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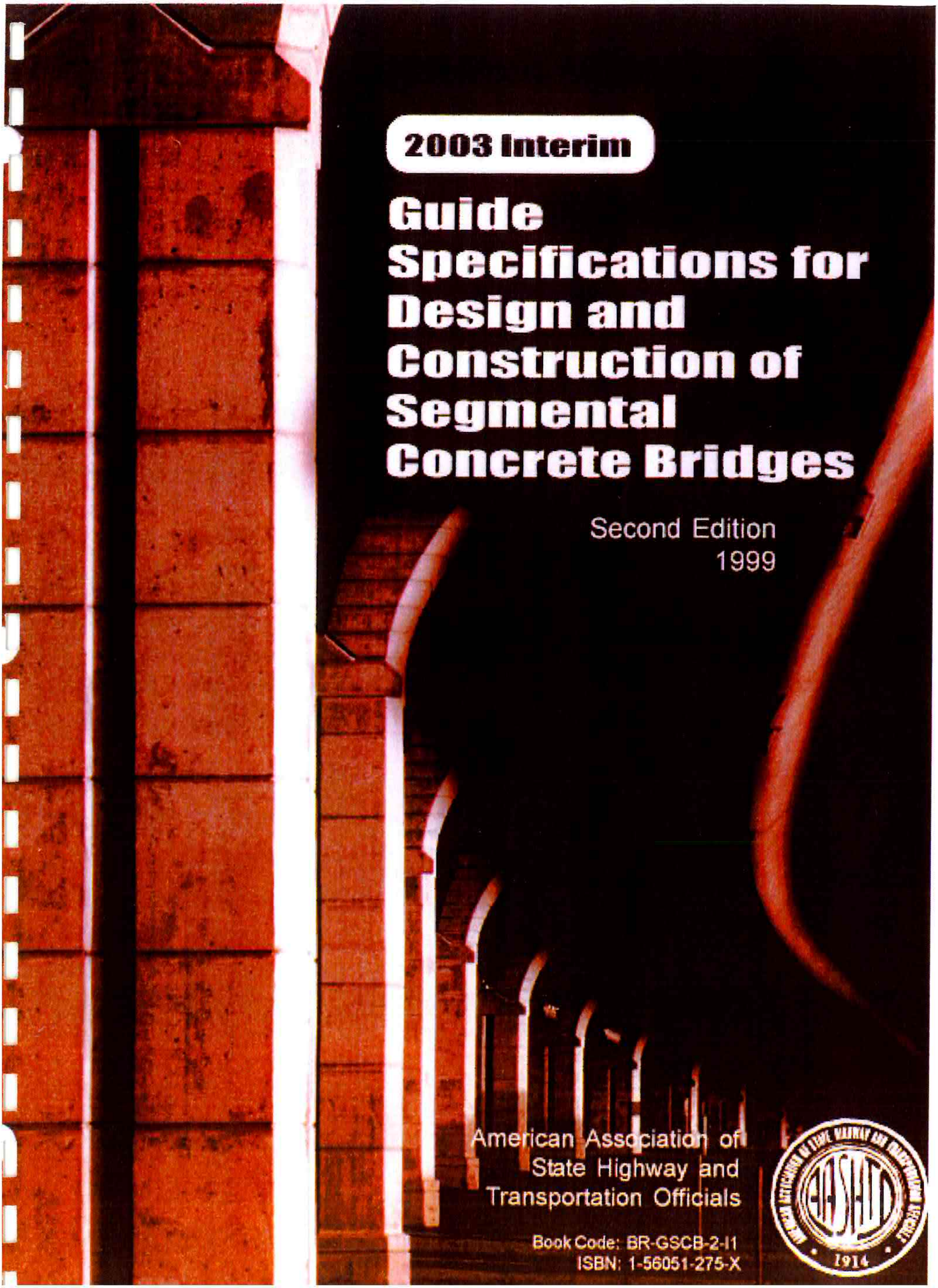

2003 Interim

Guide Specifications for Design and Construction of Segmental Concrete Bridges

Second Edition
1999

American Association of State Highway and Transportation Officials

Book Code: BR-GSCB-2-I1
ISBN: 1-56051-275-X

To recipients of the *Guide Specifications for Design and Construction of Segmental Concrete Bridges, Second Edition* (1999):

Instructions

Interim revisions have been made to the *Guide Specifications for Design and Construction of Segmental Concrete Bridges, Second Edition* (1999). This packet contains the revised pages. They have been designed to replace the corresponding pages in the book and are numbered accordingly.

A vertical line in the margins indicates revisions that were approved in 2003 by the AASHTO Subcommittee on Bridges and Structures.

All revised pages also display a box in the lower outside corner indicating the interim year(s). Any non-technical changes in page appearance will be indicated by this revision box alone to differentiate such changes from those which have been approved by the AASHTO Subcommittee on Bridges and Structures.

To keep your Specifications correct and up-to-date, please replace the appropriate pages in the book with the pages in this packet.

2003 Interim Revisions to the

Guide Specifications for

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Second Edition (1999)

Published by the American Association of
State Highway and Transportation Officials
444 North Capitol Street, N.W., Suite 249
Washington, DC 20001
www.transportation.org

ISBN: 156051-275-X

GUIDE SPECIFICATIONS FOR DESIGN AND CONSTRUCTION OF SEGMENTAL CONCRETE BRIDGES 1999

American Association of State Highway and Transportation Officials
444 North Capitol Street, N.W., Suite 249
Washington, D.C. 20001

Printed in the United States of America.

ISBN 1-56051-103-6

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PREFACE

This Second Edition of the *Guide Specifications for Design and Construction of Segmental Concrete Bridges* was prepared for use in conjunction with the *Standard Specifications for Highway Bridges, Sixteenth Edition, 1996,* and subsequent interim revisions to these specifications. The second edition of the guide specifications was developed by a broad-based committee organized by the American Segmental Bridge Institute. The committee included representatives of State Departments of Transportation, the Federal Highway Administration, academicians, consulting engineers, contractors, and suppliers. The second edition of the guide specifications was subsequently studied and approved by the Highway Subcommittee on Bridges and Structures, as part of the 1998 AASHTO ballot process.

DIVISION I
DESIGN SPECIFICATIONS

SECTION A
GENERAL REQUIREMENTS AND MATERIALS

1.0 APPLICATION

1.1 General

These specifications are intended for design of longitudinally and/or transversely post-tensioned bridges utilizing normal weight or sand-lightweight concrete constructed with either precast or cast-in-place box segments of single or multiple cells, or combinations thereof, as well as simple span and continuous segmental beam-type bridges. The specifications include provisions for design of segmental substructures. The specifications pertain to bridges of all sizes and are not restricted to bridge span lengths of 500 feet or less. Unless otherwise stated or superseded by these specifications, the provisions of the Sixteenth Edition of the AASHTO *Standard Specifications for Highway Bridges* are intended to apply to the design of segmental concrete bridges.

1.2 Notations

Notations are in accordance with AASHTO *Standard Specifications for Highway Bridges* and the following:

A = area of concrete surrounding a bar, sq. in. (See Commentary, Section 18.2.)
A = static segment weight (Section 7.4.3)
A = reaction (See Fig. 26-2.)
A_{cc} = area of concrete in compression chord, sq. in.
A_{cn} = area of one face of a truss node region, sq. in.
A_{cp} = area enclosed by outside perimeter of concrete cross section, sq. in.
A_{cs} = area of inclined compression strut, sq. in.
A_g = gross area of concrete cross section, sq. in.
A_k = area of the base of all shear keys in the failure plane, sq. in. (Section 12.2.21)
$A\ell$ = total area of additional longitudinal reinforcement to resist torsion, sq. in. (Section 12.3.8)
A_o = area enclosed by shear flow path, sq. in. (Section 12.2.10)
A_s = area of nonprestressed tensile reinforcement, sq. in.
A_s' = area of compression reinforcement, sq. in.
A_s^* = area of prestressed reinforcement in tension zone, sq. in.
A_t = area of one leg of continuous, closed transverse torsion reinforcement within a distance s, sq. in.
A_v = area of transverse shear reinforcement within a distance s, sq. in.
a = portion of single span, end span, or span adjacent to cantilever arm subject to shear lag effects (See Fig. 3-1.)
b = web width, in. (See Section 11.2.)
b = top or bottom flange width either side of web (See Section 3.3.2.)
b_e = minimum effective shear flow web or flange width to resist torsional stresses, in. (See Sections 12.2.10 and 12.3.1.)
b_m = effective width of flange (See Figs. 3-1 and 3-2.)
b_n = effective flange width for lateral distribution of post-tensioning force (See Fig. 3-4.)
b_{no} = web width at anchorage of post-tensioning force (See Fig. 3-4.)
b_o = web width (See Fig. 24-1.)
b_s = effective top or bottom flange width coefficient at supports and for cantilever arms (See Figs. 3-1 and 3-2.)
b_w = minimum web width, in. (See Section 12.2.5.)
b_w = web width, in. (See Section 11.2.)
b_1 = width of cantilever flange of box girder (See Fig. 3-3.)
b_2 = width of half of interior top flange of box girder (See Fig. 3-3.)
b_3 = width of half of interior bottom flange of box girder (See Fig. 3-3.)
CE = weight of specialized construction equipment
CLE = longitudinal construction equipment load
CLL = construction live load, normally taken as 10 psf
c = portion of continuous span adjacent to interior support subject to shear lag effects (See Fig. 3-1.)
D = sum of dead load of structure (DL), superimposed dead load (SDL), and permanent effects of erection loads (EL)
DIFF = differential (unbalanced) dead load from one cantilever
DL = dead load of structure only
d = distance from the extreme compression fiber to the centroid of the longitudinal tension reinforcement, in. For prestressed concrete members, the greater of the distance from the extreme compression

fiber to the centroid of the prestressed tension reinforcement or 0.8h may be used.

d = shear key dimension (See Fig. 24-1.)

d_c = thickness of cover from tension fiber to center of bar, in. (See Commentary, Section 18.2.)

d_o = total depth of section (See Fig. 24-1.)

d_p = distance from extreme compression fiber to centroid of prestressing reinforcement, in. (See Section 11.2.)

E_{cm} = secant modulus of elasticity, psi (See Commentary, Section 6.5.)

E_{eff} = effective modulus of elasticity for long term loads considering creep deformations, psi (See Commentary, Section 6.5.)

EL = permanent effect of erection loads (final state)

e = eccentricity of reaction (See Fig. 26-2.)

e = base of Naperian logarithms

F_r = radial force due to tendon curvature, kips per ft. (See Section 16.3.)

f'_c = specified compressive strength of concrete at 28 days, psi

$\sqrt{f'_c}$ = square root of specified compressive strength of concrete, psi (See Section 12.2.6 for limit.)

f'_{ci} = compressive strength of concrete at time of initial prestress, psi

f_{cn} = compressive stress in the concrete node regions, psi. (See Section 12.4.6.)

f_{cu} = crushing strength of diagonally cracked concrete, psi. (See Section 12.4.4.)

f_{pc} = compressive stress in concrete after allowance for all prestress losses, psi Critical stress to be determined at:

a) the centroid of the cross section resisting external loads, or

b) the junction of the web and compression flange when the centroid lies within the flange, or

c) in composite members, the stress at (a) or (b) for stresses due to both prestress and the moments resisted by the precast member acting alone.

f_s = stress in nonprestressed reinforcement under erection loads (See Commentary Section 18.2.)

f'_s = ultimate strength of prestressing steel, psi

f_{st} = steel stress at beginning of time intervals t_1, psi

f_{su} = average stress in nonprestressed reinforcement at ultimate load, psi

f^*_{su} = average stress in prestressed reinforcement at ultimate load, psi

f_{sy} = Specified yield strength of nonprestressed reinforcement

f'_y = specified minimum yield strength of compression reinforcement, psi (See Section 11.2.)

f^*_y = yield point stress of prestressing steel, psi

H = horizontal force in bottom flange and web (See Fig. 26-2.)

h = overall thickness of member, in. (See Section 12.2.18.)

h = depth of shear key, in. (See Fig. 24-1.)

IE = impact load from equipment

K = $\sqrt{1+f_{pc}/(2\sqrt{f'_c})}$ (See Section 12.2.10.)

k = friction wobble coefficient, per foot (See Section 10.2.)

ℓ = span length (See Fig. 3-1.)

ℓ = tendon length from jacking end to point x (See Section 10.2.)

ℓ_i = effective span length (See Fig. 3-1.)

ℓ_i = length of tendon between anchorages (See Section 11.2.)

M_u = factored moment at section, in.-lb.

N_s = Number of support hinges crossed by the tendon (draped tendons only) (See Section 11.2.)

N_{uc} = factored compressive axial force normal to cross section, lb. (See Section 12.2.10.)

N_{ut} = factored tensile axial force normal to cross section, lb. (See Section 12.2.10.)

P_{cp} = outside perimeter of the concrete cross section, in. (See Section 12.2.10.)

P_h = perimeter of centerline of outermost continuous closed transverse reinforcement, in.

P_j = tendon jacking force, kips (See Sections 16.3 and 16.6.1.)

R = force in plane of web (See Fig. 26-2.)

R = tendon radius of curvature, ft. (See Section 16.3.)

R = rib shortening and creep effects (See Sections 7.2 and 7.4.1.)

R_{lr} = loss of prestress due to steel relaxation, low relaxation strand, psi

R_{sr} = loss of prestress due to steel relaxation, stress relieved steel, psi

S = shrinkage effects (See Section 7.4.1.)

S_u = force in a truss member due to factored loads, lb.

SDL = superimposed dead load

s = spacing of shear or torsion reinforcement measured parallel to the longitudinal axis of the member, in.

s = bar spacing, in. (See Commentary, Section 18.2.)

T = sum of effects of thermal rise or fall (TRF) and thermal gradient (TG)

TG = thermal differential from top flange to bottom flange, degrees Fahrenheit

TG = temperature gradient, degrees Fahrenheit

TRF = thermal rise or fall, degrees Fahrenheit

T_c = torsional cracking moment, in.-lb. (See Section 12.2.10.)

T_n = nominal torsion resistance, in-lb. (See Section 12.3.7.)

T_o = tendon stress at jacking end. (See Section 10.2.)

T_u = factored torsion at section, in-lb.

T_x = tendon stress at x feet from jacking end (See Section 10.2.)

t = thickness of wall, in. (See Section 23.3.)

t = time at end of time interval, hour (See Section 10.4.)

t_1 = time at anchorage of prestressing steel taken as 1/24 of a day, or any time at the beginning of a time interval, hours (See Section 10.4.)

U = load due to segment unbalance on opposite cantilever ends

V_c = nominal shear strength provided by concrete, lb.

V_n = nominal shear force resisted by member, lb.

V_{nj} = nominal strength of Type B joint (See Section 12.2.21.)

V_p = vertical component of the effective prestressing, lb. (See Sections 12.1.5.1 and 12.1.5.2.)

V_s = nominal shear resisted by the 45° truss model as measured by the stirrup capacity, lb.

V_u = factored shear force at section, lb.

V_{uj} = strength of Type B joint (See Section 12.2.21.)

WUP = wind uplift on cantilever

X_u = the clear length of the constant thickness portion of a wall between other walls or fillers between walls, in. (See Section 23.3.)

Z = correction dimension for location of center of gravity of tendon bundle in duct, in. (See Section 11.5.)

Z = quantity for detailing of reinforcement to control flexural cracking during erection (See Section 18.2 and Commentary Section 18.2.)

ϕ = strength reduction factor (See Section 7.3.)

ϕ_b = strength reduction factor for bearing (See Section 7.3.)

ϕ_c = creep coefficient, ratio of creep strain to elastic strain

ϕ_f = strength reduction factor for flexure (See Section 7.3.)

ϕ_j = strength reduction factor for design of Type B (dry) joints (See Sections 7.3 and 12.2.21.)

ϕ_v = strength reduction factor for shear and diagonal tension (See Section 7.3.)

μ = friction coefficient (per radian)

α = total angular deviation from jacking end to point x, radians

β_1 = factor on concrete strength. β_1 shall be taken as 0.85 for concrete strengths f'_c up to 4000 psi. For strengths above 4000 psi, β_1 shall be reduced continuously at a rate of 0.05 for each 1000 psi of strength in excess of 4000 psi, but β_1 shall not be taken as less than 0.65.

1.3 Definitions

Definitions are in accordance with AASHTO *Standard Specifications for Highway Bridges* and the following:

Anchorage Blister - Build-out in the web, flange, or web-flange junction to provide area for one or more tendon anchorages.

Closure - Cast-in-place concrete segment or segments used to complete a span.

Deviation Saddle - Build-out in the web, flange, or web-flange junction to provide for change of direction of an external tendon.

External Tendon - Tendon located outside the flanges or webs of the structural member, generally inside the box girder cell.

Internal Tendon - Tendon located within the flanges or webs (or both) of the structural member.

General Bursting Forces - Bursting forces due to all of the tendons anchored at a cross section. Dependent on the overall concrete dimensions, and the magnitude direction and location of the total prestressing force anchored.

Low Relaxation Steel - Prestressing strand in which the steel relaxation losses have been substantially reduced by additional manufacturing procedures (stretching at elevated temperature).

Partially Bonded Tendons - Tendons initially external to the concrete cross section which are encased in the cross section over a portion of their length by use of a supplementary concrete pour and reinforcement ties.

Sand-Lightweight Concrete - Concrete mix with lightweight coarse aggregate and a normal weight sand fine aggregate.

2.0 MATERIALS

2.1 Concrete

Structural concrete used in segmental construction shall have a minimum 28-day strength of 4500 psi or greater as specified by the Engineer. The required concrete strength at the time of stressing shall be determined in accordance with Section 9.2.

2.2 Reinforcement

2.2.1 Prestressing Steel

Prestressing steel shall conform to the requirements of AASHTO *Standard Specifications for Highway Bridges* Article 9.3.1.

2.2.2 Reinforcing Steel

Reinforcing steel shall be in accordance with AASHTO *Standard Specifications for Highway Bridges*, Article 8.3, or as otherwise noted in Division II, Sections 3.2.2 and 3.2.3.

In aggressive environments, special design and construction measures shall be taken for corrosion protection of all bridge deck reinforcement, including any reinforcement projecting from the web into the deck.

SECTION B
ANALYSIS

3.0 METHODS OF ANALYSIS

3.1 General

Elastic analysis and beam theory may be used to determine design moments, shears, and deflections. The effects of creep, shrinkage and temperature differentials shall be considered, as well as the effects of shear lag. Shear lag shall be considered in accordance with the provisions of Section 3.3. Analysis for earthquake loads shall be in accordance with Division I-A, Seismic Design, Section 4 of the AASHTO *Standard Specifications for Highway Bridges*.

3.2 Strut-and-Tie Models

Strut-and-tie models may be used for analysis in areas of load or geometrical discontinuity.

3.3 Effective Flange Width

3.3.1 General

Effective flange width for service load stress calculations may be determined by elastic analysis procedures,[5, 6, 52] by the provisions of Section 3.3.1 of the 1991 Ontario Highway Bridge Design Code,[7] or by the provisions of Section 3.3.2.[8, 44]

3.3.2 Effective Flange Width

Section properties for analysis and for calculation of the effects of bending moments and shear forces may be based on the flange widths specified in this section, or may be based on flange widths determined by other procedures listed in Section 3.3.1. The effects of unsymmetrical loadings on effective flange width may be disregarded.

The effective flange width, b_e, (see Figure 3-3) may be assumed equal to the full flange width, b, if:

1. $b \leq 0.1\, \ell_i$ (ℓ_i = effective span length)

2. $b \leq 0.3d_o$ (d_o = web height)

For flange widths, b, greater than $0.3d_o$ or $0.1\, \ell_i$, the effective width may be determined in accordance with Figures 3-1 and 3-2. The value of b_{es}, the effective flange width in the support area, shall be determined using the greater of the effective span lengths adjacent to the support. If b_{em}, the effective flange width in the mid-span area, is less than b_e in a span, the pattern of the effective width within the span may be determined by the connecting line of the effective support widths, b_e at adjoining support points. However, the effective width, b_e, shall not be greater than b.

If the construction procedure is such that the configuration of a span within a unit changes, the final configuration may be used in the determination of the effective flange widths.

The section properties for normal forces may be based on the pattern according to Figure 3-4, or may be determined by more rigorous analytical procedures.

Stresses due to bending, shear, and normal forces may be determined by using their corresponding section properties.

The capacity of a cross section may be determined by considering the full flange width effective.

4.0 TRANSVERSE ANALYSIS

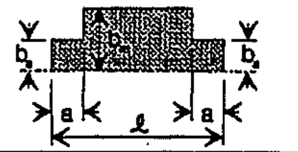
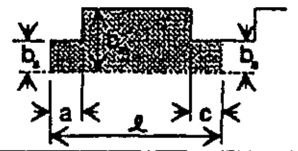
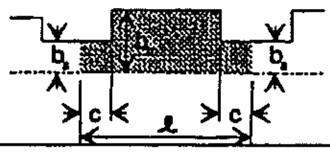
The transverse design of box girder segments for flexure shall consider the segment as a rigid box frame. Flanges shall be analyzed as variable depth sections considering the fillets between the flange and webs. Wheel loads shall be positioned to provide maximum moments, and elastic analysis shall be used to determine the effective longitudinal distribution of wheel loads for each load location. Consideration shall be given to the increase in web shear and other effects on the cross section resulting from eccentric loading or unsymmetrical structure geometry.

Influence surfaces[(11)(12)(13)] or other elastic analysis procedures may be used to evaluate live load plus impact moment effects in the top flange of the box section.

Transverse elastic and creep shortening due to prestressing and shrinkage shall be considered in the transverse analysis.

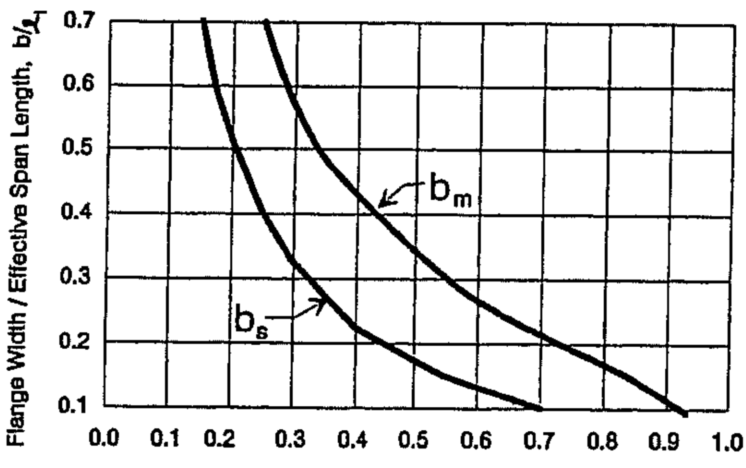
The effect of secondary moments due to prestressing shall be included in stress calculations under service loads. In calculating flexural and shear capacity under factored loads, the secondary moments or shears induced by prestressing (with a load factor of 1.0) shall be added algebraically to the moments and shears due to factored dead and live loads.

The transverse design of beam type segmental bridge decks may be in accordance with the provisions of the AASHTO *Standard Specifications for Highway Bridges*, Section 3.24.3.1.

System	Pattern of Effective Flange Widths	Effective Span Length for use with Figure 4-2
Single-Span Girder		$\ell_i = \ell$
End Span of Continuous Girder		$\ell_i = 0.8\,\ell$
Inner Span of Continuous Girder		$\ell_i = 0.6\,\ell$
Cantilever Arm		$\ell_i = 1.5\,\ell$

a = largest b, but not exceeding 0.25ℓ
c = $0.1\,\ell$

Fig. 3-1



For $b/\ell_i \geq 0.7$:
$b_m = 0.173\,\ell_i$
$b_s = 0.104\,\ell_i$

Where b_m = effective flange width near mid-span
b_s = effective flange width near support

Fig. 3-2

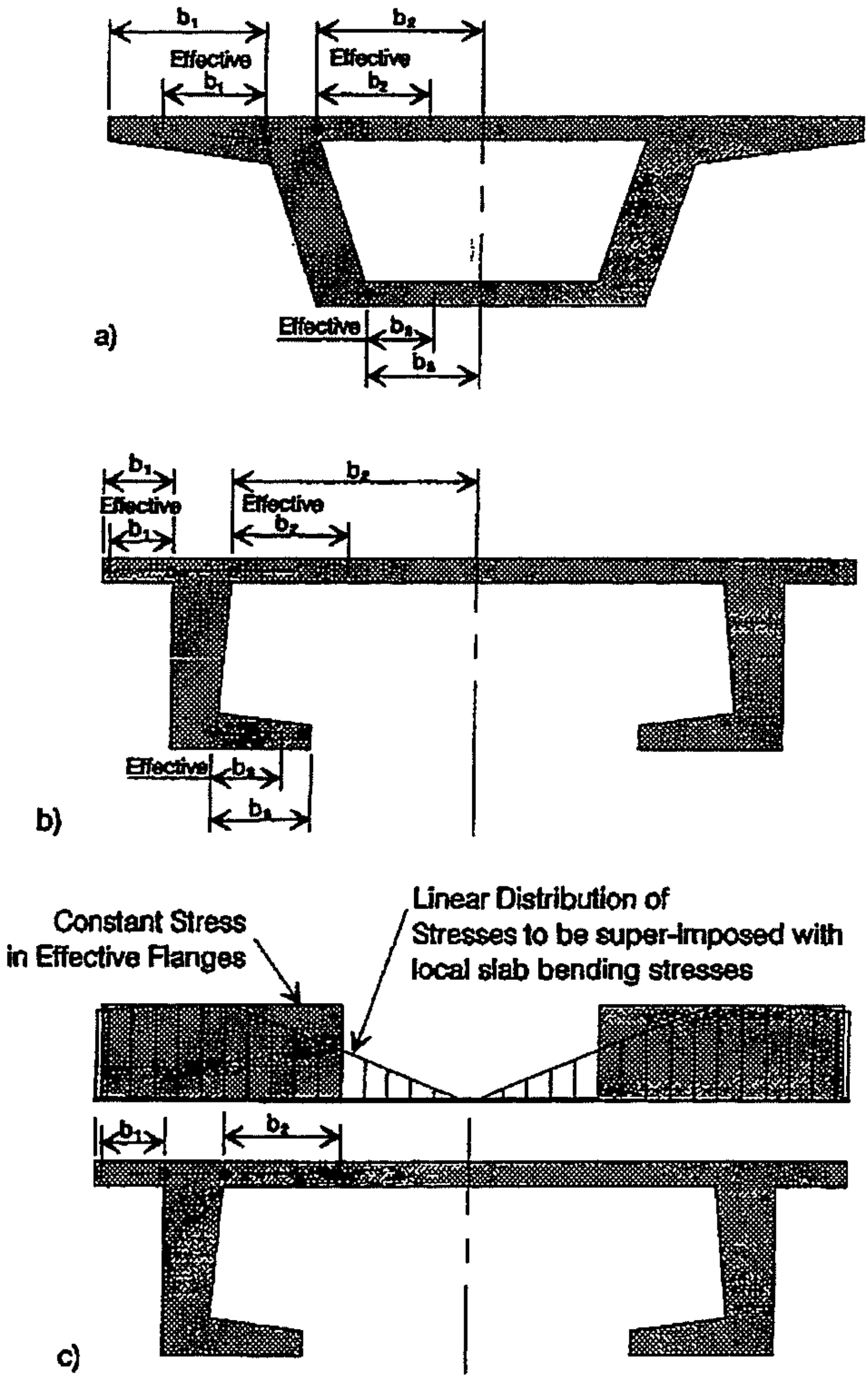


Fig. 3-3. Cross Sections and Corresponding Effective Flange Widths for Bending and Shear

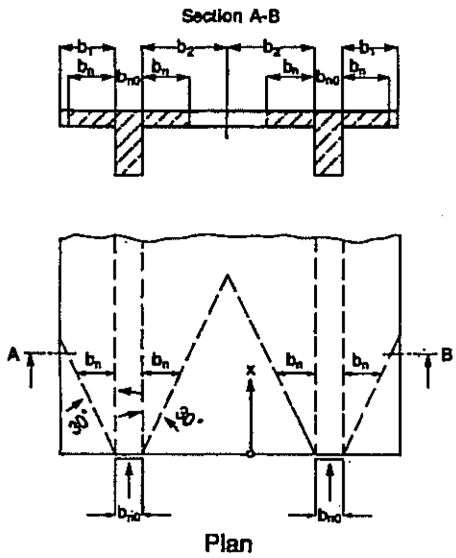


Fig. 3-4. Effective Flange Widths b_n for Normal Forces

5.0 LONGITUDINAL ANALYSIS

5.1 General

Longitudinal analysis shall be in accordance with the provisions of Section 3.0. Longitudinal analysis of segmental concrete bridges shall consider a specific construction method and construction schedule, as well as the time-related effects of concrete creep, shrinkage, and prestress losses.

