after galvanizing, overtapping, and lubricating.

~~AASHTO M 253 (~~ASTM A490~~)~~, high-strength steel bolts are not used in aluminum structures because they may become embrittled by galvanizing. Galvanizing is typically required to prevent galvanic corrosion of aluminum in contact with steel.

Type 2 bolts have been withdrawn from ~~AASHTO M 164 (~~ASTM A325~~)~~ and ~~AASHTO M 253 (~~ASTM A490~~)~~ and, therefore, are no longer manufactured. However, Type 2 bolts manufactured before this discontinuation may still be in inventory and are considered acceptable.

31.3.2.1.2—Identifying Marks

Back to 2010 Edition

Delete AASHTO M 164 citations as follows:

Bolts shall be identified by marks on their heads in accordance with ~~AASHTO M 164 (~~ASTM A325~~)~~. Nuts shall be identified by marks on one face in accordance with the specifications referenced in ~~AASHTO M 164 (~~ASTM A325~~)~~. Bolt head markings shall identify the grade by the symbol "A325" and the Manufacturer, if Type 3. Nut markings must identify the property class and the Manufacturer. Markings on washers shall identify the Manufacturer.

Page left intentionally blank.

Back to 2010 Edition

C31.3.2.1.4

Delete AASHTO M 164 citation as follows:

~~AASHTO M 164 (~~ASTM A325~~)~~ Type 2 bolts shall be mechanically galvanized only.

31.3.2.1.5—Alternative Fasteners

Delete AASHTO M 164 citations in bullets 2 and 3 as follows:

Subject to the approval of the Engineer, other fastener or fastener assemblies may be used provided they:

- meet the materials, manufacturing, and chemical composition requirements of ~~AASHTO M 164 (~~ASTM A325~~)~~,
- meet the mechanical property requirements of ~~AASHTO M 164 (~~ASTM A325~~)~~ in full-size tests, and
- have body diameter and bearing areas under the head and nut, or their equivalent, not less than those provided by a bolt and nut of the same nominal dimensions prescribed in Article 31.3.2.1.3, "Dimensions."

Such alternate fasteners may differ in other dimensions from those of the specified bolts and nuts.

2014 Revision

SECTION 32: SHOCK TRANSMISSION UNITS

TABLE OF CONTENTS

SECTION 32

SHOCK TRANSMISSION UNITS

32.1—GENERAL

This work shall consist of furnishing and installing shock transmission units (STUs) and hardware for attaching to bridge structures.

STUs and hardware shall be constructed in accordance with the details shown in the contract documents and these specifications. When complete details are not provided, STUs and hardware shall be furnished that conform to the limited details shown in the contract documents and shall provide the design capacities for minimum and maximum loads and movements, and the performance characteristics specified.

C32.1

A Shock Transmission Unit (STU) is a fabricated component that is designed to be connected between bridge members which moves freely when loads are slowly applied such as would be caused by temperature changes, but acts as a rigid link capable of transmitting a force under rapidly applied loads caused by vehicle braking or impact and earthquakes.

32.2—WORKING DRAWINGS

Whenever complete details for STUs and hardware are not shown in the contract documents, the Contractor shall prepare and submit working drawings for the STUs and hardware. Such drawings shall show the external details and dimensions of the STUs and hardware proposed for use and shall be approved by the Engineer prior to fabrication. Such approval shall not relieve the Contractor of any responsibility under the contract documents for successful completion of the work.

The following shall be specified on the working drawings:

- the total number of STUs required, grouped according to rated capacity and rated travel limits,
- the weight (mass) of each STU,
- the plan view and section elevation view showing all relative dimensions, including dimensions for the assumed installation temperature, of each STU and attachment hardware,
- the minimum and maximum design temperatures of the STUs,
- the maximum drag force at the specified rate of movement caused by the specified temperature changes for each STU,
- the total movement capacity of each STU,
- the maximum rated load capacity for each STU,
- the maximum movement anticipated for each cycle during dynamic loadings,
- the type of materials to be used for all STUs and attachment hardware,
- painting or coating materials to be applied,

2010 Edition

- alignment plans for the STUs showing tolerances for alignment in which the STUs must be installed,
- installation schemes,
- design calculations for the attachment hardware verifying conformance with the loading requirements, if required by the contract documents,
- anchorage details of the attachment hardware,
- the place of manufacture of the STUs and the fabricator of the hardware, and
- the Manufacturer's name and the name of the representative who will be responsible for coordinating production, inspection, sampling, and testing.

32.3—MATERIALS

32.3.1—Steel

2014 Revision

Unless noted otherwise in the contract documents, attachment hardware shall meet the requirements of AASHTO M 270M/M 270 (ASTM A709/A709M), Grade 50 (Grade 345) steel. All other metal components of the STU, except for the piston rod, shall meet the requirements of SAE 1026 (*SAE Handbook*, 2004), or equivalent.

The piston rod shall meet the requirements of ASTM A240/A240M, Type 304L, stainless steel, or equivalent.

Bolts shall meet the requirements of AASHTO M 164 (ASTM A325), Type 1, unless noted otherwise in the contract documents.

C32.3.1

The corrosion protection system used on STUs should be the same as used on the bridge. Uncoated weathering steel is preferred to reduce maintenance requirements. However, STUs can be furnished with a standard galvanized finish on all exposed parts. The boot protecting the piston rod is made of a durable reinforced neoprene material. The stainless steel piston rod is protected by grease packed into the neoprene boot. It is the responsibility of the designer to determine if this protection "package" is satisfactory for the exposure conditions expected over the service life of the STU. If additional protection is required, it shall be so noted in the project special provisions. If specific durability testing is required as a condition of acceptance, the requirements shall be so specified by the designer as a part of the order. Particular attention must be directed to whether corrosion protection beyond that provided for other bridge components is necessary for the anchorage system(s).

Environmental conditions to be considered are:

- coastal (marine salts),
- industrial,
- potential for periodic immersion due to flooding,
- deicing salts and mud (under bridge joints),
- inadvertent sandblasting during repainting,
- pigeon nests, and
- ultraviolet light and ozone exposure of Neoprene.

32.3.2—Internal Fluid

The Manufacturer shall determine the fluid used inside the STU to provide the shock resistance capacity of the STU. Where this material is proprietary in nature, the contract documents shall specify that the Manufacturer demonstrate through the testing program that the design requirements of the STU can be met. The operating fluid used in the STU shall be OSHA approved, nontoxic, nonflammable, silicone-based fluid or putty. The Manufacturer shall provide a certification that the material in each STU is the same material as was used in the testing program.

32.3.3—Packaging, Handling, and Storage

Prior to shipment from the point of manufacture, STUs shall be packaged in such a manner to ensure that during shipment and storage, the STUs will be protected against damage from handling, weather, or any normal hazard. Each package shall be marked to clearly note the STU identification number, the rated capacity, the Manufacturer's name, and the project identification.

All STUs shall be stored at the work site in an area that provides protection from environmental and physical damage. When installed, STUs and hardware shall be clean and free of all foreign substances.

Dismantling of STUs at the site shall not be done unless absolutely necessary for inspection or installation. STUs shall not be opened or dismantled at the site, except under the direct supervision of, or with the approval of, the Manufacturer.

32.3.4—Manufacture/Fabrication

32.3.4.1—General

The Manufacturer shall certify that each STU satisfies the requirements of the contract documents and these specifications. Each STU shall have stamped on the body the STU identification number, the rated capacity, the Manufacturer's name, and the project identification. STUs, including the attachment hardware, shall be preassembled in the shop by the Manufacturer and checked for proper completeness, tolerances, and geometry before shipping to the site.

32.3.4.2—Connecting Hardware

Hardware used to connect the STU to the substructure and/or superstructure shall be fabricated in accordance with Section 11, "Steel Structures."

32.3.4.3—Connection Tolerances

Differences between pin and hole diameters shall be 0.010 in. maximum.

32.4—TESTING AND ACCEPTANCE

All shock transmission units shall be tested to verify their performance and design properties under slow movement, fast movement, and cyclic loading. In general, there are three types of tests to be performed on an STU:

- prequalification test (system characterization tests), described in Article 32.4.1,
- prototype tests, described in Article 32.4.2, and
- proof tests (quality control tests), described in Article 32.4.3

C32.4

The testing requirements and service life of the device are to be chosen by the designer and can be 25 to 75 yr, subject to the appropriate maintenance being performed by the Owner. If a particular application requires a significantly longer or shorter service life, appropriate modifications to the test requirements for a fatigue life should be considered.

32.4.1—Prequalification Tests

The performance and fundamental properties of the STU shall be verified by testing before it can be adapted for use. These tests include component tests of individual units, assembled units, and scaled structure complete with STUs. At a minimum, these tests shall be conducted according to the testing guidelines developed by the Highway Innovative Technology Center (HITEC) evaluation panel.

