NEAL BETTIGOLE RITA ROBISON

BRIDGE DECKS

Design ✖ Construction ✖ Rehabilitation ✖ Replacement

ASCE PRESS

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Abstract:

This book is a comprehensive reference for the evaluation, testing, selection, and examination of relevant design criteria and alternatives for bridge decks, which appear in the AASHTO/LRFD design specifications. Important challenges to civil engineers, such as life cycle cost analysis, and constructability, particularly as related to maintaining traffic during deck replacement, are discussed. The authors discuss why the use of standard bridge deck designs is not always possible on bridge rehabilitation projects. This practical reference will aid busy engineers in dealing with the major changes that will mandate much greater attention to deck selection and design in the future. For example, most future bridge projects will involve rehabilitation or replacement--which makes traffic maintenance a major issue--and life cycle cost analysis is quickly becoming mandatory in the U.S. This guide is intended to be used throughout the development of any construction project involving bridges.

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Dedication

Rita Robison, an engineering journalist whose impressive body of work included numerous feature stories on some of the biggest U.S. construction projects of modern times, died in August, after a short illness, in Albuquerque, N.M., where she had retired in 1991.

After eight years of working for ASCE, where she held several posts, including senior editor of *Civil Engineering* magazine, Robison retired in 1991. But that was hardly the end of her career. Until shortly before her death at age 70, she was a contributing editor to the magazine, and was most recently represented in its July issue by a lead feature article, "Boston's Home Run," which detailed construction of the Ted Williams Tunnel, recipient of the Society's Outstanding Civil Engineering Achievement (OCEA) for 1996. (In fact, she had written the July cover story on OCEA winners for the past 10 years.)

In another current writing project, Robison was a co-author, with Neal Bettigole, of a forthcoming ASCE Press book, *Bridge Decks: Design, Construction, Rehabilitation, Replacement,* which is scheduled for publication early next year.

In her 13-year association with ASCE, Robison wrote more than 90 stories for CE, covering a wide range of engineering topics such as bridges, highways, and structures. Says CE Editor-in-Chief Virginia Fairweather of Robison's work, "Rita could take any manuscript, no matter how technical or dense, and turn it into crystal clear and lively prose. As a reporter, she was thorough and responsible with facts. It will be nearly impossible to replace her."

Before joining the Society in 1983, Robison reported on engineering and architectural topics for three magazines: *American School and University,* of which she was editor, *Progressive Architecture* and *Architecture and Engineering News,* where she was managing editor. In those jobs, she covered and sometimes broke news stories on innovations in building materials such as fabric roofs and weathering steel. Robison held a bachelor's degree in journalism from the University of Colorado.

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CONTENTS

PREFACE

This book is a compilation of ideas contributed by a long list of bridge engineers who have been generous with their time and knowledge. We acknowledge their help with profound gratitude and hope that the book will be as useful as possible to the group of people with whom we have both spent most of our professional lives: civil engineers in the practice of their profession.

Neal H. Bettigole, P.E., F.ASCE, Upper Saddle River, N.J.
Rita R. Robison, Albuquerque, N.M.

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And thanks are due also to hundreds of other bridge engineers who cannot be named for reasons of brevity. Their conversations and opinions, so freely offered, are embedded in this book.

CHAPTER 1

INTRODUCTION TO BRIDGE DECKS AND HOW WE GOT HERE

The purpose of this text is to provide an overview of bridge decks. Little has been written about decks in the past, which indicates more than a measure of complacency about them throughout the highway and bridge design/construction industry. This text deals with how future bridge decks should be designed, constructed, and maintained, both for new bridges and for deck replacements. It also gives specific information about the design, details, construction, evaluation, and maintenance of existing bridge decks.

Owners and engineers should begin to think about bridge projects in a new way, starting with a complete picture of the history of the project. They should consider, in an organized way, the probable future use of the bridge, and replace the casual "type, size, and location" thinking for new bridges with a vastly expanded check/job list. This book intends to help to expand the user's thinking about the decks on bridges, and also to provide the tools for competent consideration of alternatives and execution of the deck decision.

Life-cycle costs, experience with the performance of bridge decks, and a methodology to deal with the many additional factors that must be considered in selecting the deck type for a specific project are now requirements for all bridge construction, rehabilitation, and repair projects. One section of this book deals with selecting the optimum deck for new and existing bridges of various types. Others deal with evaluation of existing decks, options for repair by overlay and other techniques, partial and full replacement, and descriptions of the various deck types. Maintenance and repair, materials, and techniques are also covered.

Access to information in the text is through a composite index, which we have made as exhaustive as possible. Chapter 7 discusses items relating to bridge decks appearing in the 15th edition of the *AASHTO Standard Specification for Highway Bridges* (1992), the first edition of the *AASHTO/LRFD Bridge Design Specifications* (1994), and the *Ontario Highway Bridge Design Code,* third edition (1991). The AASHTO/LRFD specification contains a

Commentary on the right-hand side of each page, which adds greatly to its usefulness as a reference. We have made every effort to make the index of this work as useful as possible.

When using the AASHTO specifications, an engineer is well advised to read the Introduction to both the AASHTO and AASHTO/LRFD (load and resistance factor design), which are not at all identical. The statements describing these works as guides differ in several ways, such as the AASHTO note that spans exceeding 152.4m (500 ft) may require consideration of additional factors.

This book is intended as a technical reference, to be used throughout the development of any construction project involving bridges. Attention to the deck of a bridge deserves greater focus as engineers strive for longer-life projects at reasonable life-cycle costs. With this book, it should be possible for an engineer to review efficiently any matter concerning bridge decks that his or her work requires, gaining a depth of understanding without exceeding the time normally allocated to that task.

SOME HISTORICAL NOTES

As a rule, some understanding of the way it used to be gives a good foundation for understanding what goes into a design today. The history of bridge decks doesn't really begin until the 20th century. Until then, from earliest settlements in colonial times up to development of roads and bridges suitable for automobile traffic, timber was the material of choice for decks and stringers. A plentiful material, timber was used as decking for stone and steel bridges well into the 1920s. Concrete bridges, however, seemed to demand concrete decks.

Reinforced concrete became the most common material for bridge deck construction because of its apparent advantages over timber when motor vehicle use took over as the most prevalent mode of transportation in America. By 1930, reinforced concrete was well established as a building material, and knowledge of how to design and build with this material was widespread.

As stated in the introduction to the AASHTO specification, a Committee on Bridges and Structures was organized in 1921. A complete specification, largely devoted to concrete, was first available in mimeographed form in 1926, revised in 1928, and finally printed for the first time in 1931. Revisions were published every four years or so, with many annual interim specifications reflecting changes voted on by association member (state) departments.

In March 1930, *Public Roads Magazine* published an article that became the foundation for all later design of reinforced concrete bridge decks. It was titled *"Computation of Stresses in Bridge Slabs due to Wheel Loads,"* written by H. M. Westergaard, professor of theoretical and applied mechanics at the University of Illinois. His theory is discussed further in Chapter 6.

Deck type	No. of bridges	% of total
Cast-in-place concrete	330,063	68.8
Precast concrete	37,887	7.9
Open grid	3,447	0.7
Closed grid (filled)	1,467	0.3
Steel plate (orthotropic)	1,412	0.3
Corrugated (plate)	9,887	2.1
Aluminum (incl. corrug)	443	0.1
Timber	12,973	2.7
Other	82,084	17.1
Totals:	479,663	100

Fig. 1.1. Deck Structure Types

Most of the nearly 600,000 bridges that currently exist in the United States have been constructed since 1930. The bridge deck used on 68.8% of these bridges (and 85.78% of the total deck area) is cast-in-place reinforced concrete slabs. This figure is from the 1994 National Bridge Inventory, supplied by the Bridge Management Branch of the Federal Highway Administration.

The often quoted figure of the total number of the nation's bridges, 580,695, includes 101,027 single- or multiple-barrel culverts, in which the total centerline of the road dimension exceeds 6.096 m (20 ft). From the National Bridge Inventory (NBI), the figures for all deck types recorded under Item 107, Deck Structure Type, by number of bridges and percentages, are shown in Figure 1.1.

Also from the NBI, and perhaps more significant for a text on bridge decks, are statistics based on deck area. There are approximately 286,226,310 sq m (3,080,902,000 sq ft) of highway bridge decks in the United States. The breakdown of deck area by type of material is shown in Figure 1.2.

STANDARDIZATION

Standardization of construction of the elements of a bridge became a matter of particular national concern in 1956, when the Interstate Highway System launched a program to build 66,000 km (41,000 mi) of new highways and bridges.

There was little or no discussion, however, about standardization of the bridge deck, a major element of bridge construction. The basis for this significant policy decision was that it had already been done: one of the earliest standards had been the design, detailing, and construction of reinforced concrete bridge decks.

In the 1950s, the Bureau of Public Roads published a set of typical

Deck type	Deck area 000's sq ft	Area % of total	% No. of bridges
CIP concrete	2,643,082	85.78	68.80
Precast concrete	121,032	3.93	7.90
Timber	90,474	2.94	2.70
Open grid	25,937	0.84	0.70
Closed grid (all types)	15,524	0.50	0.30
Corrugated metal	14,861	0.48	2.10
Steel plate (orthotropic)	5,625	0.18	0.30
Aluminum (orthotropic)	871	0.03	0.10
Other types	61,249	1.99	17.10
101,066 culverts	302,247	3.33	*
TOTALS	3,080,902	100.00	100.00

*Culverts not included In percentage of total

Fig. 1.2. Deck Structure Type

drawings for several types of bridges. These drawings included the deck construction with typical details, all of which made it virtually mandatory for bridge engineers to use reinforced concrete for their bridge decks.

Individual states gradually developed their own standards for the design of bridges, building on and referencing the AASHTO specifications. These standards increasingly restricted bridge design and continue to do so. However, in defense of the policy of using a reinforced concrete deck as a standard, it is true that it was, and is, an invaluable aid for an engineer who must determine the dead load and its distribution on the structure as early as possible during design of a new bridge.

Standardization is a two-edged sword. Saving money in the initial construction is of course a desirable achievement, provided, as is now known, it is compatible with obtaining the lowest life-cycle cost. If, however, a standard is deficient and is used over a long enough period of time, the magnitude of the effect of the error can be enormous. Standardization is also a throttle on future innovation.

This is why what now appears to have been a questionable national policy to use only reinforced concrete bridge decks, in reality was an easy, conservative conclusion, reached by most states to expedite the design of new bridges: There was a need to standardize the decks. Dr. Westergaard's theory was well known, and convenient design procedures became widely available. The real problem was that the standard deck design was frozen before its deficiencies were noted, researched, and solved. Many believe that the standard design was a blunder for which our society will pay untold billions of dollars over the next 30 or so years.

Virtually 100% of the existing reinforced concrete and precast concrete bridge decks will fail a major test of life-cycle cost. They will fail, by many, many years, to serve out the useful life of the bridge, thus requiring at least one complete deck replacement before replacement of the superstructure.