The effect of secondary moments due to prestressing shall be included in stress calculations at service limit states. In calculating flexural and shear capacity requirements under factored loads, the secondary moments or shears induced by prestressing (with a load factor of 1.0) shall be added algebraically to the moments and shears due to factored dead and live loads.

5.2 Erection Analysis

Analysis of the structure during the construction stage shall consider the construction load combinations, stresses, and stability considerations outlined in Section 7.4.

5.3 Analysis of the Final Structural System

The final structural system shall be analyzed for redistribution of erection stage moments resulting from the effects of creep and shrinkage, and from any change in the statical system, including the closure of joints. Thermal effects on the final structural system shall be considered in accordance with the AASHTO *Standard Specifications for Highway Bridges*, Section 3.16, and Section 6.4 of these guide specifications. The effect of prestress losses occurring after closure shall be evaluated in accordance with Section 10.0. Multiple presence of live load shall be evaluated in accordance with the provisions of AASHTO *Standard Specifications for Highway Bridges*, Section 3.12. The maximum moments resulting from the above analyses shall be utilized for stress calculations under service loads, and shall be utilized in conjunction with the combinations of loads specified in the AASHTO *Standard Specifications for Highway Bridges*, Section 3.22, for determination of required flexural strength.

SECTION C
DESIGN

6.0 LOADS

6.1 General

All loadings shall be in accordance with the AASHTO *Standard Specifications for Highway Bridges* except as provided below.

6.2 Dead Loads

Unit weight of concrete (including rebar)—155 pounds per cubic foot or as determined for the project. Weight of diaphragms, anchor blocks, or any other deviations from the typical cross section shall be included in the dead load calculations.

6.3 Erection Loads

6.3.1 Erection loads comprise all loadings arising from the designer's anticipated system of temporary supporting works and/or special erection equipment to be used in accordance with the assumed construction sequence and schedule. The assumed erection loads and acceptable closure forces due to misalignment corrections shall be stated on the drawings. Due allowance shall be made for all effects of any changes of the statical structural scheme during construction and the application, changes or removal of the assumed temporary supports of special equipment taking into account residual "built-in" forces, moments, deformations, secondary post-tensioning effects, creep, shrinkage and any other strain induced effects.

6.3.2 All elements of the bridge shall be designed for the anticipated construction system assumed by the Engineer and shown on the plans. Any changes by the Contractor in the construction method or in the design shall comply with Sections 28.2 or 28.3.

6.4 Thermal Effects

6.4.1 Normal Mean Temperature

Unless more precise local data are available, normal mean temperature for the location shall be taken as the average of the January and July values from Figures 6-1 and 6-2,[20] respectively.

6.4.2 Seasonal Variation

a. For the purposes of design of the structure, the minimum and maximum overall temperatures for the location shall be taken from Figures 6-1 and 6-2, respectively, unless more precise local data are available.

b. The temperature setting variations for bearings and expansion joints shall be stated on the bridge plans.

6.4.3 Thermal Coefficient

The coefficient of thermal expansion used to determine temperature effects shall be taken as 6.0×10^{-6} per degree Fahrenheit for normal weight concrete and 5.0×10^{-6} for sand-lightweight concrete unless more precise data are available.

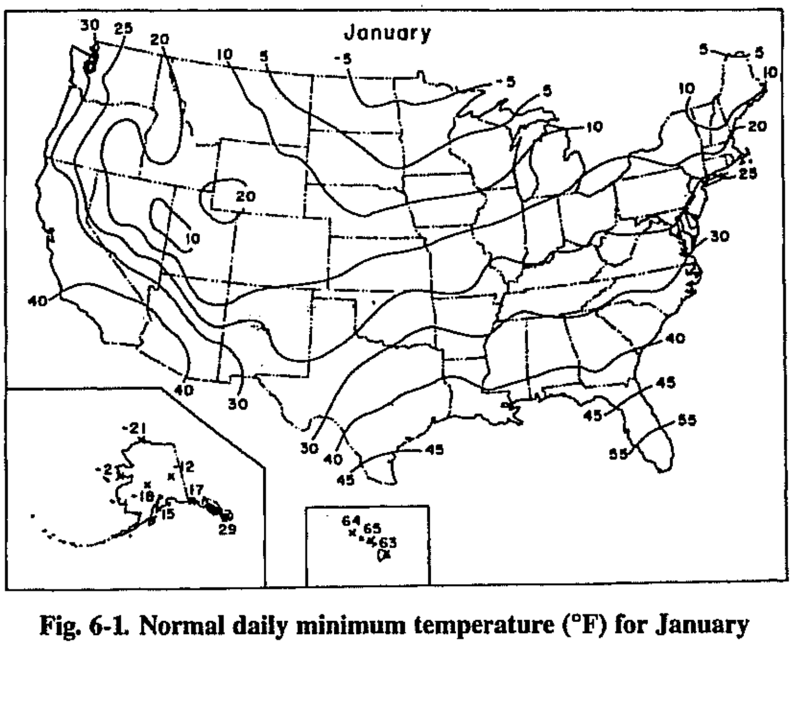


Fig. 6-1. Normal daily minimum temperature (°F) for January

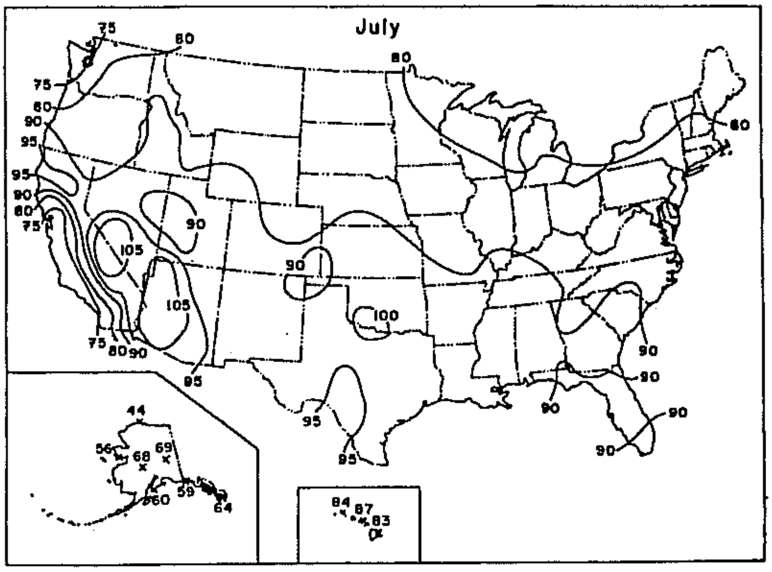


Fig. 6-2. Normal daily maximum temperature (°F) for July

6.4.4 Temperature Gradient

For the purpose of this article, the country shall be subdivided into zones as indicated in Figure 6-3. Positive temperature gradient (TG) values for the zones shall be taken as specified for various deck surface conditions in Table 6-1. Negative temperature values shall be obtained by multiplying the values specified in Table 6-1 by -0.30 for plain concrete surfaces and -0.20 for surfaces with 2-inch asphalt topping.

The vertical temperature gradient shall be taken as shown in Figure 6-4.

The dimension "A" in Figure 6-4 shall be taken as:

- 12.0 inches for superstructures that are 16.0 inches or more in depth
- for sections shallower than 16.0 inches, "A" shall be 4.0 inches less than the actual depth

Temperature value T_3 shall be taken as 0.0, unless a site-specific study is made to determine an appropriate value, but shall not exceed 5°F.

Where temperature gradient is considered, internal stresses and structure deformations due to both positive and negative temperature gradients may be determined in accordance with the provisions of the AASHTO Guide Specifications—*Thermal Effects in Concrete Bridge Superstructures*, Appendix A, 1989.[20]

Table 6-1. Basis for Temperature Gradients

Plain Concrete Surface or Asphalt Topping		
Zone	T1(F°)	T2(F°)
1	54	14
2	46	12
3	41	11
4	38	9

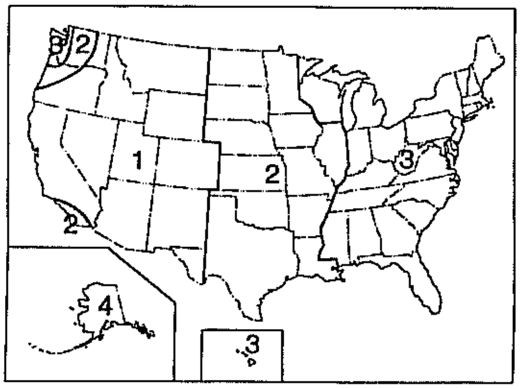


Fig. 6-3. Solar Radiation Zones for the United States

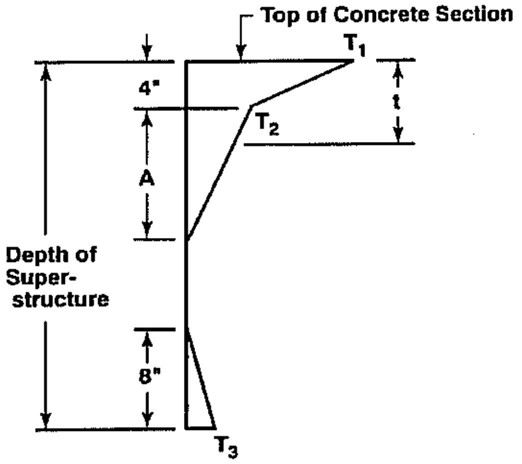


Figure 6-4. Positive Vertical Temperature Gradient

6.5 Creep and Shrinkage

Effects due to creep and shrinkage strains shall be calculated in accordance with the provisions of Section 5.3. The creep coefficient ϕ_c may be evaluated in accordance with the provisions of the ACI Committee 209 Report,[17] the CEB-FIP Model Code,[18] or by a comprehensive test program. Creep strains and prestress losses which occur after closure of the structure cause a redistribution of the forces. Stresses shall be calculated for this effect based on an assumed construction schedule stated on the plans.

6.6 Post-Tensioning Force

The structure shall be designed for both initial and final post-tensioning forces. For determining the final post-tensioning forces, prestress losses shall be calculated for the construction schedule stated on the plans. The final post-tensioning forces used in services load stress calculations shall be based on the most severe condition at each location along the structure.

7.0 LOAD FACTORS

7.1 General

In the final working condition, service or strength load combinations shall be in accordance with AASHTO *Standard Specifications for Highway Bridges*, Section 3.2.2, with consideration of the additional load combinations of Section 7.2. Strength reduction factors, ϕ shall be in accordance with Section 7.3.

During construction, load combinations, allowable stresses, and stability shall be in accordance with Section 7.4. Prior to grouting, the allowable concrete tensile stress during construction shall be zero for structures utilizing internal grouted tendons.

7.2 Additional Load Combinations

The permanent effects of creep and shrinkage shall be added to all AASHTO loading combinations with a load factor of 1.0.

7.2.1 Erection Loads at End of Construction

The final state erection loads (EL) are defined as the final accumulated "built-in" forces and moments resulting from the construction process.

7.2.2 Additional Thermal Loading Combination

7.2.2.1 For existing service load combinations which include full live load plus impact, a load factor of 0.50 shall be applied for temperature gradient, when temperature (T) is included.

7.2.2.2 In addition to AASHTO service load combinations, the following load combination shall apply:

$$DL+SDL+EL+\beta_E E+B+SF+R+S+TG$$

$$\leq 100\% \text{ Allowable Stress}$$

LOADING	
Dead Load Structure Only:	DL
Superimposed Dead Load:	SDL
Erection Loads (final state):	EL
Thermal-Rise or Fall	TRF
Thermal-Differential	TG
Creep Effects	R

See AASHTO *Standard Specifications for Highway Bridges*, Section 3.22.1, for other definitions.

AASHTO – D = (DL+SDL+EL)
AASHTO – T = (TRF+TG)

7.2.2.3 For all factored load combinations, a load factor of zero (0) shall be applied to differential temperature effects (TG).

7.3 Strength Reduction Factors

7.3.1 The strength reduction factors, ϕ_f and ϕ_v, for flexure and shear, respectively, shall consider both the type of joint between segments and the degree of bonding of the post-tensioning system provided. The appropriate value of Φ_v from Section 7.3.6 shall be used for shear and torsional effect calculations in Section 12.

7.3.2 Since the post-tensioning provided may be a mixture of fully bonded tendons and unbonded or partially bonded tendons, the strength reduction factor at any section shall be based upon the bonding conditions for the tendons providing the majority of the prestressing force at the section.

	ϕ_f Flexure	ϕ_v Shear	ϕ_j Joint
Normal Weight Concrete			
Fully Bonded Tendons Type A Joint	0.95	0.90	-
Unbonded or Partially Bonded Tendons			
Type A Joint	0.90	0.85	-
Type B Joint	0.85	0.85	0.75
Sand-Light Weight Concrete			
Fully Bonded Tendons Type A Joint	0.90	0.70	-
Unbonded or Partially Bonded Tendons			
Type A Joint	0.85	0.65	-
Type B Joint	0.80	0.65	0.60

7.3.3 In order for a tendon to be considered as fully bonded to the cross section, it must be bonded beyond the critical section for a development length not less than that required by AASHTO *Standard Specifications for Highway Bridges*, Section 9.28.1. Sorter embedment lengths are permissible if demonstrated by full size tests and approved by the Engineer.

7.3.4 Cast-in-place concrete joints, and wet concrete or epoxy joints between precast units, shall be considered as Type A joints.

All new structures shall have only Type A joints.

7.3.5 Dry joints between precast units shall be considered as Type B joint.

Reference to Type B joints is retained for the purpose of load rating existing bridges.

7.3.6 Strength reduction factors, ϕ, shall be taken as presented in Table 7-1, and the following provisions of this section.

The strength reduction factor for direct shear capacity of dry joints, ϕ_j, shall be used in conjunction with Section 12.2.21.

The strength reduction factor for bearing, ϕ_b, shall be taken as 0.70 for all types of construction. This value shall not be applied to bearing stresses under anchorage plates for post-tensioning tendons.

7.4 Construction Load Combinations, Stresses and Stability

7.4.1 Erection Loads During Construction

Erection loads as defined by AASHTO and as stated on the plans shall be as follows:

a. Dead load of structure (DL): Unit weight of concrete (including rebar) – 155 pcf or as determined for the project. Weight of diaphragms, anchor blocks, or any other deviations from the typical cross section shall be included in the dead load calculations.

b. Differential load from one cantilever (DIFF): This only applies to balanced cantilever construction. The load is 2% of the dead load applied to one cantilever.

c. Superimposed dead load (SDL): This does not normally apply during construction. If it does, it should be considered as part of the dead load (DL).

d. Distributed construction live load (CLL): This is an allowance for miscellaneous items of plant, machinery and other equipment apart from the major specialized erection equipment. Distributed load allowance is 10 psf. In cantilever construction, distributed load shall be taken as 10 psf on one cantilever and 5 psf on the other. For bridges built by incremental launching, construction live load may be taken as zero.

e. Specialized construction equipment (CE): This is the load from any special equipment such as a launching gantry, beam and winch, truss or similar major item. This also includes segment delivery trucks and the maximum loads applied to the structure by the equipment during the lifting of segments.

f. Impact load from equipment (IE): To be determined according to the type of machinery anticipated. For very gradual lifting of segments, where the load involves small dynamic effects, the impact load may be taken as 10 percent.

g. Longitudinal construction equipment load (CLE): The longitudinal force from the construction equipment.

h. Segment unbalance (U): This applies primarily to balanced cantilever construction but can be extended to include any "unusual" lifting sequence which may not be a primary feature of the generic construction system.

Table 7-2
ALLOWABLE TENSILE STRESSES FOR CONSTRUCTION LOAD COMBINATIONS

Load Combinations Coefficients

Superstructure and Substructure

Combination	Dead Loads DL	Dead Loads DIFF	U	Live Loads CLL	Live Loads CE	Live Loads IE	Live Loads CLE	Wind Loads W	Wind Loads WUP	Wind Loads WE	(1) **** Allowable Stress	(R + S + T)	(2)**** Superstructure Only Including (R + S + T) Allowable Stress	β_E***	B	SF	Segmental Substructure Only (3)**** Excluding (R + S + T) Allowable Stress	Segmental Substructure Only (4)**** Including (R + S + T) Allowable Stress	Comments
a	1	1	0	1	1	1	0	0	0	0	$6\sqrt{f'_c}$	1	$7\sqrt{f'_c}$	β_E	1	1	$6\sqrt{f'_c}$	$7\sqrt{f'_c}$	
b	1	0	1	1	1	1	0	0	0	0	$6\sqrt{f'_c}$	1	$7\sqrt{f'_c}$	β_E	1	1	$6\sqrt{f'_c}$	$7\sqrt{f'_c}$	
c	1	1	0	0	0	0	0	0.7*	0.7	0	$6\sqrt{f'_c}$	1	$7\sqrt{f'_c}$	1	1	1	$7\sqrt{f'_c}$	$7\sqrt{f'_c}$	
d	1	1	0	1	1	0	0	0.7*	1	0.7	$7\sqrt{f'_c}$	1	$7\sqrt{f'_c}$	β_E	1	1	$7\sqrt{f'_c}$	$7\sqrt{f'_c}$	Equipment not working
e	1	0	1	1	1	1	0	0.3**	0	0.3	$7\sqrt{f'_c}$	1	$7\sqrt{f'_c}$	β_E	1	1	$7\sqrt{f'_c}$	$7\sqrt{f'_c}$	Normal Erection
f	1	0	0	1	1	1	1	0.3**	0	0.3	$7\sqrt{f'_c}$	1	$7\sqrt{f'_c}$	β_E	1	1	$7\sqrt{f'_c}$	$7\sqrt{f'_c}$	Moving Equipment

The allowable stresses in Columns 1 and 2 apply to the summation of all the loads multiplied by their tabulated coefficients in all the columns to the left. Similarly for Columns 3 and 4 with the exceptions of (R + S + T) as noted.

* Reduction is to allow for lesser probability of maximum wind during construction period.

** Reduction is to allow for limiting wind beyond which construction is halted.

*** The β_E term is as defined in AASHTO Section 3.22.

**** When less than 60% of the tendon capacity is provided by internal tendons, the maximum allowable construction stresses shall be $3\sqrt{f'_c}$ for Type A joints, and 0 for Type B joints.

The load "U" is the effect of any out of balance segments or other unusual condition as applicable.

i. Wind uplift on cantilever (WUP): 5 psf (balanced cantilever construction applied to one side only).

j. Accidental release or application of a precast segment load or other sudden impact from an otherwise static load of A: Impact force = 2A.

k. Creep (R)

In accordance with Section 6.5. Creep effects shall be considered as part of rib shortening (R).

l. Shrinkage (S)

In accordance with Section 6.5.

m. Thermal (TRF) and Thermal (TG)

The sum of the effects due to thermal rise and fall (TRF) and differential temperatures (TG) from Section 6.4.

7.4.2 Construction Load Combinations and Allowable Stresses

Stresses shall be checked under the construction load combinations given in Table 7-2. The distribution and application of the individual erection loads (Section 7.4.1) appropriate to a construction phase shall be such as to produce the most unfavorable effects. Table 7-2 is a guide; if more unfavorable conditions may occur with the particular construction system, these shall be taken into account. The maximum allowable construction load compressive stress shall be $0.5f'_c$.

Load factor design need not be used for construction conditions with the exception of Section 7.4.3 below.

7.4.3 Construction Load Combinations Load Factor Design Check

Using capacity reduction factors (ϕ) in accordance with Section 7.3, the strength provided shall not be less than required by the following load factor combinations.

For maximum forces and moments:

$$1.1(DL + DIFF) + 1.3CE + 2A$$

For minimum forces and moments:

$$DL + CE + 2A$$

8.0 SEISMIC DESIGN

Segmental superstructure design with moment resisting column to superstructure connections shall consider the inelastic hinging force (lateral shear force) from columns. The lateral shear force from the column shall be determined from the inelastic flexural capacity of the column by multiplying the nominal flexural capacity by 1.30. Bridge superstructures in seismic zones C and D with moment resisting column to super structure connections shall be reinforced with ductile details to resist longitudinal and transverse flexural demands produced by column plastic hinging.

Type A joints shall be used in seismic zones C and D. Segment joints shall provide capacity to transfer seismic demands.

Internal tendons shall be provided to support the superstructure dead load with a factor of safety of 1.3 in seismic zones C and D. The average stress in the prestressing steel of the internal tendons for this load case shall be calculated in accordance with Section 11.2. Not more than 50 percent of the total post-tensioning steel force shall be provided by external tendons.

9.0 ALLOWABLE STRESSES

9.1 Prestressing Steel

Tensile stresses in prestressing tendons shall not exceed the following.

9.1.1 Due to tendon jacking force

$$0.80f'_s$$

but not greater than $0.90\ f^*_y$

9.1.2 At anchorages and couplers immediately after tendon anchorage

$$0.70f'_s$$

9.1.3 At internal tendon locations immediately after prestress transfer

$$0.83f^*_y$$

but not greater than $0.74f'_s$

9.2 Prestressed Concrete

9.2.1 Temporary stresses before losses due to creep and shrinkage, at the time of application of the prestress (transfer) for both normal weight and sand-light weight concrete:

9.2.1.1 Compression

$$0.60f'_{ci}$$

9.2.1.2 Longitudinal stresses in the precompressed tensile zone:

a. Type A joints with minimum bonded auxiliary reinforcement through the joints sufficient to carry the calculated tensile force at a stress of $0.5\ f_y$; internal and external tendons.

$3.0\sqrt{f'_{ci}}$ maximum tension

b. Type A joints without the minimum bonded auxiliary reinforcement through the joints; internal tendons: no tension allowed.

c. Type B joints external tendons, not less than:

100 psi minimum compression

9.2.1.3 Transverse tension in the precompressed tension zone: $3.0\sqrt{f'_{ci}}$ maximum tension.

9.2.1.4 Tension in other areas without bonded non-prestressed reinforcement: zero tension.

Where the calculated tensile stress exceeds this value, bonded reinforcement shall be provided at a stress of $0.5f_{sy}$ to resist the total tensile force in the concrete computed on the assumption of an uncracked section. In such cases the maximum tensile stress shall not exceed $6.0\sqrt{f'_{ci}}$.

9.2.2 Stresses at the service load level after losses have occurred:

9.2.2.1 Compression:

a. The compressive stress under all service load combinations, except as stated in (b), shall not exceed $0.60f'_c$. When the flange or web slenderness ratio, calculated in accordance with Section 23.3, is greater than 15, the compressive stress shall be reduced by a factor calculated by the equations presented in Section 23.4.3.

b. The compressive stress due to effective prestress plus permanent (dead) loads shall not exceed $0.45f'_c$.

9.2.2.2 Longitudinal stresses in the precompressed tensile zone:

a. Type A joints with minimum bonded auxiliary reinforcement through the joints sufficient to carry the calculated tensile force at a stress of $0.5f_{sy}$; internal tendons:

$3.0\sqrt{f'_c}$ maximum tension

b. Type A joints without minimum bonded auxiliary reinforcement through joints:

Superstructures: zero tension.
Substructures: 200 psi tension plus overstress provision, as specified in Table 3.22.1A (computed on the basis of the gross section)

c. Type B joints, external tendons, not less than:

100 psi minimum compression

9.2.2.3 Transverse tension in the precompressed tensile zone:

$3.0\sqrt{f'_c}$ maximum tension

9.2.2.4 Tension in other areas without bonded reinforcement:

Superstructures: zero tension
Substructures: 200 psi tension plus overstress provision, as specified in Table 3.22.1A (computed on the basis of the gross section)

Where the calculated tensile stress exceeds this value, bonded reinforcement shall be provided at a stress of $0.5f_{sy}$ to resist the total tensile force in the concrete computed on the assumption of an uncracked section. In such cases, the maximum tensile stress shall not exceed $6.0\sqrt{f'_c}$.

10.0 PRESTRESS LOSSES

10.1 General

Lump sum losses shall only be used for preliminary design purposes. Losses due to creep, shrinkage, and elastic shortening of the concrete as well as friction, wobble, anchor set and relaxation in the tendon shall be calculated for the construction method and schedule shown on the plans in accordance with time-related procedures for calculation of prestress losses and in accordance with the following sections.

10.2 Duct Friction and Wobble

The loss of prestress force due to friction and wobble within an internal tendon duct shall beobtained using the equation:

$$T_x = T_o e^{-(\mu\alpha + k\ell)}$$

For tendons in webs of curved bridges, or in inclined webs of straight bridges, α shall be calculated as the total vector accumulation of the horizontal and vertical angle changes, and ℓ shall be the total tendon length.

The loss of prestress force in an external tendon due to friction across a single deviator pipe shall be obtained using the equation:

$$T_x = T_o e^{-\mu(\alpha + 0.04)}$$

Friction and wobble coefficients may be estimated using the values in Table 10-2. However, these values do not consider misalignment of internal ducts at joints. The inadvertent angle change of 0.04 radians per deviator may vary depending on job specific tolerances on deviator pipe placement, and need not be applied in cases where the deviation angle is strictly controlled or precisely known, as in the case of continuous ducts passing through separate longitudinal bell-shaped holes at deviators. Where large discrepancies occur between measured and calculated tendon elongations, in place friction tests are required.

Table 10-2. Estimated Values for Friction and Wobble Coefficients

	Friction Coefficient (μ) (1/rad)	Wobble Coefficient (k) (1/ft)
1. For strand in galvanized metal sheathing	0.12 - 0.25	0.0002
2. For deformed high strength bars in galvanized metal sheathing	0.30	0.0002**
3. For strand in internal polyethylene duct.	0.23	.0002
4. For strand in straight polyethylene duct (external to concrete)	0	0
5. Rigid steel pipe deviators for external tendons	0.25*	0.0002
6. Continuous external tendon polyethylene ducts through a deviator	0.15	0

* Lubrication will probably be required
** Wobble coefficient for duct installed with preplaced bars may be taken as 0.0001.

The inadvertent angle change need not be considered for calculation of losses due to wedge seating movement.

10.3 Anchorage Seating

For strand tendons anchored with two or three piece wedges, anchorage seating may be approximated as ¼ inch. Anchor seating for bar tendons may be approximated as 1/16 inch. The value of anchorage seating used in the design shall be stated on the design drawings with the provision that it shall be verified during construction.

10.4 Steel Relaxation

Loss of prestress due to steel relaxation over the time interval t_1 to t may be estimated in accordance with the following sections.

10.4.1 Stress-relieved Steel

$$R_{st} = f_{st}\{[\log 24t - \log 24t_1]/10\} \times [f_{st}/f_y^* - 0.55]$$

where: $f_{st}/f_y^* - 0.55 \geq 0.05$

$f_y^* = 0.85f_{su}^*$

f_{st} = steel stress level at beginning of time interval (t_1)

10.4.2 Low Relaxation Steel

$$R_{lr} = f_{st}\{[\log 24t - \log 24t_1]/45\} \times [f_{st}/f_y^* - 0.55]$$

where: $f_{st}/f_y^* - 0.55 \geq 0.05$

$f_y^* = 0.90f_{su}^*$

f_{st} = steel stress level at beginning of time interval (t_1)

10.4.3 Bars

Steel relaxation loss for bar tendons conforming to ASTM A722 may be estimated as 3000 psi.