C32.4.1

These prequalification tests usually are not project specific. They are conducted to establish the properties of the unit when a new STU is being developed or a substantially different version of an existing unit is being evaluated.

32.4.2—Prototype Tests

32.4.2.1—General

The resistance and the stroke under slow movement and the lock-up force under fast movement of the STU used in the design and in the analysis shall be verified by prototype tests. All testing shall be performed at an independent testing laboratory approved by the Engineer. All testing shall be performed in the presence of the Engineer unless otherwise approved in writing by the Engineer. These tests can be conducted at ambient temperature.

The prototype tests identified herein shall be performed on at least one STU of each type.

C32.4.2.1

The objectives of these tests are to evaluate the performance of STUs for two design conditions consisting of slow movements that will not lock up the device, and fast movements that will lock up the device within temperature and loading conditions at least equal to those at the project site.

32.4.2.2—Hydrostatic Pressure Test

The STU shall be tested for at least 3 min at 150 percent of the maximum computed internal pressure to verify the structural integrity of the high pressure boundary. The STU shall be pressurized and sealed for the duration of the test. Initial and final pressure readings shall be recorded.

ACCEPTANCE: No signs of leakage under pressure. Hydrostatic pressure shall not drop more than five percent during the test.

32.4.2.3—Slow Movement Test (Thermal)

The STU shall be cycled for three complete and continuous cycles at a low velocity and a maximum stroke specified by the Engineer to verify the operation and travel of the unit. A continuous plot of the load and deflection shall be recorded.

C32.4.2.3

Nominal rated force of the STU is defined as P_R.

ACCEPTANCE: No sign of leakage under operation. No sign of structural binding under operation. The force required to cycle the unit shall not exceed ten percent of the nominal rated force or as specified by the Engineer. The STU shall not lock up during this test.

32.4.2.4—Fast Movement Test

The STU shall have the full nominal rated force applied to it at a fast travel rate specified by the Engineer. The STU shall be tested in both compression and tension, but testing need not to be cyclic. A continuous plot of load versus deflection shall be recorded.

ACCEPTANCE: The STU shall lock up within 0.5 in. or as specified by the Engineer of the point of zero movement. The lock-up deflection shall be taken as the deflection at which a constant stiffness is achieved. The deflection from the point of lock-up to the maximum test load shall not exceed 0.5 in. or as specified by the Engineer. The stiffness of each STU throughout the force range from lock-up to maximum test load shall not vary by more than ten percent. Unit shall show no sign of leakage or binding.

32.4.2.5—Simulated Dynamic Test

The STU shall be tested to determine the ability of the unit to lock up during dynamic loads. Each unit will have tension force applied in less than 0.5 s or as specified by the Engineer; this force will be sustained for the period of 5 s. At the end of the 5 s of tension load, the unit shall be put into compression within 1 s or as specified by the Engineer. The compression force will be held for 5 s. The tension and compression force shall be equal and will be at least three times the lock-up force determined in Article 32.4.3.4, Fast Movement Test, but no more than the nominal rated force. A continuous plot of force versus deflection shall be recorded.

ACCEPTANCE: Deflection between the point of zero load and the point of maximum load shall not exceed 0.5 in. or as specified by the Engineer, in either the initial loading stage or in the force reversal stage. Deflection during the sustained load portion of the test shall not exceed 0.5 in. or as specified by the Engineer.

32.4.2.6—Overload Test

The STU shall be loaded by 1.5 times the nominal rated force at a rate fast enough to make the unit lock up, then hold the load for 30 s or as specified by the Engineer.

ACCEPTANCE: Unit shall show no sign of leakage or binding.

C32.4.2.6

The purpose of this test is to ensure the STU will perform properly should the nominal rated force be exceeded while in service.

32.4.2.7—Fatigue Load Test

The STU shall be cycled for 100,000 complete and continuous cycles at a velocity (frequency) specified by the Engineer and a load equal to the nominal rated force (P_R) of the STU.

C32.4.2.7

The purpose of this test is to determine if the STU can withstand as many cycles of load as could occur from braking on a highway bridge with high vehicle braking actions.

ACCEPTANCE: Unit shall show no sign of leakage or binding.

The worst-case scenario for service loading of STUs is an application of braking loads equal to the lock-up load four times a day, for the AASHTO LRFD specified design life of 75 yr. This is roughly equivalent to 100,000 load cycles.

(4 cycles/day × 365 days/yr × 75-yr service life = 109,500; use 100,000).

See *AASHTO LRFD Bridge Design Specifications*, Article 3.6.4, for *BR* force and application.

32.4.3—Proof Testing (Quality Control)

32.4.3.1—General

These tests shall be conducted on every STU that is going to be incorporated into the structure. All testing shall be performed at an independent testing laboratory approved by the Engineer. All testing shall be performed in the presence of the Engineer unless otherwise approved in writing by the Engineer. These tests can be conducted at ambient temperature.

The proof tests shall consist of those described in Articles 32.3.2 through 32.4.3.4.

32.4.3.2—Hydrostatic Pressure Test

The STU shall be tested for at least 3 min at 150 percent of the maximum computed internal pressure to verify the structural integrity of the high pressure boundary. The STU shall be pressurized and sealed for the duration of the test. Initial and final pressure readings shall be recorded.

ACCEPTANCE: No signs of leakage under pressure. Hydrostatic pressure shall not drop more than five percent during the test.

32.4.3.3—Slow Movement Test (Thermal)

The STU shall be cycled for three complete and continuous cycles at a low velocity and a maximum stroke specified by the Engineer to verify the operation and travel of the unit. A continuous plot of the force versus deflection and force versus time shall be recorded.

ACCEPTANCE: No sign of leakage under operation. No sign of structural binding under operation. The force required to cycle the unit shall not exceed ten percent of the nominal rated force or as specified by the Engineer. The STU shall not lock up during this test.

32.4.3.4—Fast Movement Test

The STU shall have the full nominal rated force applied to it at a fast travel rate specified by the Engineer. The STU shall be tested in both compression and tension, but testing need not be cyclic. A continuous plot of load versus deflection shall be recorded.

ACCEPTANCE: The STU shall lock up within 0.5 in. or as specified by the Engineer of the point of zero movement. The lock-up deflection shall be taken as the deflection at which a constant stiffness is achieved. The deflection from the point of lock-up to the maximum test load shall not exceed 0.5 in. or as specified by the Engineer. The stiffness of each STU throughout the force range from lock-up to maximum test load shall not vary by more than ten percent. Unit shall show no sign of leakage or binding.

32.5—MANUALS

32.5.1—Installation Manuals

The Manufacturer shall provide an installation manual which includes specific instructions to ensure proper installation procedures for the STU. The following items shall be included:

- name of the Manufacturer's representative who will provide assistance and advice during installation of the STUs,
- alignment dimensions, installation temperature, and the "ideal" installation length of the device,
- details of any required installation equipment and complete procedures, including installation tolerances, as well as instructions on its use,
- shop drawings of the STUs and the connections to the bridge components, including tolerances for critical components and connection details,
- STU job site storage requirements while awaiting installation, and
- STU electrical isolation procedures where there is a possibility of galvanic or electrolytic corrosion occurring.

32.5.2—Maintenance and Inspection Manual

The Manufacturer shall provide a manual that will include specific instructions to ensure proper maintenance and inspection procedures for the STUs while in service. The following items should be included:

- Information as to what to inspect, and what to look for (i.e., pull back piston rod covering, if used, and look for evidence of leakage). Include any precautions necessary to avoid damage to the boot and device.
- Information on type of grease to apply to the exposed piston rod and frequency of application.

- Instructions on how to check the inspection holes, greasing points, etc.
- Instructions for removal and replacement of the STU, including schematics of any special equipment needed.
- When required by the contract documents, details of any necessary tools and equipment shall be provided to allow the STU to be proof tested to verify movement capability while on the structure.
- Instructions for each inspection (generally assumed to be at a frequency of 2 yr) and after each significant seismic event shall include at least the following:
    - Perform a visual inspection of the anchorage system to ensure it is not damaged.
    - Determine need for maintenance forces to clean the STU and its anchorages to prevent accelerated corrosion.
    - Determine need for repainting/recoating of the device.
    - All STUs on the bridge shall be closely inspected for signs of leakage. If leakage is noted when inspecting the boot, notify maintenance forces to remove boot and inspect piston rod for damage.
    - Inspect for unexpected changes in alignment or condition that would indicate the STU experienced unexpected force application; etc.
    - Every 6 yr, or as determined by the Owner, notify maintenance forces to unclasp the protective boot and examine the condition of the piston rod, as well as determine if there is evidence of leakage of the internal fluid.
    - Every 10 yr, notify maintenance forces to load STU(s) as selected by the Owner, and in a manner to ensure the STU can develop the rated load.