It is only since January 3, 1994, that life-cycle cost has been included in bridge design criteria. On that date, the Interim Final Rule for 23CFR500, subpart C (Bridge Management Systems) went into effect. This section of the Code of Federal Regulations includes reference to, and definition of, life-cycle cost for the first time.

The 1994 surface transportation bill included the following language when it was sent from the House to the Senate (as HR 4385): "SEC. 103. QUALITY IMPROVEMENT. (a) Life-Cycle Cost Analysis.—Section 106 of title 23, United States Code, is amended by adding at the end the following: (e) Life Cycle Cost Analysis.—(1) Establishment.—The Secretary shall establish a program to require States to conduct an analysis of the life-cycle costs of all projects on the National Highway System."

However, the 1994 legislation, which included a proposed National Highway System, failed to be voted into law. (The final language of the 1995 funding bill for surface transportation did mandate life-cycle costing, but only for federally aided projects costing more than $25 million.)

Is it therefore correct to assert that no standards for bridge deck design were adopted for reasons of life-cycle cost consideration before 1995? The answer is no. Although life-cycle cost was not often referred to by decision makers, a number of bridge deck construction practices can, in fact, only be attributed to a concern about the useful service life of decks.

NEW TASKS FOR ENGINEERS

The concept of life-cycle cost has finally been recognized by Congress as fundamental to the long-term best interests of society, even if there is no mandate for it on bridges costing less than $25 million. Life-cycle cost, as opposed to first-construction cost, computes all costs of a project during its lifetime, including operation, maintenance, and repair. It is easier, however, to give lip service to this concept than to put it into practice.

Value engineering was mandated by Congress in 1995 for all projects on the National Highway System, although its present definition departs significantly from recent practices in bridge construction. The new definition requires a *"systematic process of review and analysis of a project or activity during its design phase by a multidisciplined team of persons not originally involved in the project or activity."* This clearly does not constitute an invitation for entertaining construction cost-cutting proposals by the successful low bidder, as has been the rule, rather than the exception, in the past. Both life-cycle costing and value engineering are discussed in Chapter 2.

As with all other federal agencies, the Federal Highway Administration has been mandated since January 1994 to adopt the metric system. Deadline for constructing new highways in the metric system is October 1, 1996. Almost all states are also preparing to convert their standard specifications, and at least 39 states will have completed pilot projects before the 1996 deadline, according to estimates.

The Ontario specification is, of course, set forth in metric units. The AASHTO material is in English units, although metric AASHTO/LRFD specification can be purchased. Meanwhile, further information is available from the Construction Metrication Council, National Institute of Building Sciences, 1201 L St., N.W., Suite 400, Washington, DC 20005.

CHAPTER 2

THE ECONOMY OF BETTER BRIDGE DECKS

If life-cycle costing is to become a fundamental part of bridge and bridge deck design, it follows that the economy of better bridge decks will become apparent. It's obvious that we need better decks than those that are failing throughout the country. The place to begin life-cycle costing is by reviewing the relationship of the deck to the bridge structure.

The deck of any highway bridge, short span or long span, fixed or movable, has one primary function: to deliver the reaction from vehicle tires to the framing system that supports the deck. Additional functions include the following:

- Serving as an important addition to the top flange of stringers, floor beams, and sometimes the main girders, thereby behaving compositely with framing members.
- Improving structural capacity and/or serving as part of the stiffening system for the bridge (composite with the top or bottom chord of trusses). In long-span bridges, the deck may be used as part of a system to limit torsion deformation of the bridge superstructure.
- Extending the width of usable roadway by design as a continuous cantilever beyond the fascia members.
- Acting as a slab structure, without framing members in the case of simple-span structures up to approximately 40 ft in length. A deck can also be designed as a continuous ribbon over multiple transverse supports.
- Acting as a horizontal diaphragm; the deck may be the most stiff element of the entire bridge superstructure.

When decks are designed as replacements for existing decks, the requirement to maintain traffic during construction may dictate designs that can be completed under severe construction "window" restraints. These decks may have to carry traffic loads within hours of installation, and may require a design that maintains the structural integrity of the bridge superstructure without interruption.

Central to the design of bridge decks is the question of how they are loaded. The National Cooperative Research Project (NCHRP) 12–26 examined the distribution of wheel loads on highway bridges. Much of the findings of this research were incorporated in the AASHTO/LRFD (load and resistance factor design) specifications, the table of contents of the bridge deck portions appear in Chapter 7. There is an important issue here: are bridge decks only flexural elements, or does "arching action" (also known as a "compression membrane") more accurately portray the real behavior of decks?

The Ontario Ministry of Transport specifications for bridge decks are based on the "arching action" concept of deck performance. This conclusion led to the development of "isotropic" reinforcement for concrete decks, which is included in the AASHTO/LRFD specification as "empirical design" (Article 9.7.2). Most recently, this thinking has led to the construction of one span of a bridge in which the concrete deck has no internal reinforcement at all. Flat steel straps are welded to stringer top flanges, spanning transversely to traffic under the unreinforced concrete slab. Cantilevered slabs are, of course, not feasible.

It appears that there is a fundamental question about the design of reinforced concrete bridge decks: Can we assume that flexure of the deck alone is not the significant factor in the deterioration of these decks? An independent researcher, John Allen, of Boulder, Colo., has documented apparent flexural cracking of isotropic decks. Allen's research has led him to advocate the elimination of the top mat of rebar except from the edges of the deck to the middle of the first bay between stringers. He believes that the deflection of floor-system framing members eliminates most of the negative bending moment that would occur in the slab if it was supported on unyielding supports (as was assumed by Westergard). Since the corrosion of rebar has been identified as the culprit in the destruction of reinforced concrete bridge decks, he deduces that eliminating most of the top rebar will save money initially and in the long term.

CRITERIA FOR BETTER DECKS

Satisfactory bridge decks must be both constructable and maintainable. Durability is a must, because the deck is the most frequently overloaded component of a highway bridge. Weigh-in-motion studies have shown that axle loads between two and three times the legal limit are not uncommon on our highways and, of course, on our bridges.

Stiffness is essential for deck longevity. Regardless of the wear characteristics of the top surface of the deck, stiffness is a sine qua non for long deck performance, as well as for long service performance of any overlay.

Stiffness of a deck is achieved in three ways: by the superstructure (particularly against torsion); by the floor system and stringers (if used); and by the deck itself.

It is certain that a mandatory limit of truck tire pressure, if such a regulation could be proposed and adopted, would reduce the deterioration rate of road pavements as well as bridge decks. However, lobbying by tire and trucking interests is likely to send truck tire pressures up, not down, in the future.

Making the deck composite with floor-system members is a fundamental requirement for long-lasting bridge deck construction. Composite action adds to the stiffness of both the floor system and deck. Without composite action, almost all types of bridge decks fail, as noted in these examples:

- Cast-in-place concrete decks. Absence of shear connectors in most decks built prior to the early 1950s exacerbates damage to these decks. Water infiltration at transverse joints, between the bottom of the slab and the top of steel stringers, causes the slab to lift in freezing weather and is followed by accelerated rubbelizing of the slab by the pounding of traffic.
- Precast concrete decks. Non-composite modules installed on intermittent polymer concrete pedestals failed at the Woodrow Wilson Bridge, in Washington, D.C.,
- Half-filled steel grid decks. At the Tobin Bridge, in Boston, the area between the top of the floor-system members and the bottom of the concrete filling was left open. The unstiffened webs of the main bearing bars of the grid were insufficient to achieve composite behavior, and field welding of longitudinal joints between grid modules failed.
- Full-depth filled steel grid decks. One of the many problems on the Williamsburg Bridge, New York, was caused by inadequate attachment of the deck to the framing of the outer roadway floor system. Another problem was caused by grid "growth" resulting in a convex deck shape, separating the deck from continuous contact with the floor system.
- Steel orthotropic decks. Failure to achieve adhesion of the paving course to stiffened steel plates was a form of lack of composite behavior on bridges such as the Ben Franklin Bridge in Philadelphia, the Throgs Neck Bridge in Queens, N.Y., and the upper deck of the George Washington Bridge. All suffered early failure. Fatigue failure of welds between the deck plate and stiffening webs or other elements of this type of welded deck construction also resulted in loss of composite behavior, thereby accelerating deck damage.

- Aluminum orthotropic decks. Failure to achieve adhesion of a paving course to this type of bridge deck led to very limited use before abandonment of the effort by the aluminum industry several years ago to introduce an aluminum bridge deck technology.
- Timber decks. These lose any composite action as a result of timber shrinkage, which causes nails or other connectors to loosen.

CHECK THE STRESSES

Another aspect of bridge deck performance to keep in mind is that maximum theoretical stresses in negative bending (over a floor-system member) are not as damaging to the deck as the same level of stress in positive bending (between floor-system members). The reason is that there is a line support in areas of negative bending, whereas in positive bending areas (between floor-system members), tire paths wander to some extent. The result is that the deck is subjected to a kneading action by traffic.

The following items are considered by the New Jersey Department of Transportation (DOT) when evaluating alternative bridge deck options. Their base of comparison is success with their own tried and proven reinforced concrete bridge decks:

- Estimated service life—goal is 75 years, with maintenance
- Proven service-life history
- Load-carrying capacity
- Compliance with AASHTO and New Jersey DOT specifications
- Resistivity to deicing salts
- Abrasion resistance, leaving skid resistance intact. (Some old pavements are very strong and hard, but the coarse aggregate has become polished by traffic, reducing skid resistance.)
- Maintainability
- Repairability
- Inspectability (Stay-in-place forms are permissible except if the top surface of the deck is covered, as in ballasted railroad bridges and asphaltic overlays. No asphaltic concrete overlays are used.)
- Whether the Federal government will pay its share

COPING WITH LIFE-CYCLE COSTING

Life-cycle costing (LCC) is the answer to problems brought on by decades of low-first-cost construction. In principle, LCC is obvious: consider every factor over the life of a structure in determining the true cost of ownership. In practice, LCC will revolutionize the entire construction industry, from design to the final paint job, and beyond to maintenance and operations procedures throughout the life of the structure.

A Federal Highway Administration (FHWA) publication in 1994 defined LCC for highway and bridge engineers: "Life cycle cost analysis is the evaluation of agency, user, and other relevant costs over the life of investment alternatives. Evaluating total costs over the life of an alternative is essential if improvements that minimize long-term costs are to be identified. Improvements with the lowest initial costs are often more costly in the long run than alternatives with higher initial costs, especially if costs of traffic delay during maintenance and rehabilitation activities in congested areas are considered."

Putting LCC into practice—and this need is clear if one attends almost any discussion on the subject—requires a redefinition of the mission of all public works agencies at all levels of government. The rules for funding projects must be worked out in light of each project's lifetime, not just for its first cost.

That is why LCC scares some people. Rather than being the designer's problem, LCC principles must be put into practice by every person responsible for the project—owner, designer, specifier, and purchasing agent. If any one of these people opts for a low first cost regardless of quality or durability, the project is thrown back into the dollar-competitive pot.