10.5 Stressing Hardware

A loss in tendon force occurs through the stressing hardware and anchorage device. This loss shall be considered in design.

11.0 FLEXURAL STRENGTH

11.1 General

Flexural strength shall be calculated in accordance with the provisions of Sections 9.17, 9.18, and 9.19 of the AASHTO *Standard Specifications for Highway Bridges*, and/or as provided in this Section.

The flexural capacity required by the load factor provisions of Section 7.1 shall be less than or equal to Φf times the flexural capacity calculated in accordance with AASHTO *Standard Specifications for Highway Bridge*, Sections 9.17, 9.18, and 9.19, except that the average stress in unbonded prestressing steel shall be determined in accordance with Sections 11.2 or 11.3. The values of Φf shall be taken from Section 7.3.

11.2 Stress in Unbonded Prestressing Steel at Factored Load

For calculation of average stress in unbonded prestressing steel at factored loads, equation 9-18 of the AASHTO *Standard Specifications for Highway Bridges* shall be replaced by the following formula:

$$f_{ps}^* = f_{pe} + 900\left(\frac{d_p - c_y}{\ell_e}\right)$$

where:

$$\ell_e = \left(\frac{\ell_i}{1 + \frac{N_s}{2}} \right)$$

d_p = distance from extreme compression fiber to centroid of prestressing reinforcement.

f_{pe} = effective stress in prestressing steel after losses (ksi)

ℓ_i = length of tendon between anchorages

N_s = number of support hinges crossed by the tendon (draped tendons only)

c_y = neutral axis depth calculated as follows:

for T-section behavior:

$$c_y = \frac{A_s^* f_{su}^* + A_s f_{sy} - A_s' f_y' - 0.85\beta_1 f_c'(b-b_w)h_f}{0.85 f_c' \beta_1 b_w}$$

for rectangular section behavior:

$$c_y = \frac{A_s^* f_{su}^* + A_s f_{sy} - A_s' f_y'}{0.85 f_c' \beta_1 b}$$

11.3 Strain Compatibility

As an alternative to use of AASHTO *Standard Specifications for Highway Bridges,* Sections 9.17, 9.18, and 9.19, flexural strength of bonded tendon bridges may be calculated in accordance with the strain compatibility provisions of Section 10.2 of the ACI 318-95 Building Code.(28) Strain compatibility analysis may also be used for computation of flexural resistance of bridges with unbonded or partially bonded tendons provided that the analysis correctly recognizes the differences in strain between the tendons and the concrete section, and the effect of deflection geometry changes on the effective stress in the tendons.

11.4 Minimum Reinforcement

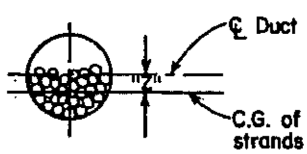
11.4.1 The minimum amount of flexural reinforcement shall comply with Section 9.18.2.1 of AASHTO *Standard Specifications for Highway Bridges.*

11.4.2 The requirements of Section 11.4.1 may be waived if the area of reinforcement at a section is at least one-third greater than that required by analysis based on the loading combinations specified in Article 3.22 of AASHTO *Standard Specifications for Highway Bridges.*

11.5 Center for Gravity Correction for Strand Tendons.

Draped strand tendons shall be assumed to be at the bottom of the duct in negative moment areas, and at the top of the duct in positive moment areas. For both strength and allowable stress calculations, the location of the tendon center of gravity with respect to the center line of the duct shall be assumed as illustrated below (negative moment area shown).

Duct Size	Z
3" OD and Less	½
Over 3" OD to 4"	¾
Over 4" OD	1"



12.0 SHEAR AND TORSION

12.1 Scope

The provisions of Section 12 shall apply to the design of prestressed concrete segmental bridges subjected to shear or combined shear and torsion. The design shall be based on factored load conditions. The provisions of Section 12.2 shall apply to all parts of Section 12.

12.1.1 Regions with one-way beam or thin plate type action or similar conditions in which the plane sections assumption of flexural theory can be applied shall be designed for shear or shear and torsion according to Section 12.1, and either the modified traditional approach of Section 12.3 or the strut-and-tie model provisions of Section 12.4. Detailing of all shear and torsion reinforcement must meet the requirements of Section 12.2. For shear design of structures using sand-lightweight concrete, $0.85\sqrt{f_c'}$ shall be substituted for $\sqrt{f_c'}$ in Sections 12.2.12 and 12.2.21.

12.1.2 Discontinuity regions where the plane sections' assumption of flexural theory is not applicable such as regions adjacent to abrupt changes in cross sections, openings, dapped ends, regions where large concentrated loads, reactions, or post-tensioning forces are applied or deviated, diaphragms, deep beams, corbels or joints shall be designed for the applied forces causing shear or shear and torsion according to Section 12.2 and the strut-and-tie model approach of Section 12.4. In addition, special discontinuity regions like deep beams, brackets and corbels should be designed for the applicable parts of Section 12.5.

12.1.3 Interfaces between elements such as webs and flanges, between dissimilar materials, between concretes cast at different times, or at an existing or potential major crack shall be designed for shear transfer in accordance with Section 12.6.

12.1.4 Slab type regions subjected to local concentrated forces such as concentrated loads or column reactions shall be designed for two-way punching shear in accordance with Section 12.7.

12.1.5 The applied shear on a cross section shall consist of the shear due to factored ultimate dead load ($V_{u\ DL}$) including continuity effects, factored ultimate live load ($V_{u\ LL}$) and any other factored ultimate load cases specified. Torsional moments (T_u) shall be included in design for factored ultimate load when their magnitude exceeds the value specified in Section 12.2.10.

12.1.5.1 The applied shear due to the component of the effective longitudinal prestress force which acts in the direction of the section being examined (V_P) shall be considered as a load effect.

12.1.5.2 The vertical component of inclined tendons (V_p) shall only be considered to reduce the applied shear on the webs for tendons which cross the webs and are anchored or fully developed by anchorages, deviators, or internal ducts located in the outer one-third of the webs.

12.2 General Requirements

12.2.1 For members subjected to combined shear and torsion, the resulting shear forces in the different elements of the structure from the combined shear flows from shear and from torsion shall be considered. The individual elements shall be designed for the resultant shear forces.

12.2.2 The effects of axial tension due to creep, shrinkage and thermal effects in restrained members shall be considered wherever applicable.

12.2.3 The component of the effective prestressing force in the direction of the shear force shall be considered in accordance with Section 12.1.5.

12.2.4 Components of inclined flexural compression or tension in variable depth members shall be considered.

12.2.5 The effects of any opening or ducts in members shall be considered. In determining the effective web width, b_w or b_e, the diameters of ungrouted ducts or one-half the diameters of grouted ducts shall be subtracted from the web width at the level of these ducts.

12.2.6 The values of $\sqrt{f'_c}$ used in any part of Section 12 shall not exceed 100 psi.

12.2.7 The design yield strength of non-prestressed transverse shear or torsion reinforcement shall not exceed 60 ksi. The shear and torsion resistance contribution of prestressed transverse shear or torsion reinforcement shall be based on substitution of the effective stress after allowance for all prestress losses plus 60 ksi, but not to exceed f_y^* in place of f_{sy} in transverse reinforcement expressions.

12.2.8 In pretensioned elements, the reduced prestress in the transfer length of the prestressing tendons shall be considered when computing f_{pc} and V_p. The prestress force due to a given tendon shall be assumed to vary linearly from zero at the point at which bonding commences to a maximum at a transfer length which may be assumed as 50 diameters for 1/2 inch diameter strand.

12.2.9 Shear effects may be neglected in areas of members where the factored shear force V_u is less than $\phi V_c/2$ (V_c is defined in Section 12.2.12). Nominal minimum stirrup capacity of not less than the equivalent of two #4 Grade 60 bars at 1 ft. on centers shall be provided per web in such areas.

12.2.10 Torsional effects may be neglected in members where the factored torsional moment T_u is less than $\phi_v T_c/3$.

In lieu of a more detailed calculation, T_c may be taken as

$$T_c = 2K\sqrt{f'_c}\,(2A_o b_e)$$

K shall be computed as

$$\sqrt{1+f_{pc}\,/\,(2\sqrt{f'_c})} \text{ but } K \leq 2.0$$

However, K shall not exceed 1.0 at any section where the stress in the extreme tension fiber due to factored load and effective prestress force exceeds $6\sqrt{f'_c}$ in tension. The influence of axial tension, N_{ut}, shall be accounted for by replacing f_{pc} by $f_{pc}\ N_{ut}/A_g$. The influence of axial compression, N_{uc}, shall be accounted for by replacing f_{pc} by $f_{pc} + N_{uc}/A_g$. A_o is the area enclosed by the shear flow path defined by the centroids of the longitudinal chords of the space truss model resisting the applied torsion. In lieu of a more precise analysis, A_o may be taken as 85% of the area enclosed by the centerline of the exterior closed transverse torsion reinforcement. The term, b_e, is the effective width of the shear flow path of the elements making up the space truss model resisting torsion. In box girders, b_e may not exceed the minimum effective width, b_w, of the thinnest web or flange comprising the closed box section. In lieu of a more precise analysis, b_e may be taken as A_{cp}/p_{cp}, where A_{cp} is the area enclosed by the outside perimeter of the concrete cross section and P_{cp} is the outside perimeter of the concrete cross section. The effects of openings and ducts must be considered as required in Section 12.2.5.

12.2.11 In a statically indeterminate structure where significant reduction of torsional moment in a member can occur due to redistribution of internal forces upon cracking, the factored torsion moment T_u may be reduced to $\phi_v T_c$ (T_c is defined in Section 12.2.10), provided that moments and forces in the member and in adjoining members are adjusted to account for the redistribution. In lieu of a more exact analysis, the torsional loading from a slab may be assumed as linearly distributed along the member.

12.2.12 Transverse reinforcement shall be provided in all elements except for slabs and footings, and elements where V_u is less than 0.5 $\phi_v V_c$. In lieu of more detailed calculations, V_c may be taken as:

$$V_c = 2K\sqrt{f'_c}b_w d$$

K shall be computed in accordance with Section 12.2.10.

12.2.13 Where transverse reinforcement is required, the minimum tensile capacity of the transverse reinforcement shall be 50 $b_w s$, where b_w and s are in inches. Greater amounts may be required to carry shear and torsion to meet the requirements of Section 12.3 or 12.5.

12.2.14 Transverse reinforcement may consist of:

a. Stirrups perpendicular to the axis of the member or making an angle of 45° or more with the longitudinal tension reinforcement, inclined to intercept potential cracks.

b. Welded wire fabric sheets or cages with wires located perpendicular to the axis of the member.

c. Longitudinal bars bent to provide an angle of 30° or more with the longitudinal tension reinforcement and inclined to intercept potential diagonal cracks.

d. Well-anchored prestressed tendons which are carefully detailed and constructed to minimize seating and time dependent losses.

e. Combinations of stirrups, tendons, and bent longitudinal bars.

f. Spirals.

12.2.15 Transverse reinforcement shall be detailed so that the shear forces between the different elements or zones of a member are effectively transferred. Transverse shear or torsion reinforcement shall extend as a continuous tie from the extreme compression fiber (less cover) to the outermost tension reinforcement. All transverse reinforcement shall be fully anchored according to AASHTO *Standard Specifications for Highway Bridges*, Section 8.27.

12.2.16 Torsion reinforcement shall consist of longitudinal bars or tendons and:

a. closed stirrups or closed ties, perpendicular to the axis of the member;

b. a closed cage of welded wire fabric with transverse wires perpendicular to the axis of the member;

c. spirals.

Transverse torsion reinforcement shall be made fully continuous and shall be anchored according to AASHTO *Standard Specifications for Highway Bridges*, Sections 8.27.2.1 or 8.27.2.3, where the concrete surrounding the anchorage is restrained against spalling by flange or slab or similar element. Anchorage shall be by 135° standard hooks around longitudinal reinforcement where the concrete surrounding the anchorage is unrestrained against spalling. Spacing of closed stirrups or closed ties shall not exceed one-half of the shortest dimension of the cross section, nor 12 inches.

12.2.17 At any place on the cross section where the axial tension due to torsion and bending exceeds the axial compression due to prestressing and bending, either supplementary tendons to counter the tension must be added or local longitudinal reinforcement which is continuous across the joints between segments is required.

If supplementary tendons are added, they shall be distributed around the perimeter of the precompressed tension zone inside the closed stirrups. At least one tendon shall be placed near each corner of the stirrups in the precompressed tension zone.

If longitudinal reinforcement is added, the bars shall be distributed around the perimeter formed by the closed stirrups. Perimeter bar spacing shall not exceed 18 inches. At least one longitudinal bar shall be placed in each corner of the stirrups. The minimum diameter of the corner bars shall be 1/24 of the stirrup spacing, but no less than that of a #5 bar.

12.2.18 Maximum spacing of transverse reinforcement shall not exceed 0.5d in nonprestressed elements and 0.75h in prestressed elements nor 36 in.

When V_u exceeds $6\phi_v\sqrt{f'_c}b_wd$, these maximum spacings shall be reduced one-half.

12.2.19 Flexural reinforcement, including tendons, shall be extended beyond the theoretical termination or deviation points for a distance of at least h/2. Transverse reinforcement for shear and torsion shall be provided for a distance at least h/2 beyond the point theoretically required.

12.2.20 Shear keys in webs of precast segmental bridges shall extend for as much of the web height as is compatible with other detailing requirements. Details of the shear keys shall be similar to Figure 24-1. Alignment shear keys shall also be provided in top and bottom flanges.

12.2.21 For structures utilizing dry joints, the nominal capacity of the joint shall be calculated as:

$$V_{uj} = \phi_j V_{nj}$$

$$V_{nj} = A_k\sqrt{f'_c}(12 + 0.017f_{pc}) + 0.6A_{sm}f_{pc}$$

Where: A_k = Area of the base of all keys in the failure plane, in^2.

f'_c = Compressive strength of concrete, psi.

f_{pc} = Compressive stress in concrete after allowance for all prestress losses, psi, determined at the centroid of the cross-section (existing definition).

A_{sm} = Area of contact between smooth surfaces on the failure plane, in^2.

12.3 Traditional Shear and Torsion Design for Plane Section Type Regions

12.3.1 The design of beam-type members or regions for shear and torsion may be carried out according to Section 12.3 provided:

a. V_n does not exceed :

$$12\sqrt{f'_c}b_wd$$

b. $\sqrt{(V_n / bwd)^2 + (T_n / 2A_obe)^2}$

does not exceed $15\sqrt{f'_c}$

c. There are no significant discontinuities such as abrupt changes in cross section or openings.

d. No concentrated load located within 2d of a support causes more than one-third of the shear at that support.

e. Where required, shear reinforcement consists of tendons and stirrups perpendicular to the axis of the member or welded wire fabric sheets or cages with wires perpendicular to the axis of the member, and conforms to Section 12.2.

f. Where required, torsion reinforcement consists of longitudinal bars, and closed stirrups perpendicular to the axis of the member, and conforms to Section 12.2.

12.3.2 The design of cross sections subject to shear shall be based on $V_u \le \phi_v V_n$ where V_u is the factored shear force and V_n is the nominal shear strength. V_u shall consider any unfavorable effect of prestressing V_p and may consider favorable effects of prestressing in accordance with Section 12.1.5. For the purposes of this section, V_n may be computed as

$$V_n = V_c + V_s$$

where V_c may be determined from Section 12.2.12 and V_s may be determined from Section 12.3.4. In equations for V_c and V_s, d shall be the distance from the extreme compression fiber to the centroid of the prestressed reinforcement in the tension chord or 0.8h, whichever is greater.

12.3.3 The applied shear V_u in regions near supports may be reduced to the value computed at a distance h/2 from the support when both of the following conditions are satisfied:

a. The support reaction, in the direction of the applied shear, introduces compression into the support region of the member, and

b. No concentrated load occurs within a distance h from the face of the support.

12.3.4 The nominal shear contribution of the truss model with concrete diagonals at 45° inclination as determined by the shear reinforcement perpendicular to the axis of the member is

$$V_s = A_vf_{sy}d/s$$

12.3.5 Where required by Section 12.2.10, torsion reinforcement shall be provided in addition to the reinforcement required to resist the factored shear, flexure and axial forces that act in combination with the torsion.

12.3.6 The longitudinal and transverse reinforcement required for torsion shall be determined from $T_u \le \phi_vT_n$.

12.3.7 The nominal torsional resistance provided by a space truss with concrete diagonals at 45° inclination and the indicated transverse reinforcement for torsion is

$$T_n = 2A_oA_tf_{sy} / s$$

where A_o is as defined in Section 12.2.10.

12.3.8 The additional longitudinal reinforcement for torsion shall not be less than

$$A_\ell = \frac{T_n P_h}{2A_o f_{sy}}$$

where P_h is the perimeter of the polygon defined by the centroids of the longitudinal chords of the space truss resisting torsion. P_h may be taken as the perimeter of the centerline of the outermost closed stirrups.

A_ℓ shall be distributed around the perimeter of the closed stirrups in accordance with Section 12.2.17.

12.3.9 The area of additional longitudinal torsion reinforcement in the flexural compression zone may be reduced by an amount equal to $M_u/(9d\ f_{sy})$ where M_u is the factored bending moment acting at that section except that the reinforcement provided shall not be less than required by Section 12.2.17.

12.4 Strut-and-Tie Truss Model Design for Either Beam Type or Discontinuity Regions

12.4.1 The design of any region for shear and torsion may be carried out according to Section 12.4 based on an analysis of the internal load paths for all forces acting on the member region. The effects of the prestress force shall be included in accordance with Section 12.1.5. The internal load paths shall be idealized using appropriate strut-and-tie or space truss models consisting of:

a. concrete and compressive reinforcement compression chords
b. inclined concrete compressive struts
c. longitudinal reinforcement tension chords or ties
d. transverse reinforcement tension members or ties
e. node regions at all joints of chords, struts and ties.

12.4.2 The proportions of the elements and the reinforcement shall be selected so that the tension ties yield before the compression chords or struts crush. Chord capacities shall be based on under-reinforced sections for flexure.

12.4.3 The sizes of the members and joint regions in the truss shall be chosen so that the computed forces in the struts, ties, and truss members, S_u, due to factored loads shall satisfy:

a. Compression chords

$$\phi_f(0.85f'_c A_{cc} + A'_s f'_c) \geq S_u$$

where ϕ_f is the appropriate ϕ value for flexure.

b. Inclined compressive struts

$$\phi_v\ (f_{cu} A_{cs}) \geq S_u$$

where ϕ_v is the appropriate ϕ value for shear and diagonal tension and f_{cu} is the limiting strut compressive stress from Section 12.4.4.

c. Reinforcement tension chords

$$\phi_f\ (A_s f_{sy} + A_s^* f_{su}^*) \geq S_u$$

where ϕ_f is the appropriate ϕ value for flexure and f_{su}^* is the average stress in prestressing steel at ultimate load considering the anchorage and bonding conditions.

d. Transverse reinforcement tension members or ties

$$\phi_v(A_v f_{sy}) \geq S_u$$

where ϕ_v is the appropriate ϕ value for shear and diagonal tension. When such members or ties are prestressed, the effective stress after prestress losses shall be used in place of f_{sy}.

e. Node regions

$$\phi_b\ (f_{cn} A_{cn}) \geq S_u$$

where ϕ_b is the appropriate ϕ value for bearing and f_{cn} is the limiting compressive stress in a node region from Section 12.4.6.

12.4.4 The compressive stress in an inclined compressive strut, f_{cu} shall not exceed:

a. For essentially undisturbed, uniaxial compressive stress states. $0.6f'_c$

b. For compressive stress states where tensile strains in the cross-direction or transverse tensile reinforcement may cause cracking of normal crack width parallel to the strut. $0.45\ f'_c$

c. For compressive stress states with skew cracking or skew transverse reinforcement. $0.35\ f'_c$

d. For compressive stress states with very wide skew cracks when the strut orientation differs appreciably from the elastic orientation of the internal load path. $0.25\ f'_c$

12.4.5 The tension chord and all tension ties shall be effectively anchored to transfer the required tension to the truss node regions in accordance with the ordinary requirements of AASHTO for development of reinforcement (AASHTO *Standard Specifications for Highway Bridges*, Sections 8.24 to 8.33) and shall be detailed to satisfy the stress limits of Section 12.4.6.

12.4.6 Unless special confining reinforcement is provided, the concrete compressive stress f_{cn} in the node regions shall not exceed:

a. 0.85 f'_c in node regions bounded by compressive struts and bearing areas

b. 0.70 f'_c in node regions anchoring only one tension tie, or

c. 0.6 f'_c in node regions anchoring tension ties in more than one direction.

12.5 Special Requirement for Diaphragms, Deep Beams, Corbels and Brackets

12.5.1 All discontinuity regions must be proportioned using the strut-and-tie model approach of Section 12.4.

Special discontinuity regions like diaphragms, deep beams, corbels, and brackets must also satisfy the special provisions of Section 12.5.

12.5.2 Diaphragms are ordinarily required in pier and abutment superstructure segments to distribute the high shear forces to the bearings. Vertical and transverse post-tensioning shall be analyzed using the strut and tie model of Section 12.4, and the effective prestress forces of Section 12.1.5. The diaphragm tendons must be effectively tied into the diaphragms with bonded non-prestressed reinforcement to resist tendon forces at the corners of openings in the diaphragms.

12.5.3 Deep beams are members in which the distance from the point of zero shear to the face of the support is less than 2d or members in which a load causing more than one-third of the shear at a support is closer than 2d from the face of the support.

12.5.3.1 The strut-and-tie model of Section 12.4 shall be used to analyze and design deep beams.

12.5.3.2 The minimum tensile capacity of transverse reinforcement shall be 120 $b_w s$, and s shall not exceed d/4 nor 12 inches.

12.5.3.3 Bonded longitudinal bars shall be well distributed over each face of the vertical elements in pairs. The minimum tensile capacity of this bonded reinforcement pair shall be 120 $b_w s$. The vertical spacing between each pair, s, shall not exceed d/3 nor 12 inches.

12.5.3.4 In deep beam vertical elements with a width less than 10 inches, the pairs of bonded bars required by Section 12.5.3.3 may be replaced by a single bar with the required tensile capacity.

12.5.4 Brackets and Corbels

12.5.4.1 The strut-and-tie model of Section 12.4 shall be used to analyze and design brackets and corbels.

12.5.4.2 The depth at the outside edge of the bearing area shall be at least half the depth at the face of the support.

12.5.4.3 Corbels and brackets shall be designed to resist the calculated external tensile force N_{ut} acting on the bearing area, but N_{ut} shall not be less than 0.2 V_u unless special provisions are made to avoid tensile forces. Therefore, N_{ut} shall be regarded as a live load even when tension results from creep, shrinkage or temperature change.

12.5.4.4 The steel ratio A_s/bd at the face of the support shall be at least 0.04 f'_c / f_{sy}, where d is measured at the face of the support.

12.5.4.5 Closed stirrups or ties parallel to the primary tensile tie reinforcement A_s with a total area not less than 0.5 A_s shall be uniformly distributed within 2/3 of the effective depth adjacent to A_s.

12.5.4.6 At the front face of a bracket or corbel, the primary tension reinforcement A_s shall be effectively anchored to develop the specified yield strength, f_{sy}, by:

a. a structural weld to a transverse bar of at least equal size, or;

b. bending the primary bars, A_s, back to form a continuous loop, or;

c some other positive means of anchorage.

12.5.4.7 The bearing area of the load on a bracket or corbel shall not project beyond the straight portion of the primary tension bars, A_s, nor project beyond the interior face of any transverse anchor bar.

12.6 Shear Transfer at Interfaces

Shear transfer at interfaces shall be designed in accordance with Section 11.7 of ACI 318-95 using the ϕ_v values from this Specification.

12.7 Two-Way Punching Shear

Two-way punching shear slab type elements shall be designed in accordance with AASHTO *Standard Specifications for Highway Bridges*, Section 8.16.6.6, using the ϕ_v values from this specification.

13.0 FATIGUE STRESS LIMITS

13.1 Fatigue Stress Limits for Bonded Non-Prestressed Reinforcement

Design of bonded non-prestressed reinforcement for fatigue shall conform to the provisions of AASHTO *Standard Specifications for Highway Bridges*, Section 8.16.8.3.

13.2 Fatigue Stress Limits for Prestressed Reinforcement

Fatigue of prestressed reinforcement need not be considered for bridges designed in accordance with this Specification.

14.0 POST-TENSIONED ANCHORAGE ZONES

All anchorage zone design provisions shall be in accordance with Section 9.21 of the AASHTO *Standard Specifications for Highway Bridges.*

15.0 PROVISIONAL POST-TENSIONING DUCTS AND ANCHORAGES

15.1 General

In accordance with Section 15.2, the design of ducts and anchorages for bridges with internal tendons shall provide for increases in the post-tensioning force at selected locations along the bridge during construction to compensate for excessive friction and wobble losses during stressing. In addition, for bridges with either internal or external tendons, the design shall provide for future installation of external unbonded tendons in accordance with the provisions of Section 15.3 as an allowance for addition of future dead load, or to adjust for cracking or deflection of the bridge.

15.2 Bridges with Internal Ducts

Provisional anchorage and duct capacity for negative and positive moment tendons located symmetrically about the bridge centerline shall provide for an increase in the post-tensioning force of not less than 5 percent of the total positive and negative primary moment forces, respectively. At least one empty duct per web shall be provided. Except for non-continuous bridges, and the minimum empty duct capacity noted above, provisional positive moment duct and anchorage capacity shall not be required for 25 percent of the span length either side of pier supports. Any provisional ducts not utilized for adjustment of the post-tensioning force shall be grouted at the same time as other ducts in the span.

15.3 Provision for Future Dead Load or Deflection Adjustment

Specific provision shall be made for access and for anchorage attachments, pass through openings, and deviation block attachments to permit future addition of unbonded external tendons (inside the box section) symmetrically about the bridge centerline for a post-tensioning force of not less than 10 percent of the positive moment and negative moment primary post-tensioning forces.

SECTION D
DETAILING

16.0 DUCT DETAILS

16.1 Material Thickness

16.1.1 Metal Ducts

Metal ducts shall be galvanized corrugated semi-rigid conduit. For strand and wire tendons, the duct thickness shall be 26 gauge up to 2 5/8 inches diameter. Ducts larger than 2 5/8 inches diameter shall be 24 gauge. For bar tendons, the duct thickness shall not be less than 31 gauge. Steel deviation pipe thickness shall be equal to or greater than 0.120 inches. Steel deviation pipe shall be fabricated with bend tolerances of +2 degrees and -0 degrees.

16.1.2 Polyethylene Duct

Corrugated polyethylene duct used as internal duct shall be high density polyethylene and conform to the requirements of ASTM D 3350 with ultra violet stabilizers added. Rigid polyethylene pipe used for external duct shall conform to the requirements of ASTM D 2447, Grades P33 or P34; D 3350 or F 714 with a Cell Classification of PE345433C. Material thickness and other requirements for polyethlene duct shall be as follows:

a. Internal polyethylene duct shall be corrugated and shall have a minimum material thickness of 0.050 IN ± 0.010 IN.

b. External polyethylene duct shall be black, smooth HDPE pipe with a minimum wall thickness of 1/21 of the outside diameter, and a minimum hydrostatic design basis (HDB) of 1.25 KSI.

16.2 Duct Area

16.2.1 For tendons made up of a number of wires, bars, or strands, duct area shall be at least twice the net area of the prestressing steel. Placement of tendons by the pull-through method requires duct area of 2 1/2 times the prestressing steel area.

16.2.2 For tendons made up of a single wire, bar, or strand, the duct diameter shall be at least 1/4 inch larger than the nominal diameter of the wire, bar, or strand.