32.6—MEASUREMENT AND PAYMENT

STUs shall be measured and paid for by the number of units installed and accepted as shown in the contract documents or ordered by the Engineer.

C32.6

Some agencies prefer to pay for testing separately, especially if waiver of testing is a consideration.

The contract unit price paid for STUs shall include full compensation for furnishing all labor, materials, tools, equipment, and incidentals; and for doing all the work involved in installing STUs (including testing), complete in place, as specified in these Specifications, the contract documents, and as directed by the Engineer.

32.7—REFERENCES

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2009. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 29th Edition, HM-29, American Association of State Highway and Transportation Officials, Washington, DC. Includes AASHTO M, R, and T standards, which are also available individually in downloadable form.

Highway Innovative Technology Evaluation Center (HITEC), a service center of the Civil Engineering Research Foundation (CERF).

SAE. 2004. "Chemical Composition of SAE Carbon Steels," SAE J403, *SAE Handbook*, Society of Automotive Engineers, Warrendale, PA.

Back to 2010 Edition

SECTION 32: SHOCK TRANSMISSION UNITS

32.3.1—Steel

Delete AASHTO M 164 citation in paragraph 3 as follows:

The piston rod shall meet the requirements of ASTM A240/A240M, Type 304L, stainless steel, or equivalent. Bolts shall meet the requirements of ~~AASHTO M 164 (~~ASTM A325~~)~~, Type 1, unless noted otherwise in the contract documents.

SECTION 33: MICROPILES

TABLE OF CONTENTS

Page left intentionally blank.

SECTION 33: MICROPILES

[*Insert this new section in its entirety.*]

33.1—DESCRIPTION

This work shall consist of furnishing and constructing micropile foundations of the type and dimensions designated in the contract documents and these specifications. This specification also covers providing test piles and performing load tests.

C33.1

The term “micropiles” as used in this section means a small (generally 4 to 12 in.) diameter bored, cast-in-place composite pile, in which the applied loads are resisted by steel reinforcement, cement grout, and frictional grout-to-ground bond.

Type A micropiles are constructed by placing a neat cement grout or sand-cement grout in the pile under a gravity head only.

Type B micropiles are constructed by injecting a neat cement or sand-cement grout under pressure (typically 0.07 to 0.15 ksi) into the drilled hole while the temporary drill pipe/casing or auger is withdrawn.

Type C micropiles are grouted as for Type A, followed 15 to 25 min after primary grouting by injection of additional grout under pressure (typically greater than 0.15 ksi) via a preplaced sleeved grout tube.

Type D micropiles are grouted similar to Type C, but the primary grout is allowed to harden before injecting the secondary grout under pressure (typically 0.3 to 1.2 ksi) with a packer to achieve treatment of specific pile intervals or material horizons. Several phases of injection are possible at selected horizons. There is usually an interval of at least 24 hr between successive phases.

Type E micropiles are constructed by drilling with grout injection through a continuous-thread, hollow-core steel bar. The grout injection serves to flush cuttings, achieve grout penetration into the ground, and stabilize the drill hole. Often the initial grout has a higher water-to-cement ratio and is then replaced with thicker structural grout near the completion of drilling.

Primary grout is a portland cement-based grout injected into the micropile hole before or after reinforcement installation. Primary grout provides direct load transfer along the micropile to the surrounding ground.

Post grouting is the injection of additional grout into the load transfer length of the micropile after hardening of primary grout, also known as regrouting or secondary grouting.

33.2—SUBMITTALS

At least four weeks prior to the start of micropile installation, the Contractor shall submit four copies of a project reference list to the Engineer for approval, verifying the successful completion by the Contractor of at least three separate foundation projects within the last

C33.2

Electronic versions of all submittals should be encouraged. All submissions should be made concurrently to all on the distribution list.

2015
Revision

five years with micropile installations of similar size and difficulty to those shown in the Plans, and with similar subsurface geotechnical conditions. A brief description of each project and the Owner's contact person's name and current phone number shall be included for each project listed.

33.2.1—Experience and Personnel

At least two weeks prior to the start of micropile construction, the Contractor shall submit four copies of a list identifying the on-site supervisors, drill rig operators, and load testers assigned to the project to the Engineer for approval. The list shall contain a detailed summary of each individual's experience in micropile construction and testing.

- On-site supervisors shall have a minimum of two years' experience in supervising construction of micropile installations of similar size and difficulty to those shown in the Plans and similar geotechnical conditions to those described in the geotechnical report. The work experience shall be direct supervisory responsibility for the on-site micropile construction operations. Project management level positions indirectly supervising on-site micropile construction operations shall not be considered to be acceptable for this experience requirement.
- Drill rig operators shall have a minimum of one year's experience in construction of micropile foundations.
- Load testers shall have a minimum of one year's experience in testing of micropile foundations.

The Engineer shall approve or reject the Contractor's qualifications and field personnel within ten working days after receipt of the submission. Work shall not be started on any micropile installation until the Contractor's qualifications and field personnel are approved by the Engineer. The Engineer may suspend the micropile installation if the Contractor substitutes unapproved field personnel without prior approval by the Engineer. The Contractor shall be fully liable for the additional costs resulting from the suspension of work and no adjustments in contract time resulting from such suspension of work shall be allowed.

33.2.2—Construction Submittals

Working drawings and relevant structural and geotechnical design calculations prepared by the Contractor for the planned micropile system or systems shall be submitted for review and approval prior to the start of construction and shall bear the seal of a Professional Engineer.

A detailed description of the construction procedures shall be submitted for review, including a schedule of major required equipment resources, and drilling and grouting procedures.

The working drawings shall include micropile installation details giving:

- Micropile type, number, location and pattern;
- Micropile batter and orientation;
- Micropile factored design load;
- Maximum deflection of piles at factored design load;
- Type and size of reinforcing steel;
- Details of central reinforcing steel centralizers;
- Minimum total bond length;
- Total micropile length;
- Tip elevation;
- The proposed mix design for the cement grout;
- The proposed test results for the cement grout;
- Grouting volumes and maximum pressures;
- Micropile top connection details;
- Micropile cut-off elevation; and
- Anticipated ground conditions.

Shop drawings shall be submitted for review and approval for all structural steel, including micropile components, and corrosion protection system and shall bear the seal of a Professional Engineer.

For steel pipe used as permanent pipe/casing, or as core steel, a minimum of two representative coupon tests or mill certifications shall be submitted on each truckload delivered to the project. Certified mill test reports for the reinforcing steel shall be submitted for record purposes as the materials are delivered. The ultimate strength, yield strength, elongation, and chemical analyses shall be included.

Typically the pipe suppliers and thread fabricators perform unique coupon tests for each truckload of new “Mill Secondary” pipe delivered from the pipe manufacturers. A truckload is defined as up to 44,000 lb.

For steel pipe/casing that is in tension and requires threaded joints, data demonstrating the adequacy of the joint under tensile loads shall be submitted.

The Contractor shall submit for approval the grout mix designs, including details of all materials to be incorporated, and the procedure for mixing and placing the grout. The submittal shall include certified test results verifying the acceptability of the proposed grout mix designs.

The Contractor shall submit for review and approval detailed plans for the method proposed for testing of micropiles prior to any testing. The plans shall include all necessary drawings and details to clearly describe the test method and equipment proposed.

The Contractor shall submit for review and acceptance the proposed micropile load testing procedure. The testing procedure shall be provided in accordance with project specified schedules and shall bear the seal of a Professional Engineer. The micropile verification load testing procedure shall be in general conformance with the latest versions of ASTM D1143 and/or ASTM D3689, and shall indicate the minimum following information:

- Type and accuracy of apparatus for measuring load;
- Type and accuracy of apparatus for applying load;
- Type and accuracy of apparatus for measuring the pile deformation;
- Type and capacity of reaction load system; and
- Calibration reports for hydraulic jack and load and deformation devices.

The Contractor shall submit calibration reports for each test jack, pressure gauge, load cell, and master pressure gauge to be used. The calibration tests shall be performed by an independent testing laboratory. Calibration of pressure gauges shall be within six months prior to the date submitted. Testing shall not commence until these calibrations have been approved.

All Contractor construction submittals shall be submitted at least four weeks prior to the start of micropile construction. The Contractor shall submit the number of copies required by the contract documents of the required construction submittals for acceptance by the Engineer. Work shall not begin until the appropriate submittals have been approved in writing.