LCC has many critics who say it may be a fine practice for others but not for construction. The fact is, LCC turns everything any engineer has learned in a lifetime upside down. Yet LCC has been the law, at least at the federal level, since 1991, when the the Intermodal Surface Transportation Efficiency Act (ISTEA) mandated consideration of "the use of life-cycle costs in the design and engineering of bridges, tunnels, or pavement." Optimum life-cycle cost became the new policy, but FHWA has yet to reduce this requirement to actual procedures.

The theme song of many objectors is "Not Here, Not Yet." Some dismiss LCC as leading to increased first cost, claiming that there is not enough money to do all the needed work anyway, and ask how we can justify spending more on any single job than is absolutely necessary. The answer is that if any job is worth doing with public funds, it is worth doing well.

Other objectors appear to believe that the first step towards LCC must be development of an acceptable methodology for determining two factors: (1) the remaining useful life of a specific bridge; and (2) the predicted useful life of materials and structural systems or components. They assume that using probability-based methodologies to develop an enormous database will provide tools that will permit precise evaluation of a specific bridge, system, or component, "some day."

The plain fact is that such precision is unobtainable because a large part of any life-cycle cost equation—*maintenance*—is extremely unpredictable for bridges, highways, and other public works. Some states, notably Pennsylvania and Ohio, have succeeded in building maintenance into their overall bridge programs. New York City engineers embarked on an extensive and expensive bridge maintenance program several years ago, only to see it gutted by succeeding mayoral administrations. The reasoning is the universal, "We can't afford it."

In January 1994, President Clinton signed Executive Order 12893, "Principles for Federal Infrastructure Investments," which requires systematic analysis of expected benefits and costs over *the full life cycle* of each project. The order notes that some benefits and costs are uncertain, requiring consideration of qualitative measures to reflect values that are not readily quantifiable. It also requires agencies to consider "design standards that incorporate new technologies and construction techniques."

The final language of the 1995 funding bill for surface transportation limits the mandate for life-cycle cost analysis by the Secretary of Transportation to all federally aided projects that cost over $25 million. This last minute emasculation of life-cycle cost analysis for bridges will certainly be reversed in the future. If a fundamental change in the method of evaluating options in the design of bridge projects is to apply to all projects costing over $25 million, it would be nonsense to assume that the change has no relevance for the vast majority of bridge projects which cost less than this amount.

FHWA, the Transportation Research Board, and other groups have begun to schedule conferences and seminars where civil engineers may learn to apply LCC as a rational thought process rather than wait for a formula to be worked out in the future.

ANALYZING LIFE-CYCLE COSTS

It is indeed rational to prepare a life-cycle cost analysis as an extension of construction cost analysis. Materials and systems may not perform as hoped, traffic volumes may grow at an unanticipated rate, funding for maintenance may not be available at times, but the LCC-minded owner will still be better off than the low-first-cost–minded owner.

Even though their circumstances are different, civil engineers can learn from their colleagues, electrical and mechanical engineers. The energy crunch of the 1970s forced mechanical and electrical engineers to recommend building systems on the basis of LCC. Owners bought into the idea, for instance, that furnace A, if more efficient than furnace B, would cost less in the long run. In addition to escalating energy prices, inflation and rising interest costs helped focus attention on total long-term costs. Several states passed legislation requiring LCC for all public building projects, and at the federal level, the General Accounting Office (GAO) made a series of studies for the Department of Energy.

Owners, public and private alike, were surprised by GAO analyses that put the construction cost of an office building at just 2% of the total life-cycle cost of owning, maintaining, and operating that building. The figures for hospitals were a bit different: operating costs in the first three to five years exceeded construction costs.

However, LCC was a part of the building process long before the energy crunch. For federal and state projects, it entered into the benefit/cost ratios required for approval. For industry, it was, and is, an important element in decision making by profit-oriented businesspeople. Whether or not they call it life-cycle costing, they have developed formulas and equations that can now be brought over directly to civil engineering's roads and bridges.

There are several financial considerations that go into these equations, and the terminology is the same for any analysis, be it a building, highway, or bridge. Instruction in the methodology is available through books, seminars, and special courses that will take the engineer through concepts that begin with the cost of capital and the time value of money. Various calculations lead to comparing lifetime expenditures by either the present worth method or the uniform annual cost method.

There are, however, several very real differences between using LCC to determine a building's optimum HVAC system and using it to determine the service life of a bridge deck. There are infinite variables that affect future costs and maintenance needs.

Even determining what the life span should be can baffle engineers. In the United States there is a tendency to see 70 years as a desired useful life of a bridge. In England and some parts of Europe, the project design goal is 120 years. While the difference in goals says a good deal about the difference in attitudes, the important point is to begin LCC by selecting a life cycle that will be the measure for all investment alternatives.

Another factor, which has had no discussion so far among highway and bridge owners, is safety. Who can put a number on safety as a factor in the LCC equations? But just because there are such difficulties is no excuse for not complying with the directives to use LCC in planning and designing bridges and bridge decks.

It is not enough to declare that life-cycle cost analysis is important, or even paramount. The analysis must show why and how the data are compiled.

Defining the life cycle for the specific project to be analyzed is the first step. Arbitrary or not, this selected life cycle will be the measure for all investment alternatives.

A life-cycle cost analysis should simply present as many investment alternatives as the project manager selects, with a level playing field established consisting of all factors deemed relevant. For instance, if the project involves the rehabilitation of a bridge deck (or its replacement) an obvious item of input must be an estimate of the remaining useful life of the bridge itself.

Consideration of the useful life may include the substructure and superstructure only or, more properly, the function of the bridge: capacity (traffic volume and live load) and its relationship to the existing and future area transportation system.

The items to be included, such as maintenance of traffic, user costs, discount rate, and others are discussed in FHWA and Office of Management and Budget (OMB) publications. Some items, such as maintenance of traffic, demolition and removal, and salvage value, will vary from project to project. For a life-cycle cost analysis of a deck rehabilitation or replacement, the input for each viable deck option must include

- First cost (i.e., construction contract)
- Estimated remaining future life of the superstructure
- Anticipated traffic demand growth plotted on a time scale
- Anticipated maintenance schedule requirements and costs plotted on a time scale
- Estimated future life of the deck
- Deck replacement cost if applicable (include in maintenance graph)
- Discount rate plotted on a time scale
- Estimated vehicle delay cost per hour plotted on a time scale

THE NEW YORK EXPERIENCE

New York DOT has formalized the life-cycle cost analysis of various deck treatments. Even though the treatment is dictated primarily by technical considerations, the engineers are directed to perform a life-cycle cost analysis as follows. It uses the present worth method with a 4% discount rate to convert expenditures occurring at different times into equivalent amounts occurring at the present.

Present worth is computed as

$$PW = C \times SPPWF \qquad (2.1)$$

where PW = present worth of the expenditure; C = future cost of the expenditure; and SPPWF = single-payment present worth factor for an expenditure at year Y.

The total present worth for all treatments needed to extend deck life through the planning horizon equals the sum of the individual values.

Other economic analyses required are the uniform series of present worth factor for Y payments (WSPWF) and the capital recovery factor (CRF) over Y years.

In an example, given by the New York DOT, two treatment sequences are technically appropriate for a deteriorated deck. Assume the following costs and service lives:

Asphalt overlay	$1.92	4 years
Select deep removal	15.27	25 years
Replacement	39.60	40 years

Treatment A involves an immediate asphalt overlay with a deck replacement in 4 years.

$$\begin{aligned}\text{Treatment A} &= 1.92 + 39.60 \times \text{SPPWF [sub4]} \\ &= \$385.02/\text{m}^2\ (\$35.77/\text{sq ft})\end{aligned} \tag{2.2}$$

Treatment B involves a select deep removal and overlaying at 25-year intervals. For treatment B, two applications provide 50 years of service compared to the 44 years provided by treatment A. Because of this difference in years, the costs of Treatment B must be adjusted to a planning horizon of 44 years:

$$\begin{aligned}\text{Treatment B} &= 15.27 + 15.27 \times \text{CRF [sub25]} \\ &\quad \times \text{USPWF [sub19} \times \text{SPPWF [sub25]} \\ &= \$216.25/\text{m}^2\ (\$20.09/\text{sq ft}).\end{aligned} \tag{2.3}$$

Treatment B clearly provides the lower life-cycle cost.

We believe the procedure for performing all life-cycle cost analyses will involve the use of ranges of values, rather that simple integers. This will make possible the evaluation of materials and structural systems for which long-term experience records are not yet available.

To do otherwise would be to rule out all new ideas and discoveries. Besides, the mechanical and physical properties of all construction materials that we use in bridge deck construction are, in fact, far from uniform and may themselves be considered approximations.

CHAPTER 3

SELECTION OF BRIDGE DECKS

In the past, when lowest first cost was the only purchasing determinant for any bridge project, cast-in-place concrete decks were the most commonly used. In fact, these decks became, and still are, the only deck type that has ever been standardized by the Bureau of Public Roads, the Federal Highway Administration (FHWA), and state DOTs.

Now that life-cycle cost provisions have been written into federal highway legislation, this limited evaluation approach to bridge project design will surely end within the next few years. Deck rehabilitation and replacements, especially, require an in-depth selection process because of the need to maintain traffic during reconstruction, a factor not included in a new bridge design checklist.

Deciding on a deck type is an integral part of the development process of any bridge project. Indeed, the checklist for deck selection is not much different than the checklist for the entire project, which is worth repeating here. The steps, or phases, are

1. Recognition of the need for a specific bridge
2. Definition of the scope of work to be accomplished for the project
3. Selection of the design entity (group) by the bridge owner (agency)
4. Confirmation of the scope of work by the design group
5. Development of a schematic design for the project; this may be for internal use by the design group
6. Preliminary design and estimate of construction cost; this allows the owner to compile cash flow projections
7. Preliminary report on life-cycle cost issues for the project
8. Final design, drawings, specifications, and estimates of construction cost and life-cycle cost
9. Advertisement for bids
10. Receipt of bids

11. Award of construction contract
12. Construction and inspection (by owner or its agent)
13. Completion of as-built drawings and final program for future maintenance

There are additional activities or phases, such as funding, authorizations and permits, public hearings, and so forth, that complete the picture, but they are beyond the scope of this chapter. Without steps 1–2 taking place in some form, no bridge construction project has ever been completed. Phase 13 is often ignored, but the push for life cycle costing now makes it mandatory for most projects.

THE NEW BRIDGE

In new bridge projects, the deck fits into several phases of the project planning sequence, beginning with phase 2. This deals with the scope of work, which should include information about the construction timetable, that must be met. Actual selection of the deck type must be made during the schematic design phase (phase 5). The reasons for this are that the deck width and weight must be established before phase 6 can proceed.