16.3 Minimum Radius of Curvature

Tendon ducts shall preferably be installed with a radius of curvature of 20 feet or more. Ducts with sharper curvature down to a minimum of 10 feet shall have confinement reinforcement detailed to tie the duct into the concrete. Duct curvature with radii less than 10 feet may be approved by the Engineer based on review of test data. The minimum radius for corrugated polyethylene duct shall be 30 feet unless curvature with radius less than 30 feet is approved by the Engineer based on review of test data. The confinement reinforcement shall be proportioned to resist radial forces calculated as

$$F_r = \frac{P_j}{R}$$

where P_j is the tendon force in kips, and R is the radius of curvature, in feet, and F_r is the radial force in kips per foot. Confinement reinforcement shall be proportioned at 0.6 f_{sy} where f_{sy} shall not exceed 60 KSI. Spacing of confinement reinforcement shall not exceed 12 inches. Closer spacing shall be used for duct with radius of curvature less than 15 feet.

16.4 Duct Supports

16.4.1 Internal Ducts

Internal ducts shall be rigidly supported by ties to reinforcing steel as follows:

a. Transverse slab tendons in metal duct: 2 feet. Preplaced strand in metal duct = 4 feet.

b. Transverse slab tendons in polyethylene duct: 2 feet. Preplaced strand in polyethylene duct = 4 feet.

c. Longitudinal slab or web tendons in metal duct: 4 feet.

d. Longitudinal slab or web tendons in polyethylene duct: 2 feet.

16.4.2 External Ducts

External ducts shall have a maximum unsupported length of 25 feet unless a vibration analysis is made.

16.5 Duct Size, Clearance and Detailing

Maximum size of internal ducts shall not exceed 0.4 x web thickness.

Where two curved tendons run parallel such that the outer one is bearing inwards toward the inner one, a minimum clearance of one duct diameter shall preferably be maintained between the ducts. If this is not possible, reinforcement shall be provided between the ducts to fully restrain the outer tendon if it has to be stressed before the inner tendon has been stressed and grouted. In cases where longitudinal tendons cross each other at least one-half duct diameter but not less than

2 inches clear space shall be provided. This restriction does not apply to transverse ducts crossing longitudinal ducts at approximately 90°.

Curved tendons should not be placed around re-entrant corners or voids. If this is unavoidable, then the tendons must be provided with well anchored, full reinforcement restraint proportioned as per Section 16.6.1. In no case shall the distance between the re-entrant corner or void and the edge of the duct be less than 1.5 duct diameters.

16.6 Duct Confinement Reinforcement

16.6.1 Effects of Curved Tendons

When curved tendons are located in thin webs or close to internal voids, reinforcement shall be provided to prevent the tendon from bursting through the concrete into the void whenever the nominal shear stress in the concrete cover beside the tendon due to tendon jacking forces exceeds $2\sqrt{f'_c}$.

The area of steel required may be estimated from:

$$A_s = \frac{P_j}{R \times 0.6 f_{sy}}$$

Where A_s = Area of steel required, sq.in./ft. and f_{sy} shall not exceed 60 ksi.

The lateral force exerted on the concrete by the tendons may be calculated by dividing the tendon force by the radius of curvature in accordance with Section 16.3.

16.6.2 Ducts in Flanges

Ducts in bottom slabs shall be located between top and bottom layers of transverse and longitudinal slab reinforcement. For ducts in the bottom flanges of variable depth segments, nominal confinement reinforment shall be provided around the duct at each segment face. The reinforcement shall not be less than two rows of #4 hairpin bars at both sides of each duct with vertical dimension equal to the slab thickness less top and bottom cover dimensions.

When closely spaced transverse or longitudinal ducts are located in top or bottom flanges, the top and bottom non-prestressed reinforcement mats shall be tied together with vertical reinforcement consisting of #4 hairpin bars with spacing not to exceed 18 inches or 1 1/2 times the slab thickness in each direction, whichever is the lesser.

17.0 COUPLERS

Not more than 50 percent of the longitudinal post-tensioning tendons shall be coupled at one section. The spacing between adjacent coupler locations shall not be closer than the segment length or twice the segment depth. The void areas around couplers shall be deducted from the gross section area and moment of inertia when computing stresses at the time of application of the post-tensioning force.

18.0 CONCRETE COVER AND REINFORCEMENT SPACING

18.1 Cover and Spacing

Reinforcement cover shall be in accordance with AASHTO *Standard Specifications for Highway Bridges*, Section 9.26.1. Reinforcement spacing shall be in accordance with AASHTO *Standard Specifications for Highway Bridges*, Section 9.26.2. Reinforcement details for erection loads shall be in accordance with Section 18.2 of these specifications.

18.2 Reinforcement Details for Erection Loads

The transverse analysis of the box girder shall include an evaluation of the quantity Z of equation 8-61 of the AASHTO *Standard Specifications for Highway Bridges*, Section 8.16.8.4, for any loads applied prior to attainment of full design strength. The value of Z calculated for flanges and webs shall not exceed 130.

19.0 INSPECTION ACCESS

Inspectability of the structure shall be assured by providing secured access hatches with minimum dimensions of 2'-6" x 4'-0". Interior diaphragms shall be provided with openings larger than the dimensions specified for access hatches. The box section shall be vented by drains or screened vents in webs at intervals not greater than 50 ft. Such venting is to prevent the build-up of potential hazardous gas which might endanger inspection personnel.

20.0 BOX GIRDER CROSS SECTION DIMENSIONS AND DETAILS

20.1 Minimum Flange Thickness

Top and bottom flange thickness shall not be less than any of the following:

1. 1/30 the clear span between webs or haunches; a lesser dimension will require transverse ribs at a spacing equal to the clear span between webs or haunches.

2. Top flange, 9 inches in anchorage zones where transverse post-tensioning is used, and 8 inches minimum thickness beyond anchorage zones or for pretensioned slabs. Transverse post-tensioning or pretensioning shall be used where the clear span between webs or haunches is 15 feet or larger.

20.2 Minimum Web Thickness

1. Webs with no longitudinal or vertical post-tensioning tendons - 8 inches.

2. Webs with only longitudinal or vertical post-tensioning tendons - 12 inches.

3. Webs with both longitudinal and vertical post-tensioning tendons - 15 inches.

20.3 Length of Top Flange Cantilever

The cantilever length of the top flange measured from the centerline of the web should preferably not exceed 0.45 times the interior span of the top flange measured between the centerline of the webs.

20.4 Overall Cross Section Dimensions

Overall dimensions of the box girder cross section should preferably not be less than required to limit live load plus impact deflection calculated using the gross section moment of inertia and the secant modulus of elasticity to 1/1000 of the span. The live loading shall consist of all traffic lanes fully loaded with adjustment for number of loaded lanes as specified in the AASHTO *Standard Specifications for Highway Bridges*, Section 3.12. The live loading shall be considered to be uniformly distributed to all longitudinal flexural members.

21.0 BRIDGE BEARINGS

21.1 General

Bridge bearings shall conform to Sections 14, 19, or 20 of the AASHTO *Standard Specifications for Highway Bridges*.

21.2 Replacement or Adjustment

Provisions shall be made in the design for the future replacement of bridge bearings and adjustment of support reactions.

21.3 Bursting and Spalling

Concrete surfaces in contact with bearings as well as jacking pockets and jack locations provided for bearing replacement or adjustment shall be reinforced to prevent bursting (splitting) and spalling.

21.4 Settings

Expansion bearings shall be sized and set at time of construction for the following conditions.

21.4.1 Allowance for movement away from the fixed pier based on the total anticipated movement resulting from the maximum temperature rise taken from Figure 6-2 with respect to the mean shade temperature for the 48-hour period prior to the time at which the bearings are set.

21.4.2 Allowance for movement toward the fixed pier based on 1.3 times the total anticipated movement resulting from the combined effect of temperature fall, creep, shrinkage and elastic shortening. Temperature fall shall be based on the maximum value taken from Figure 6-1 with respect to the mean shade temperature for the 48-hour period prior to the time at which the bearings are set. Creep, shrinkage and elastic shortening shall be computed from the time the bearings are installed through day 4000. Installation of bearings shall be estimated based on the early construction schedule stated on the plans.

21.4.3 A table of setting adjustments shall be provided to account for variation of the mean shade temperature for the 48-hour period prior to the time of installation and the mean temperature given in Section 6.4. The table shall indicate the mean shade air temperature for the 48-hour period prior to the time of installation and adjustments shall be calculated for the difference between the mean shade temperature for the 48-hour period prior to the installation and the mean temperature from Section 6.4.

22.0 EXPANSION JOINTS

22.1 Design

Expansion joints shall be designed for the full range of movement anticipated due to creep, shrinkage, and temperature effects, and shall comply with the provisions of Section 19, Division II, of the AASHTO *Standard Specifications for Highway Bridges*, or approved special joint design provisions.

22.2 Settings

The setting of expansion joint recesses and expansion joint devices, including any precompression, shall be clearly stated on the drawings. Expansion joints shall be sized and set at time of construction for the following conditions.

22.2.1 Allowance for closure movements based on the maximum temperature rise taken from Figure 6-2 with respect to the mean shade temperature for the 48-hour period prior to the time at which the joints are set.

22.2.2 Allowance for opening movements of modular compression sealed expansion joint devices shall be based on 1.3 times the total anticipated movement resulting from the combined effects of temperature fall, creep and shrinkage. Temperature fall shall be based on the maximum value taken from Figure 6-1 with respect to the mean shade temperature for the 48-hour period prior to the time at which the joints are set. Creep and shrinkage shall be computed from the time the expansion joints are installed through day 4000. All expansion joints shall be installed after superstructure segment erection is completed for the entire bridge. Time of installation shall be estimated based on the early construction schedule stated on the plans.

22.2.3 To account for the larger amount of opening movement, expansion devices should be set precompressed to the maximum extent possible.

Expansion devices shall be sized and set such that after 4000 days the device shall not remain in tension through the full range of design temperature.

22.2.4 A table of setting adjustments shall be provided in accordance with Section 21.4.3.

22.3 Expansion Joint Location

Expansion joints shall generally be located at piers, abutments, or within one segment length of points of dead load contraflexure. Mid-span expansion joints may be used when calculated deflections are within acceptable limits, and/or when moment carrying beams are used to provide for deflection adjustment across expansion joints.

SECTION E
SUBSTRUCTURE DESIGN

23.0 SPECIAL PROVISIONS FOR HOLLOW RECTANGULAR SEGMENTAL BRIDGE SUBSTRUCTURES

23.1 General

Pier and abutment design shall conform to Section 7 of the AASHTO *Standard Specifications for Highway Bridges*, and to the provisions of this Section. Consideration shall be given to erection loads, moments, and shears imposed on piers and abutments by the construction method shown on the plans. Auxiliary supports and bracing shall be shown on the plans as required. Hollow, rectangular precast segmental piers shall be designed in accordance with the provisions of this section.[53]

The load factor for temperature gradient shall be taken as zero (0) in design of segmental substructures for service, strength, and construction load combinations.

23.2 Construction Load Combinations

The capacity of hollow, rectangular precast segmental piers to support the construction load combinations of Table 7-2 shall be investigated in accordance with the provisions of this Section.

Tensile stresses in segmental substructures during construction shall be computed for load combinations *a* through *f* of Table 7-2.

Load combination *d* is applicable to conditions when erection equipment is not working, load combination *e* relates to normal erection, and load combination *f* applies when erection equipment is moving.

23.3 Wall Slenderness Ratio

The wall slenderness ratio of a hollow rectangular cross-section shall be taken as:

$$\lambda w = \frac{X_u}{t}$$

where

X_u = the clear length of the constant thickness portion of a wall between other walls or fillers between walls (in.) (see Fig. 23-1).

t = thickness of wall (in.) (see Fig. 23-1).

Wall slenderness greater than 35 may be used only when the behavior and resistance of the wall is documented by analytic and experimental evidence acceptable to the owner.

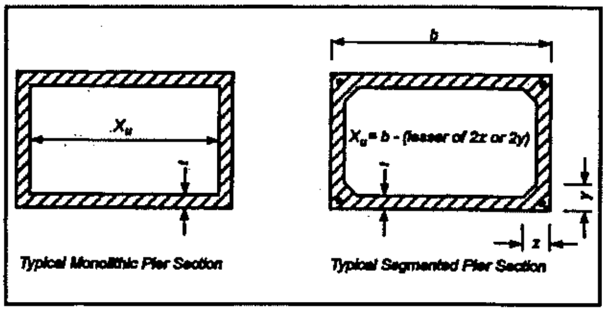


Fig. 23-1. Illustration of X_u

23.4 Limitations on the Use of the Rectangular Stress Block Method

23.4.1 General

Except as specified in Article 23.4.3, the equivalent rectangular stress block method shall not be employed in the design of hollow rectangular compression members with a wall slenderness ratio greater than or equal to 15.

Where the wall slenderness ratio is less than 15, the rectangular stress block method may be used based on a compressive strain of 0.003.

23.4.2 Refined ϕ Method for Adjusting Maximum Usable Strain Limit

Where the wall slenderness ratio is 15 or greater, the maximum usable strain at the extreme concrete compression fiber is equal to the lesser of the computed local buckling strain of the widest flange of the cross-section, or 0.003.

The local buckling strain of the widest flange of the cross-section may be computed assuming simply supported boundary conditions on all four edges of the flange. Non-linear material behavior shall be considered by incorporating the tangent material moduli of the concrete and reinforcing steel in computations of the local buckling strain.

Discontinuous, non-post-tensioned reinforcement in segmentally constructed hollow rectangular compression members shall be neglected in computations of member strength.

Flexural resistance shall be calculated using the principles of Section 9.17 of the AASHTO *Standard Specifications for Highway Bridges*.

23.4.3 Approximate Method for Adjusting Factored Resistance

The provisions of this article and the rectangular stress block method may be used in lieu of the provisions of Articles 23.4.1 and 23.4.2.

Factored strength of a hollow column, determined using a maximum usable strain of 0.003 and the strength reduction factors specified in Section 7.3.6 shall be further reduced by a factor taken as:

- if $\lambda_w \leq 15$, then $\phi_w = 1.0$
- if $15 < \lambda_w \leq 25$, then $\phi_w = 1.0 - .025\,(\lambda_w - 15)$
- if $25 < \lambda_w \leq 35$, then $\phi_w = 0.75$

23.5 Reinforcement for Hollow Rectangular Compression Members

23.5.1 General

The area of longitudinal reinforcement in the cross-section shall not be less than 0.0018 times the gross area of concrete.

Two layers of reinforcement shall be provided in each wall of the cross-section, one layer near each face of the wall. The areas of reinforcement in the two layers shall be approximately equal.

23.5.2 Spacing of Reinforcement

The center-to-center lateral spacing of longitudinal reinforcing bars shall be no greater than the lesser of 1.5 times the wall thickness or 18 inches.

The center-to-center longitudinal spacing of lateral reinforcing bars shall be no greater than the lesser of 1.25 times the wall thickness, or 12 inches.

23.5.3 Ties

Cross-ties shall be provided between layers of reinforcement in each wall. The cross-ties shall include a standard 135° hook at one end, and a standard 90° hook at the other end. Cross-ties shall be located at bar grid intersections, and the hooks of all ties shall enclose both lateral and longitudinal bars at the intersections. Each longitudinal reinforcing bar and each lateral reinforcing bar shall be enclosed by the hook of a cross-tie at a spacing no greater than 24 inches.

For segmentally constructed members, additional cross-ties shall be provided along the top and bottom edges of each segment. The cross-tie shall be placed so as to link the ends of each pair of internal and external longitudinal reinforcing bars in the walls of the cross-section.

23.5.4 Splices

Lateral reinforcing bars may be joined at the corners of the cross-section by overlapping 90° bends. Straight lap splices of lateral reinforcing bars shall not be permitted unless the overlapping bars are enclosed over the length of the splice by the hooks of at least four cross-ties located at intersections of the lateral bars and longitudinal bars.

23.5.5 Hoops

Where details permit, the longitudinal reinforcing bars in the corners of the cross-section shall be enclosed by closed hoops. If closed hoops cannot be provided, then pairs of "U" shaped bars with legs at least twice as long as the wall thickness, and oriented 90° to one another, may be used.

Post-tensioning ducts located in the corners of the cross-section shall be anchored into the corner regions with closed hoops, or by stirrups having a 90° bend at each end which encloses at least one longitudinal bar near the outer face of the cross-section.

SECTION F
SPECIAL PROVISIONS FOR BRIDGE TYPES

24.0 PRECAST SEGMENTAL

24.1 General

Precast segmental bridges are normally erected by balanced cantilever, by use of erection trusses, or by progressive placement. To limit construction deflections to values consistent with design calculations, precast segments shall be a minimum of 14 days old at the time of erection unless earlier erection is specifically approved by the Engineer.

24.2 Stresses for Construction Practices

24.2.1 Removal of Forms

Before stripping the forms or moving the segment while resting on its soffit form in the casting bed, the concrete shall have a minimum compressive strength of 2500 psi unless analysis requires a higher value. For transverse post-tensioning, the concrete shall have a minimum compressive strength of 3500 psi. For handling of the segment, the concrete shall have a minimum compressive strength of 3000 psi.

24.2.2 Erection

The following concrete strengths shall be obtained before post-tensioning:

1. When considered necessary to avoid cracking, partial stressing of post-tensioning across longitudinal or transverse closure strips may be permitted when the concrete has attained a minimum strength of 1800 psi. Full post-tensioning of closure strips shall normally be permitted when the concrete has attained a minimum strength of 2500 psi, unless analysis indicates that a higher value is required.

2. Before stressing permanent longitudinal post-tensioning tendons, the concrete segments shall attain the minimum 28-day compressive strength.

24.2.3 Temporary Stress on Epoxy Joints

A minimum compressive stress of 40 psi shall be provided for the closure stress on an epoxied joint until the epoxy has set.

24.3 Shear Keys

Shear keys in webs of precast segmental bridges shall extend for as much of the web depth as is compatible with other detailing requirements. Details of the shear keys shall be similar to Figure 24-1. Shear keys shall also be provided in top and bottom slabs, but these may be larger single element keys.

24.4 Joints

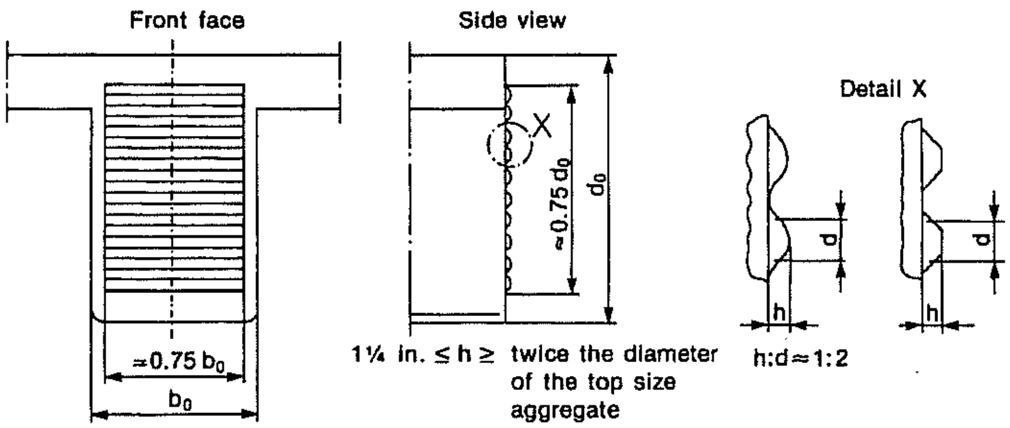
24.4.1 General

Precast segmental bridges are usually match-cast with Type A or epoxied joints. Epoxy on both faces serves as a lubricant during placement of the segments, prevents water intrusion, provides a seal to prevent crossover during grouting and provides some tensile strength across the joint. Closure pours are cast-in-place. Type B joints or dry joints are not allowed for new construction. Reference has been retained for the purpose of load rating existing structures.

24.4.2 Type A Joints

Type A (epoxied) joints shall be utilized for all bridges utilizing internal tendons, and for all bridges

Fig. 24-1. Example of design of fine indentation joint faces



exposed to severe climatic conditions where freeze/thaw cycles do not occur, and where de-icer chemicals are not used.

24.5 External Tendons

External tendons shall be permanently protected against corrosion.

25.0 CAST-IN-PLACE SEGMENTAL

25.1 General

Cast-in-place segmental bridges may be erected on falsework, by the free cantilever technique, by span-by-span lifting of spans cast at the bridge site, or by incremental launching. The special design requirements for incremental launching are presented in Section 26.0. Special design requirements for other cast-in-place segmental construction methods are presented in the following sections.

25.2 Joint Details

Contact surfaces between cast-in-place segments shall be clean, free of laitance, and shall be intentionally roughened to expose coarse aggregate. The use of shear keys is optional.

25.3 Closure Pours

Closure pours shall have sufficient width to permit coupling of tendon ducts.

25.4 Form Traveler Weight

The form traveler weight assumed in stress and camber calculations shall be stated on the design drawings.

25.5 Segment Lengths

Segment lengths may vary with the construction method, with the span length, and with location in the span.

25.6 Diaphragms

Diaphragms shall be provided at abutments, piers, hinge joints, and at bottom flange angle points in structures with straight haunches. Diaphragms shall be substantially solid at piers and abutments except for access openings and utility holes. Diaphragms shall be sufficiently wide as required by design with a minimum overhang over bearings of not less than 6 inches.

26.0 INCREMENTAL LAUNCHING

26.1 Loads and Stresses During Launching

26.1.1 Friction n Launching Bearings

The friction on launching bearings shall be assumed to vary between 0 percent and 4 percent, whichever governs the determination of hold-back or pushing forces. The upper value may be reduced by 0.5 percent, if the pier deflections and the launching jack forces are monitored. The design shall consider that inclined launching bearings (as opposed to the horizontal permanent bearings) create additional forces at the launching jacks and at the pier tops.

26.1.2 Moments Due to Construction Tolerances

The moments due to the following permissible construction tolerances shall be superimposed onto those from external loads:

In longitudinal direction between two adjacent bearings: 0.2 inches

In transverse direction between two adjacent bearings: 0.08 inches.

26.1.3 Combination of Construction Tolerances and Temperature Gradient

During construction, one-half of the moments due to construction tolerances as per Section 26.1.2 and one-half of the moments due to temperature in accordance with Section 6.4.4 shall be superimposed onto those from external loads. Tensile stresses resulting from the combined moments shall not exceed $7.0\sqrt{f'_c}$.

26.1.4 Permissible Stresses

Permissible stresses shall be in accordance with Section 9.2.1.2.a for members with bonded reinforcement, through the joint and internal tendons. Temporary piers and/or a launching nose may be used to reduce launching stresses.

26.2 Bridge Design

26.2.1 Piers and Superstructure Diaphragms

Piers and superstructure diaphragms at piers shall be designed in such a way that during all launching stages, and after launching for the installation of the permanent bearings, the superstructure can be lifted with hydraulic jacks. Pier designs shall consider frictional forces during launching in accordance with Section 26.1.1.

26.2.2 Bottom Edges of Superstructures

At the underside of the webs above the launching bearings high local stresses occur. The design shall take into account:

26.2.2.1 The launching pads shall be placed with a clear distance of at least 3 1/8 inches to the outside, as shown in Figure 26-1.

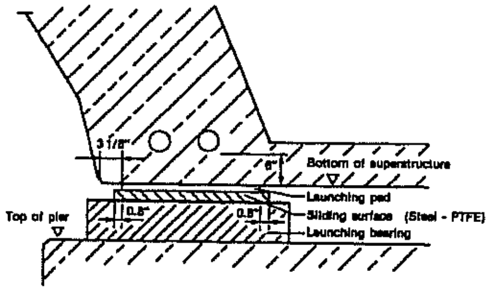


Fig. 26-1. Location of launching pads

26.2.2.2 Post-tensioning ducts shall have concrete cover of at least 6 inches to the underside, as shown in Figure 26-1.

26.2.2.3 The bearing pressures at bottom edges of superstructures shall be investigated, and shall consider any eccentric location of the support reaction and any ungrouted ducts, as shown in Figure 26-2.

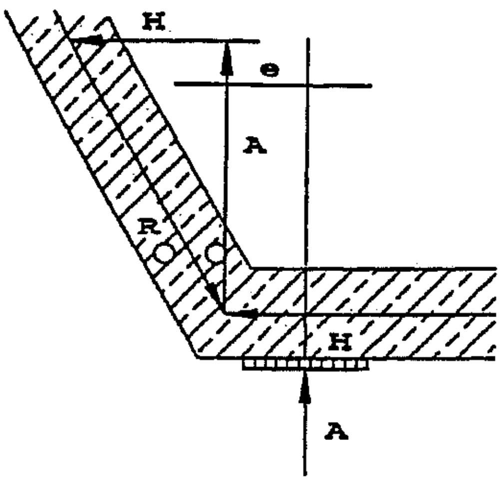


Fig. 26-2. Introduction of support reaction of launching bearings

26.2.3 Tendons

The straight tendons required during launching should preferably be placed in the top and bottom slabs (for T-sections in the lower one-third of the web). Not more than 50 percent of these straight tendons shall be coupled in a construction joint.

The concrete strength at the time of stressing the straight tendons shall be taken into account when designing the size and locations of the tendon anchorages.

26.2.4 Construction Joints Between Girder Segments

The faces of construction joints shall be provided with shear keys or other suitable means for providing a distinctly roughened surface. The minimum bonded, nonprestressed reinforcement in both directions and at all concrete surfaces across the joint and over a distance of 7 feet on either side of the joint shall be #4 bars at 5 inches on centers.

26.3 Design of Construction Equipment

26.3.1 Launching Nose

The construction tolerances in the sliding surface at the bottom of the launching nose are limited to those of the super-structure; see Section 26.1.2.

The introduction of the support reactions in the launching nose shall be carefully investigated to prevent buckling of the web of the launching nose.

26.3.2 Launching Bearings

Launching bearings shall be designed in such a way that they can compensate for local deviations of the sliding surface of up to 0.08 inches by elastic deformation.

26.3.3 Launching Equipment

The launching equipment shall be sized for friction in accordance with Section 26.1.1, and the actual superstructure gradient. The launching equipment shall be designed to assure that a power failure will not lead to uncontrolled sliding of the superstructure.

The friction coefficient between concrete and the hardened profiled steel surfaces of the launching equipment shall be taken as 60 percent. A safety factor of 1.3 shall be provided against uncontrolled sliding.

26.3.4 Forms

The forms for the sliding surfaces underneath and outside the web shall be wear resistant and sufficiently stiff so that their deflection during casting does not exceed 0.08 inches.

27.0 PRECAST SEGMENTAL BEAM BRIDGES

27.1 General

Precast segmental beam bridges shall be designed in accordance with the provisions of these specifications and in accordance with the additional provisions of this section.

27.2 Segment Reinforcement

Segments of segmental beam bridges shall preferably be prestressed for dead load and all construction loadings to limit the tensile stress in the concrete to $3.0\sqrt{f'_c}$. When segments without prestressed reinforcement are used, attention shall be given to detailing of reinforcement to comply with the requirements of Section 18.2.

27.3 Joints

27.3.1 Cast-in-Place Joints

The width of cast-in-place joints shall permit access for coupling of conduits and thorough vibration of the concrete. The compressive strength of the joint concrete at a specified age shall be compatible with design stress limitations. The face of the precast segments shall be intentionally roughened to expose coarse aggregate, or shear keys shall be utilized in accordance with Section 24.3.

27.3.2 Epoxied Match-Cast Joints

Epoxied match-cast joints for segmental beam bridges shall be designed in accordance with Section 24.3. A minimum of 40 psi compression shall be applied to the joint during the "open time" of the epoxy.

27.4 Post-Tensioning

Post-tensioning may be applied either before and/or after placement of deck concrete. Part of the post-tensioning may be applied to provide beam continuity prior to placement of the deck concrete, with the remainder placed after deck concrete placement. When couplers are used, the use of couplers shall conform to Section 17.0 with special attention given to the effect of voids for the couplers on the section properties at the time of post-tensioning.

Where tendons terminate at the top of the beam section, the duct opening shall be protected during construction to prevent debris accumulation. Drains shall be provided at tendon low points.