33.2.3—Installation Records

As a minimum, the installation records shall be submitted within 24 hr after each pile installation is completed. As a minimum, the records shall include the following:

- Pile drilling duration, rate, and observations;
- Description of soil and rock encountered;
- Approximate final tip elevation;
- Cut-off elevation;
- Nominal resistance;
- Description of unusual behavior/conditions;
- Deviations from planned parameters;
- Grout pressures attained (where applicable) including target pressures and pressures attained;

2015
Revision

- Grout volumes pumped;
- Pile materials and dimensions;
- Load test records, analysis and details; and
- Project information, pile location (or ID number), inspector name, drill method, drill rig operator.

In addition, as-built drawings showing the locations of micropiles, their depths and inclination, and the details of their composition shall be submitted.

33.3—MATERIALS

33.3.1—Water

Water for mixing grout shall be potable, clean and free from substances that may be in any way deleterious to grout or steel. If water is not potable, it shall be tested in accordance with AASHTO T 26 for acceptability.

33.3.2—Admixtures

Admixtures shall conform to the requirements of AASHTO M 194. Expansive admixtures shall only be added to grout used for filling sealed encapsulations or micropile top connections. Admixtures shall be compatible with the grout and mixed in accordance with the manufacturer's recommendations. Their use shall only be permitted after field tests on fluid and set grout properties. Admixtures with chlorides shall not be permitted. Accelerators shall not be permitted.

Admixtures that control bleed, improve flowability, reduce water content, and retard set may be used in the grout with prior written approval by the Engineer.

33.3.3—Cement

All cement shall be portland cement conforming to AASHTO M 85 Type I, Type II, Type III, or Type V, and shall be the product of one manufacturer. If the brand or type of cement is changed during a project, additional grout mix tests shall be conducted to ensure consistency of quality and performance in situ.

33.3.4—Fillers

Inert fillers such as sand may be used in the grout in special situations with prior written approval by the Engineer.

C33.3.4

Special situations may include the presence of large voids in the ground or when grout take and travel must be limited.

33.3.5—Bar Reinforcement

Reinforcing steel shall be either:

1. Solid deformed bars conforming to AASHTO M 31 Grade 60 or Grade 75, or AASHTO M 275 Grade 150; or
2. Continuous-thread, hollow-core steel bars (hollow injection rods) conforming to the quality, ductility and deformation requirements of AASHTO M 31

C33.3.5

Larger bars than listed in AASHTO M 31 with higher yield strength may also be used, provided they meet the quality, ductility and deformation requirements of AASHTO M 31.

2015 Revision

Grade 60, Grade 75, Grade 85, Grade 95, or Grade 150.

Bar couplers, if required, shall meet the requirements specified in Article 5.11.3 of the *AASHTO LRFD Bridge Design Specifications*.

33.3.6—Pipe/Casing

If pipe or casing is required to support loads or reduce deflection, the permanent steel pipe/casing shall meet the tensile requirements of API 5L Grade X52 or better or API 5 CT Grade N80 or better.

Pipe/casing shall be either:

1. New "mill secondary" steel pipe/casing without Mill Certification, provided it is free from defects (dents, cracks, tears) and has a minimum of two unique coupon tests per truckload, or
2. "Prime" steel pipe/casing meeting the requirements of API 5L Grade X52 or better or API 5CT Grade N80 or better (API, 1998).

Coupon testing shall meet the requirements of ASTM A 370-14.

If welding of high strength steel pipe/casing is required, a welding procedure conforming to AWS D1.1 *Structural Welding Code—Steel* specifications and/or recommendations shall be submitted for review and written approval by the Engineer, prior to any welding operation.

Pipe/casing splices or threads shall develop the required nominal strength of the pile cross section and shall provide proper alignment so that no eccentricity or angle occurs between the axes of the two lengths spliced.

C33.3.6

It is commonplace in the micropile industry to utilize "mill secondary" (also referred to as "structural grade") steel pipe/casing as an economical means to satisfy the structural requirements of the design and installation. It should be noted that on Federal-Aid highway construction projects it is the Contractor's responsibility to comply with the provisions of Buy America when selecting materials for the project, which can be more complicated for "mill secondary" material than for new "prime".

33.3.7—Plates and Shapes

Structural steel plates and shapes for pile top attachments shall conform to AASHTO M 183 Grade 36 or AASHTO M 223 Grade 50 as specified in the contract documents or on the working drawings.

33.3.8—Centralizers

Centralizers shall be fabricated from plastic or material that is nondetrimental to the reinforcing steel. Wood shall not be used.

Centralizers shall provide for the grout cover specified in Article 33.4.1 and shall permit the free flow of grout without misalignment of the reinforcement.

33.3.9—Corrosion Protection

Coating requirements shall be as shown on the Drawings.

If epoxy coating is used, the minimum thickness of electrostatically applied coating in reinforcing steel shall be 0.007 in. Epoxy coating shall be in accordance with ASTM A775/AASHTO M 284 (ASTM, 2006b;

C33.3.9

Coatings are typically not applied to pipe/casing because the coating may be damaged or removed by drilling during casing advancement.

Sacrificial steel may be used in design of the pile and bearing plates, subject to approval by the Engineer.

Epoxy coating thickness in excess of 0.007 in. may

2015
Revision

AASHTO) or ASTM A934 (ASTM, 2006b). Bend test requirements shall be waived. Epoxy coating shall be factory-applied only; on-site epoxy coating shall not be permitted, except for minor field touch-up. Any field touch-up shall be completed per manufacturer's specifications.

result in problems with connection of bar and hardware resulting from inability to thread couplers and nuts onto bars.

Galvanization shall not be allowed as a means of corrosion protection for bars with yield strengths higher than Grade 75.

Galvanization is not standard industry practice. If casing is used, galvanization should not be used.

33.4—CONSTRUCTION

33.4.1—Installation

Restrictions on drill methods or installation procedures shall be as specifically defined in the project specifications or drawings.

The micropile installation technique and sequence shall be consistent with the geotechnical, logistical, environmental, and load-carrying conditions of the project. The micropile contractor shall select the drilling method and the grouting procedures used for the installation of the micropiles.

The drilling equipment, methods, and sequence shall be suitable for drilling through the conditions to be encountered, with minimal disturbance to these conditions or any overlying or adjacent structure or service.

The borehole must be open to the defined nominal diameter, full length; and prior to placing grout and reinforcement unless the installation is completed by grouting through hollow injection rods.

All installation techniques shall be determined and scheduled such that there will be no interconnection or damage to previously installed piles.

Centralizers shall be provided at 10 ft. maximum spacing on central reinforcement. The uppermost and lowermost centralizers shall be located a maximum of 5 ft. from the ends of the micropile. Centralizers shall permit the free flow of grout without misalignment of the reinforcement.

C33.4.1

Water, polymer slurries, and compressed air are typical micropile hole flushing media. Air should be used with caution when in the vicinity of existing structures or services, and in granular soils below the water table.

The requirement for an open borehole of defined nominal diameter may be met in stable, competent materials without the need for temporary borehole support. Often, some form of borehole wall support will be required for all or part of the pile length. Such methods include temporary steel pipe/casing, hollow stem augers, or the use of a hole-stabilizing drilling fluid (including grout) provided it has no deleterious effect on geotechnical bond development.

Spacing of centralizers on pipe reinforcement (other than pipe/casing used for drilling) may be larger than 10 ft.

2015
Revision

Minimum grout cover shall be as shown in Table 33.4.1-1.

Table 33.4.1-1—Minimum Grout Cover for Steel Reinforcement

Condition	Minimum Cover on Bar (in.)	Minimum Cover on Coupler (in.)
Micropiles in Soil	1	$^1/_4$
Micropiles in Rock	$^1/_2$	$^1/_4$
Coated or Encapsulated Bars	$^1/_2$	$^1/_4$

Table 33.4.1-1 provides the reinforcement steel minimum required grout coverage so that the designer can develop the minimum drill hole diameters required for corrosion protection only. Drill hole diameter should be determined based on the structural and geotechnical load requirements, corrosion protection requirements, and constructability issues.

For Type A, B, C, and D micropiles, the central reinforcement steel with centralizers shall be lowered into the stabilized drill holes to the desired depth without difficulty. Partially inserted reinforcing bars shall not be driven or forced into the hole such that there will be no interconnection or damage to piles in which the grout has not achieved final set.

Grout shall be injected in accordance with Article 33.4.2.

The Contractor shall check pile top elevations and adjust all installed micropiles to the planned elevations.

For Type E micropiles, the hollow injection rods serve as the central pile reinforcement.

33.4.2—Grouting

The Contractor shall provide calibrated systems and equipment to measure the grout quality (including, at a minimum, compressive strength according to AASHTO T 106/ASTM C109 and grout density), quantity, and pumping pressure during the grouting operations.

Grouting of the bond zone shall occur within a 24-hour period after completion of drilling when installing micropiles in materials such as consolidated clays and weak rock, which can deteriorate and soften from exposure.