In a new bridge project, the maximum number of deck-type options are available because floor-system framing members and their spacing are established at the same time the deck type is selected. The only general rule of thumb is that the costs of a steel superstructure increase with deck weight, but the cost of the deck itself decreases as its weight increases. Cost/weight comparisons are not as critical with concrete as with steel superstructures, but there is great value in careful comparison of deck types for any bridge design.

With the advent of life-cycle costing and total quality management, first cost is only part of the question. Decisions must now factor in longevity and ease of eventual repair, renovation, or replacement. The logical way to approach decision making is to set up a matrix and assign weighted numbers to each of the attributes—from critical through desired to unimportant and not applicable. A more complete sample matrix is included at the end of this chapter, with arbitrary weighting values and commentary. It was first published in *Steel Bridges,* a publication of the Steel Bridge Forum, in 1989.

To use such a matrix, the engineer lists all review criteria and assigns a weight to each according to its importance. For instance, if deck weight is critical, it is assigned a high "importance factor" number on a 1–10 scale. If first cost is important to the owner, it also rates a high number. Ease of inspection might be rather unimportant and thus rate a low number.

Deck types are listed across the top of the matrix form, then rated for each criterion listed in the first column. Ratings, on a scale of 1–5, are multiplied by the importance factors and totaled for each deck type. Finally, weighted ratings for each type are obtained by the sum of importance factors. As an example, if the importance factors add up to 60, and the individual ratings add up to 206, the weighted rating of a deck type would be 3.43.

Deck types considered are cast-in-place concrete, precast concrete, prestressed concrete, exodermic, half-filled grid with overlay, filled grid with overlay, open grid, timber, steel orthotropic, and aluminum orthotropic.

COMPATIBILITY WITH THE SUPERSTRUCTURE

The selection of a deck obviously must incorporate a global view of the project. Compatibility with the superstructure can mean many things. Questions to consider include the following:

- Can the deck be widened in the future?
- Does the required stiffness of the superstructure neccessitate attachment of the deck to the main supporting members?
- Will future inspection and/or maintenance of the bridge and the deck be affected adversely by the deck selection?
- Will the selected deck survive anticipated superstructure movement, vibration, and deflection due to wind loading, seismic events, and live load?
- Will the selected deck, its installation details, and attached appurtenances be compatible with any anticipated future maintenance, repair, or replacement needs?

REHABILITATION PROJECTS

Just as in the design of a new bridge, for which a type, size, and location study must be completed, the design of a rehabilitated or replacement bridge deck should be preceded by a similar study. A list of options, and the consequences of each, should be analyzed. Bridge rehabilitation projects will almost always benefit from use of a lighter, and composite, deck replacement.

For a rehabilitation project, the scope of work should include traffic requirements. Will the bridge be closed during construction, or will maintenance of traffic limit deck selection options? Also, is live-load capacity to be increased, and how does it pertain to weight and/or number of traffic lanes?

The remaining life of the bridge superstructure and substructure must be assessed. Any loss of floor system and main structural member capacities due to corrosion, erosion, or fatigue must be determined. The scope of work (phase 2) will have established the live-load criteria to be used in the project. Light weight of the deck may very well be the most important criterion in the deck selection matrix in order to minimize the need to replace or strengthen existing superstructure members.

Before compiling a deck selection matrix for a rehabilitation project, however, several questions must be considered. These include

- What is the remaining useful life of the superstructure?
- Are there any substructure problems that might supersede other considerations?
- Can the existing deck be repaired to serve for the remaining useful life of the bridge?
- Does the existing floor-system layout limit the choice of deck options (i.e., maximum span required)?
- Should the new deck be made composite with the floor-system?
- Can the new deck be made composite with the main bridge members, if desired?
- Can joints, scuppers, railing, and barriers be readily accommodated by the new deck?
- Are there availability, time, or location constraints for new deck options? If so, what will be their effect?
- Are there any considerations about constructability or speed of construction that may limit deck options?
- What are the desired live-load and impact criteria?
- Can the existing superstructure and floor system support the heaviest deck options without reinforcement?
- What is the weight per square meter (average), including all joints, haunch material, etc., that can be accommodated by the superstructure and floor system? (This is actually an output of the previous considerations.)
- What is the required future useful life and estimated Average Annual Daily Track Traffic (AADTT) demand growth?
- Is there adaptability to staged replacement of the existing deck?
- Will there be adaptability of the deck to future widening?
- What maintenance efforts will be required? The top portion of the traffic surface will require periodic resurfacing to replace worn areas and restore a smooth cross section, profile, and skid resistance.
- What about skid resistance, and future renewal thereof?
- What is the resistance to permanent damage from occasional overload, which should be expected to occur?
- What is the replaceability and cost of removal of the original deck (for the life-cycle cost analysis)?
- What is the life-cycle cost of each deck option (to estimate the future useful life of the bridge)?
- What will the initial cost of construction be, for current year budget purposes?

When the matrix has been completed, there are still questions to be considered, particularly those involving the required schedule and sequence of deck construction. If traffic must continue to use the deck replacement on a daily (or other) basis during the project, then the cross section and profile of the new deck modules, as they are placed, must match the existing deck.

PLANNING PHASE FIVE—SCHEMATIC DESIGN

Before design can begin for either a new bridge or a rehabilitation/replacement project, generic questions must be resolved about the future bridge deck. Some of these items are required by code, others depend on local accepted practices, but all affect the ultimate design of the deck.

- Is a centerline barrier required? Most bridges do not have centerline barriers, but if a bridge is part of an approach/exit roadway to and from another roadway, a barrier will enhance safety.
- Are shoulder barriers required? If the shoulder is for vehicle use only during emergencies, there should be no barriers. If the shoulder accommodates lanes for pedestrians and/or bicycle riders, it should be separated from the vehicle lanes by a barrier.
- Should the sidewalks (pedestrian and/or bicycle) be designed for different loads than the vehicle lanes? The design loads can be lighter if a barrier excludes vehicles and thus their wheel loads.
- Are railings required? In almost all cases, the requirement for railings is that they be designed for vehicle impact loads unless there is a shoulder barrier, in which case separate loads are calculated. (Chapter 13 of the AASHTO/LRFD *Specifications* presents a worthwhile discussion on this subject).
- Are there lighting and/or sign supports? These can pose significant structural and space requirements.
- How is surface drainage to be handled? The profile grade may or may not be sufficient to meet maximum ponding limitations (the distance within the travel lane that may be covered by water during a storm of a stated frequency).
- Where are the superstructure expansion joints located? What type are they? Must they all be retained? What type of joint is to be used in the deck? What amount of movement must be accommodated?
- What live load and impact criteria are desired for the deck?
- What considerations should be made for aerodynamic stability? Will spoilers or other devices be needed at deck boundaries?
- What is the present condition of floor-system framing members? What materials are used?

Consideration of the above items will establish the following: deck cross section; location and condition of floor system framing members; maximum dead-load capacity of the superstructure, after deducting live load and impact requirement, less an allowance for loads imposed by utilities, snow, ice, and any future wearing surface; required composite properties of floor-system framing members; and life-cycle costs of the selected deck type, including user costs during deck replacement (i.e., costs to motorists of traffic tie-ups, diversions, detours, etc.).

CONSIDER THE DECK JOINTS

Deck joints should be selected at the same time as the deck type so that they will be compatible. When it comes to bridge deck joints, the best design is no joint at all. That's easy with very short decks on very short bridges. It requires more effort on other spans to leave the deck intact.

More than a decade ago, Tennessee DOT embarked on an ambitious program to eliminate deck joints from the design of new bridges. The technique places expansion joints behind the abutments and treats piers and abutment breast walls as "rocker bents" in some bridges. Results have been good, according to the agency, although some problems have developed with the asphalt paving at the ends of several decks. Adding compressible material to those areas seems to have alleviated the problem.

Tennessee DOT sticks to its original limit on the length of jointless bridges—245 m (800 ft) for concrete and 120 m (400 ft) for steel bridges. Other states have been far more cautious, and most use 45 m (150 ft) as the limit for either type.

Where there must be deck joints, however, the designer must choose from an array of commercially available joints, none of which is entirely satisfactory. The three basic types are (1) modular expansion joints, including finger and sliding plate dams, providing movement of 50–660 mm (2–26 in.); (2) metal reinforced expansion joints, providing up to 330 mm (13 in.) movement; and (3) strip seals and armored expansion joints, including preformed neoprene seals, providing up to 100 mm (4 in.) of movement.

Each type is produced by one or more manufacturers under various trade names. All are purported to meet the criteria set forth by AASHTO design practices, which state: "The design shall be such as to allow for total thermal movement at the rate of 1-1/4 in. in 100 ft for steel. Provisions shall be made for changes in length of span resulting from live-load stresses. In spans more than 300 ft long, allowance shall be made for expansion and contraction in the floor. The expansion end shall be secured against lateral movement."

AASHTO believes that a good expansion joint should

- Accommodate all movements of the structure
- Withstand all loadings
- Have good riding qualities
- Not present a danger to cyclists and other types of traffic

- Not impart undue stress to the structure unless it has been designed accordingly
- Be reasonably silent and vibration free
- Give reliable service through all expected temperatures
- Resist corrosion
- Facilitate maintenance and repair
- Control deck drainage to prevent damage to the structure below

RATING THE JOINTS

In the mid-1980s, an FHWA–sponsored research project was conducted by Pennsylvania DOT (PennDOT), which evaluated the various joints being used in that state. It paid special attention to the performance of several reinforced elastomeric expansion dams and gland-type expansion dams, and included modular, metal-reinforced, and gland systems, as well as the more common finger dams, armored neoprene, and preformed neoprene compression seals. The study was prompted by a nationwide rash of damage to bridge deck joints in relatively new bridges. Earlier, PennDOT had studied damage due to deicing chlorides, snowplows, and heavy truck traffic. The agency found that 76% of the joints studied were either completely open or leaking water onto the superstructure.

In the expansion joint study, PennDOT engineers looked for compliance with FHWA objectives, substituting "need of maintenance" for the too vague "ease of maintenance" criterion. Although drainage was not considered a part of the study, the engineers noted that in many cases drainage maintenance had probably contributed to the joint problems.

At the time, PennDOT was using three basic types of joints: (1) open joints protected by armored neoprene or preformed neoprene compression seals; (2) strip seals, metal plates with neoprene strips; and (3) finger dams, toothed bearing supports that transmit traffic loads across the joints.

Some of these joints were equipped with drainage troughs to direct water away from the structure below. The most common problems were related to poor construction (especially misalignment) and lack of maintenance. Debris blocking the dams or troughs was laid to slopes flatter than 8%, although periodic maintenance flushing can help prevent corrosion by the blocked contaminated water.

Cantilevered finger joints are often simply not strong enough or well enough anchored to withstand heavy pounding by trucks. These problems may be traced to design, manufacture, and construction. In armored neoprene and preformed neoprene compression seals, corrosion may occur if plates are not well protected by paint or epoxy.