28.0 SPECIFICATIONS, CONTRACT DRAWINGS, AND ALTERNATE CONSTRUCTION METHODS

28.1 Specifications and Contract Drawings

Specifications and contract drawings for segmental concrete bridges shall contain all information necessary to allow the Contractor to construct the bridge by at least one method shown on the drawings. The specifications or contract drawings shall provide tolerances on construction equipment weights and permissible variations in concrete properties consistent with aggregates available near the place of construction.

Contract plans shall make allowances for construction loads and construction stages and provide the Contractor with camber information. The camber information shall be adequate for the development of casting curves or data at significant cross section points.

Scaffolding or local strengthening of the structure to carry construction loads, if such loads are not shown on the drawings, shall be the Contractor's responsibility.

Contract drawings shall follow either one of two methods of presentation, as outlined below.

Method A: Fully detailed drawings, structurally and dimensionally, including: segment length, construction joints, tendon sizes and geometry, jacking forces, nonprestressed reinforcement details, camber information, and one construction method.

Method B: Drawings showing: concrete dimensions, segment length, nonprestressed reinforcement details, stressing force and eccentricity requirements in the form of force and/or moment diagrams after friction losses at time of construction, camber information, and one construction method.

28.1.1 Contract Drawings Prepared According to Method A

If the Engineer prepares a set of contract drawings as outlined under Method A, he or she is fully responsible for all information provided on the drawings, including dimensional accuracy and interference-free construction. The Engineer is also responsible for the feasibility of the construction stages on which the design is based.

The Contractor shall be permitted to revise the tendon sizes and layout shown on the plans, provided the post-tensioning forces and moment due to post-tensioning forces (forces times eccentricities) are not less than those represented by the fully detailed plans and not more than 5 percent in excess at any section. In such case, the Contractor is required to prepare dimensional layout drawings, establish losses at time of stressing due to friction, and accept responsibilities for those modifications.

28.1.2 Contract Drawings Prepared According to Method B

If the Engineer prepares a set of contract drawings according to Method B, he or she is responsible for overall structural adequacy, including the construction stages on which the design is based. The Contractor may choose tendon sizes and tendon layout. The Contractor shall meet the moment diagram due to prestress after friction losses, but before creep, shrinkage, and relaxation losses, and not exceed them by more than 5 percent. The Contractor is then fully responsible for detailing the tendon layout. The Contractor is also responsible for preparing detailed nonprestressed reinforcement cutting and bending shop drawings, based on the non-prestressed reinforcement requirements shown on the contract drawings.

28.2 Alternate Construction Methods Not Requiring Value Engineering

When permitted by the contract special provisions, the Contractor may be allowed to choose alternate construction methods and a modified post-tensioning layout suitable for the selected construction method. In such case, the Contractor shall supply a structural analysis, documenting that the post-tensioning forces and eccentricities shown on the construction plans meet all requirements of the design specifications. If additional post-tensioning is required for construction stages or other reasons, it shall be demonstrated that the stresses at critical sections in the final structure meet the allowable stress provisions of the design specifications. Removal of temporary post-tensioning to achieve such conditions is permissible. Use of additional non-prestressed reinforcement for construction stages will be permitted. All extra materials required for construction stages shall be provided by the Contractor at no cost to the Owner.

28.3 Value Engineering

Value Engineering provisions may be included in the special contract provisions permitting alternate construction methods which require a complete re-design of the final structure. The Contractor's engineering expenses for preparing the Value Engineering Design and the Owner's engineering expenses for checking the design shall be considered part of the cost of the re-design structure.

Pier spacing, alignment, outside concrete appearance and dimensions shall not be changed under Value Engineering proposals, except when contract documents define such changes as being permitted.

The Contractor shall provide for the Value Engineering a complete set of design computations and revised contract drawings. The Value Engineering re-design shall be prepared by a Professional Engineer experienced in segmental bridge design. Upon acceptance of a Value Engineering re-design, the Professional Engineer responsible for the re-design shall become the Engineer of Record.

28.4 Shop Drawings

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. Whenever contract drawings prepared according to Method A are modified, or where contract drawings do not provide detailed dimensional information, as for those prepared according to Method B, it is the Contractor's responsibility to coordinate the placement of all embedments and to correct any interferences. The post-tensioning layout shall govern over the layout of non-prestressed reinforcement. Where necessary, location of nonprestressed reinforcement shall be adjusted to clear tendons.

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COMMENTARY
DIVISION I—DESIGN SPECIFICATIONS

SECTION A—GENERAL REQUIREMENTS AND MATERIALS

1.1 General

Segmental bridges contemplated under this section include but are not limited to those erected by the following methods:

1. Balanced cantilever
2. Span-by-span with truss or falsework
3. Span-by-span lifting
4. Incremental launching
5. Progressive placement
6. Segmental beam bridges

The span length of bridges considered by these specifications ranges to approximately 800 feet. Bridges supported by stay cables are not specifically covered although many of the specification provisions are applicable to cable-stayed bridges.

Although there is discussion of corrosion protection of conventional reinforcement and post-tensioning tendons in various sections, comprehensive treatment of this subject is beyond the scope of this report. This guide specification does not consider the subject of total replacement of the bridge deck.

A segmental bridge condition survey was published by the American Segmental Bridge Institute in 1995 based on state bridge inspection reports.[54] This survey concluded "...that segmental concrete construction performs well over time, with consistently high ratings for bridges that have been in use for up to 30 years."

In aggressive environments, use of a sacrificial bridge deck overlay is recommended, as well as use of other special durability provisions.

Two monumental segmental bridges have now been constructed in very aggressive environments over sea water with design service life of 100 years and without provision for deck replacement.[57, 58] The design criteria and concrete mix design for these bridges incorporated special durability provisions.

2.2 Reinforcement

Special consideration of corrosion protection is considered necessary for all bridge deck reinforcement (post-tensioning materials and rebar) in areas where deicer chemicals are used. Corrosion protection should also be provided for all reinforcement of bridges located in coastal areas or over sea water, or in corrosive industrialized areas. See ACI Committee 222 report, *Corrosion of Metals in Concrete*[1] for a comprehensive discussion of methods of corrosion protection.

A research report, *Durability of Post-Tensioned Bridge Decks*[43] includes the following in the conclusions section: *"For the exposure conditions of the durability tests, the prestressed specimens in which crack widths were limited to approximately 0.002 in. (0.05 mm) under load resulted in virtually no corrosion of nonprestressed reinforcement. In general, both the grease-filled plastic duct and the grouted galvanized duct adequately protected the prestressing tendons in the length between the anchorages."*

SECTION B—ANALYSIS

3.1 General

For spans in excess of 250 feet, results of elastic analyses should be evaluated with consideration of possible variations in the modulus of elasticity of the concrete, and variations in the concrete creep and shrinkage properties, as well as the impact of variations in the construction schedule on these (and other) design parameters.

3.2 Strut-and-Tie Models

Strut-and-tie models provide a means of analyzing areas near concentrated loads, bearing areas, diaphragms, corners, bends, openings, anchorage zones for post-tensioning tendons, and other areas where nonlinear strains exist, as well as the cracked global structural system. References 2, 3 and 4 on extension of this concept originally proposed by Mörsch in 1899 are included under References.

3.3 Effective Flange Width

The procedures of Section 3-10.2 of the 1983 Ontario Highway Bridge Design Code provide an equation for determining the effective flange width for use in calculating bending resistances and bending stresses.

3.3.2 Effective Flange Width for Analysis, and for Calculation of Section Capacity and Stresses

Note that b as used in this section is the flange width either side of the web. (b_1, b_2 or b_3 in Figure 3-3a).

The pattern of stress distribution in Figure 3-4 is intended only for calculation of stresses due to anchorage of post-tensioning tendons, and may be disregarded in the general analysis to determine design moments, shears and deflections.

Superposition of local slab bending stresses due to wheel loads (two-way slab action) and the primary longitudinal bending stresses is not normally required.

4.0 TRANSVERSE ANALYSIS

See references 9 and 10 for background on transverse analysis of concrete box girder bridges.

5.0 LONGITUDINAL ANALYSIS

5.1 General

Analysis of concrete segmental bridges requires consideration of variation of design parameters with time, as well as a specific construction schedule and method of erection. This, in turn, requires the use of a computer program developed to trace the time-dependent response of segmentally erected prestressed concrete bridges through construction, and under service loads. Among the many programs developed for this purpose, several are in the public domain, and may be purchased for a nominal amount.[14, 15, 16]

A comprehensive series of equations for evaluating the time-related effects of creep and shrinkage is presented in the ACI Committee 209 report, "Prediction of Creep, Shrinkage and Temperature Effects in Concrete Structures."[17] A procedure based on graphical values for creep and shrinkage parameters is presented in the CEB-FIP Model Code.[18] Comparisons of the effects of application of the provisions of references 17 and 18 are presented in the Appendix of the first edition of the AASHTO *Guide Specifications for Design and Construction of Segmental Concrete Bridges*, and in reference 14.

One research report[19] suggests that the ACI 209 predictions underestimate the creep and shrinkage strains for the large scale specimens used in segmental bridges. The ACI 209 creep predictions were consistently about 65 percent of the experimental results in these tests. The report suggests modifications of the ACI 209 equations based on the size or thickness of the members.

5.3 Analysis of the Final Structural System

The Appendix to the first edition of the AASHTO *Guide Specifications for Design and Construction of Segmental Concrete Bridges* presents results of analyses of a segmental concrete superstructure with values of the creep coefficient, ϕ_c, of 1, 2, and 3, and using both the ACI 209 and CEB-FIP creep models. Review of the results of these analyses indicates that final stresses are essentially unchanged for creep coefficients of 1, 2, and 3 using the ACI 209 creep provisions. Although the analyses with the CEB-FIP creep model show somewhat more variation in final stresses, the range of stresses is still small for a large variation in creep coefficients. The selection of the ACI 209 or CEB-FIP creep model has a larger impact on the final stress values than the creep coefficients. However, it is doubtful that the full range of stresses reflected in the six analyses would be of practical significance with respect to the performance of the structure. Since the creep coefficient will be known or determined with reasonable accuracy under the requirements of these specifications, analysis using a single value of the creep coefficient is considered satisfactory, and use of low and high values of the creep coefficient in analysis is generally considered unnecessary. However, further research is recommended to reconcile the differences in the ACI 209 and CEB-FIP creep models. Further, it is not intended to infer that creep values should not be determined accurately. These values do have significant impact on the prestress losses, deflections, and the axial shortening of the structure.

SECTION C—DESIGN

6.2 Dead Loads

The value of 155 pcf for normal weight concrete is intended to provide for more heavily reinforced sections than would be anticipated in more conventional concrete superstructures. The unit weight of lightweight concrete used in design calculations shall be stated in the General Notes of the Contract Drawings.

6.3 Erection Loads

Erection loads may be imposed on opposing cantilever ends by use of the Formtraveler, diagonal alignment bars, a jacking tower, or by external weights. Cooling of one cantilever with water has also been used to provide adjustment of misalignment. Any misalignment of interior cantilevers should be corrected at both ends before constructing either closure. The frame connecting cantilever ends at closure pours should be detailed to prevent differential rotation between cantilevers until the final structural connection is complete. The magnitude of closure forces should not induce stresses in the structure in excess of those tabulated in Table 7-2 in the design specifications (including the footnotes).

6.4.3 Thermal Coefficient

For major bridges, tests or use of previous test data to determine more precise thermal coefficients is recommended.

6.4.4 Temperature Gradient

The temperature gradients recommended in NCHRP Report 276 have been modified in the second edition of the segmental guide specifications based on review of field measurements of temperature gradients of segmental bridges. Further field measurements of temperature gradients in segmental concrete bridges are recommended.

Transverse analysis for the effects of differential temperature outside and inside box girder sections is not considered generally necessary. However, such an analysis may be necessary for relatively shallow bridges with thick webs.[10, 20, 21, 22] In that case, a plus or minus 10 degree Fahrenheit temperature differential is recommended. Additional field research is recommended to determine temperature differentials between the inside and outside surfaces of segmental concrete box girder sections in U.S. temperature zones.

6.5 Creep and Shrinkage

A variety of computer programs and analytical procedures have been published to evaluate creep and shrinkage effects in segmental concrete bridges.[14, 15, 16, 17, 18-23]

For permanent loads, the behavior of segmental bridges after closure may be approximated by use of an effective modulus of elasticity, E_{eff}, which may be calculated as:

$$E_{eff} = \frac{E_{cm}}{\phi_c}$$

where ϕ_c is the creep coefficient, and E_{cm} is the 28-day secant modulus of elasticity of the concrete calculated from:

$$E_{cm} = 1802\sqrt{f'_c}$$

where E_{cm} and f'_c are in ksi.

6.6 Post-Tensioning Force

Prestress losses vary significantly with different values of the creep coefficient, type of prestressing steel (low relaxation steel is recommended), and with the creep model (ACI 209 or CEB-FIP). Further, the prestress losses vary significantly at different sections along the superstructure.

7.2.2 Additional Thermal Loading Combination

Restrained deformations due to temperature change, shrinkage, creep, and support displacements induce sectional forces that are directly proportional to stiffness, and hence highly sensitive to any change in stiffness. At ultimate limit state, the sectional forces due to restrained deformations disappear completely with the formation of plastic hinges. Restrained deformations need therefore only be considered in systems of limited ductility, where the assumption of plastic deformations is not valid.[60] Segmental bridge research demonstrates that plastic hinges develop in segmental bridges designed in accordance with these guide specifications.[27, 55, 56]

7.3 Strength Reduction

The values of ϕ_f and ϕ_v presented in Section 7.3 have been modified based on research results.[24, 25, 26, 44, 55] A new design equation for shear capacity of dry joints has been introduced in Section 12.2.21, and a strength reduction factor for dry joints, ϕ_j, has been added to Section 7.3.

7.3.6

The proposed ϕ_f values for flexure for segmental bridges with fully bonded tendons with cast-in-place concrete joints, wet concrete joints or epoxy joints are based on the current AASHTO value of 0.95 for monolithic post-tensioned construction. Comprehensive tests[27] of a large continuous three-span model of a twin cell box girder bridge built from precast segments with fully bonded internal tendons and epoxy joints indicated that cracking was well distributed through the segment lengths, no epoxy joint opened at failure, and the load-deflection curve was identical to that calculated for a monolithic specimen. The complete ultimate strength of the tendons was developed at failure. The model had substantial ductility and full development of calculated deflection at failure. Tests[24, 25] on single span segmental girders with varied tendon arrangements (internal, mixed and external tendons) and with dry joints indicate that the deflection at failure was less than would be expected for monolithic girders. Flexural cracking concentrated at joints, and final failure came with a central joint opening widely and crushing occurring at the top of the joint.

The most recent tests of a large segmental bridge model[55] with external tendons and both Type A and Type B joints resulted in a flexural failure in the span with dry joints at factored loads of 1.3 DL+6.8 (LL+I), substantially above the AASHTO LRFD factored load requirements. Under service loads, the deflections were L/6000 for the exterior spans, and L/7500 for the interior span. A full scale test of a prototype test span of the Bangkok second stage expressway span-by-span design with dry joints and external tendons was reported in 1995.[56] The 45.25 m span had a deck width of 10.2 m and consisted of 14 segments. The span carried 118 percent of the capacity predicted by the AASHTO *Guide Specification for Design and Construction of Segmental Concrete Bridges* (first edition). The increase in stress in the unbonded tendons at failure was 389 Mpa (55,400 psi) as compared to the 103 Mpa (15,000 psi) value permitted by the AASHTO Guide Specifications. The midspan deflection at failure was 326 mm. The ϕ factors for flexure and shear in the second edition of the segmental guide specifications have been increased to reflect the results of the more recent tests discussed above, as well as the addition of the capacity reduction factor, ϕ_j, to provide for separate calculation of the shear capacity of dry joints. A comparison of shear behavior of various types of joints is presented in Fig. C-1.[26]

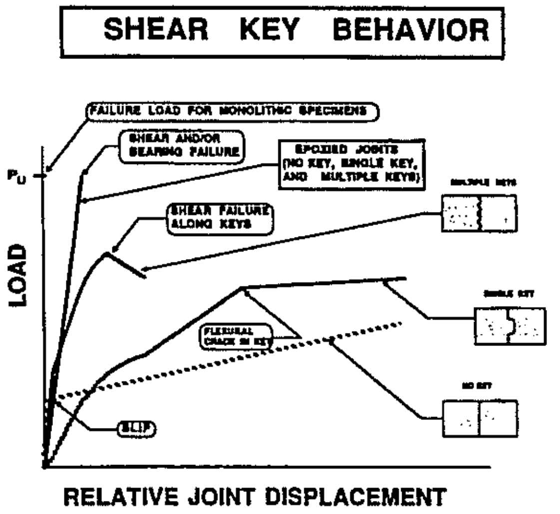


Fig. C-1[26]

8.0 SEISMIC DESIGN

The distinction between internal tendons and external tendons with respect to seismic behavior reflects the general condition that internal tendons are effectively bonded at all sections along the span, while external tendons are effectively bonded at only their anchorages and one or two intermediate deviators. Hence, the overall strength and ductility achieved with internal tendons are substantially larger than with external tendons. However, tests[61] have shown that by increasing the number of locations along the span where the external tendons are discretely bonded to the superstructure, substantial increases in ductility and strength are possible. In fact, when the external tendons were discretely bonded at ten points along the span, the ultimate strength and corresponding ductility of epoxy joint spans was 95 percent of that computed for spans with all internal tendons. Such discrete bonding of external tendons can easily be done in follow-up operations after erection as long as provisions are made in the design and casting phases.

9.1 Prestressing Steel

These allowable stresses in prestressing steel are in accordance with the ACI 318-95.[28] The maximum tendon jacking force of 0.80 f'_s is reduced from 0.85 f'_s for low relaxation strand in ACI 318-83. This change reflects concern about possible increased wire breakage at higher stressing levels, and the fact that jacking stresses of 0.85 f'_s are not compatible with the maximum stress of 0.74 f'_s immediately after prestress transfer, or the maximum stress of 0.70 f'_s at anchorages.

10.1 General

Although various degrees of approximation are involved in calculations of prestress losses, a number of time-related procedures for loss-calculation have been published which are quite accurate.[4, 16, 17, 18, 23, 32, 33, 34, 35, 45]

Elastic shortening losses may be calculated in accordance with methods presented in previously published guidelines.[32,46] Elastic shortening losses for external tendons may be calculated in the same manner as for internal tendons.

10.2 Duct Friction and Wobble

The friction and wobble coefficients shown for strand tendons in galvanized duct were developed for conventional cast-in-place box girder bridges based on job-site tests of various sizes and lengths of tendons. The values are reasonably accurate for 12 1/2 inch diameter strand tendons in 2 5/8 inch diameter galvanized metal sheathing. Tests and experience indicate that the values are conservative for larger tendons and duct diameters. However, experience with segmental concrete bridges to date has often indicated higher friction and wobble losses due to movement of ducts during concrete placement, and misalignment at segment joints. For this reason, in-place friction tests are recommended at an early stage in major projects as a basis for modifying friction and wobble loss values. No reasonable values for friction and wobble coefficients can be recommended to account for gross duct misalignment problems. As a means of compensating for high friction and wobble losses or for provisional post-tensioning tendons as well as other contingencies, additional ducts are required in accordance with Section 15.

See reference 36 for a general development of friction loss theory for bridges with inclined webs, and for horizontally curved bridges.

Field tests conducted on the external tendons of a segmental viaduct in San Antonio, Texas, indicate that the loss of prestress at deviators is higher than the usual friction coefficient ($\mu = 0.25$) would estimate.

This additional loss seems due in part to the tolerances allowed in the placement of the deviator pipes. Small misalignments of the pipes can result in significantly increased angle changes of the tendons at the deviation points. The inadvertent angle change of 0.04 radians added to the theoretical angle change accounts for this effect based on a typical deviator length of 3 feet and placement tolerance of ±3/8 inch. The 0.04 value is to be added to the theoretical value at each deviator. The value may vary with tolerances on pipe placement. This additional friction loss does not apply in cases where the deviation angle is strictly controlled or precisely known, as in the case of continuous ducts passing through separate longitudinal bell-shaped holes at deviators.

The measurements also indicated that the friction across the deviators was higher during the stressing operations than during the seating operations.

10.3 Anchorage Seating

For wedge-type strand tendons anchorage seating may vary between 1/8 inch and 3/8 inch, depending on the type of equipment used. For short tendons (less than 100 feet), a small anchorage seating value is desirable and equipment with power wedge seating should be used. For long tendons the effect of anchorage seating on tendon forces is insignificant, and power seating is not necessary. The 1/4 inch anchorage seating value assumed in the elongation computations is adequate but approximate only. Actual values of anchorage seating somewhat smaller or larger have insignificant influence on the tendon force for long tendons.

10.5 Stressing Hardware

The loss across stressing hardware and anchorage devices has been measured from 2 to 6 percent[44] of the force indicated by the ram pressure times the calibrated ram area. The loss varies depending on the ram and the anchor. An initial design value of 3 percent is recommended.

11.2 Stress in Unbonded Prestressing Steel at Factored Load

A recent paper[63] provides a general review of various formulas for stress in unbonded tendons at factored load.

11.4 Minimum Reinforcement

A comprehensive proposal for revision of the ACI minimum reinforcement requirements, including elimination of the 1.2 times the cracking moment provision, has been published in the *ACI Structural Journal*.[59]

12.1 Scope

All design for shear and torsion of prestressed concrete segmental bridges is based on ultimate load conditions because little information is available concerning actual shear stress distributions at working or service load levels.

Some engineers calculate principal tensile stresses in webs under service load in proportioning web thickness.

12.1.1

Regions with beam-type action are basically those where the Bernoulli hypothesis that linear strain profiles exist is valid.

12.1.2

Discontinuity regions, where the assumption that strain profiles are linear is invalid, usually exist for about a distance h from a concentrated load or point of geometrical discontinuity. Moving wheel loads need not be considered as large concentrated loads. The use of strut-and-tie models in design is well described in J. Schlaich, K. Schaefer, and M. Jennewein.[2] Note that a structure can be made up of both beam-type and discontinuity regions. The strut-and-tie model procedures must be used in the discontinuity regions. Either the traditional beam approach or the strut-and-tie approach can be used in the beam-type regions.

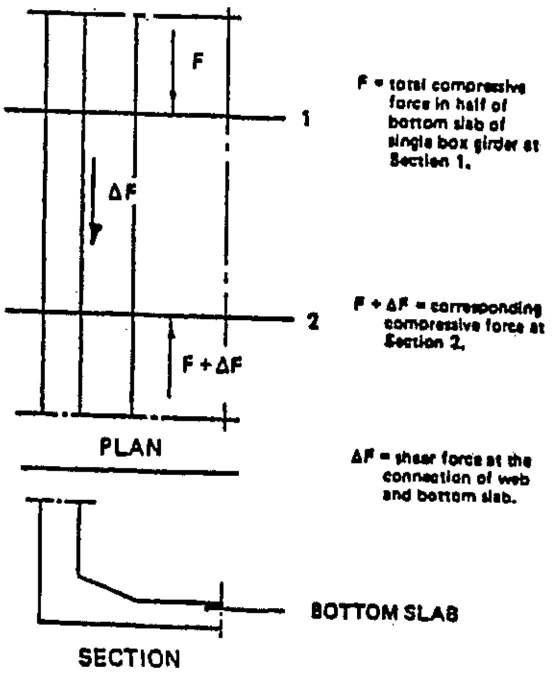


Fig. C-2. Longitudinal shear transfer by bottom slab to web haunches

12.1.3

In addition to the obvious checks for shear transfer when dissimilar materials are utilized, adequate shear transfer reinforcement must be provided perpendicular to the vertical planes of web/slab interfaces to transfer flange longitudinal forces at ultimate conditions. This shear transfer shall account for the shear force, ΔF, as shown in Figure C-2, as well as any localized shear effects due to prestress anchorages at that section.

Section 11.7 of ACI 318-95 is generally termed the "shear-friction" method but does provide in Section 11.7.3 that a wide range of shear transfer design methods may be utilized. In some cases, the designer may find the strut-and-tie method of Section 12.4 useful in proportioning transverse reinforcement to assist in transfer of horizontal shear between elements.

12.1.5

The shear effect of moving vehicle loads may be considered by development of maximum factored shear envelopes and the use of these values in determining the factored ultimate live load shear on the section.

Prestressing is considered as an applied load with a carefully controlled magnitude and direction. The components of the prestress force can add to or subtract from the shear on a cross section. In cantilevered segmental construction the prestress vertical component can reverse the applied shear direction near the supports.

12.2.6

The limitation on the effective diagonal tension and aggregate interlock components of shear strength contributed by the concrete has been recently adopted by ACI Committee 318.

12.2.12

A simplified determination of V_c is presented which eliminates the need to check V_{ci} and V_{cw} as in the present AASHTO Standard Specifications and which eliminates the complex $V_u d/M_u$ term. This expression has been checked against a wide range of test data and has been found to be a conservative yet simpler expression.

12.2.19

In place of requiring additional longitudinal reinforcement for shear as indicated by the mechanics of the truss model, the requirement of extending all flexural reinforcement beyond the theoretical bend or cut-off points for a distance of h/2 automatically satisfies this need. Since actual shear and tension may vary from the assumed calculation, it is also recommended that transverse reinforcement be provided for the same distance beyond the zone theoretically required.

12.2.20

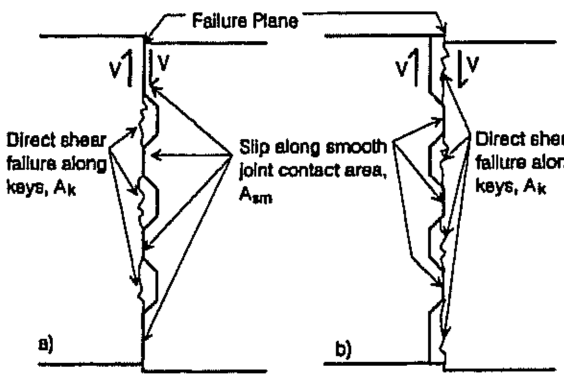
Preference for multiple shear keys is based on superior behavior in tests as illustrated in Figure C-1.

12.2.21

The joint shear capacity must be checked in dry jointed structures to ensure the integrity of the joint.

The equation was derived with guidance from work by Mattock[46], and confirmed by test data from the experimental programs of Koseki and Breen[26] and Bakhoum, Buyukozturk and Beattie[47]. The equation may also be used to determine the acceptable number of broken keys to be allowed before repair is deemed necessary.

Figure C-3 illustrates a typical failure plane of a keyed joint in shear. The areas of the base of the sheared keys, A_k, and the smooth areas, A_{sm}, are shown. The critical failure plane will have the greatest ratio of A_{sm} to A_k (this means the greatest area of slip and the least area of key breakage).



For the same joint configuration, the failure plane in a) is most critical because it contains a larger smooth contact surface and a smaller area of key breakage.

Fig. C-3. Joint shear failure plane

12.3 Traditional Shear and Torsion Design for Plane Section Type Regions

This section is a simplified version of the present AASHTO Standard Specification approach for section design in beam-type regions. It is based on the simplified V_c term introduced in Section 12.2.12. Provision of a "traditional" but less complex approach for beam-type regions is desirable since designers may find its application easier than strut-and-tie models for moving loads and because efforts to more precisely calculate the concrete shear contribution to shear capacity may involve design costs which exceed the potential savings in the cost of shear reinforcement.