After drilling of Type A, B, C, or D micropiles, the holes shall be flushed with water or air, or both to remove drill cuttings and/or other loose debris. Type E micropiles may be flushed with grout.

The Contractor shall provide a stable, homogenous neat cement grout or a sand-cement grout with a minimum 28-day unconfined compressive strength of 4 ksi. The grout shall not contain lumps or any other evidence of poor or incomplete mixing. Admixtures, if used, shall be mixed in accordance with manufacturer's recommendations. The pump shall be equipped with a pressure gauge to monitor grout pressures. The pressure gauge shall be capable of measuring pressures of at least 0.15 ksi or twice the actual grout pressures used by the Contractor, whichever is greater. The grouting equipment shall be sized to enable the grout to be

C33.4.2

2015 Revision

pumped in one continuous operation. The grout shall be kept in constant agitation prior to pumping.

The grout must be placed within 1 hr after mixing the grout, or within the time recommended by the manufacturer, if admixtures are used. Grout not placed within the allowed time limit shall be rejected.

The grout shall be injected from the lowest point of the drill hole (by tremie methods) until clean, pure grout flows from the top of the micropile. The tremie grout may be pumped through grout tubes, hollow stem augers, or drill rods. Subsequent to tremie grouting, all grouting operations for example, those associated with extraction of drill pipe/casing and pressure grouting, must ensure complete continuity of the grout column. The use of compressed air to directly pressurize the fluid grout is not permissible. The grout pressures and grout takes shall be controlled to prevent excessive heave in soil or fracturing of soil or rock formations. The entire pile shall be grouted to the design cut-off level.

Drilling with grout may be considered as meeting the tremie grout requirement, provided that the grout spoils at the completion of grouting meet the grout design strength.

Upon completion of grouting of Type A and B piles, the grout tube may remain in the hole, but it shall be filled with grout. For Type C and D piles, post-grout tubes shall be installed prior to the tremie grouting.

Grout within the micropiles shall be allowed to attain the minimum design strength prior to being loaded.

If the Contractor uses a post-grouting system, all relevant details including grouting pressure, volume, location, and mix design, shall be submitted as part of Article 33.2.2.

33.4.3—Pile Splices

Pile splices shall be constructed to develop the required factored design strength of the pile cross section.

Lengths of pipe/casing and reinforcing steel to be spliced shall be secured in proper alignments and in such a manner that no eccentricity or change in angle results between the axis of the spliced.

C33.4.3

Casing joints must be carefully and properly aligned to ensure the threaded joints connect correctly and cross threading is avoided, which may damage casing joints. When proper threading is verified, the casing can be spun tight so the shoulders of adjoining pieces of casing butt against one another tightly.

For reinforcing steel that is spliced or coupled, the bars and couplers should be tightened in accordance with the manufacturer's recommendations.

33.4.4—Tolerances

Centerline of piling shall not be more than 3 in. from indicated plan location.

Pile-hole alignment of vertical piles shall be within 2 percent of design alignment.

Pile-hole alignment of piles inclined up to 1:6 shall be within 4 percent of design alignment.

Pile-hole alignment of piles inclined greater than 1:6 shall be within 7 percent of design alignment.

Top elevation of pile shall be within +1 in. to –2 in. of the design vertical elevation.

Centerline of core reinforcement shall not be more than $^{3}/_{4}$ in. from centerline of piling.

C33.4.4

These are maximum tolerances for control of construction.

2015
Revision

33.5—LOAD TESTING

33.5.1—General

Pile load tests shall be performed to verify the adequacy of the design and construction of the pile system.

C33.5.1

Micropiles are field tested to verify that the micropile design loads can be carried without excessive movements. In addition, testing is used to verify the adequacy of the Contractor's drilling, installation, and grouting operations prior to and during construction of production piles. Therefore the soil/rock conditions, as well as the method, equipment, and operator used for installing production piles must be similar to those used for installing test piles.

Micropile testing is conducted by incrementally loading (and if specified, unloading) the pile and measuring the movement of the pile head at each load increment. Typically, the pile head movement reading is recorded just after the next load increment has been applied. The loading increments, the time that each load increment is held, and the number of measurements for each load increment are determined by the type of test being performed and will be specified in the contract documents. If not specified, recommended practice is to obtain a pile-head movement reading just after the load has been applied, and a second reading after the load has been maintained for a sufficient amount of time to ensure that pile-head movement has stabilized.

The total number of load tests, maximum test load capacities, and load test schedules will vary based on ground type, pile ultimate capacity, type of loading (e.g., axial, lateral, or cyclic), structure sensitivity, and Owner/Contractor experience.

In creep susceptible soils, Extended Creep Tests beyond the tests described herein should be based on Sabatini, et al. (2005) and as described on the Drawings or in the Project Specifications.

Load testing should be monitored by the Engineer in conjunction with the Contractor.

33.5.2—Verification Testing

The number of test piles, their location, acceptable load and movement criteria, and the type(s) of loading direction shall be as identified in the contract documents.

The micropile load test results shall verify the suitability of the design and installation methods, and shall be reviewed and accepted by the Engineer prior to the Contractor's initiation of production micropile installation.

The drilling and grouting methods, pipe/casing and other reinforcement details, and depth of embedment for the test pile(s) shall be identical to the production piles, except where approved otherwise by the Engineer.

C33.5.2

2015 Revision

The tested micropiles shall be loaded to 150 percent of the factored design load (FDL). The jack shall be positioned at the beginning of the test such that the unloading and repositioning of the jack during the test shall not be required. Piles shall be tested under tension loads prior to testing under compression loads. An alignment load (AL) may be applied to the pile prior to setting the movement recording devices. This alignment load shall be no more than 0.04 FDL. Dial gauges shall be zeroed at the first setting of AL.

Verification pile load tests are typically performed on sacrificial test piles to verify the adequacy of the design of the pile system and the proposed construction procedures prior to installation of production piles.

The load tested piles should be of the same design as the production piles for the most meaningful results. If the maximum test load exceeds the structural yield load of the pile, the pipe/casing wall thickness and/or bar size may be increased for the test pile; this will result in a decrease in the elastic pile deflection, which may then be estimated by calculation. Alternatively, the bond length may be shortened and the test load reduced when approved by the Engineer.

Axial pile load tests shall be made by loading the micropile in the steps shown in Table 33.5.2-1 and recording the head movement at each step.

For sacrificial test piles, further load cycles may be conducted to failure.

The cyclic test procedure is similar to ASTM D1143 (ASTM, 2006d). This loading method permits fundamental analysis of load test performance because it allows elastic and permanent movements to be studied separately, which can reduce shear/bond strengths to residual values due to continued alternating extension and compression.

Measurement of pile movement shall be obtained at each increment. The load hold period for creep test at 0.975 FDL measurement shall start as soon as the test load is applied and shall be recorded at 1, 2, 3, 4, 5, and 10 min (load cycle maxima only).

The acceptance criteria for micropile verification load tests shall be:

1. The pile shall sustain the compression and tension design loads (0.75 FDL) with no more than the specified permissible total vertical movement at the top of the pile as measured relative to the top of the pile prior to the start of testing. If an alignment load is used, then the allowable movement shall be reduced by multiplying by a factor of (0.75 FDL – AL)/0.75 FDL.

2. Test piles shall have a creep rate at the end of the 0.975 FDL increment that is not greater than 0.04 in./log cycle time (1 to 10 min) or 0.08 in./log cycle time (6 to 60 min) and has a linear or decreasing creep rate.

3. Failure does not occur by 1.50 FDL. Failure is defined as a slope of the load versus deflection curve (at end of increment) exceeding 0.025 in./kip.

The adjustment for allowable movement with an alignment load conservatively accounts for the movement needed to reach AL.

For compression piles with diameters greater than 10 in. and tested in compression, the acceptance criteria may be amended as suggested in the *AASHTO LRFD Bridge Design Specifications*, Article 10.9.3.5.4.