Strip-seal and gland-type devices are prone to debris accumulation and leakage, whereas problems with metal-reinforced elastomeric and continuous-belt dam systems range from poor anchorage to damage from traffic and snowplows. If the neoprene tears or its anchorage becomes loose, leaks and debris accumulation follow.

The PennDOT engineers found defects in all types of deck joints because of poor design and/or construction, and poor maintenance. Skewed joints are particularly troublesome, and it is possible to blame heavy truck traffic for much of the damage to any deck joint. The high number of failures related to anchorage problems led to a recommendation that anchors be cast as part of the concrete construction, securely fastened to the reinforcing steel.

The general lessons for bridge deck designers are spelled out in a series of questions to ask before specifying any joint:

- Can the system stand the continous pounding of traffic, especially heavy truck traffic? Can it be protected against damage from snowplows?
- Is the system designed so that water, deicing chemicals, sand, and other debris will not collect in it?
- Will the system perform if maintenance is neglected? (Remember, preventive maintenance is still an oxymoron.)
- How easy will it be to maintain the system with minimal traffic delay?

Those questions appear in the study report "*Bridge Deck Expansion Joints*," published in December 1985 by the National Technical Information Service. Since the study was made, PennDOT has led the way in establishing computerized maintenance procedures for all bridges. Other states have also adopted better maintenance as a policy, even under severe budgetary restraints.

In the decade since that study, little has been done to improve commercially available deck joints. The type known as "compression seals" (longitudinally honeycombed extrusions squeezed into the transverse gap in the deck) has fallen out of favor. Too many popped out onto the roadway like gigantic rubber bands. These have generally been replaced by "strip seals," or "glands," which are shaped like a dumbbell in cross section, with the V-shaped web pointed downward. The strip seal "gland" is inserted into steel female extrusions anchored to the deck structure on each side of the gap. This "gland" arrangement is also used in multiple arrays in the construction of "modular" roadway joints, where the amount of movement to be accommodated is large.

Bridge engineers must still use caution. The most commonly used material in the strip seal is neoprene, which has a tensile strength of about 17 kPa (2,500 psi). Recently, a competitor has appeared with a tensile strength half that, and the results are predictable. Unless the seals are cleaned out frequently (which simply does not happen), road debris that falls into the V stays there, abrading and tearing the material as the joint opens and closes.

DRAIN THAT DECK

Another item to be considered early in design of a bridge deck is its drainage. What is the best way to get rain and snowmelt off the riding surface quickly and safely?

Bridge drainage is a complex problem that must be solved by committee—or at least cooperation among a number of experts: the structural engineer, hydraulic engineer, traffic/transportation engineer, and maintenance engineer, as well as the administrator and staff concerned with design, operation, and maintenance of the bridge and the highway or road it carries. Lack of coordination between the bridge and highway designers, for instance, leads to unacceptable conflicts such as guardrail supports or other structures blocking the flows to drains. Such problems are most often caused by conflicting design schedules that complete the roadway drainage design well ahead of the bridge design.

On the bridge itself, placement of drainage components may conflict with placement of the reinforcing steel on a concrete bridge deck or with the structural members of the bridge. Hangers that carry pipe may seem inconsequential, but can be difficult to place and secure properly without damage to either the pipe or the structure.

The first consideration about drainage depends on the nature of the bridge. If it spans water, free fall through open drains may be sufficient; if it spans another road or highway, more elaborate precautions must be taken to divert flows safely away from the traffic below.

To some bridge deck engineers, decisions about drainage begin with how much ponding is acceptable in a storm. The answer is usually none—no puddling, no surface water, running or standing, that could cause vehicle hydroplaning. The object is to get the water off the deck as quickly and completely as possible, without causing erosion or contamination below.

Various state highway departments have their own criteria for drainage systems that take weather, traffic, and other conditions into account. Most specify a 1–2% cross slope, with a steeper slope at the shoulders. This is a case, however, where more is not better: too much slope endangers cars when the surface becomes icy. Longitudinal slope is usually 0.5–1%, but, if less, the gutter must be sloped enough to transport water to inlet boxes from high points between them.

Drainage through expansion joints—whether designed as such or accidental—is a major problem that still defies solution after decades of research. The best systems are equipped with neoprene or metal gutters to catch water that comes through them, carrying it safely away. Even these "best" systems are susceptible to debris clogs, leaks, and damage from freeze-thaw cycles, deicers, and sand used to provide traction when the surface becomes icy. Precipitation is not the only factor that can impact bridge drainage systems. Desert conditions, which include lengthy dry spells in any part of the country, can produce blowing sand and grit that can fill drains. Yet the drainage system must handle rains that eventually come, even in the desert. Preventive maintenance and periodic cleaning with back-flushing are the only solutions to these problems.

Screening the drainage system from all debris is impossible, but properly designed grates can keep out large chunks. "Proper design" also includes a metal-to-void proportion that is durable enough to carry traffic loads without snagging bicycle tires or allowing enough water to sheet right across it. In addition to the structural strength of the grate itself, that strength must be carried over to the fastening mechanisms that secure it to the deck.

The next problem in drainage design is transporting the runoff. Most state DOTs specify pipe sizes of at least 150 mm (6 in.) [some say 200 mm (8 in.)], used in slopes "as steep as possible." Sharp bends, tees, and rough interior joints must be avoided, and clean-out plugs should be placed where maintenance crews can reach them easily and safely. Care must be taken so that runoff does not run down the faces of girders and piers, nor drip onto the superstructure. Take advantage of the surface tension of water when designing details.

See Fig. 3.2 below.

TALKING ABOUT DRAINAGE

It's important for everyone to use the same terms during design, construction and maintenance of bridge drainage systems.

Drainage system: entire arrangement of grates, drains, inlet boxes, pipes, gutters, ditches and outfalls that collect water for disposal.

Drain: the receptacle that receives water.

Inlet box: drain set into the bridge deck.

Catch basin or drop inlet: drain set away from the deck.

Grate: ribbed or perforated cover of an inlet box or catch basin.

Outlet pipe: pipe that leads water away from the inlet box, catch basin or drop inlet.

Cleanout plug: removable plug for access to a run of pipe, usually in a wye.

Runoff, drainage, water: Rain or other water (including contaminating liquids) that collect on the deck surface.

Scupper: horizontal opening in the curb or barrier through which water flows.

Sewer: underground piping system that connects to a disposal system.

Fig. 3.2. Talking about Drainage (Source: NCHRP Synthesis No. 67, Bridge Drainage Systems, Dec. 1979.)

BRIDGE DECK RATING MATRIX

		BRIDGE DECK RATINGS														
		CONCRETE				METAL DECK										
REVIEW CRITERIA	IMPORTANCE	A	B	C	D	E	F	G	H	J	K	L	M	N	O	P
Weight of deck	10	5(50)	3	5	3	5	3	5	3	5	3	5	3	5	3	5
Degree of composite action	9	4(36)	5	4	5	4	5	4	5	4	5	4	5	4	5	4
Cost of traffic maintenance	8	2(16)	2	2	2	2	2	2	2	2	2	2	2	2	2	2
Ease of construction	7	3(21)	1	3	1	3	1	3	1	3	1	3	1	3	1	3
Ease of future replacement	6	1(6)	1	5	1	1	5	1	1	5	1	1	5	1	1	5
Quality control	5	4(20)	2	2	4	2	2	4	2	2	4	2	2	4	2	2
First cost of deck installed	4	5(20)	5	5	5	5	5	5	5	5	5	5	5	5	5	5
Deck service life	3	5(15)	5	5	5	5	5	5	5	5	5	5	5	5	5	5
Annualized life cycle cost	2	4(8)	4	4	4	4	4	4	4	4	4	4	4	4	4	4
Conform to AASHTO std	1	4(4)	4	4	4	4	4	4	4	4	4	4	4	4	4	4
Ease of inspection	1	3(3)	3	3	3	3	3	3	3	3	3	3	3	3	3	3
Ease/cost of maintenance	1	3(3)	3	3	3	3	3	3	3	3	3	3	3	3	3	3
Resist corrosion/ deterioration resistance	1	2(2)	2	2	2	2	2	2	2	2	2	2	2	2	2	2
Substructure improvement	1	1(1)	1	1	1	1	1	1	1	1	1	1	1	1	1	1
Availability of fabrication	1	1(1)	1	1	1	1	1	1	1	1	1	1	1	1	1	1
TOTALS	60	(206)														

HOW TO USE THE BRIDGE DECK RATING MATRIX

STEP 1

ESTABLISH REVIEW CRITERIA

Select and list the criteria that should be reviewed to produce the appropriate deck type. Include both major and minor considerations. The list in the matrix is not all-inclusive. Add or delete criteria according to the specific job. Other criteria that might be considered include riding quality, overload capacity and suitability of the deck for jointless bridge construction.

STEP 2

ASSIGN IMPORTANCE FACTORS

For each criterion, assign an importance factor on a scale of 1 to 10, with 10 being the most important. These factors are based on the particular conditions of the structure under consideration and, of course, sound engineering judgement.
Total all the assigned factors at the bottom of the column.

STEP 3

RATE DECK TYPES

List all possible deck types across the top of the matrix. Give each deck type a rating for each criterion listed in the first column. Ratings should be based on past experience and performance, information contained in manufacturer's catalogues and, again, engineering judgement. Ratings are assigned on the following scale:
1 = Least favorable;
2 = Below average;
3 = Average;
4 = Above average;
5 = Most favorable.

Fig. 3.1. Rating Matrix (Source: Steel Bridge Forum, c/o American Iron & Steel Institute, 1133 15th St. NW, Suite 300, Washington, DC 20005-2701. Used by permission.)

FINAL WEIGHTED RATINGS BY DECK TYPE

DECK TYPE	WEIGHTED RATING
CONCRETE	
A. Conventionally reinforced CIP	3.43
B. Isotropic reinforced CIP	2.85
C. Transversely post-tensioned CIP	3.66
D. Precast concrete panel	3.01
METAL	
E. Full-depth filled steel grid, CIP	3.26
F. Full-depth filled steel grid, precast	3.25
G. Half-depth fllled steel grid, CIP	3.43
H Half-depth filled steel grid, precast	2.85
J. Exodermic, CIP	3.66
K. Exodermic, precast	3.01
L. Open steel grid, riveted	3.26
M. Open steel grid, welded	3.25
N. Steel orthotropic deck, closed ribs	3.43
O. Steel orthotropic deck, open ribs	2.85
P. Aluminum orthotropic dock	3.66

STEP 4
MULTIPLY IMPORTANCE FACTORS BY DECK RATINGS
Under each deck type, multiply the importance factor for each criterion by the deck ratings. These products for each deck type are then totaled, as shown in the example by the parentheses for Deck A.

STEP 5
OBTAIN WEIGHTED RATINGS
To obtain the final weighted rating for each deck type, divide the product total (step four) by the sum of the importance factors in column two. Example: Weighted rating for Deck A = 206/60 = 3.43. Ratings for other deck types are found in the same manner.