12.3.1

In this Second Edition of the AASHTO *Guide Specifications for Design and Construction of Segmental Concrete Bridges*, a minor revision has been made in 12.3.1 to more correctly reflect realistic limits on shear reinforcement based on web crushing. In the 1989 AASHTO *Guide Specification for Design and Construction of Segmental Concrete Bridges*, a limit on $V_n = V_c + V_s$ of $10\sqrt{f'_c}b_w d$ (with f'_c in psi) with a maximum $\sqrt{f'_c}$ of 100 psi was adopted (curve D in Figure C-4). This value was chosen very conservatively using $V_c = 2\sqrt{f'_c}b_w d$ and $V_s = 8\sqrt{f'_c}b_w d$ which were traditional limits for reinforced concrete. In actuality, for shear limited by web compression in prestressed concrete, the 1996 AASHTO Standard Specification and earlier editions allow a value of $V_{cw} = 3.5\sqrt{f'_c}b_w d$ without any additional allowance for f_{pc} and V_p which would increase the value for prestressed members. In addition, it allows a maximum V_s value of $8\sqrt{f'_c}b_w d$. Thus, when web compression governs under the 1996 Standard Specifications, a value higher than $V_{cw} + V_s = 11.5\sqrt{f'_c}b_w d$ (curve C in Figure C-4) would apply. The relatively unsophisticated 1978 edition of the CEB-FIP Code allowed the much greater values shown by Curve A in Figure C-4. Substantial refinement in the 1990 CEB-FIP Code resulted in the values shown by Curve B in Figure C-4. Both CEB-FIP values include a ϕ factor so that they are very conservative approaches. Note that curves A, B, and C all allow substantially higher values than the 1989 Guide Specification (Curve D).

Assuming a minimal contribution of f_{pc} to the traditional V_{cw} term of $0.5\sqrt{f'_c}b_w d$ (psi units), the AASHTO Standard Specification maximum limit would be $V_{cw} + V_s = (3.5+0.5)\sqrt{f'_c}b_w d + 8\sqrt{f'_c}b_w d = 12\sqrt{f'_c}b_w d$. Thus, V_n max would be $12\sqrt{f'_c}b_w d$ to protect against web crushing. Experience with this limit in the Standard Specifications has shown it to be conservative and hence the Revised 1997 Guide Specifications for Segmental Bridges uses a limit on $V_n = 12\sqrt{f'_c}b_w d$ (psi) or $0.38\sqrt{f'_c}b_w d$ in ksi units.

12.3.8

In determining the required amount of longitudinal reinforcement, the beneficial effect of longitudinal prestressing may be taken into account by considering it equivalent to an area of reinforcing steel with a yield force equal to the effective prestressing force.

13.1 Fatigue Stress Limits for Bonded Nonprestressed Reinforcement

Calculation of fatigue stress limits in bonded reinforcement is necessary only for cracked sections under service loads. In segmental bridge superstructures designed in accordance with these specifications, cracking will occur at load levels substantially above service load.

13.2 Fatigue Stress Limits for Prestressed Reinforcement

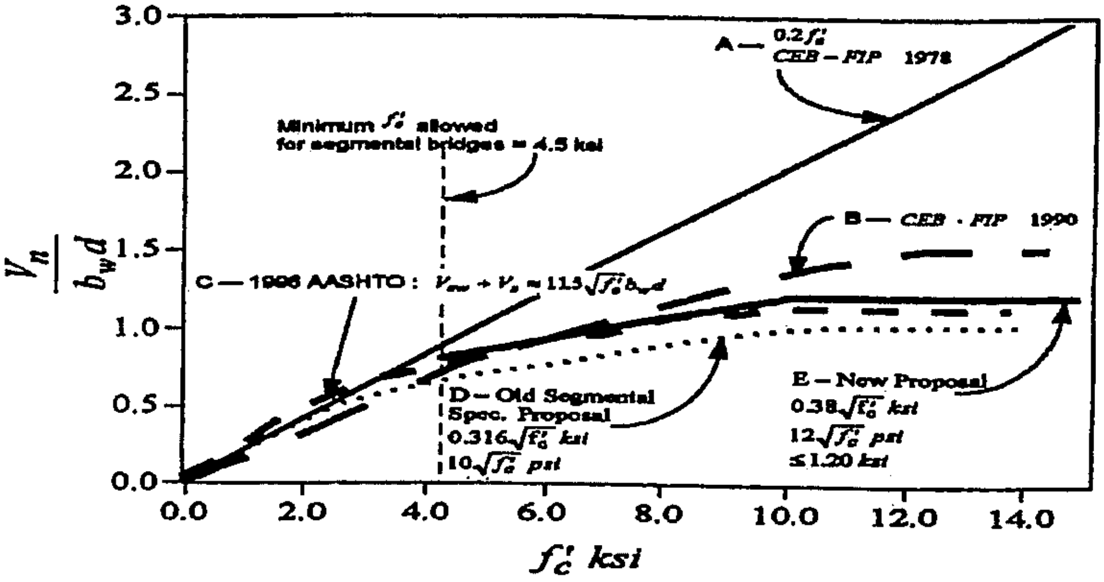


Fig. C-4. Maximum shear limits based on web compression

Bridges designed under the allowable stresses of this specification should be uncracked at service load levels. Fatigue of prestressed reinforcement will not occur in uncracked sections due to the related small stress range. Fretting fatigue due to rubbing between duct and strand also does not occur in uncracked sections.

14.0 POST-TENSIONED ANCHORAGE ZONES

Comprehensive anchorage zone design recommendations are included in the AASHTO *Standard Specifications for Highway Bridges*. These recommendations are adopted by reference in the second edition of the segmental guide specifications

15.2 Bridge with Internal Ducts

Excess capacity may be provided by use of oversize ducts and oversize anchorage hardware at selected anchorage locations.

15.3 Provision for Future Dead Load or Deflection Adjustment

This provides for future addition of internal unbonded post-tensioning tendons draped from the top of the diaphragm at piers to the intersection of the web and flange at midspan. Tendons from adjacent spans are to be lapped at opposite faces of the diaphragm to provide negative moment capacity. The requirement of a force of 10 percent of the positive moment and negative moment post-tensioning force is an arbitrary value. Provision for larger amounts of post-tensioning might be developed as necessary to carry specific amounts of additional dead load as considered appropriate for the structure.

16.6.1 Metal Ducts

Thickness of metal duct material is related to duct diameter and the method of installing the tendon. Strand tendons are normally installed in the duct after the concrete is placed, requiring a stiffer duct. Bar tendons are normally preassembled inside small diameter ducts and placed as a unit. The bar fills most of the void and helps to prevent duct damage. The use of epoxy coated metal duct is not recommended due to questionable bond characteristics.

16.3 Minimum Radius of Curvature

Polyethylene duct abrades at curvature radii less than 30 feet.

16.4.1 Internal Ducts

It is recommended that duct support requirements be stipulated or shown in the contract documents.

16.4.2 External Ducts

External ducts are normally polyethylene.

16.6.2 Ducts in Flanges

The hairpin bars tie the slab together in event of spalling forces at slab joints.

Ducts spaced closer than 12 inches on center in either direction should be considered as closely spaced.

The hairpin bars are provided to prevent slab delamination along the plane of the post-tensioning ducts. The hairpin bars are not required in areas where duct congestion does not exist.

17.0 COUPLERS

European experience indicates that the prestressing force decreases locally in the region of a coupler. This is believed to result partially from increased creep caused by high compressive stresses in the reduced concrete section due to coupling of tendons. Cracking has not been observed in bridges where the number of tendons coupled at a section has been limited to 50 percent of the total number of tendons.

18.0 CONCRETE COVER AND REINFORCEMENT SPACING

18.1 Cover and Spacing

At least 2 inches of cover is recommended for all reinforcement in bridge decks exposed to use of de-icer chemicals or other corrosive conditions.

18.2 Reinforcement Details for Erection Loads

The quantity Z provides reinforcement detailing that will reasonably control flexural cracking. Crack potentials are largest when handling and storing segments for precast construction and when stripping forms and supports from cast-in-place construction.

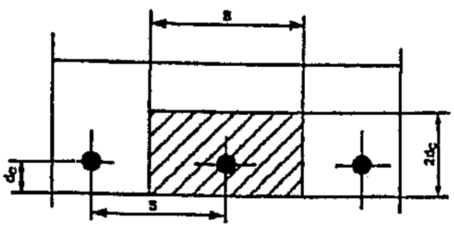
The value of Z is calculated as:

$$Z = f_s x \sqrt[3]{d_c A}$$

where f_s = stress in non-prestressed reinforcement ksi

A = area of concrete surrounding a bar = 2 x d_c x s (bar spacing), square inches

d_c = thickness of cover from tension fiber to center of b



20.1 Minimum Flange Thickness

A top flange thickness of 9 inches is preferable in the area of anchorages for transverse post-tensioning tendons. A minimum flange thickness of 8 inches is recommended.

20.2 Minimum Web Thickness

Ribbed webs may be reduced to 7 inch thickness.

20.4 Overall Cross Section Dimensions

With four lanes of live load (only three lanes required by live load reduction factors) the live load deflection of the model of the Corpus Christi Bridge was approximately L/3200 in the main span. The deflection limit of L/1000 was arbitrarily chosen to provide guidance as to the maximum live load deflections anticipated for segmental concrete bridges with normal dimensions of the box girder cross section.

Girder depth and web spacing determined in accordance with the following will generally provide satisfactory deflection behavior:

A. Constant depth girder

$$1/5 > d_o/L > 1/30$$
optimum 1/18 to 1/20

where d_o = girder depth, ft.

L = span length between supports, ft.

In case of incrementally launched girders the girder depth should preferably be between the following limits:

L = 100',	1/15	$< d_o/L$	<1/12
L = 200',	1/13.5	$< d_o/L$	<1/11.5
L = 300',	1/12	$< d_o/L$	<1/11

B. Variable Depth Girder with Straight Haunches at pier $1/16 > d_o/L > 1/20$

optimum 1/18

at center of span $1/22 > d_o/L > 1/28$

optimum 1/24

(Note: a diaphragm will be required at the point where the bottom flange changes direction.)

C. Variable Depth Girder with Circular or Parabolic Haunches at pier $1/16 > d_o/L > 1/20$

optimum 1/18

at center of span $1/30 > d_o/L > 1/50$

D. Depth to Width Ratio

A single cell box should preferably be used when

$$d_o/b \geq 1/6$$

A two cell box should preferably be used when $d_o/b < 1/6$

where b = width of the top flange.

If in a single cell box the limit of depth to width ratio given above is exceeded, a more rigorous analysis is required and may require longitudinal edge beams at the tip of the cantilever to distribute loads acting on the cantilevers. An analysis for shear lag should be made in such cases. Transverse load distribution is not substantially increased by the use of three or more cells.

23.3 Longitudinal Reinforcement of Hollow, Rectangular Precast Segmental Piers

Research conducted at the University of Texas at Austin by A. W. Taylor, R. B. Rowell and J. E. Breen on the subject, *Design and Behavior of Thin Walls in Hollow Concrete Bridge Piers and Pylons*, provides guidance for these members.[53] This test program, however, was limited to the case of loading under simultaneous axial and uniaxial bending about the weak axis of the section. The results of the study have not been confirmed for the case of biaxial bending. Until such study is completed, the designer should investigate the effects of biaxial loading on hollow sections.

Minimum longitudinal reinforcement of hollow, rectangular precast segmental piers is based on the *AASHTO LRFD Bridge Design Specifications*, Section 5.10.8, for shrinkage and temperature reinforcement. This provision reflects the satisfactory performance of segmental piers constructed between 1982 and 1995 with this amount of longitudinal reinforcement.· (See Table C-1). In this context, it is noted that the discontinuous longitudinal bars in precast-segmental piers do not carry significant loads. Tensile reinforcement of precast segmental piers is provided by post-tensioning tendons.

Table C-1. Longitudinal Reinforcement in Completed Bridges with Precast Segmental Piers

[illegible]	[illegible]	Wall Thickness	[illegible]
James River (1990)	#3's @ 10" E.F.	10"	0.0020
Sunshine Skyway (1987)	#4" @ 12" E.F.	24"	0.0014
Wando River (1989)	#3's @ 10" E.F.	10"	0.0020
Neches River (1990)	#3's @ 10"E.F.	10"	0.0020
Linn Cover Viaduct (1983)	#3's @ 9" E.F.	12"	0.0019
Seven Mile Bridge (1982)	#4's @ 12"	10"	0.0020
C&D Canal (1995)	#4's @ 12" E.F.	12"	0.0028
Albemarle Sound (1989)	#3's @ 10" E.F.	12"	0.0017
Escatawpa River (1987)	#4's @ 10"	8"	0.0025
Choctawhatchee Bay (1993)	#3's @ 9.5"E.F.	12"	0.0018

SECTION D—SPECIAL PROVISIONS FOR BRIDGE TYPES

24.1 General

Bridges erected by balanced cantilever or progressive placement normally utilize internal tendons. Bridges built with erection trusses may utilize internal tendons, external tendons, or combinations. Due to considerations of segment weight, span lengths for precast segmental box girder bridges rarely exceed 400 feet. Large stress changes on epoxy joints should be avoided during the initial curing period.

24.2.2 Erection

One agency has partially stressed closure pours at cylinder strengths as low as 500 psi in an attempt to avoid cracking due to thermal movement of the superstructure. In such cases, this agency has limited the compressive stress on the green concrete to 0.15 f'_{ci}.

24.2.3 Temporary Stress on Epoxy Joints

Closure stresses should be as uniform as possible to avoid epoxy thickness variations, especially if this could lead to a systematic accumulation of geometric error. Large stress changes on epoxy joints should be avoided during the initial curing period.

24.4.2 Type A Joints

Epoxy serves as a lubricant during placement of segments, provides waterproofing of the joints for durability, and provides a seal to avoid cross-over during grouting of internal tendons, and provides some tensile strength across the joint. (See Figure C1).

24.5 External Tendons

Some early external tendon bridges with unprotected tendons experienced substantial problems due to corrosion of the tendons.[40] Modern bridges with external tendons have successfully utilized cement grout in polyethylene tubing or in metal tubing. Tendons have also been protected by heavy grease or other anti-corrosion medium if future replacement is envisioned.[41] Tendon anchorage regions must be suitably protected against corrosion by encapsulation or other effective means. This is of critical importance in unbounded construction since any failure of the anchorage can release the entire tendon.

25.4 Form Traveler Weight

A form traveler for a typical two lane bridge with 15 to 16 feet segments may be estimated to weigh 160 to 180 kips. Weight of form travelers for wider double-celled box sections may range up to approximately 280 kips. Consultation with contractors or subcontractors experienced in free cantilever construction with respect to the specific bridge geometry under consideration is recommended to obtain a design value for form traveler weight.

25.5 Segment Lengths

Segments cast on falsework are usually 40 to 60 feet in length. Segments for free cantilever construction usually range between 10 and 16 feet.

27.1 General

AASHTO Type III and Type IV segmental beam designs have been proposed for spans up to 180 ft.[42] Longer segmental beam bridge spans (up to 250 ft.) are feasible with deeper Bulb Tee sections.

27.3.1 Cast-in-Place Joints

Cast-in-place joints eliminate the need for match-casting of segments.

DIVISION II
CONSTRUCTION SPECIFICATIONS

1.0 GENERAL

Cast-in-place and precast segmental construction shall conform to applicable provisions of Division II — Construction of the current edition of the AASHTO Standard Specifications except as expressly modified or revised in the following provisions. Where reference is made to the "Standard Specifications" the reference is to the current AASHTO Division II, Construction Specifications.

2.0 CONCRETE

2.1 General

Concrete shall conform to applicable provisions of Division II, Section 4 of the AASHTO Specifications and the additional provisions presented below.

2.2 Concrete Quality

Concrete proportioning, production and testing shall conform to the requirements of Chapter 4 of the ACI 318-86(I) Building Code. The water content of the concrete mix shall be the minimum consistent with the necessary workability for concrete placement. The maximum water-cement ratio shall not exceed 0.45.

2.3 Test Cylinders

All test cylinders representing tests for removal of forms and/or falsework and for "Release Strength" shall be cured under the same conditions, be subjected to the same curing materials and to the same weather conditions as the concrete represented. Coupled maturity meters may be used, in accordance with ASTM C 1074, to determine concrete strength for removal of forms and/or falsework. "Design Strength" cylinders for acceptance of concrete shall be cured with the member which the cylinders represent and transported to the laboratory as soon as handling strength is obtained. These cylinders shall then be cured for the remainder of the test period in accordance with AASHTO T 23 (ASTM C 31). Compressive strength of test cylinders shall be evaluated in accordance with Chapter 5 of the ACI 318-95 Building Code.

2.4 Tests for Modulus of Elasticity, and Creep and Shrinkage Coefficients

In most cases, values of modulus of elasticity and creep and shrinkage coefficients can be estimated with sufficient accuracy by reference to the ACI Committee 209 Report[7] or the CEB-FIP Model Code for Concrete Structures.[8] For large projects involving bridges sensitive to creep and shrinkage effects, and for bridges constructed of sand-lightweight concrete, tests shall be performed in accordance with the provisions of this section to determine concrete modulus of elasticity, and creep and shrinkage coefficients for the selected mix design. The test data shall be obtained at the earliest possible time during the contract period for use in adjusting the design values and the related calculations of structural deflection and geometry control.

The Contractor shall prepare concrete specimens in the presence of the Engineer and shall arrange for shipment to an approved laboratory for testing when handling strength is obtained. All specimens in a given sample shall be taken from the same batch of concrete. The report of the test results shall be in accordance with the following sections.

2.4.1 Modulus of Elasticity Tests

Tests shall be performed in accordance with the requirements of ASTM C 469. Ages of specimens (cylinders) at time of test shall be 3, 28 and 90 days. Number of specimens per test shall be three cylinders or a total of nine cylinders. All specimens in a given sample shall be taken from the same batch of concrete.

2.4.2 Creep and Shrinkage Test

Tests shall be performed in accordance with requirements of ASTM C 512.

Ages of specimens (cylinders) at time of initial loading shall be 3, 28 and 90 days.

Number of specimens per test shall be in accordance with ASTM C 512.

Duration of load shall be 90 days.

Specimens shall be cured and stored in accordance with the standard curing requirements of Section 6.1 of ASTM C 512, except that the specimens shall be moist cured for a period of 14 days or until age of test, whichever comes first. Thereafter, specimens shall be stored at 73°F and 50 percent humidity

2.4.3 Test Results

Certified results of the above specified tests shall be furnished to the Engineer within one week from the time of the performance of each test.

2.5 Admixtures

Admixtures shall conform to the provisions of Section 3.6 of the ACI 318-95 Building Code. The use of admixtures shall be rigidly controlled to avoid undesirable variations in strength and modulus of elasticity of the concrete.

2.6 Vibration of Concrete

All concrete shall be compacted and thoroughly consolidated to the surface of the forms by working with approved high frequency mechanical vibrators. Mechanical vibration shall conform to Section 8.7.3 of the AASHTO Specification.

2.7 Construction Joints

Construction joints shall have roughness of 1/4 inch amplitude. Surface laitance shall be removed with water or sand blasting.

2.8 Accelerated Curing with Low Pressure Steam or Radiant Heat

Accelerated curing of concrete with low pressure steam or radiant heat shall be in accordance with AASHTO Specifications, Division II, Section 8.11.3.5, with the following modification: The last sentence of the fourth paragraph of Section 8.11.3.5 shall be deleted and the following added:

After attaining the desired strength, the temperature within the curing enclosure shall be decreased at an average rate not exceeding 40°F per hour until the inside temperature is within 20°F of the outside ambient temperature. The curing enclosure shall be of such size as to totally enclose both the cast against segment and the segment being cast, and any rigid appendages thereto.

2.9 Curing for Cast-in-Place Segmental Construction

Cast-in-place segmental concrete structures shall be cured with transparent, self-degrading curing compound, by water curing; by forms insulated with styrofoam and provided with heat cables, activated if need be; or some other method that has been submitted and approved by the Engineer.

3.0 NON-PRESTRESSED REINFORCEMENT

3.1 General

Non-prestressed reinforcement shall be deformed reinforcement, except that plain reinforcement may be used for spirals. Reinforcing bars to be welded shall be indicated on the drawings and welding procedure to be used shall be specified. ASTM reinforcing bar specifications, except for ASTM A 706, shall be supplemented to require a report of material properties necessary to conform to welding procedures specified in "Structural Welding Code-Reinforcing Steel" (AWS DI.4) of the American Welding Society.

3.2 Deformed Reinforcement

3.2.1 Deformed reinforcing bars shall conform to one of the following specifications:

a. Specification for Deformed and Plain Billet Steel Bars for Concrete Reinforcement, including Supplementary Requirements SI (AASHTO M 31 or ASTM A 615 including SI).

b. Specification for Low-Alloy Steel Deformed Bars for Concrete Reinforcement (ASTM A 706). Bars shall be free of loose rust, form oil, dirt or other contaminants.

3.2.2 Deformed reinforcing bars with a specified yield strength f_y exceeding 60,000 psi may be used, provided f_y shall be the stress corresponding to a strain of 0.35 percent and the bars otherwise conform to one of the ASTM specifications listed in Section 3.2.1.

3.2.3 Welded deformed wire fabric for concrete reinforcement shall conform to the "Specification for Welded Deformed Wire Fabric for Concrete Reinforcement" (AASHTO M 221 or ASTM A 497), except that for wire with a specified yield strength f_y exceeding 60,000 psi, f_y shall be the stress corresponding to a strain of 0.35 percent. Welded intersections shall not be spaced farther apart than 16 inches in the direction of calculated stress.

3.2.4 If required by the contract documents, reinforcing bars shall be galvanized or epoxy coated in accordance with "Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement" (ASTM A 767) or "Specification for Epoxy-Coated Reinforcing Steel Bars" (ASTM D 3963 or AASHTO M 284). Zinc or epoxy-coated reinforcing bars shall conform to one of the specifications listed in Section 3.2.1.

3.3 Plain Reinforcement

3.3.1 Plain bars for spiral reinforcement of post-tensioning tendon anchorage zones or for use as column reinforcement shall conform to the specifications listed in Section 3.2.1.

3.3.2 Smooth wire for spiral reinforcement of post-tensioning tendon anchorage zones or for use as column reinforcement shall conform to "Specification for Cold-Drawn Steel Wire for Concrete Reinforcement" (ASTM A 82), except that for wire with a specified yield strength f_{sy} exceeding 60,000 psi, f_{sy} shall be the stress corresponding to a strain of 0.35 percent.

3.4 Substitution of Grade or Type of Reinforcement

Non-prestressed steel reinforcement shall normally be Grade 60 as required by the design. Grade 40 may be substituted on approval of the Engineer. If Grade 40 is substituted, an increase of 50 percent in area of reinforcing shall be provided over that shown on the plans for Grade 60. If approved by the Engineer, welded wire fabric may be substituted for reinforcement bars on the underside of the cantilever roadway flange and in the bottom flange of the box girder provided the area of steel supplied is equivalent to or greater than that detailed on the contract plans. Any substitution made in the grade of nonprestressed steel reinforcement shall be clearly shown on the shop drawings.

3.5 Special Corrosion Protection

Special corrosion protection shall be provided for the nonprestressed reinforcement for bridges in corrosive environments, in accordance with requirements of the contract documents.

4.0 POST-TENSIONING MATERIALS

4.1 General

Post-tensioning materials shall conform to the PostTensioning Institute "Guide Specification for Post-Tensioning Materials,"[3] with the exception of special anchorage assemblies which shall comply with the provisions of Section 4.2. Additional requirements for post-tensioning materials shall be in accordance with the provisions of Sections 4.3, 4.4 and 4.5.

Except for special anchorage devices tested in accordance with Section 4.2, the Contractor shall furnish for testing one specimen of each size of prestressing tendon, including couplings, of the selected type, with end fittings attached, for strength tests only. These specimens shall be 5 feet 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. 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.

4.2 Special Anchorage Devices

Special anchorage devices shall be tested in accordance with AASHTO Standard Specifications Section 9.21.2.3.2

4.3 Identification and Testing of Tendon Material

All strand from each manufactured reel and all bars of each size from each mill heat to be shipped to the site shall be assigned a lot number and shall be tagged in such a manner that each lot can be positively identified at the job site. Each reel of prestressing 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 the chemical composition, the yield and ultimate strengths, elongation at rupture, modulus of elasticity, and the stress strain curve for the actual prestress reinforcing intended for use. All values certified shall be based on test values and actual sectional areas of the material being certified. All unidentified prestressing steel received at the site will be rejected and loss of positive identification prior to installation of these items at any time will be cause for rejection.

Samples from each size and each heat of prestressing bars and from each manufactured reel of prestressing steel strand shall be furnished to the Engineer for testing. With each sample of prestressing strand or bar, there shall be included therewith a manufacturer's certification stating the minimum guaranteed ultimate tensile strength of the sample furnished. The test samples shall be 7 feet long. If results of tests indicate the necessity of check tests, additional specimens shall be furnished without cost. Samples shall be identified by lot number.

Samples shall be submitted in ample time to allow for testing, for tabulating results and, if necessary, in case of unsatisfactory finds, to call for and retest substitute samples. The Contractor shall have no claim for additional compensation because of delay while awaiting approval of the materials furnished for testing. The Engineer shall be allowed a period not less than 21 calendar days prior to the beginning of installation to perform the tests and to provide approval of the materials furnished.

Relaxation characteristics of strand may be evaluated by a 30 minute test.[5] Low-relaxation strand subjected to the 30 minute test shall be required to show a relaxation loss of not more than 1.20 percent of the initial tension with an adjustment for test temperatures over 72°F. If the initial 30 minute test exceeds 1.20 percent, two additional tests shall be made of strand from the same pack and both tests shall be equal to or less than 1.20 percent for acceptance. The 30 minute test shall be used only for the purpose of identifying low relaxation strand for conformance with ASTM Specification A 416, and shall not be interpreted in reference to the magnitude of the 1000 hour steel relaxation loss.

4.4 Protection of Post-Tensioning Materials

All prestressing steel shall be protected against physical damage and rust or other results of corrosion at all times from manufacture to grouting. See Table 5-1 for permissible intervals between tendon installation and grouting without use of a corrosion inhibitor. 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, when ordered by the Engineer. Prestressing steel exhibiting rust stains may be examined, to determine if pitting exists, prior to use.

Prestressing strand 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 strand. The corrosion inhibitor shall have no deleterious effect on the strand or concrete or bond strength of steel to 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 strand, and the type of corrosion inhibitor used, including the date packaged.

Bar tendons shall be protected from corrosion at the job site by spraying with a rust preventative, and by use of an approved protective covering over the bars.

4.5 Duct Materials and Placement

4.5.1 Metal Ducts

Metal ducts shall be the corrugated galvanized semi-rigid conduit. For strand tendons, the duct thickness shall be as follows: 26 gauge for ducts less than or equal to 2 5/8 inches diameter, 24 gauge for ducts greater than 2 5/8 inches diameter. For bar tendons, the duct thickness shall not be less than 31 gauge.

Duct shall be sized to comply with the provisions of AASHTO Section 9.26.4. Placement of tendons by the pull-through method requires duct area of 2 1/2 times the prestressing steel area.

4.5.2 Polyethylene Duct

Corrugated polyethylene duct used as internal duct shall be high density polyethylene and conform to the requirements of ASTM D 3350 with ultra violet stabilizers added. Rigid polyethylene pipe used for external duct shall conform to the requirements of ASTM D 2447, Grades P33 or P34, D 3350 or F 714 with a Cell Classification of PE345433C. Material thickness shall be as follows:

a. Corrugated, internal polyethylene duct = 0.050 inches ± 0.010 in.

b. Smooth, external polyethylene duct shall have an external diameter to wall thickness ratio of 21 or less.

Internal ducts shall be mortar tight, and capable of withstanding concrete pressures without excessive deformation or permitting the entrance of cement paste during the placing of concrete. The ducts shall have sufficient rigidity to maintain the required profile between points of supports.

4.5.3 Steel Pipe Deviators

Steel pipe duct shall be galvanized steel pipe conforming to the requirements of ASTM A 53, Type E, Grade B. The nominal wall thickness of the pipe shall be not less than 1/8 inch. The pipe shall be bent so as to accurately conform to the alignment of the tendon, taking into consideration the minimum bending radius shown in the shop drawings.

4.5.4 Duct Placement

Duct shall be rigidly supported at the proper location in the forms by ties to 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 feet. Polyethylene duct in webs for longitudinal post-tensioning shall be tied to stirrups at intervals not to exceed 2 feet, and metal duct for longitudinal post-tensioning in webs shall be tied to stirrups at intervals not to exceed 4 feet. 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 feet into the corresponding duct of the previously cast segment. The mandrels shall be of sufficient rigidity to maintain the duct geometry within a 1/2 inch tolerance within the top flange, 1 inch in webs, and within a 1/8 inch tolerance at the segment joints. High point grout vents shall be provided for continuous longitudinal tendons over 400 feet long. Low point grout vents shall be provided for ducts of continuous longitudinal tendons or for ducts of web tendon in areas where freezing weather can be anticipated. Low point vents shall remain open until grouting is started.