Table 33.5.2-1—Load Steps for Verification Testing

Int.	Load	Hold Time (min)
1	AL	-
2	0.075 FDL	4
3	0.15 FDL	4
4	0.225 FDL	4
5	0.30 FDL	4
6	0.375 FDL	4
7	AL	1
8	0.15 FDL	1
9	0.30 FDL	1
10	0.375 FDL	1
11	0.45 FDL	4
12	0.525 FDL	4
13	0.60 FDL	4
14	0.675 FDL	4
15	0.75 FDL	4
16	AL	1
17	0.30 FDL	1
18	0.60 FDL	1
19	0.675 FDL	1
20	0.75 FDL	1
21	0.825 FDL	4
22	0.90 FDL	4
23	0.975 FDL	10 or 60 (Creep Test)
24	AL	1
25	0.30 FDL	1
26	0.60 FDL	1
27	0.90 FDL	1
28	0.975 FDL	1
29	1.05 FDL	4
30	1.125 FDL	4
31	1.20 FDL	4
32	1.275 FDL	4
33	1.35 FDL	4
34	1.425 FDL	4
35	1.50 FDL	4
36	1.20 FDL	4
37	0.90 FDL	4
38	0.60 FDL	4
39	0.30 FDL	4
40	AL	15

The failure load, or the maximum test load if the test is not taken to failure, needs to exceed the required nominal geotechnical resistance, which is equal to FDL/φ. Since the geotechnical resistance factor from the *AASHTO LRFD Bridge Design Specifications*, Article 10.5.5.2 is 0.7, then maximum test load needs to be FDL/0.7 = 1.43 FDL; this is rounded up to 1.50 FDL. ASTM requires loading increments at 5 percent of failure load so the increments are 5 percent of 1.50 FDL equal to 7.5 percent FDL.

ASTM requires each load increment to be added in a continuous fashion immediately following the completion of movement readings for the previous load interval. At the end of the load increment keep the load constant for a time interval of not less than 4 min and not more than 15 min. The same time interval is to be used for all loading increments throughout the test.

ASTM suggests considering longer time intervals for the failure load to assess creep behavior and for the final zero load to assess rebound behavior.

The Contractor shall submit a written report providing micropile geometry and construction details within seven working days after the completion of the verification tests.

If the micropile load tests fail to meet the design requirements, the cause(s) shall be established and the micropile design and/or installation methods shall be modified by the Contractor, and the new system shall be retested as directed by the Engineer.

Modifications include, but are not limited to, installing replacement micropiles, modifying the installation methods, increasing the bond length, regrouting via preplaced regrout tubes, or changing the micropile type.

Any modification that requires changes to the structure shall have prior review and acceptance of the Engineer. The cause for any modifications of design or construction procedures shall be decided in order to appropriately determine any additional cost implications.

33.5.3—Proof Testing

The Contractor shall proof test the specified minimum number of production micropiles. The piles to be tested will be selected by the Engineer. At the Contractor's suggestion, but with the Engineer's concurrence, tension tests may be performed to 1.00 FDL with sufficient structural tension capacity.

Axial pile load tests shall be made by loading the micropile in the steps shown in Table 33.5.3-1 and recording the head movement at each step.

C33.5.3

The total number of load tests, maximum test load capacities, and load test schedules will vary on a project by project basis. They are dependent on ground type, pile ultimate capacity, pile loadings type (i.e., static or seismic), structure sensitivity, and Owner/Contractor experience.

In creep susceptible soils, extended creep tests beyond the tests described herein should be based on Sabatini et al. (2005) and as described on the drawings or in the project specifications.

2015 Revision

Table 33.5.3-1—Load Steps for Proof Testing

Int.	Load	Hold Time (min)
1	AL	—
2	0.10 FDL	4
3	0.20 FDL	4
4	0.30 FDL	4
5	0.40 FDL	4
6	0.50 FDL	4
7	0.60 FDL	4
8	0.70 FDL	4
9	0.80 FDL	4
10	0.90 FDL	4
11	1.00 FDL	10 or 60 (Creep Test)
12	0.75 FDL	4
13	0.50 FDL	4
14	0.25 FDL	4
15	AL	4

The acceptance criteria for micropile proof load tests shall be:

1. The pile shall sustain the tension and compression design loads (0.75 FDL) with no more than the specified minimum permissible total vertical movement at the top of the pile as measured relative to the pile prior to the start of testing. If an alignment load is used, then the allowable movement shall be reduced by multiplying by a factor of (0.75 FDL – AL)/0.75 FDL.

2. Test piles shall have a creep rate at the end of the 1.00 FDL increment that is not greater than 0.04 in./log cycle time (1 to 10 min) or 0.08 in./log cycle time (6 to 60 min) and has a linear or decreasing creep rate.

3. Failure does not occur by 1.00 FDL test load.

2015 Revision

If a production micropile fails to meet the acceptance criteria, modifications shall be made to the design, the construction procedures, or both.

Any modification that requires changes to the structure shall have prior review and acceptance of the Engineer. The cause for any modifications of design or construction procedures shall be decided in order to appropriately determine any additional cost implications.

Modifications include, but are not limited to, installing replacement micropiles, modifying the installation methods, increasing the bond length, regrouting via preplaced regrout tubes, or changing the micropile type. The Engineer may elect to proof test additional piles depending on the circumstances of the modifications.

33.5.4—Lateral Loading

If required, lateral load testing shall be conducted in accordance with the latest version of ASTM D3966. Lateral load testing shall be conducted before axial load testing. During both verification and proof test phases, care must be exercised to not cause permanent structural damage to the pile that will subsequently reduce its axial load capacity.

C33.5.4

The specified acceptance criterion, expressed as a maximum total movement at a certain load, must be carefully selected so as not to potentially damage the structure.

33.6—MEASUREMENT AND PAYMENT

33.6.1—Method of Measurement

33.6.1.1—Mobilization and Demobilization

Mobilization and demobilization shall be measured on a lump sum basis.

33.6.1.2—Micropiles

Project-specific criteria such as minimum pile lengths and maximum top of bond zone elevation shall be as indicated in the project drawings or project specifications.

Micropiles, installed and accepted including test piles, shall be measured per linear foot in soil and per linear foot in rock. Micropiles, complete and in place, shall be of the types and shall provide the load resistances indicated in the contract documents or accepted in writing by the Engineer, and furnished in compliance with the material requirements of these Specifications.

Such measurement shall not include micropiles damaged prior to completion of the work unless remedied to the satisfaction of the Engineer.

C33.6.1.2

This provision assumes the Owner has specified the required micropile load resistances and quantity. Measurement may be stipulated as lump sum if micropile Contractor determines the micropile quantity.

For projects with soil pile design by the Contractor, micropiles, including test piles, should be measured per each, installed and accepted.

For projects with rock pile design by the Contractor, micropiles, including test piles, should be measured per linear foot to rock, installed and accepted. Since the actual linear feet in rock may vary by Contractor the cost of the rock socket is not a separate item.

33.6.1.3—Verification Load Tests

Verification load tests shall be measured by the number of load tests performed and accepted for each designated pile load capacity. Load tests made at the option of the Contractor shall not be included in the quantity measured for payment.

33.6.1.4—Proof Load Tests

Proof load tests shall be measured by the number of load tests performed and accepted for each designated pile load capacity. Load tests made at the option of the Contractor shall not be included in the quantity measured for payment.

33.6.1.5—Obstruction Drilling

Obstruction drilling shall be measured by the hour where obstructions, as defined in the contract documents, are encountered.

C33.6.1.5

An obstruction is generally defined as a specific object (including, but not limited to, boulders, logs, and man-made objects) encountered during the drilling operation that prevents or hinders the advance of the borehole.

33.6.2—Basis of Payment

33.6.2.1—Mobilization and Demobilization

Mobilization and demobilization shall be paid on a lump sum basis in accordance with the contract documents. Such payment shall be considered full compensation for providing all labor, equipment, and materials needed to complete micropile installation and testing.

33.6.2.2—Micropiles

Micropiles shall be paid for at the contract prices per linear foot in soil and per linear foot in rock for piles of the types specified and approved by the Engineer.

The Contractor shall be responsible for estimating the grout take. There shall be no extra payment for grout overruns, except as otherwise specified in the contract documents.

C33.6.2.2

Payment may be stipulated as lump sum if the micropile contractor determines the micropile quantities. See also Article C33.6.1.2 for Contractor designed piles in soil.

Payment for grout overrun may be appropriate on projects where such overrun is beyond the control of the Contractor such as voided ground (e.g., karst, urban fill, mines) as addressed in the contract documents.

33.6.2.3—Verification Load Tests

Verification load tests shall be paid for at the contract price per test specified, satisfactorily completed, and meeting load carrying requirements.

Payment for verification load tests includes full compensation for providing all labor, equipment, and materials needed to perform load tests and submitting reports as specified.

33.6.2.4—Proof Load Tests

Proof load tests shall be paid for at the contract price per test specified and satisfactorily completed and meeting load carrying requirements.

Payment for proof load tests shall include full compensation for providing all labor, equipment, and materials needed to perform load tests and submitting report(s) as specified.

33.6.2.5—Obstruction Drilling

Payment for obstruction drilling shall include full compensation for providing all labor and equipment needed to drill though or remove obstructions.

33.7—REFERENCES

AASHTO. 2006. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing. Part I—Specifications,* American Association of State Highway and Transportation Officials, Washington, DC.

American Petroleum Institute. (1998) *Specification for Casing and Tubing API Specification 5CT, 6th Edition*, API, Washington, DC.