Note:
This Bridge Deck Rating Matrix is presented as an example. All numbers are arbitrary, for illustration purposes only. No attempt was made to rate one deck type over another, or to list one review criterion as more important than the other criteria.

CHAPTER 4

EVALUATING EXISTING BRIDGE DECKS

Engineers are called on to evaluate existing bridge decks far more often than they are called on to design new decks. Of the approximately 600,000 highway bridges in the United States, most are examined every two years, as required by law. Most state highway departments have well-developed manuals to guide the inspectors, who are responsible for evaluating the entire bridge from foundations through the superstructure to the deck.

Because concrete has been the material of choice for many decades, the bulk of information in these manuals refers to concrete decks. In its introduction, the Federal Highway Administration (FHWA) *Bridge Inspector's Training Manual/90,* for instance, alerts readers to the variety of deck defects they may encounter, but the passage assumes a concrete deck. The manual, widely used to train bridge inspectors, is dated July 1991 and does deal with other deck types as well as concrete. Some of that information is included in this chapter.

CONCRETE

Concrete decks suffer from wear and abrasion, impact damage (especially from snowplows), and overloads. Environmental hazards include freezing/thawing, and chlorides from seawater and deicing chemicals. The inspector should also be on the lookout for design and construction deficiencies such as poor concrete work and insufficient or improperly located reinforcement.

Evaluation of the deck itself should always begin with a field examination of the top and bottom surfaces by an experienced bridge design engineer. A visual inspection will reveal cracking, scaling, spalling, corroding of reinforcement, and delaminations. These most often occur in areas exposed to traffic or drainage.

Flexure cracks occur in the bottom of the slab between the supports, and in the top of the slab in negative moment regions of the deck. Stay-in-place forms may be hiding evidence of moisture and chlorides that penetrate full-depth cracks, as well as the cracks themselves. In precast decks, cracks may be seen in shear key joints between the panels and in anchorage zones of the tie rods, where grout may have deteriorated.

Coring is the most common, and probably most important, diagnostic tool for concrete decks. It establishes a reference base for interpretation of the results of rapid nondestructive testing (NDT) deck survey methods. An engineer from the inspecting organization's office should be present throughout the entire coring operation so that each core may be examined as it is removed from the deck, in addition to subsequent laboratory evaluation.

It is also strongly recommended that the inside of the hole from which the core is removed be examined with the aid of a dentist's mirror as soon as the core has been extracted. This affords the opportunity to spot delamination, determine deck thickness and rebar location (i.e., concrete cover of top rebar), and check the dispersion of aggregate. For instance, if all coarse aggregate has settled to the bottom, leaving sand at the top, the inspector will look for weaknesses and damage as a consequence of the slab being poured as a too soupy mix.

Inspection of the underside of the deck should include an evaluation of the composite behavior of the deck and floor system. Does the intended composite behavior exist or not? Spalling of the bottom concrete cover over embedded rebar will be a significant factor in the estimated future service life of the deck.

Coring locations on the deck should be selected at random, provided that at least one core is taken immediately behind a transverse roadway joint and, in snowbelt states, at the face of the curb and other locations where salt-laden runoff tends to accumulate. The total number of corings should be determined on the spot, not by some arbitrary number chosen back at the office.

When an inspecting engineer sees a problem in one core, he or she will take more cores to determine if the problem is local or occurs throughout the deck. When such problems do not appear, only a few corings are taken; a 450 m^2 (5,000 sq ft) deck, for instance, may have as few as three or four.

The inspector takes close-up photos of each core for permanent documentation, along with notes that will adequately describe the condition of the deck. Cores are then laboratory-tested for compressive strength, air content, freeze-thaw indications of poor durability, and chloride content. The chloride tests are critical in snowbelt states, areas over or near coastal waters, and where the aggregate sources may have included chloride. This determination is made from concrete specimens taken at several depths in a core boring, as well as with nondestructive half cell-equipment used when the concrete top surface is exposed. The initial visual examination of the deck, perhaps confirmed by a limited core boring program, however, may eliminate any reason to carry out any of the expensive NDT alternative methods.

NONDESTRUCTIVE TESTING

Use of half-cell equipment is one of several methods of examining concrete bridge decks without taking cores or otherwise disturbing the slab, hence the term nondestructive, or NDT. The chain drag is the most common, but can be used only on decks that have not been covered with any type of wearing course. Analysis of the echoes made by the chains can identify areas of delamination. Sounding can also be done with a hammer. Some researchers have developed high-tech sonic devices for locating delaminations and other flaws, but these have not been put into general use. The more conventional chain drag and hammer methods are far less expensive and do not require experienced technicians and operators.

Another nondestructive method for decks that have not been overlaid with asphalt or any other surfacing material is thermographic mapping. This is done with special equipment housed in mobile units that operate over large areas at fairly high speeds. On-board computers provide the data that identify distressed areas.

For decks that have been overlaid with a wearing course, ground penetrating radar can evaluate the concrete deck. This method, too, carries a "however." Installation of a cathodic protection system using either a grid or a conductive aggregate in the asphalt will probably interfere with the use of radar.

Within the past few years, "pavement management" has become a buzzword at federal, state, and county levels, and a network of private engineering firms has sprung up to provide computerized pavement management systems (PMS). These firms usually include bridge deck evaluation in their repertoire, and all have acquired high-tech equipment to gather data on the condition of pavements and decks. The firm that won the PMS contract at Chicago's O'Hare Airport, for instance, operates a special vehicle that it says can evaluate all types of reinforced concrete pavement with or without overlays, and, like other such firms, uses its own proprietary computer programs for analyzing data and determining existing conditions.

Several state DOTs are developing in-house pavement management systems, some of which may include bridge decks. Most of this expensive development work, however, goes into the computer analysis software rather than the methods of condition-data collection. PMS, the departments are finding, encompass far more than measuring something and determining what, if any, corrective measures to take. Training staff members to use a new software system can be an ongoing project in itself.

WHAT CAN GO WRONG

Several decades ago, Texas DOT became alarmed at the extent of deterioration of its bridge decks. They were cracking, scaling, suffering from delamination and in some instances even falling to the ground under and over traffic. The agency had just come through a period of constructing the thinnest decks ever allowed by the AASHTO specifications, and was anxious to discover the cause—or causes—of the failures. Speculations included the following comments:

- Decks too thin
- Insufficient concrete cover over reinforcing
- Slabs composite with steel beams
- Slabs not composite with steel beams
- Steel beams too limber
- Dirty concrete aggregates
- Reactive concrete aggregates
- Insufficient cement in concrete
- Water-cement ratio too high
- Magic ingredients in concrete
- Ready-mix concrete
- Excessive concrete placement temperature
- Incomplete consolidation of concrete
- Slow finishing methods
- Lack of uniform curing
- Insufficient long-term curing
- No protective coating provided on reinforcing
- Concrete too young when traffic allowed
- Deicing salts

According to Texas agency officials, most of these factors "probably" had some influence on the deterioration, and "most" have been addressed by various corrective measures over the years. Officials tightened up concrete specifications to increase cover over the top reinforcing and require more cement, less water, cleaner aggregates, air entrainment, controlled placement temperature, better consolidation, improved curing, and longer curing before allowing traffic on the slab.

For areas where deicing salts are used, they developed two alternatives: a two-course asphaltic surface treatment, and a single course of latex asphalt and lightweight aggregate covered with a layer of asphaltic concrete. The latter is said to be less expensive than membrane systems used elsewhere while adequate for moisture protection. Epoxy-coated reinforcing steel is widely used in the top reinforcing and often in the bottom mat as well. Where decks are not topped with asphalt or other coating, the standard protection is a 50-50 mixture of linseed oil and kerosene or mineral spirits.

About 99% of all bridge decks in the Texas system are concrete, although a few experimental bridges have been constructed with decks designed according to the Ontario empirical method.

REPORTING DECK CONDITIONS

All states and agencies have developed their own reporting requirements, usually designating a specific form to be used by the inspector. The documentation must be complete and detailed enough to back up recommendations on repair, rehabilitation, or replacement. Descriptive language must be precise—there's no room for creative writing in a deck condition report. The manual developed by the New York State DOT is reproduced at the end of the book as a good example of procedures and forms developed to aid in the deck evaluation process.

Sketches and color photos should reinforce written data about the core locations; cracks; damp, efflorescent, and rusty areas; spalls; patches; delaminations; and other indications of deterioration. Rebar is checked for corrosion, and concrete rated from sound to rubble. (Note that standard deck inspection forms assume that the deck is reinforced concrete.) Also noted are the framing system, connections, and presence (and condition) of a wearing course.

Finally, the report includes a recommendation for further action, from "No Action" for a completely sound deck to "Repair," "Rehabilitate," or "Replace". Will patching be recommended? An overlay? Or is a partial or full deck replacement in order? At this point, some of the questions outlined in Chapter 3 come into play. No decision can be made about repair unless one knows the remaining useful life of the superstructure and any substructure problems that might influence the deck work. Joints, scuppers, railings, and barriers must be taken into account, and traffic demands reconciled with construction plans.

The probable remaining useful life of the deck should be compared to that of the bridge superstructure. This comparison may indicate clearly that the existing deck, even if serviceable, cannot be expected to last as long as the superstructure. Yet a replacement deck can be expected to serve significantly longer than the probable remaining life of the superstructure. A simile for this thinking is the fact that no one intentionally waits to fill an automobile's gasoline tank until the next-to-last drop is all that is left. Another example, also related to automobiles, is that whatever the replacement schedule is for tires, say 65,000 km (40,000 mi), optimal vehicle trade-in time will be at 55,000, 125,000, or 190,000 km (35,000, 75,000, or 115,000 mi).

TIMBER

Timber decks—plank, nailed, glued, or prestressed laminated—should be examined visually for signs of excessive wear, weathering, and impact damage. If there is an asphaltic wearing surface, both it and the timber below should be checked. Outside areas and those exposed to drainage may show decay, as do bearing areas where the timber deck contacts the supporting floor system. Flexure damage may cause, or contribute to, splitting, sagging, and cracks.

Suspect areas are examined further by a variety of techniques: sounding and probing, drilling, core sampling, and electrical testing. All fasteners should receive thorough attention during the inspection.

STEEL GRID DECKS

Whether unfilled or filled with concrete, steel grid decks should be visually inspected for broken welds, failed fasteners, broken grids, and section loss. Special attention should be paid to bearing areas, where primary bars may be broken, cracked, or even missing, and to areas where trapped water could cause corrosion. Broken connections may often be spotted by listening for rattles as traffic passes over the deck.

Unfilled steel grids may have become slippery because of excessive wear, and concrete-filled grids may have experienced expansion at panel boundaries because of corrosion of the grid, which is not completely restrained by the floor system attachment.

EXODERMIC DECKS

These decks consist of two components that are bonded together. The reinforced concrete upper component, 90–130 mm (3.5–5 in.) thick, should be inspected using techniques described for concrete decks. The open steel grid lower component may be visually inspected from underneath the deck.