5.0 INSTALLATION AND STRESSING OF TENDONS

5.1 Installation of Tendons

The total number of strand in an individual tendon may be pulled through the duct as a unit, or individual strand may be pulled or pushed through the duct.

Prestressing steel which is installed in members prior to placing and curing of the concrete, shall be continuously protected against rust or other corrosion, until grouted, by means of a corrosion inhibitor placed in the ducts or applied to the steel in the duct.

When steam curing is used, prestressing steel for longitudinal post-tensioning shall not be installed until the steam curing is completed. Prestressing steel for transverse top flange post-tensioning, transverse bottom flange post-tensioning, or for vertical web post-tensioning may be in place during steam curing. 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.

5.2 Corrosion Protection of Tendons Prior to Grouting

Prestressing steel installed in members prior to placing and curing of the concrete, or installed in the duct but not grouted within the time limit specified in Table 5-1, 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 encasement in concrete. Prestressing steel installed and tensioned in members after placing and curing of the concrete and grouted within the time limit specified in Table 5-1 will not require the use of a corrosion inhibitor described herein.

5.3 Corrosion Protection of Anchorage Devices

Within 15 days after the ducts are completely grouted, all blockouts for anchorages shall be grouted or filled with a non-shrink grout, a special mortar to protect the anchorage, or protected by other approved means. The minimum strength of grout or mortar used for filling anchorage recesses shall be the same as the strength of the concrete in the adjacent portion of the structure. If blockouts are visible on the sides or bottom of the box girders, the grout or mortar used shall match the color and texture of the concrete of the box girder.

5.4 Jack Calibration

Within 30 days prior to stressing, the Contractor shall calibrate all jacks and gauge systems to be used in the work and shall furnish the Engineer certified copies of load calibration curves. Each jack and its gauge shall be calibrated as a unit or checked against a master gauge with the cylinder extension in the approximate position it will be in at the final jacking force. Stressing equipment shall be recalibrated at six month intervals and at other times when required by the Engineer. Jacks and gauges shall not be interchanged without recalibration or proof loading using load cells, master gauges or other methods approved by the Engineer.

Permissible Intervals between Tendon Installation and Grouting without Special Corrosion Protection of Post-Tensioning Steel

Exposure	Permissible Intervals between Tendon Installation and Grouting without Use of a Corrosion Inhibitor
Very Damp Atmosphere or over salt water (Humidity > 70%)	7 days
Moderate Atmosphere (Humidity < 70% > 40%)	20 days
Very Dry Atmosphere (Humidity < 40%)	40 days

Table 5-1

5.5 Stressing Equipment

All strands of tendons comprised of more than four 1/2 inch or 0.6 inch diameter strands shall be stressed simultaneously with a multi-strand jack. The Contractor shall provide a positive means of marking total elongation on the strands. Jacks shall be equipped with proper ports or windows for adequate visual examination and measurement of tendon movement. They shall also be capable of slow release of stress to allow relaxation from temporary overstress to the proper seating force. All prestressing tendons shall be tensioned by the use of equipment allowing actual elongation to be measured directly and using a hydraulic ram equipped with a method of determining the tensioning force applied using either a gauge measuring the internal hydraulic pressure in the ram or force exerted by the ram, or a load cell. Readings taken from any one of these gauges shall be converted to actual tensioning forces through the use of calibrated values from a calibration chart. All gauges shall be of sufficient size and adequately made to allow readings to be made to an accuracy of 100 psi.

If a pressure gauge is used, it shall have an accurately reading dial at least 4 1/2 inches in diameter. If a load cell is used, it 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 10 percent of the manufacturer's rated capacity will not be used in determining the jacking stress.

5.6 Modulus of Elasticity of Tendon Material

The modulus of elasticity of strand or bar tendons shall be based on test reports on single strand or bar elements supplied by the strand producer, and/or on tests performed on single strand or bar elements by a qualified laboratory with experience in testing post-tensioning materials. The modulus of elasticity test and elongation calculations shall be based on the nominal area of the strand or bar.

5.7 In-Place Friction Test

When specified by the Engineer, the Contractor shall test early in the project, in place, two representative tendons of each size and type shown on the plans, for the purpose of accurately determining the friction loss in a strand and/or bar tendon.

The test procedure shall consist of stressing the tendon at an anchor assembly with load cells at the dead end and jacking end. The test specimen shall be tensioned to 80 percent of ultimate in 10 increments. For each increment, the gauge pressure, elongation and load cell force shall be recorded. The data shall be furnished to the Engineer. The theoretical elongations and post-tensioning forces shown on the post-tensioning shop drawings shall be re-evaluated by the Contractor using the results of the tests and corrected as necessary. Revisions to the theoretical elongations shall be submitted to the Engineer for evaluation and approval. Apparatus and methods used to perform the tests shall be proposed by the Contractor and be subject to the approval of the Engineer.

Testing by the Contractor shall not be paid for separately but shall be incidental to the price paid for the various prestressing steel items.

5.8 Tendon Force and Elongation Tolerances

Elongation shall be measured to an accuracy of ±1/16 inch. Elongation tolerances for individual tendons based on friction coefficients determined in accordance with Section 5.7, and based on material properties determined from laboratory tests, shall be within ±7 percent of the theoretical value, or as deemed necessary by the Owner.

When friction must be reduced on post-tensioning tendons, water soluble oil or graphite may be used subject to the approval of the Engineer. This oil or graphite shall be flushed from the duct by use of water under pressure or as part of the grouting process. Water used to flush ducts may contain slack lime (calcium hydroxide) or quick lime (calcium oxide) in the amount of 0.1 pounds per gallon. After flushing, excess water shall be blown out of the ducts with compressed air, and the ducts shall be grouted immediately to prevent the formation of rust.

The total post-tensioning force anchored at a segment face shall be within ±5 percent of the design value. Adjustments of the tendon force shall be made at not more than three segment intervals using provisional post-tensioning ducts and anchorages so that the total force and eccentricity provided at each location along the span is not less than the value required by the design plans. Two percent wire failures in individual tendons shall be permitted without replacement.

5.9 Stressing Sequence

Tendons shall be stressed in accordance with the sequence shown on the shop drawings for post-tensioning materials. The stressing sequence shall be such that not more than one tendon shall be eccentric about the centerline of the segment at any time, and the lateral bending stresses imposed on the structure shall not exceed $6\sqrt{f'_c}$.

5.10 Post-Tensioning Supervision

The Contractor shall provide qualified personnel skilled in prestressing systems to supervise or provide appropriate surveillance of the work and give the Engineer such pertinent information as he or she may require for inspecting the work. Such personnel shall be available full-time on all days during which the stressing and grouting of tendons is in progress.

A record of gauge pressures and tendon elongations for each tendon shall be provided by the Contractor for review and approval by a representative of the Engineer of Record. Stressing tails of tendons shall not be cut off until the stressing records have been approved.

6.0 GROUTING

Grouting shall be in accordance with the Post-Tensioning Institute's "Recommended Practice for Grouting of Post-Tensioned Prestressed Concrete."[3] If any grouting is done before any span is completely tensioned, great care shall be exercised to assure that grout does not cross over into any duct.

It is recommended that grouting be delayed until the entire span or area is stressed. During this period, the Contractor shall take all necessary precautions to prohibit any foreign material from entering the ducts, and to prevent corrosion of the prestressing steel. Corrosion protection of tendons prior to grouting shall be in accordance with Section 5.2.

7.0 EPOXY

Epoxy material, placement, inspection and testing shall be in accordance with AASHTO Materials Specification M 235, and with Division II, Construction, Sections 9.2, 9.5 and 9.6.

8.0 GEOMETRY CONTROL

8.1 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.

8.2 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 actions proposed 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 inch.

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 ±0.012 inch. For all other types of segmental construction, and for erection of segmental bridges, surveying shall be provided to an accuracy of 0.01 feet.

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.

9.0 TOLERANCES

Unless otherwise specified, reinforcement shall be fabricated and placed within the following tolerances:[6]

a. for depth, d, and minimum clear concrete protection in flexural members, walls and compression members where d is

	d	Min. Cover
8 in. or less	± 3/8 in.	- 3/8 in.
More than 8 in.	± 1/2 in.	- 1/2 in.

but the tolerance on the clear distance to formed soffits shall be -1/4 inch, and in no case shall the tolerance on cover exceed one-third of the minimum cover stipulated on the structural drawings or in the specifications. *Note:* "d" is specified effective depth.

b. For longitudinal location of bends and ends of bars: ±2 inches except at discontinuous ends of members where tolerance shall be ±1/2 inches.

c. As long as the total number of bars specified is maintained, a reasonable tolerance in spacing individual bars is ±1 inch, 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 for precast segmental bridges in Section 4.5.4.

Tolerances for completed segments shall be in accordance with Table 9-1 and the related drawing. For bridges without an overlay, the flatness of the top slab shall be 1/8 inch in 10 feet 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 on the design. 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 7-2 of the 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.

10.0 SHOP DRAWINGS AND DESIGN CALCULATIONS FOR CONSTRUCTION PROCEDURES

10.1 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 super-structure 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 Sections 10.2 and 10.3. 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.

10.2 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:

1. 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.

2. Computations of jacking forces required at joints during temporary post-tensioning.

10.3 Shop Drawings

The Contractor shall submit detailed shop drawings for approval in accordance with the construction special provisions. The shop drawings shall include but not necessarily be limited to the following information.

1. Fully and accurately dimensioned views showing the geometry of the segments including all projections, recesses, notches, openings, blackouts and other pertinent details.

2. 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.

3. 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.

4. 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.

5. Prestressing details shall include 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.

6. A table shall be provided giving jacking sequence, jacking forces and initial elongation of each tendon at each stage of erection for all post-tensioning.

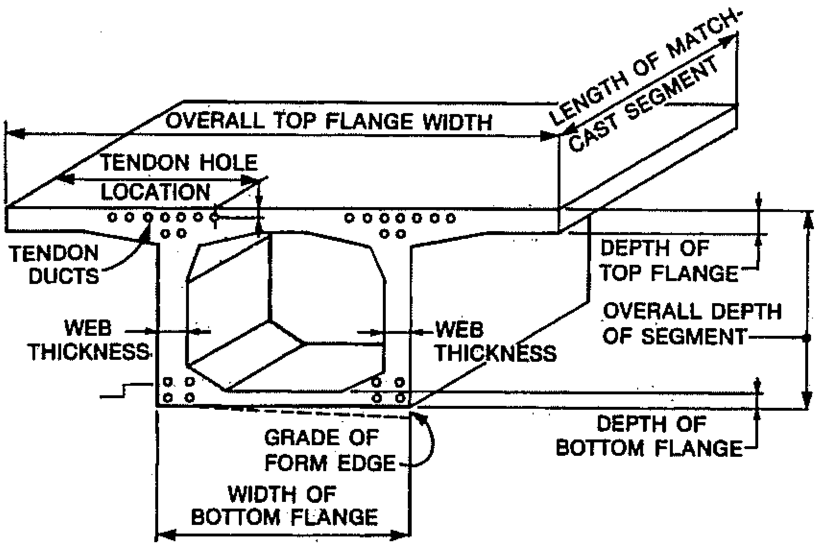
7. A table shall be provided showing elevations and geometry to be used in positioning the forms for the next segment to be cast.

8. 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 he described.

Table 9-1
Completed segment tolerance for segmental box girder bridge construction.
(To correlate tolerances, see sketches below.)

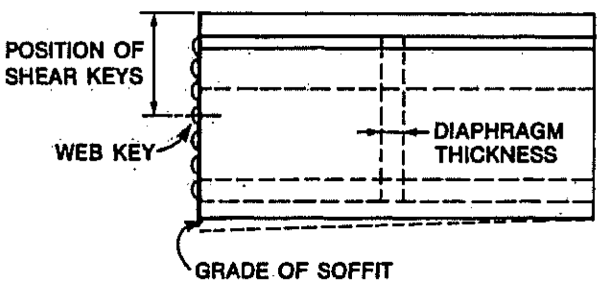
Finished segment tolerances should not exceed the following:

Length of match-cast segment (not cumulative) (10.4 mm/m, +1 in. max. (25 mm)	± 1/8 in/ft.
Length of cast-in-place segment	± 1/2 in. but not greater than + 2 in. per span
Web thickness	± 3/8 in. (9.5 mm)
Depth of bottom slab	± 3/8 in. (9.5 mm)
Depth of top slab	± 3/8 in. (9.5 mm)
Overall top slab width	± 1/16 in/ft (5.2 mm/m), ± 3/4 in. max (25 mm)
Diaphragm thickness	± 1/2 in. (12.5 mm)
Grade of form edge and soffit	± 1/8 in. in 10 ft (1.0 mm/m)
Tendon hole location	± 1/8 in. (3.2 mm)
Position of shear keys	± 1/4 in. (6.3 mm)



Note: Shear keys not shown

SEGMENTAL BOX GIRDER



LONGITUDINAL SECTION

9. Details of tie down tendons, and temporary and permanent bearing assemblies as required.

10. Details of grouting equipment, grout mix design, and method of mixing and placing grout shall be provided.

11.0 FORMS

11.1 General

Shop drawings shall be submitted for forms and form travelers as required by the construction special provisions.

In addition to the requirements of the Standard Specifications, the forms used to cast the concrete segments shall be capable of:

a. Match casting (for precast segmental construction).

b. Producing the segments within the tolerances permitted.

c. Accommodating blackouts, openings and protrusions.

d. Adjusting to changes in segment geometry as shown in the plans, or for correcting previous minor casting errors to prevent accumulation.

e. Stripping without damage to the concrete.

f. The form design must provide 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 1/16 of an inch for flat surfaces and 1/8 of an inch for corners and bends will not be permitted. Offsets between adjacent matching faces of cast-in-place segments shall not exceed 1/4 of an inch.

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 instrumented 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.

11.2 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 Section 9.0.

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 Section 14.2. 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 shall not be used to facilitate segment removal.

12.0 PERMANENT BEARINGS

This work shall consist of the furnishing of all materials, the 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 tests 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 Standard Specifications. The adequacy of the design and installation details shall meet with the approval of the Engineer, whose decision shall be final.

13.0 SPECIAL PROVISIONS FOR CAST-IN-PLACE SEGMENTAL CONSTRUCTION

13.1 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 plans and special provisions.

13.2 Forming System

In addition to the submittals required in Section 10, 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.

13.3 Superstructure Construction

This work consists 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, 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 insure the accuracy of alignment of the completed superstructure.

Work equipment shall 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 the erected superstructure at any stage of construction other than which specifically meets the requirement of total working load per segment, as allowed by the plans, and/or approved by the Engineer. This includes the post-tensioning hardware, jointing, jacking, grouting equipment and any other equipment whatsoever, and workers and materials of any kind.

In addition to segment unbalanced loads which are permitted for the construction method, a 10 pound per square foot load is permissible. This load includes: (1) workers; (2) miscellaneous equipment; and (3) stored material. It is the Contractor's responsibility to insure that this allowable load is not exceeded.

Stressing may be performed in accordance with the following schedule:

Fifty percent of the post-tensioning force may be applied when field cured compression cylinders indicate the compressive strength of the segment concrete is 2500 psi, and 18 hours 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 the remainder stressed when tendons in the new segment are stressed. The form support system must be designed to avoid overstressing the top slab in the area of the partially stressed tendon.

Construction joints will be limited to locations shown on the contract plans 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 plans 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 checklist) 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 per plans, at the check points listed below, or an alternate at his or her option, and submit the same to the Engineer.

a. 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.

b. All four corners and center-line (at segment faces) of top slab of pier segments to establish grade and crown.

c. Two points on the longitudinal center line of each pier segment, one on each edge, to establish alignment.

d. One point on the longitudinal center line 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 (a) above. 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 Section 8.0, and shall maintain a record of all these checks and of all adjustments and corrections made.

14.0 SPECIAL PROVISIONS FOR PRECAST CONCRETE SEGMENTAL CONSTRUCTION

14.1 General

The superstructure shall be erected by the method designed and detailed on the contract drawings, or by an alternate method submitted by the contractor. Alternate erection methods may be permitted when specified in the contract special provisions.

When required by the contract special provisions, the stressing system, and all reinforcement and lifting details shall be successfully demonstrated on a segment designated on the plans prior to casting any permanent segments. The segment shall conform to the size and configuration required by the plans, including post-tensioning anchorage pockets, reinforcing steel, concrete and conduits (curvature and spacing). The tendons designated on the plans for this test shall be stressed to the forces shown. No additional payment will 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 procedure shall be included in the above plans and specifications for the casting bed.

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 insure 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 on the plans 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 plans, the tendons and anchors shall be in a protective enclosure capable of protecting the tendons from damage by erection equipment and 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, 0.05" for a single segment or 0.75" 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.

14.2 Fabrication

Reinforcing steel shall be fabricated and placed according to the plans and specifications. Any conflict or interference with the proper location of ducts and/or reinforcing or blackouts 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 ducts 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 Section 4.5.4, or as shown on the plans, and shall be securely fastened to prevent movement during placement of concrete.

14.3 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 feet x 4 feet 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.

14.4 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.

15.0 SPECIAL PROVISIONS FOR INCREMENTAL LAUNCHING

15.1 General

Structures built by the incremental launching method shall comply with the provisions of Section 10.0, and the additional provisions of the following sections.

15.2 Casting of Segments

Construction of incrementally launched bridges shall preferably 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.

15.3 Geometric Tolerances

The following tolerances shall not be exceeded in the region of the sliding surfaces:

a. In the Forms:

Vertical deviations in longitudinal and transverse direction: -0.04 in.

Horizontal deviation at the outside of webs: ±0.08 in.

b. On the Launching Bearings:

Vertical: longitudinally between piers ±0.08 in. transversely between bearings 0.04 in.

Horizontal deviation of lateral guides ±0.08 in.

15.4 Launching Force

The launching force shall be monitored continuously and checked against the theoretical value. A friction value between 0 and 4 percent shall be maintained. The friction value of 0 shall be considered in calculation of the force required to hold back a structure launched on a negative gradient.

15.5 Pier Monitoring

The deflection of the pier tops shall be continuously monitored. Monitoring devices are recommended which automatically switch off the launching equipment in case the permissible pier deflections are exceeded. 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.

16.0 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 2 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 will be subject to a structural review prior to acceptance. If found acceptable, the cracks shall be repaired by 'veeing' out 1/4 of an inch deep and wide and sealing with epoxy, or shall be repaired by epoxy injection.

Minor breakage, spalling or honeycombing (not over one inch 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 will be subject to structural review. If found to be satisfactory these areas will 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.

REFERENCES

1. ACI Committee 318, "Building Code Requirements for Reinforced Concrete" (ACI 318-86). American Concrete Institute, Detroit, 1987.

2. ACI Committee 222, "Corrosion of Metals in Concrete," American Concrete Institute, Detroit, 1985.

3. "Post-Tensioning Manual," Fourth Edition, "Guide Specification for Post-Tensioning Materials," and "Recommended Practice for Grouting of PostTensioned Prestressed Concrete," Post-Tensioning Institute, Phoenix, 1985.

4. "Recommendations for Acceptance and Application of Post-Tensioning Systems," Federation Internationale de la Precontrainte, 1981.

5. Preston, H. Kent, "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.

6. "Manual of Standard Practice," Concrete Reinforcing Steel Institute, Chicago, Illinois, 1980.

7. ACI Committee 209, "Prediction of Creep, Shrinkage and Temperature Effects in Concrete Structures," ACI 209R-82, American Concrete Institute, 1982.

8. CEB-FIP Model Code for Concrete Structures, Comite Euro-International de Beton (CEB), 1978, available from: Lewis Brooks, 2 Blagdon Road, New Malden, Surrey, KT3 4AD, England.

COMMENTARY
DIVISION II—CONSTRUCTION SPECIFICATIONS

SECTION 2—CONCRETE

2.2 Concrete Quality

The ACI 318-95 Building Code provides statistical procedures for production and testing of concrete to ensure that the minimum 28-day concrete strength f'_c is obtained in the structure.

Some agencies specify test cores for thick concrete members in an attempt to detect improperly compacted or weak layers of concrete.

2.4 Test for Modulus of Elasticity and Creep and Shrinkage Coefficients

These tests may be waived at the discretion of the Engineer when prior test data are available on concrete with the same or similar aggregates and mix design. Some agencies conduct tests before the project goes to contract using a mix of their own design.

2.4.2 Creep and Shrinkage Test

Specifications may require the use of selected aggregates to reduce creep and shrinkage values.

2.5 Admixtures

The use of water-reducing and air-entraining admixtures is recommended.

2.9 Curing for Cast-in-Place Segmental Construction

The construction cycle normally does not permit water curing, except for the top surface of the deck.

SECTION 3—NON-PRESTRESSED REINFORCEMENT

3.1 General

Welding of reinforcement is recommended only under conditions where rigid control of welding procedures and properties of reinforcement is possible. Reinforcement shall not be welded in the vicinity of tendons.

3.2.1(b)

ASTM A 706 covers low-alloy steel deformed bars intended for special applications where welding or bending, or both, are of importance. ASTM A 706 satisfies both exceptions which ASTM A 615 covers as Supplementary Requirements (SI).

3.4 Substitution of Grade or Type of Reinforcement

Research is recommended on more general uses of welded wire fabric in segmental box girder bridge construction.

3.5 Special Corrosion Protection

Special consideration of corrosion protection is considered necessary for all bridge deck reinforcement in areas where de-icer chemicals are used. Corrosion protection measures should be provided for all reinforcement of bridges located in coastal areas or over sea water, or in heavily industrialized areas.

See the ACI Committee 222 Report "Corrosion Of Metals in Concrete"[2] for a comprehensive discussion of methods of corrosion protection.

SECTION 4—POST TENSIONING MATERIALS

4.2 Special Anchorage Devices

Special anchorage devices usually depend on use of some type of confinement reinforcement of the anchorage zone, and usually do not meet the bearing area requirements of the Post-Tensioning Institute's *Guide Specifications for Post-Tensioning Materials.* The test requirements for special type anchorage devices are taken from the *AASHTO LRFD Bridge Construction Specifications.*

4.3 Identification and Testing of Tendon Material

Very exact testing procedures are necessary to conduct a 30-minute test for strand relaxation properties, including rigid control of the temperature in the room where the test equipment is located. See Reference 5 for discussion of test procedures. Most low relaxation strand will show less than 1.0 percent loss in the 30-minute test. However, evaluation of test results indicates that material meeting the 1.20 percent requirement will meet ASTM 416 relaxation requirements in the 1000 hour test.

4.5.1 Metal Ducts

Thickness of metal duct material is related to duct diameter and the method of installing the tendon. Strand tendons are normally installed in the duct after the concrete is placed, requiring a stiffer duct. Bar tendons are normally preassembled inside small diameter ducts and placed as a unit. In this case, the bar fills most of the void and helps to prevent duct damage. Use of 28 gauge duct is recommended when bar tendons are not preplaced with the ducts.

4.5.2 Polyethylene Duct

Specifications have been silent relative to requirements for internal pressure for duct material. Recent detection of splitting of polyethylene ducts has led to the conclusion that the Cell Classification should be specified to assure the duct material will perform for short- and long-term internal stresses. Specifying the Cell Classification provides requirements for density, melt rate, flexural modulus, tensile strength, environmental stress crack resistance, hydrostatic design basis, and color and UV stabilizer.

4.5.4 Duct Placement

When polyethylene duct is exposed to large temperature changes, white duct should be used.

The duct support spacing may be increased when mandrels or preassembled tendons are used.

SECTION 5—INSTALLATION AND STRESSING OF TENDONS

5.1 Installation of Tendons

Vapor phase inhibitor (VPI) powder may be blown into the ducts using a compressor and sand pot (sand blasting equipment).

5.4 Jack Calibration

Master gauges or hydraulic load cells may be used to calibrate jacks. Load cells are used for initial calibration of a master gauge.

5.5 Stressing Equipment

A monostrand jack may be used for individual stressing of strand in tendons of up to four 0.5 inch or 0.6 inch diameter strands in flat duct.

5.6 Modulus of Elasticity of Tendon Material

The article "Testing 7-Wire Strand for Prestressed Concrete–The State of the Art" (May-June 1985 issue of the *PCI Journal*) may be referenced for recommended testing procedures for seven wire strand. The article contains the following statements concerning strand fabrication and laboratory Modulus of Elasticity (MOE) tests:

"When wire is drawn for a prestressed concrete strand, the shop practice of the strand manufacturer will specify a diameter, but it must also permit some degree of tolerance. The wires of a 1/2 in. diameter strand have a diameter of about 0.167 in. The minimum reasonable tolerance is ±0.001 in. This variation in diameter of 0.001 in. represents a change in area, and MOE, of 1.2 percent."

"The diameter of the hole in the last die of the wire drawing operation determines the diameter of the finished wire. Since the diameter of this hole increases with wear as wire is drawn through it, it is standard practice to start a new die with a hole slightly under size and continue to use the die until the hole is producing wire that meets the maximum permissible oversize tolerance. The result is a deliberately created variation in area, and MOE, of ±1.2 percent from the nominal."

"Extremely good laboratory equipment and procedures are needed if the MOE established in the laboratory is to always be within 2.5 percent of the actual MOE of the strand being tensioned at the job site. As indicated in the following discussion, strand fabricating tolerances are such that the actual MOE of two lengths of strand fabricated on the same equipment to the same specification can differ by as much as 2.4 percent. In addition, it is difficult to attach a strain gauge to a member composed of seven individual wires so that it will measure the MOE with complete accuracy across relatively small gauge length."

In view of the above discussion concerning laboratory measurements of MOE, and the additional difficulties associated with such measurements in the field, the use of field "bench tests" to evaluate MOE is not recommended. Further, it should be assumed that a variation of MOE values on the order of 2.4 percent might routinely be encountered in field operations.

5.7 In-Place Friction Test

Tests to determine friction losses are usually specified on large projects, and when elongation measurements exceed the tolerance of Section 5.8.

5.8 Tendon Force and Elongation Tolerance

Tendon forces for strand tendons less than 30 feet in length may be verified by a lift-off test after tendons are initially stressed and wedges are seated. Shims may be used as required under the anchor head to maintain the specified tendon force.

The ±7 percent elongation tolerance is a guide, and not an absolute acceptance criteria. The reasons for larger variations should be investigated. For shorter bridge deck tendons, an elongation tolerance of ±10 percent on individual strand should be considered, provided that the average elongation of the tendons in a group (usually four) is within ±7 percent of the calculated elongation.

7.0 EPOXY

Epoxy formulations are now available that may be applied at ambient temperatures of 16 degrees Fahrenheit. This development substantially extends the construction season for bridges in cold climates, and/or eliminates the need for heating joints.

8.2 Geometry Control

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.

Twist errors are calculated as the actual twist minus the design super elevation (if any).

SECTION 10—SHOP DRAWINGS AND DESIGN CALCULATIONS FOR CONSTRUCTION PROCEDURES

10.2 Design Calculations for Construction Procedures

Load testing of special erection equipment is recommended.

Note that reinforcement for anchor blocks (as required) under this specification is not the reinforcement required for global bursting and directional forces due to post-tensioning.

SECTION 14—SPECIAL PROVISIONS FOR PRECAST CONCRETE SEGMENTAL CONSTRUCTION

14.1 General

Problems have been reported in the past when segments do not match properly at joints because of the thermal deformations induced during match casting. A design gradient has been proposed[44] which 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.