American Society for Testing and Materials (ASTM). 2006a. Volume 01.01, *Steel-Piping, Tubing, Fittings,* American Society for Testing and Materials, West Conshohocken, PA.

American Society for Testing and Materials (ASTM). 2006b. Volume 01.04, *Steel-Structural, Reinforcing, Pressure Vessel, Railway,* American Society for Testing and Materials, West Conshohocken, PA.

American Society for Testing and Materials (ASTM). 2006c. Volume 01.06, *Coated Steel Products,* American Society for Testing and Materials, West Conshohocken, PA.

American Society for Testing and Materials (ASTM). 2007. Volume 04.08, *Soil and Rock (I): D 420 - D 5876,* American Society for Testing and Materials, West Conshohocken, PA.

PTI. 2004. *Recommendations for Prestressed Rock and Soil Anchors*, 4th Edition. Post Tensioning Institute, Phoenix, AZ.

Sabatini, P. J., B. Tanyu, T. Armour, P. Groneck, and J. Keeley. 2005. "Micropile Design and Construction," *NHI Course 132078 Reference Manual*, FHWA NHI-05-039. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, December.

2015
Revision

CHANGED AND DELETED ARTICLES, 2010

SUMMARY OF AFFECTED SECTIONS

The third edition revisions to the *AASHTO LRFD Bridge Construction Specifications* affect the following sections:

3. Temporary Works
5. Drilled Shafts
11. Steel Structures
18. Bearing Devices
30. Thermoplastic Pipe

SECTION 3 REVISIONS

Changed Articles

The following Article in Section 3 contains changes or additions to the specifications, the commentary, or both:

3.1.1

Deleted Articles

No Articles were deleted from Section 3.

SECTION 5 REVISIONS

The entire Section 5 has been rewritten.

SECTION 11 REVISIONS

Changed Articles

The following Articles in Section 11 contain changes or additions to the specifications, the commentary, or both:

11.3.1.1	11.4.3.3.2	11.4.11	11.8.3.3.1
11.3.1.4	11.4.3.3.3	11.4.12.2.1	11.8.3.6.4
11.3.1.7	11.4.7	11.4.12.2.3	

Deleted Articles

No Articles were deleted from Section 11.

SECTION 18 REVISIONS

Changed Articles

The following Articles in Section 18 contain changes or additions to the specifications, the commentary, or both:

18.1.5.1.1	18.1.5.2.6	18.3.4.1	18.3.4.4.1	18.3.4.4.5
18.1.5.1.2	18.1.5.2.7	18.3.4.2	18.3.4.4.2	18.3.5
18.1.5.2.3	18.3.2.6	18.3.4.3	18.3.4.4.3	18.6.3
18.1.5.2.4	18.3.2.8	18.3.4.4	18.3.4.4.4	18.8.4.1

Deleted Articles

The following Articles were deleted from Section 18:

18.3.4.2.1	18.3.4.2.2	18.3.4.3.1	18.3.4.3.2

SECTION 30 REVISIONS

Changed Articles

The following Article in Section 30 contains changes or additions to the specifications, the commentary, or both:

30.5.2

Deleted Articles

No Articles were deleted from Section 30.

EXECUTIVE COMMITTEE
2009–2010

Voting Members

Officers:

President: Larry L. "Butch" Brown, Mississippi

Vice President: Susan Martinovich, Nevada

Secretary-Treasurer: Carlos Braceras, Utah

Regional Representatives:

REGION I:	Joseph Marie, Connecticut, One-Year Term Gabe Klein, District of Columbia, Two-Year Term
REGION II:	Dan Flower, Arkansas, One-Year Term Mike Hancock, Kentucky, Two-Year Term
REGION III:	Nancy J. Richardson, One-Year Term Thomas K. Sorel, Minnesota, Two-Year Term
REGION IV:	Paula Hammond, Washington, One-Year Term Amadeo Saenz, Jr. Texas, Two-Year Term

Nonvoting Members

Immediate Past President: Allen Biehler, Pennsylvania
AASHTO Executive Director: John Horsley, Washington, DC

HIGHWAY SUBCOMMITTEE ON BRIDGES AND STRUCTURES
2009

MALCOLM T. KERLEY, *Chair*
KEVIN THOMPSON, *Vice Chair*
M. MYINT LWIN, Federal Highway Administration, *Secretary*
RAJ AILANEY, Federal Highway Administration, *Assistant Secretary*
KEN KOBETSKY, *AASHTO Liaison*
KELLEY REHM, *AASHTO Liaison*

ALABAMA, John F. "Buddy" Black, William "Tim" Colquett, George H. Conner
ALASKA, Richard A. Pratt
ARIZONA, Jean A. Nehme
ARKANSAS, Phil Brand
CALIFORNIA, Kevin Thompson, Susan Hida, Barton J. Newton
COLORADO, Mark A. Leonard, Michael G. Salamon
CONNECTICUT, Julie F. Georges
DELAWARE, Jiten K. Soneji, Barry A. Benton
DISTRICT OF COLUMBIA, Nicolas Galdos, L. Donald Cooney, Konjit "Connie" Eskender
FLORIDA, Marcus Ansley, Sam Fallaha, Jeff Pouliotte
GEORGIA, Paul V. Liles, Jr.
HAWAII, Paul T. Santo
IDAHO, Matthew M. Farrar
ILLINOIS, Ralph E. Anderson, Thomas J. Domagalski
INDIANA, Anne M. Rearick
IOWA, Norman L. McDonald
KANSAS, Kenneth F. Hurst, James J. Brennan, Loren R. Risch
KENTUCKY, Mark Hite
LOUISIANA, Hossein Ghara, Arthur D'Andrea, Paul Fossier
MAINE, David B. Sherlock, Jeffrey S. Folsom
MARYLAND, Earle S. Freedman, Robert J. Healy
MASSACHUSETTS, Alexander K. Bardow, Shirley Eslinger
MICHIGAN, Steven P. Beck, David Juntunen
MINNESOTA, Daniel L. Dorgan, Kevin Western
MISSISSIPPI, Mitchell K. Carr, B. Keith Carr
MISSOURI, Dennis Heckman, Michael Harms
MONTANA, Kent M. Barnes
NEBRASKA, Mark J. Traynowicz, Mark Ahlman, Fouad Jaber
NEVADA, Mark P. Elicegui, Todd Stefonowicz
NEW HAMPSHIRE, Mark W. Richardson, David L. Scott
NEW JERSEY, Richard W. Dunne
NEW MEXICO, Raymond M. Trujillo, Jimmy D. Camp
NEW YORK, George A. Christian, Donald F. Dwyer, Arthur P. Yannotti
NORTH CAROLINA, Greg R. Perfetti
NORTH DAKOTA, Terrence R. Udland
OHIO, Timothy J. Keller, Jawdat Siddiqi
OKLAHOMA, Robert J. Rusch, Gregory D. Allen, John A. Schmiedel
OREGON, Bruce V. Johnson, Hormoz Seradj
PENNSYLVANIA, Thomas P. Macioce, Harold C. "Hal" Rogers, Jr., Lou Ruzzi
PUERTO RICO, (Vacant)
RHODE ISLAND, David Fish
SOUTH CAROLINA, Barry W. Bowers, Jeff Sizemore
SOUTH DAKOTA, Kevin Goeden
TENNESSEE, Edward P. Wasserman
TEXAS, David P. Hohmann, Keith L. Ramsey
U.S. DOT, M. Myint Lwin, Firas I. Sheikh Ibrahim
UTAH, (Vacant)
VERMONT, Wayne B. Symonds
VIRGINIA, Malcolm T. Kerley, Kendal Walus, Prasad L. Nallapaneni, Julius F. J. Volgyi, Jr.
WASHINGTON, Jugesh Kapur, Tony M. Allen, Bijan Khaleghi
WEST VIRGINIA, Gregory Bailey, James D. Shook
WISCONSIN, Scot Becker, Beth A. Cannestra, William Dreher
WYOMING, Gregg C. Fredrick, Keith R. Fulton

GOLDEN GATE BRIDGE, Kary H. Witt
N.J. TURNPIKE AUTHORITY, Richard J. Raczynski
N.Y. STATE BRIDGE AUTHORITY, William J. Moreau
PENN. TURNPIKE COMMISSION, James L. Stump

U.S. ARMY CORPS OF ENGINEERS—DEPARTMENT OF THE ARMY, Christopher H. Westbrook
U.S. COAST GUARD, Hala Elgaaly
U.S. DEPARTMENT OF AGRICULTURE—FOREST SERVICE, John R. Kattell, Scott F. Mitchell

ALBERTA, Tom Loo
NEW BRUNSWICK, Doug Noble
NOVA SCOTIA, Mark Pertus
ONTARIO, Bala Tharmabala
SASKATCHEWAN, Howard Yea

TRANSPORTATION RESEARCH BOARD—Waseem Dekelbab

INSTRUCTIONS AND INFORMATION

General

AASHTO has issued proposed interim revisions to *AASHTO LRFD Bridge Construction Specifications*, Third Edition (2010). This packet contains the revised pages. They are not designed to replace the corresponding pages in the book but rather to be kept with the book for fast reference.