STEEL ORTHOTROPIC DECKS

Steel orthotropic decks should be inspected for structural integrity: By definition, the deck becomes the top flange of the entire floor system, acting compositely, within itself, as a flat, thin steel plate stiffened by a series of closely spaced longitudinal ribs at right angles to the floor beams. Various means of inspection will locate broken, bent, or corroded members, as well as cracking at the rib/plate intersection welds, and inspectors also should check for delamination between the steel and the asphaltic or other wearing surface.

DECK JOINTS

Because deck joints perform so many functions, any inspection of the bridge deck must also determine that they are functioning properly. The joints must withstand all weather extremes and accommodate expansion and contraction of the deck. They also fill the gap between deck and abutment backwall to provide smooth vehicle transition on and off the bridge. The joints may be open or closed. Although they are usually not rated on state appraisal sheets, joint problems offen lead to problems elsewhere on the deck and superstructure.

The open types include formed joints and finger plates, also called tooth plate joints. There are six types of closed joints: poured joint seal, compression seal, cellular seal, sliding plate, prefabricated elastomeric seal, and modular elastomeric seal. All joints should be inspected for accumulation of dirt and debris, proper alignment, damage to seals, indiscriminate overlays, and the condition of joint supports and joint anchorage devices.

Seals may be damaged by snowplows, traffic, and debris, and may have been improperly covered by a new overlay. In a finger plate, the individual fingers should mesh together on the same plane as the deck surface—at all temperatures.

Like deck joints, deck drainage is not usually rated on the state appraisal sheet even though it may be responsible for deck or superstructure damage. The sole purpose of any deck drainage system is to transport water and the debris it may carry with it away from the deck and superstructure at a rate that avoids ponding on the roadway. Essential components are drains, outlet and downspout pipes, and clean-out plugs. Grates, designed to keep larger objects out, must be checked for clogging, deterioration, and broken parts.

Troughs, downspouts, and outlet pipes should be examined for clogging, splits, or disconnections. At no point should water be flowing onto the superstructure.

Finally, the condition of bridge barriers and guardrails is important to the condition of the deck. Vehicle barriers and pedestrian guardrails should be inspected for alignment, firm attachment to the deck, and corrosion or collision damage. End treatments—flared, buried, shields, or breakaways—should be inspected for damage, corrosion, and so forth to determine that their functions have not been impaired.

TREATMENTS AND REPAIRS

Repair and rehabilitation of a concrete bridge deck can vary from surface treatments to partial or complete concrete replacement. Surface treatments, such as a simple asphalt overlay, may only mask more serious problems. Their low cost is enticing, but this remedy should be limited to short-term situations such as keeping a deck in service until permanent measures are taken.

Repair of reinforced concrete decks by injecting grouts or sealers has had a mixed and not very encouraging history. New materials may alter this prospect, but any project that appears to be a candidate for such effort should first undergo a life-cycle analysis. If the bridge is expected to last only 10 years or less, it may be appropriate to try unusual or experimental repair procedures.

Protective treatments, such as concrete overlays or asphalt overlays with waterproof membranes, can extend deck life for several years. Because an asphalt-membrane overlay can slip or deform plastically, it should not be used on high-traffic decks, steep grades, or where vehicles abruptly accelerate or decelerate. These areas call for a rigid concrete overlay. The amount of existing material to be removed from the deck is determined by its condition.

The New York State DOT, in its *Bridge Deck Evaluation Manual,* defines service life as the length of time before additional deck work is needed, or the age at which 50% of decks develop delaminations over 40% of their surface areas. The agency has found that deep removal, plus the quality of the removal and reconstruction, have the greatest potential for extending the life of a repaired or renovated deck. The estimates are: Maintenance only, 0 years; asphalt overlay, 4 years; asphalt with membrane (resurfaced after 11 years), 22 years; concrete overlay with select deep removal, 25 years; concrete overlay with 100% deep removal, 35 years; and replacement deck, 40 years.

In general, when rehabilitation requires more than replacing the wearing course, a typical rehabilitation of a concrete deck involves several steps:

1. Remove the existing asphalt or concrete wearing course.
2. Remove all concrete as specified (deep removal) and expose the reinforcing bars so they can be blast cleaned. This removes grease, dirt, concrete, mortar, and loose rust.
3. Place bonding grout on all surfaces; place concrete around the exposed rebars to the level of the surrounding concrete.
4. Apply a protective membrane.
5. Apply the overlay, either asphalt or concrete, to match the highway pavement. Several specialized concretes are available, including high-density, latex-modified, and microsilica concretes.
6. Make required transverse saw cuts to provide skid resistance to a concrete wearing course.

As with any type of construction, concrete decks should be competently maintained to achieve the lowest life-cycle cost possible. Periodic spraying with a mixture of boiled linseed oil and mineral spirits has been done, particularly in Canada. Silicone spraying has also been done in an effort to preserve the deck surface and reduce the rate of penetration of water and chlorides into the concrete.

Clearly, the least costly and most easily justified method is periodic washing with water. This and other procedures designed to extend the useful life of the bridge deck should be scheduled by taking into account the immediate physical environment and the volume of traffic that the bridge carries.

DECK OVERLAYS

Deck overlays are generally used for repair of a deteriorated riding surface. They may be latex-modified concrete, low-slump dense concrete, and hot-mix asphaltic concrete with a preformed membrane. Their main limitations are that they will increase the dead load, and this must be evaluated before specifying any one of them. They, like the coatings, should not be applied over chloride-contaminated concrete nor where alkali aggregate reactions (silica or carbonate) are present.

To prepare a deck for any of the portland cement–based overlays, unsound concrete and all prior patches are removed, then repatched with Portland concrete cement. The entire deck surface is scarified to a depth of 13 mm (0.5 in.), grit-blasted or shot-blasted clean, and flooded with water to leave the substrate saturated but the surface dry. Screed rails are installed to ensure proper thickness. A bonding grout goes under the overlay, which is placed about 6 mm (0.25 in.) above final grade. During curing, the overlay should be protected with insulation if temperatures go below 45°F, and, after curing, it should be grooved for skid resistance.

Several other techniques for inhibiting corrosion with spray-on materials have been developed, but are still considered experimental. The sprays are named Postrite, Cortc MCI 2020, and Alox 901. These permit chloride-contaminated but sound concrete to remain in place, although damaged concrete is removed and patched with a concrete containing a corrosion inhibitor. The entire surface is dry milled, and after three applications of the inhibitor have been sprayed on the surface, a compatible corrosion-inhibitor–modified concrete is used as an overlay.

Spalls, delaminations, and other corrosion-damaged areas may be treated by polymer impregnation, another "experimental" method. After patching, the deck is grooved, dried, warmed by infrared heaters, and allowed to cool to ambient temperature. Then a monomer, methyl methacrylate, is poured into the grooves, where it soaks into the concrete. Heating again polymerizes the monomer. Finally, the grooves are backfilled with a latex-modified mortar. This produces a dense, hard, low-permeability concrete. The polymer, an electically nonconducting material, replaces the pore water and stops the corrosion process.

Regardless of the overlay to be installed or the condition of the deck surface, several items are essential for a satisfactory result. The designer must include weather conditions and preparation of the deck surface in the specifications. These must be rigidly enforced at the site. Delamination of any overlay is usually traceable to improper or inadequate surface preparation, or installation during bad weather. Repairing overlays is both expensive and uncertain because it is difficult to identify all delaminated areas.

RAPID TREATMENT

Most bridge deck repairs these days must be made under traffic conditions that prohibit long-term, or even reasonable-term, closures. When the work can only be done during off-peak hours, or by closing a single lane at a time, the engineer should consider several "rapid repair" methods that are available. Temporary patching materials such as steel plates or asphalt concrete may be used in an emergency, but permanent repair or rehabilitation can also be done with closures of less than eight hours.

Choice of materials is critical on two counts: rapid installation and cure, and early strength that will stand up under the resumed traffic. Before being put into service, hydraulic cement concretes and polymer concretes should achieve 17–27 kPa/ (2,500–4,000 psi) strengths, sealers must be tack-free, and asphalt concrete must be allowed to cool to 65°C (150°F). Steel plates may be used to protect some patches.

Latex-modified overlays are one of the the most widely used types, chosen for their durability and resistance to deicing salts. A quick-curing variation of the mix, known as LMC-III, is formulated with Type III cement, using lower water and higher cement ratios than normal. Silica fume concretes, also quick curing, offer good protection against chloride penetration as well, although there is little long-term performance data to back up short-term promises.

Polymer concrete overlays are available in several forms: dry aggregate broadcast over unfilled polymer binder, a polymer aggregate slurry covered with broadcast aggregate, and a premixed polymer concrete struck off with a vibratory screed. The basic types are epoxy, polymer, and methacrylate. The Strategic Highway Research Program review indicates that curing time can be as little as two hours at 32.2°C (90°F) or as much as eight hours at 15.6°C (60°F), whereas service life may extend to 25 years.

Asphalt overlay can also be classified as "rapid" if installed over a preformed membrane. Effort must be made, however, to protect the membrane lap joints between closures and to assure uniformity of one application to the next.

The most common patch method in use today is to remove the damaged concrete, sandblast the surface, and fill the cavity with a high-performance concrete. Asphalt patches should be used only in emergencies, then replaced with a hydraulic cement concrete. Most experts recommend ready-mix concrete because it can be ordered for "just in time" delivery at an optimum mixture for the job.

BREAKING UP OLD CONCRETE

When old concrete must be partially removed from a bridge deck, decisions must be made about how to proceed. Considerations include the depth of removal—surface only, the cover above the top reinforcing steel, the matrix that includes the concrete just below that, or the core that includes the structural concrete between the reinforcing. Other considerations are the time needed for each method and the equipment, which must not overload the deck.

For surface removal in which the objective is to provide a clean base for a new topping on reasonably small areas of sound concrete, the deck may be scrabbled, planed, sandblasted, or shot-blasted. For large areas, a concrete milling machine or hydrodemolition equipment should be brought in.

Hydrodemolition can also be used to remove cover and matrix concrete simultaneously, or can follow a milling machine that removes the cover. The equipment, which delivers a high-pressure water jet (80–240 MPa or 12,000–35,000 psi), can be calibrated to remove concrete to almost any depth. The method is most commonly used to remove matrix concrete, and its main drawback is expense: the equipment is complex and must be used by highly trained operators.

Pneumatic breakers, or jackhammers, are still valid for many concrete removal projects. While they are considered low-tech and labor intensive—the skill of the operator is important but not critical—breakers are well accepted and common enough to be easily managed. An operator can take a breaker into tight spots where high-tech equipment could never maneuver, or get in and out within short time spans.

PREPARING SPECIFICATIONS

Implementation of any rehabilitation procedure such as the above requires comprehensive documentation of the construction details and the condition of the existing bridge deck. Preparation of the contract documents for bidding, if the work is to be contracted out, requires project-specific specifications.