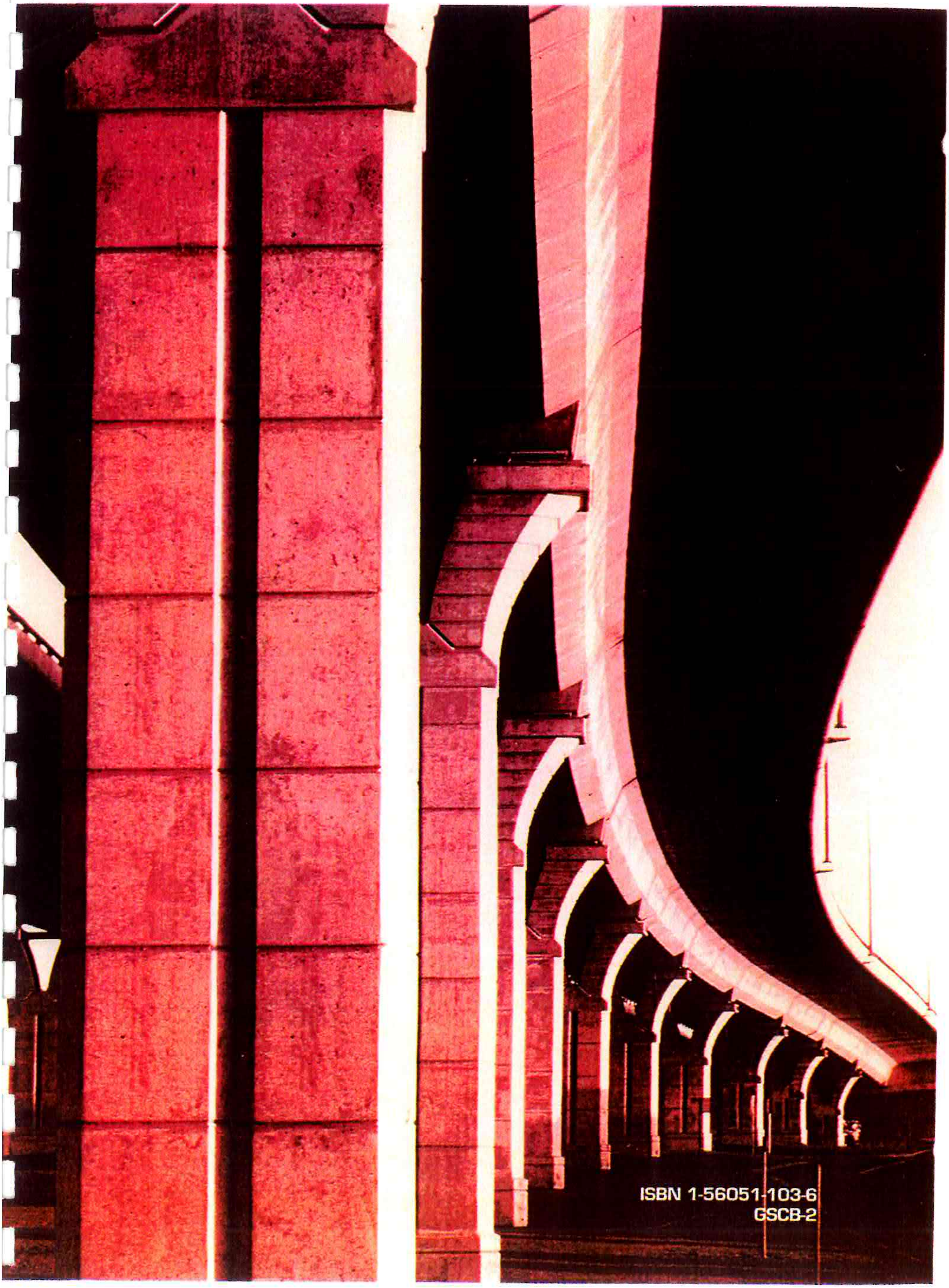
14.2 Fabrication

Use of integrated shop drawings is recommended to locate conflicts in reinforcement placement.

SECTION 15—SPECIAL PROVISIONS FOR INCREMENTAL LAUNCHING

15.4 Launching Force

Grease may be used on the sliding plates as required to maintain friction values.



ISBN 1-56051-103-6
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b. Type A joints without the minimum bonded auxiliary reinforcement through the joints; internal tendons: no tension allowed.

c. Type B joints external tendons, not less than:

100 psi minimum compression

9.2.1.3 Transverse tension in the precompressed tensile zone: $3.0\sqrt{f'_{ci}}$ maximum tension.

9.2.1.4 Tension in other areas without bonded nonprestressed reinforcement: zero tension.

Where the calculated tensile stress exceeds this value, bonded reinforcement shall be provided at a stress of $0.5f_{sy}$ to resist the total tensile force in the concrete computed on the assumption of an uncracked section. In such cases the maximum tensile stress shall not exceed $6.0\sqrt{f'_{ci}}$

9.2.2 Stresses at the service load level after losses have occurred:

9.2.2.1 Compression:

a. The compressive stress under all service load combinations, except as stated in (b), shall not exceed $0.60f'_c$. When the flange or web slenderness ratio, calculated in accordance with Section 23.3, is greater than 15, the compressive stress shall be reduced by a factor calculated by the equations presented in Section 23.4.3.

b. The compressive stress due to effective prestress plus permanent (dead) loads shall not exceed $0.45f'_c$

9.2.2.2 Longitudinal stresses in the precompressed tensile zone:

a. Type A joints with minimum bonded auxiliary reinforcement through the joints sufficient to carry the calculated tensile force at a stress of $0.5f_{sy}$; internal tendons:

$3.0\sqrt{f'_c}$ maximum tension

b. Type A joints without minimum bonded auxiliary reinforcement through joints: zero tension.

c. Type B joints, external tendons, not less than:

100 psi minimum compression

9.2.2.3 Transverse tension in the precompressed tensile zone:

$3.0\sqrt{f'_c}$ maximum tension

9.2.2.4 Tension in other areas without bonded reinforcement: zero tension.

Where the calculated tensile stress exceeds this value, bonded reinforcement shall be provided at a stress of 0.5 f_{sy} to resist the total tensile force in the concrete computed on the assumption of an uncracked section. In such cases, the maximum tensile stress shall not exceed $6.0\sqrt{f'_c}$.

10.0 PRESTRESS LOSSES

10.1 General

Lump sum losses shall only be used for preliminary design purposes. Losses due to creep, shrinkage, and elastic shortening of the concrete as well as friction, wobble, anchor set and relaxation in the tendon shall be calculated for the construction method and schedule shown on the plans in accordance with time-related procedures for calculation of prestress losses and in accordance with the following sections.

10.2 Duct Friction and Wobble

The loss of prestress force due to friction and wobble within an internal tendon duct shall be obtained using the equation:

$$T_x = T_o e^{-(\mu\alpha + k\ell)}$$

For tendons in webs of curved bridges, or in inclined webs of straight bridges, α shall be calculated as the total vector accumulation of the horizontal and vertical angle changes, and ℓ shall be the total tendon length.

The loss of prestress force in an external tendon due to friction across a single deviator pipe shall be obtained using the equation:

$$T_x = T_o e^{-\mu(\alpha + 0.04)}$$

Friction and wobble coefficients may be estimated using the values in Table 10-2. However, these values do not consider misalignment of internal ducts at joints. The inadvertent angle change of 0.04 radians per deviator may vary depending on job specific tolerances on deviator pipe placement, and need not be applied in cases where the deviation angle is strictly controlled or precisely known, as in the case of continuous ducts passing through separate longitudinal bell-shaped holes at deviators. Where large discrepancies occur between measured and calculated tendon elongations, in place friction tests are required.

The inadvertent angle change need not be considered for calculation of losses due to wedge seating movement.

10.3 Anchorage Seating

For strand tendons anchored with two or three piece wedges, anchorage seating may be approximated as 1/4 inch. Anchor seating for bar tendons may be approximated as 1/16 inch. The value of anchorage seating used in the design shall be stated on the design

Table 10-2. Estimated Values for Friction and Wobble Coefficients

	Friction Coefficient (μ) (1/rad)	Wobble Coefficeint (k) (1/ft)
1. For strand in galvanized metal sheathing	0.12 - 0.25	0.0002
2. For deformed high strength bars in galvanized metal sheathing	0.30	0.0002**
3. For strand in internal polyethylene duct.	0.23	.0002
4. For strand in straight polyethylene duct (external to concrete)	0	0
5. Rigid steel pipe deviators for external tendons	0.25*	0.0002
6. Continuous external tendon polyethylene ducts through a deviator	0.15	0

* Lubrication will probably be required

** Wobble coefficient for duct installed with preplaced bars may be taken as 0.0001.

drawings with the provision that it shall be verified during construction.

10.4 Steel Relaxation

Loss of prestress due to steel relaxation over the time interval t_1 to t may be estimated in accordance with the following sections.

10.4.1 Stress-relieved Steel

$$R_{sr} = f_{st}\ \{[\log 24t - \log 24t_1]/10\} \times [f_{st} / f_y^* - 0.55]$$

where: $f_{st} / f_y^* - 0.55 \geq 0.05$

$f_y^* = 0.85 f_{su}^*$

f_{st} = steel stress level at beginning of time interval (t_1)

10.4.2 Low Relaxation Steel

$$R_{lr} = f_{st}\{\log 24t - \log 24t_1] /45\} \times [f_{st}/f_y^* - 0.55]$$

where: $f_{st}/f_y^* - 0.55 \geq 0.05$

$f_y^* = 0.90\ f_{su}^*$

f_{st} = steel stress level at beginning of time interval (t_1)

10.4.3 Bars

Steel relaxation loss for bar tendons conforming to ASTM A722 may be estimated as 3000 psi.

10.5 Stressing Hardware

A loss in tendon force occurs through the stressing hardware and anchorage device. This loss shall be considered in design.

11.0 FLEXURAL STRENGTH

11.1 General

Flexural strength shall be calculated in accordance with the provisions of Sections 9.17, 9.18, and 9.19 of the AASHTO *Standard Specifications for Highway Bridges*, and/or as provided in this Section.

The flexural capacity required by the load factor provisions of Section 7.1 shall be less than or equal to ϕf times the flexural capacity calculated in accordance with AASHTO *Standard Specifications for Highway Bridge*, Sections 9.17, 9.18, and 9.19, except that the average stress in unbonded prestressing steel shall be determined in accordance with Sections 11.2 or 11.3. The values of ϕ_f shall be taken from Section 7.3.

11.2 Stress in Unbonded Prestressing Steel at Factored Load

For calculation of average stress in unbonded prestressing steel at factored loads, equation 9-18 of the AASHTO *Standard Specifications for Highway Bridges* shall be replaced by the following formula:

$$f_{ps} = f_{pe} + 900\left(\frac{d_p - c_y}{\ell_e}\right)$$

where:

SECTION F
SPECIAL PROVISIONS FOR BRIDGE TYPES

24.0 PRECAST SEGMENTAL

24.1 General

Precast segmental bridges are normally erected by balanced cantilever, by use of erection trusses, or by progressive placement. To limit construction deflections to values consistent with design calculations, precast segments shall be a minimum of 14 days old at the time of erection unless earlier erection is specifically approved by the Engineer.

24.2 Stresses for Construction Practices

24.2.1 Removal of Forms

Before stripping the forms or moving the segment while resting on its soffit form in the casting bed, the concrete shall have a minimum compressive strength of 2500 psi unless analysis requires a higher value. For transverse post-tensioning, the concrete shall have a minimum compressive strength of 3500 psi. For handling of the segment, the concrete shall have a minimum compressive strength of 3000 psi.

24.2.2 Erection

The following concrete strengths shall be obtained before post-tensioning:

1. When considered necessary to avoid cracking, partial stressing of post-tensioning across longitudinal or transverse closure strips may be permitted when the concrete has attained a minimum strength of 1800 psi. Full post-tensioning of closure strips shall normally be permitted when the concrete has attained a minimum strength of 2500 psi, unless analysis indicates that a higher value is required.

2. Before stressing permanent longitudinal post-tensioning tendons, the concrete segments shall attain the minimum 28-day compressive strength.

24.2.3 Temporary Stress on Epoxy Joints

A minimum compressive stress of 40 psi shall be provided for the closure stress on an epoxied joint until the epoxy has set.

24.3 Shear Keys

Shear keys in webs of precast segmental bridges shall extend for as much of the web depth as is compatible with other detailing requirements. Details of the shear keys shall be similar to Figure 24-1. Shear keys shall also be provided in top and bottom slabs, but these may be larger single element keys.

24.4 Joints

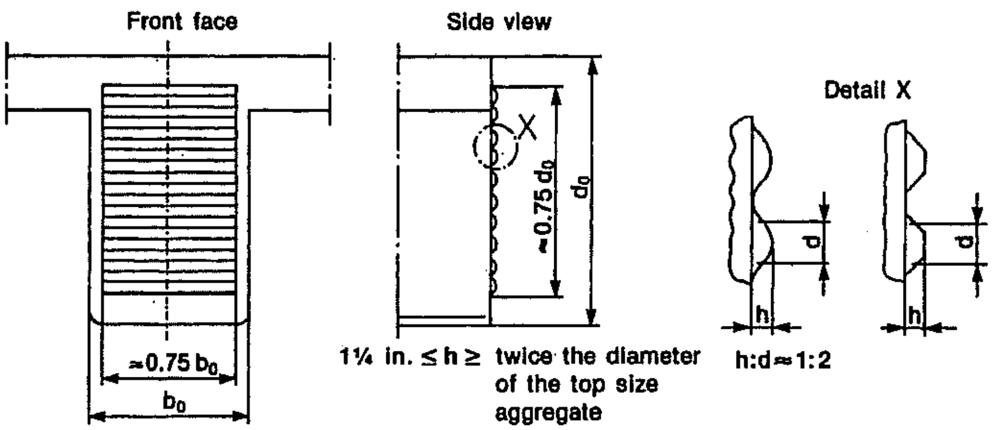
24.4.1 General

Precast segmental bridges are usually match-cast with Type A or epoxied joints. Closure pours are cast-in-place. Type B or dry joints between segments may be used under conditions stated below in Section 24.4.3.

24.4.2 Type A Joints

Type A (epoxied) joints shall be utilized for all bridges utilizing internal tendons, and for all bridges

Fig. 24-1. Example of design of fine indentation joint faces



exposed to severe climatic conditions where freeze/thaw cycles are encountered, or where de-icer chemicals are used.

24.4.3 Type B Joints

Type B (dry) joints may be used in conjunction with external post-tensioning tendons in areas where freeze/thaw cycles do not occur, and where de-icer chemicals are not used.

24.5 External Tendons

External tendons shall be permanently protected against corrosion.

25.0 CAST-IN-PLACE SEGMENTAL

25.1 General

Cast-in-place segmental bridges may be erected on falsework, by the free cantilever technique, by span-by-span lifting of spans cast at the bridge site, or by incremental launching. The special design requirements for incremental launching are presented in Section 26.0. Special design requirements for other cast-in-place segmental construction methods are presented in the following sections.

25.2 Joint Details

Contact surfaces between cast-in-place segments shall be clean, free of laitance, and shall be intentionally roughened to expose coarse aggregate. The use of shear keys is optional.

25.3 Closure Pours

Closure pours shall have sufficient width to permit coupling of tendon ducts.

25.4 Form Traveler Weight

The form traveler weight assumed in stress and camber calculations shall be stated on the design drawings.

25.5 Segment Lengths

Segment lengths may vary with the construction method, with the span length, and with location in the span.

25.6 Diaphragms

Diaphragms shall be provided at abutments, piers, hinge joints, and at bottom flange angle points in structures with straight haunches. Diaphragms shall be substantially solid at piers and abutments except for access openings and utility holes. Diaphragms shall be sufficiently wide as required by design with a minimum overhang over bearings of not less than 6 inches.

26.0 INCREMENTAL LAUNCHING

26.1 Loads and Stresses During Launching

26.1.1 Friction on Launching Bearings

The friction on launching bearings shall be assumed to vary between 0 percent and 4 percent, whichever governs the determination of hold-back or pushing forces. The upper value may be reduced by 0.5 percent, if the pier deflections and the launching jack forces are monitored. The design shall consider that inclined launching bearings (as opposed to the horizontal permanent bearings) create additional forces at the launching jacks and at the pier tops.

26.1.2 Moments Due to Construction Tolerances

The moments due to the following permissible construction tolerances shall be superimposed onto those from external loads:

In longitudinal direction between two adjacent bearings: 0.2 inches.

In transverse direction between two adjacent bearings: 0.08 inches.

26.1.3 Combination of Construction Tolerances and Temperature Gradient

During construction, one-half of the moments due to construction tolerances as per Section 26.1.2 and one-half of the moments due to temperature in accordance with Section 6.4.4 shall be superimposed onto those from external loads. Tensile stresses resulting from the combined moments shall not exceed $7.0\sqrt{f'_c}$.

26.1.4 Permissible Stresses

Permissible stresses shall be in accordance with Section 9.2.1.2.a for members with bonded reinforcement, through the joint and internal tendons. Temporary piers and/or a launching nose may be used to reduce launching stresses.

26.2 Bridge Design

26.2.1 Piers and Superstructure Diaphragms

Piers and superstructure diaphragms at piers shall be designed in such a way that during all launching stages, and after launching for the installation of the permanent bearings, the superstructure can be lifted with hydraulic jacks. Pier designs shall consider frictional forces during launching in accordance with Section 26.1.1.

26.2.2 Bottom Edges of Superstructures

At the underside of the webs above the launching bearings high local stresses occur. The design shall take into account:

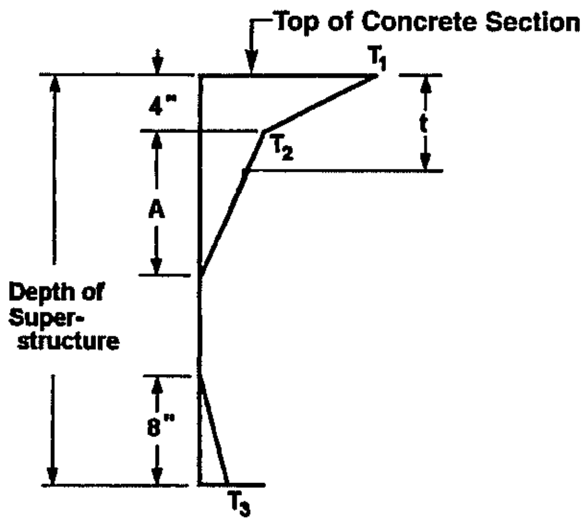


Fig. 6-4. Positive Vertical Temperature Gradient

6.5 Creep and Shrinkage

Effects due to creep and shrinkage strains shall be calculated in accordance with the provisions of Section 5.3. The creep coefficient ϕ_c may be evaluated in accordance with the provisions of the ACI Committee 209 Report,[17] the CEB-FIP Model Code,[18] or by a comprehensive test program. Creep strains and prestress losses which occur after closure of the structure cause a redistribution of the forces. Stresses shall be calculated for this effect based on an assumed construction schedule stated on the plans.

6.6 Post-Tensioning Force

The structure shall be designed for both initial and final post-tensioning forces. For determining the final post-tensioning forces, prestress losses shall be calculated for the construction schedule stated on the plans. The final post-tensioning forces used in service load stress calculations shall be based on the most severe condition at each location along the structure.

7.0 LOAD FACTORS

7.1 General

In the final working condition, service or strength load combinations shall be in accordance with AASHTO *Standard Specifications for Highway Bridges,* Section 3.2.2, with consideration of the additional load combinations of Section 7.2. Strength reduction factors, ϕ, shall be in accordance with Section 7.3.

During construction, load combinations, allowable stresses, and stability shall be in accordance with Section 7.4. Prior to grouting, the allowable concrete tensile stress during construction shall be zero for structures utilizing internal grouted tendons.

7.2 Additional Load Combinations

The permanent effects of creep and shrinkage shall be added to all AASHTO loading combinations with a load factor of 1.0.

7.2.1 Erection Loads at End of Construction

The final state erection loads (EL) are defined as the final accumulated "built-in" forces and moments resulting from the construction process.

7.2.2 Additional Thermal Loading Combination

7.2.2.1 For existing service load combinations including full live load plus impact, a load factor of 0.50 shall be applied for temperature gradient.

In addition to AASHTO service load combinations, the following load combination shall apply:

$$DL+SDL+EL+\beta_E E+B+SF+R+S+TG$$

$$\leq 100\% \text{ Allowable Stress}$$

LOADING	
Dead Load Structure Only:	DL
Superimposed Dead Load:	SDL
Erection Loads (final state):	EL
Thermal-Rise or Fall	TRF
Thermal-Differential	TG
Creep Effects	R

D=(DL+SDL+EL)
T=(TRF+TG)

See AASHTO *Standard Specifications for Highway Bridges,* Section 3.22.1, for other definitions.

7.2.2.2 For all factored load combinations, a load factor of zero (0) shall be applied to differential temperature effects (TG).

7.3 Strength Reduction Factors

7.3.1 The strength reduction factors, ϕ_f and ϕ_v, for flexure and shear, respectively, shall consider both the type of joint between segments and the degree of bonding of the post-tensioning system provided. The appropriate value of ϕ_v from Section 7.3.6 shall be used for shear and torsional effect calculations in Section 12.

7.3.2 Since the post-tensioning provided may be a mixture of fully bonded tendons and unbonded or partially bonded tendons, the strength reduction factor at any section shall be based upon the bonding conditions for the tendons providing the majority of the prestressing force at the section.

Table 7-1. Strength Reduction Factors for Segmental Construction

	ϕ_f Flexure	ϕ_v Shear	ϕ_j Joint
Normal Weight Concrete			
Fully Bonded Tendons			
Type A Joint	0.95	0.90	-
Unbonded or Partially Bonded Tendons			
Type A Joint	0.90	0.85	-
Type B Joint	0.85	0.85	0.75
Sand-Light Weight Concrete			
Fully Bonded Tendons			
Type A Joint	0.90	0.70	-
Unbonded or Partially Bonded Tendons			
Type A Joint	0.85	0.65	-
Type B Joint	0.80	0.65	0.60

7.3.3 In order for a tendon to be considered as fully bonded to the cross section, it must be bonded beyond the critical section for a development length not less than that required by AASHTO *Standard Specifications for Highway Bridges*, Section 9.28.1. Shorter embedment lengths are permissible if demonstrated by full size tests and approved by the Engineer.

7.3.4 Cast-in-place concrete joints, and wet concrete or epoxy joints between precast units, shall be considered as Type A joints.

7.3.5 Dry joints between precast units shall be considered as Type B joints.

7.3.6 Strength reduction factors, ϕ, shall be taken as presented in Table 7-1, and the following provisions of this section.

The strength reduction factor for direct shear capacity of dry joints, ϕ_j, shall be used in conjunction with Section 12.2.21.

The strength reduction factor for bearing, ϕ_b, shall be taken as 0.70 for all types of construction. This value shall not be applied to bearing stresses under anchorage plates for post-tensioning tendons.

7.4 Construction Load Combinations, Stresses and Stability

7.4.1 Erection Loads During Construction

Erection loads as defined by AASHTO and as stated on the plans shall be as follows:

a. Dead load of structure (DL): Unit weight of concrete (including rebar) - 155 pcf or as determined for the project. Weight of diaphragms, anchor blocks, or any other deviations from the typical cross section shall be included in the dead load calculations.

b. Differential load from one cantilever (DIFF): This only applies to balanced cantilever construction. The load is 2% of the dead load applied to one cantilever.

c. Superimposed dead load (SDL): This does not normally apply during construction. If it does, it should be considered as part of the dead load (DL).

d. Distributed construction live load (CLL): This is an allowance for miscellaneous items of plant, machinery and other equipment apart from the major specialized erection equipment. Distributed load allowance is 10 psf. In cantilever construction, distributed load shall be taken as 10 psf on one cantilever and 5 psf on the other. For bridges built by incremental launching, construction live load may be taken as zero.

e. Specialized construction equipment (CE): This is the load from any special equipment such as a launching gantry, beam and winch, truss or similar major item. This also includes segment delivery trucks and the maximum loads applied to the structure by the equipment during the lifting of segments.

f. Impact load from equipment (IE): To be determined according to the type of machinery anticipated. For very gradual lifting of segments, where the load involves small dynamic effects, the impact load may be taken as 10 percent.

g. Longitudinal construction equipment load (CLE): The longitudinal force from the construction equipment.

h. Segment unbalance (U): This applies primarily to balanced cantilever construction but can be extended to include any "unusual" lifting sequence which may not be a primary feature of the generic construction system.

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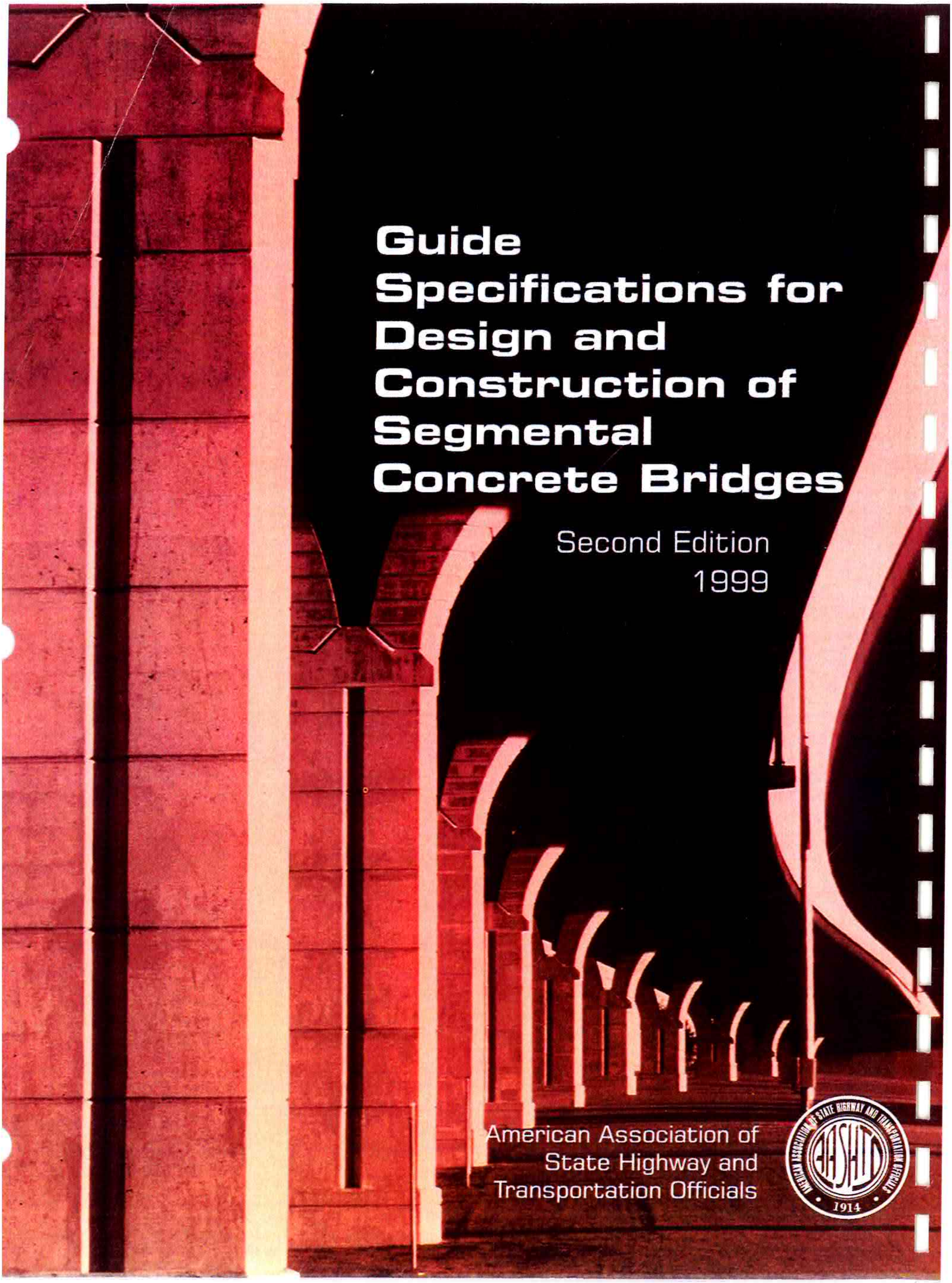

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Second Edition
1999

American Association of
State Highway and
Transportation Officials