Affected Articles

Underlined text indicates revisions that were approved in 2010 by the AASHTO Highways Subcommittee on Bridges and Structures. ~~Strikethrough text~~ indicates any deletions that were likewise approved by the Subcommittee. A list of affected articles is included below.

All interim pages are printed on blue paper to make the changes stand out when inserted in the third edition binder. They also have a page header displaying the section number affected and the interim publication year. Please note that these pages may also contain nontechnical (e.g. editorial) changes made by AASHTO publications staff; any changes of this type will not be marked in any way so as not to distract the reader from the technical changes.

Table i—2010 Changed Articles

SECTION 8: CONCRETE STRUCTURES

8.3.4	8.4.1.1	8.18

SECTION 11: STEEL STRUCTURES

11.3.2.6	11.4.12.2.6	11.5.6.4.2	11.10
11.4.9.1	11.5.6.4.1	11.5.6.4.7	

SECTION 18: BEARING DEVICES

18.1.4.2

SECTION 30: THERMOPLASTIC PIPE

Section 30 was replaced in its entirety.

SECTION 31: ALUMINUM STRUCTURES

31.4.3.3.2

INSTRUCTIONS AND INFORMATION

General

AASHTO has issued proposed interim revisions to *AASHTO LRFD Bridge Construction Specifications*, Third Edition (2010). This packet contains the revised pages. They are not designed to replace the corresponding pages in the book but rather to be kept with the book for fast reference.

Affected Articles

Underlined text indicates revisions that were approved in 2011 by the AASHTO Highways Subcommittee on Bridges and Structures. ~~Strikethrough text~~ indicates any deletions that were likewise approved by the Subcommittee. A list of affected articles is included below.

All interim pages are printed on pink paper to make the changes stand out when inserted in the third edition binder. They also have a page header displaying the section number affected and the interim publication year. Please note that these pages may also contain nontechnical (e.g. editorial) changes made by AASHTO publications staff; any changes of this type will not be marked in any way so as not to distract the reader from the technical changes.

Table i—2011 Changed Articles

SECTION 11: STEEL STRUCTURES

11.3.2.6	C11.3.2.6	11.5.6.4.7a	11.5.6.4.7b

SECTION 18: BEARING DEVICES

18.3.3.1	C18.3.3.1	18.8.3.3

SECTION 26: METAL CULVERTS

26.4.3	C26.4.3	26.5.4.5

REFERENCES

INSTRUCTIONS AND INFORMATION

General

AASHTO has issued interim revisions to *AASHTO LRFD Bridge Construction Specifications*, Third Edition (2010). This packet contains the revised pages. They are not designed to replace the corresponding pages in the book but rather to be kept with the book for quick reference.

Affected Articles

Underlined text indicates revisions that were approved in 2012 by the AASHTO Highways Subcommittee on Bridges and Structures. ~~Strikethrough text~~ indicates any deletions that were likewise approved by the Subcommittee. A list of affected articles is included below.

All interim pages are printed on pink paper to make the changes stand out when inserted in the third edition binder. They also have a page header displaying the section number affected and the interim publication year. Please note that these pages may also contain nontechnical (e.g. editorial) changes made by AASHTO publications staff; any changes of this type will not be marked in any way so as not to distract the reader from the technical changes.

Please note that in response to user concerns, page breaks are now being added within sections between noncontiguous articles. This change makes it an option to insert the changes closer to the affected articles.

Table i—2012 Changed Articles

SECTION 11: STEEL STRUCTURES

11.4.3.3.1	11.4.3.3.2	11.4.3.3.3	11.4.8.1.1	11.10

SECTION 26: METAL CULVERTS

26.1.1	26.4.1	26.5.1	26.5.7.2
26.3.8.1	26.4.2	26.5.7	26.5.7.2.1
26.3.9	26.4.2.5	26.5.7.1	26.5.7.2.2

SECTION 30: THERMOPLASTIC PIPE

30.1.1	30.6	30.7.4.1
30.3.1	30.7.4	30.7.4.2

INSTRUCTIONS AND INFORMATION

General

AASHTO has issued interim revisions to *AASHTO LRFD Bridge Construction Specifications*, Third Edition (2010). This packet contains the revised pages. They are not designed to replace the corresponding pages in the book but rather to be kept with the book for quick reference.

Affected Articles

Underlined text indicates revisions that were approved in 2013 by the AASHTO Highways Subcommittee on Bridges and Structures. ~~Strikethrough text~~ indicates any deletions that were likewise approved by the Subcommittee. A list of affected articles is included below.

All interim pages are printed on pink paper to make the changes stand out when inserted in the third edition binder. They also have a page header displaying the section number affected and the interim publication year. Please note that these pages may also contain nontechnical (e.g. editorial) changes made by AASHTO publications staff; any changes of this type will not be marked in any way so as not to distract the reader from the technical changes.

Please note that in response to user concerns, page breaks are added within sections between noncontiguous articles. This makes it an option to insert the changes closer to the affected articles.

Table i—2013 Changed Articles

SECTION 11: STEEL STRUCTURES

11.1.1	11.3.2.1	11.3.2.2	11.3.2.4	11.3.2.5
11.3.2.6	11.4.1	11.4.5	11.4.8.1.1	11.4.8.1.4
11.4.8.2.1	11.4.8.2.2	11.4.8.4	11.4.9.2	11.4.12.1
11.4.12.2	11.4.12.2.2	11.4.12.2.2a	11.4.12.2.2b	11.4.12.2.2c
11.4.12.2.2d	11.4.1.3.2	11.5.5.3	11.5.6.1	11.5.6.3
11.5.6.4.1	11.5.6.4.2	11.5.6.4.3	11.7.1	11.8.3.7.1
11.10				

SECTION 23: MISCELLANEOUS METAL

23.2

SECTION 31: ALUMINUM STRUCTURES

31.3.2.1.1	31.3.2.1.2	31.3.2.1.4	31.3.2.1.5

SECTION 32: SHOCK TRANSMISSION UNITS

32.3.1

INSTRUCTIONS AND INFORMATION

General

AASHTO has issued interim revisions to *AASHTO LRFD Bridge Construction Specifications*, Third Edition (2010). This packet contains the revised pages. They are not designed to replace the corresponding pages in the book but rather to be kept with the book for quick reference.

Affected Articles

Underlined text indicates revisions that were approved in 2013 by the AASHTO Highways Subcommittee on Bridges and Structures. ~~Strikethrough text~~ indicates any deletions that were likewise approved by the Subcommittee. A list of affected articles is included below.

All interim pages are printed on pink paper to make the changes stand out when inserted in the third edition binder. They also have a page header displaying the section number affected and the interim publication year. Please note that these pages may also contain nontechnical (e.g., editorial) changes made by AASHTO publications staff; any changes of this type will not be marked in any way so as not to distract the reader from the technical changes.

Please note that in response to user concerns, page breaks are added within sections between noncontiguous articles. This makes it an option to insert the changes closer to the affected articles.

2014 Changed Articles

SECTION 11: STEEL STRUCTURES

11.1.1	11.4.8.1	11.5.6.4.1	11.10
11.4.3.1	11.4.8.1.1	11.8.3.6.4	

SECTION 18: BEARING DEVICES

18.8.2.6.1

SECTION 33: MICROPILES

New Section

2015
Revision

INSTRUCTIONS AND INFORMATION

General

AASHTO has issued interim revisions to *AASHTO LRFD Bridge Construction Specifications*, Third Edition (2010). This packet contains the revised pages. They are not designed to replace the corresponding pages in the book but rather to be kept with the book for quick reference.

Affected Articles

Underlined text indicates additions that were approved in 2015 by the AASHTO Highways Subcommittee on Bridges and Structures. ~~Strikethrough text~~ indicates any deletions that were likewise approved by the Subcommittee. A list of affected articles is included below.

All interim pages are printed on pink paper to make the changes stand out when inserted in the third edition binder. They also have a page header displaying the section number affected and the interim publication year. Please note that these pages may also contain nontechnical (e.g., editorial) changes made by AASHTO publications staff; any changes of this type will not be marked in any way so as not to distract the reader from the technical changes.

Please note that in response to user concerns, page breaks are added within sections between noncontiguous articles. This makes it an option to insert the changes closer to the affected articles.

2016 Changed Articles

SECTION 8: CONCRETE STRUCTURES

8.2.3

SECTION 10: PRESTRESSING

10.3.1.2