Project specifications may be set forth in the traditional manner or as performance specifications. The latter, however, may be a poor choice in our litigious society because "performance" can be interpreted as strictly or as loosely as lawyers can imagine. Litigation, or even the threat of it, will not solve immediate problems nor contribute to timely completion of work.

Specifications that detail the means and method of obtaining specific results are far more useful for achievement of the desired end result than performance specifications. If the desired result does not happen, however, they expose the specifier to litigation. He or she bears the burden of proof of the adequacy of the specification, even though lacking the means or the authority to demonstrate the reason for performance failure. One positive aspect of these specifications is that they are an excellent means of preserving and transferring technology.

NDT evaluations of bridge decks are a case in point. The best available equipment, incompetently operated, the readings misinterpreted, and/or reported with errors, is worse than useless. NDT evaluations are critical because they form the basis for the information about existing conditions set forth in the construction contract documents. Therefore, use of random borings to confirm NDT survey data is most important.

In specification writing, the only answer to the litigation problem is to use practice and material specifications that have been developed by voluntary, consensus organizations such as ASTM, with full disclosure of all available information and its source.

To bring suit against the writers of such a specification would be to attack the basis of democracy. To successfully bring suit against the user of such specifications would require the demonstration of a superior alternative. ASTM and other organizations such as the American Concrete Institute, the American Welding Society, and the American Institute of Steel Construction are well established in American engineering and construction practice. Their recommended specifications serve our purpose quite well. When the method of gathering data and the results are disclosed, it would seem that the contractor must live with any discrepancies discovered during the course of the actual construction, conferring with the designers when necessary.

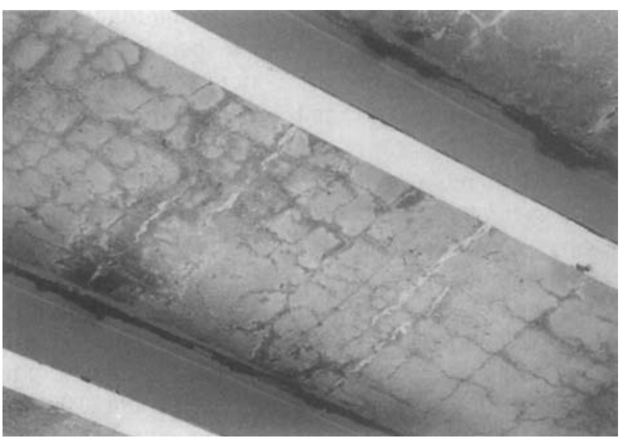

Fig. 4.1. Inspections must be thorough to note deterioration such as "alligtoring" that appeared on the underside of the reinforced concrete deck at New York's Tappan Zee Bridge.

Fig. 4.2. Concrete-filled steel grid deck exhibits deterioration known as "growth" phenomenon on the Williamsburg Bridge in New York City.

CHAPTER 5

SOLID REINFORCED CONCRETE BRIDGE DECKS

Concrete bridge decks are not only the most widely used type in the United States, they are put in place via a wide range of methods. Solid reinforced concrete decks can be cast in place, precast in any number of shapes, prestressed or post-tensioned, or formed with or without internal voids. (A nonsolid deck would be the one incorporated into a special type of box girder bridge made up of trapezoidal precast concrete segments. Such bridges are often cable stayed. Another type has 1.2 m (4 ft) wide rectangular prestressed concrete box girders.)

As with other materials, concrete poses its own set of engineering problems for deck designers, constructors, and maintenance crews. The materials lack uniformity, continuity, perfect elasticity, and other ideal physical properties that could make their mathematical analysis precise. (This is also true, of course, with other materials.) The uncertainties of bridge deck loading pose additional problems. Truck loads vary by tonnage, frequency, impact, tire spacing, and location on the deck. The maximum inflation pressure for truck tires will probably soon be increased from 700 to 875 kPa (100–125 psi). Finally, despite posted legal weight limits, significantly overweight trucks pass over our bridges daily, on secondary road bridges as well as on our superhighways.

The most common type of concrete deck, by far, is the cast in place. This type is rarely designed today; instead, the "designer" selects a deck configuration from one of the national standards or one that has been prescribed by the state or bridge-owning agency. These standards have evolved ever since Westergaard's seminal article, "*Computation of Stresses in Bridge Slabs due to Wheel Loads*," which appeared in *Public Roads* magazine in March 1930. The article formed the basis for standards that became the bridge engineer's design kit.

Westergaard's work verified that of one E.F. Kelley, who had published an article in *the* March 1926 issue of *Public Roads* that itself had referred to prior tests and discussions. Westergaard's contribution was 23 magazine pages of computations of the stresses in bridge slabs due to wheel loads, "it being assumed that the influences of the uniform loads may be estimated with sufficient accuracy by available methods."

The article presented two theories of flexure of slabs. One he called the "ordinary theory," and the other a "special theory." The first is based on the assumption that the plane cross section of a beam remains plane and normal to the elastic curve of the beam, which Westergaard called satisfactory for slabs used commonly in bridges "except for the purpose of expressing the stresses produced by a concentrated load in its immediate vicinity. The difficulty is overcome by use of the special theory."

Westergaard went into minute detail, setting forth fundamental equations and their derivation of fundamental formulas, and the derivation of formulas having direct application to the problem of bridge "floors." He visualized three types of deformations in an element of a slab, produced by the bending moments and twisting moments that would be superimposed on one another. In formulas, tables, and diagrams he charted cases of two-wheel and four-wheel loads under a variety of conditions, and elaborated on the wide range of bending and twisting moments.

What Westergaard did not do was draw conclusions or say "this is how to design a deck." He did ignore a factor that we today consider very important: the deflection of the superstructure itself. This omission has since been seized on by any number of bridge designers who proclaim that they have, by rectifying Westergaard's omission, discovered the true secret of designing better bridge decks. Nevertheless, his theories quickly became part of the various codes. His computations spoke for themselves. Because Westergaard limited the application and discussion of his theory to concrete bridge decks, it followed quite naturally that concrete became the primary material for decks. Yet it is imperative that bridge designers understand deck theory behind today's "pick one out of the spec book" practice. Deck theory is covered in many engineering textbooks for undergraduates, and need not be repeated here.

STANDARDS AND DESIGN PROCEDURES

Published standards for concrete decks are another matter. They have changed over the years from a simple 5.5 kPa (800 psi) for live loads in the Westergaard era, the 1930s, to 0.4 f_c today; from required concrete strengths of 20–27.5 kPa (3,000–4,000 psi); from 25 mm (1 in.) concrete cover over the top reinforcing mat to 51 mm (2 in.) required today—although many authorities believe that 64 mm (2.5 in.) should be the absolute minimum. Kansas, for instance, insists on 76 mm (3 in.) for many cases, such as with single-course systems or when using special coarse aggregates.

No matter what the standards, however, the design procedure behind them is to

1. Select material strengths for both yield strength of steel bar reinforcement and compressive strength of concrete (which defines "n").
2. Determine the maximum positive bending moment in the slab.
3. Use a standard deck design, if one is available, adopted by the bridge owner.

Many engineers practicing today need to be reminded of the ease with which a working stress design of a one-way reinforced concrete slab may be made.

A sample calculation, for a slab supported by steel stringers with 304.8 mm (12 in.) wide flanges, spaced 2.7432 m (9 ft) center to center of stringer webs, designed to be 215.9 mm (8.5 in.) thick and to support an HS-25 AASHTO wheel load follows:

$$
\begin{aligned}
f_c &= 4{,}000\,\text{psi} \\
A_s &= 0.614\ (\#5\ \text{bars} @ 6\ \text{in. center to center}) \\
d &= 8.5 - 1 - 0.3125 = 7.1875 \\
b &= 12 \\
p &= \frac{A_s}{bd} = 0.614/12 \times 7.1875 = 0.0071 \\
&\text{assume } n = 9 \\
pn &= 0.0071 \times 0.064
\end{aligned}
$$

$$
\begin{aligned}
k &= \sqrt{2pn + pn^2} - pn = 0.298 \\
j &= 1 - \frac{k}{3} = 0.90
\end{aligned}
$$

Stringer spacing is 9.0 ft, continuous over three or more supports.
Liveload is HS-25
$S = 9.0 - 1.0 + 0.5 = 8.5$ (for stringers with 12 in. wide flanges)

$$
\begin{aligned}
M_{DL} &= \frac{wL^2}{8} = 0.102 \times 81/8 = 1.03\ \text{ft-kips/ft} \\
M_{LL+I} &= 1.3\{[(8.5) + 2]/32\} \times 20 \times 0.8 = 6.825\ \text{ft-kips/ft} \\
M_{max} &= 1.03 + 6.825 = 7.855\ \text{ft-kips/ft}
\end{aligned}
$$

Maximum compression in the top of concrete

$$
\begin{aligned}
f_c &= 2M/kjbdd \\
&= 2 \times 7.855 \times 12/0.298 \times 0.90 \times 12 \times 7.1875 \times 7.1875 \\
&= 1{,}134\,\text{psi}
\end{aligned}
$$

Maximum tension in the bottom rebar

$$f_s = \frac{M}{A_s jd} = 7.855 \times 12/0.614 \times 0.90 \times 7.185 = 23{,}730\,\text{psi}$$

RECENT RESEARCH IN PRACTICE

There are other theories that can provide different solutions than that given above, developed by many researchers over the past few decades. As noted in Chapter 2, one deals with "isotropic" reinforcement, defined as "two identical layers of reinforcement, perpendicular to and in touch with each other." Also known as the "compression membrane" theory, it is based on a belief that bridge deck behavior is best described as "arching action."

Colorado researcher John Allen theorizes that since a concrete deck bears on *yielding* supports, the only negative moment reinforcement needed is for cantilever deck areas. Allen, however, has not yet had the chance to demonstrate his theory's performance, over time, on actual bridges in the field.

Other researchers have been studying bridge deck cracking, the phenomenon that has plagued transportation departments in every section of the country.

PRECAST CONCRETE

During the next 20–30 years, virtually all of the 300 million m^2 (3 billion sq ft) of bridge deck in the United States will be replaced, but not necessarily replaced in kind. The new decks will be lighter, wider, composite, stronger, or any combination of those factors. Most, by necessity, will be prefabricated rather than cast in place.

Prefabrication lends itself to solving one of the most serious questions about deck replacement: how to achieve it without serious disruption of traffic patterns. The work must be done in either short, off-peak-hour periods or in longitudinal strips that let traffic continue on one or more lanes. In some exurban and rural areas where traffic volumes are lighter, the work may be done while reversing traffic direction on a single lane as necessary.

Prefabrication also solves shrinkage and creep problems associated with casting new concrete and making it composite with a dimensionally stable existing superstructure. A cast-in-place replacement is subjected to the maximum tensile forces possible during its post-initial-set shrinkage. Prefabricated modules, on the other hand, will have been cured and shrinkage will have taken place long before they arrive at the bridge site.

Precast modules are not problem free, however. A successful deck requires making the modules composite with the superstructure and preventing joint leakage. Expansive grout materials, very low viscosity epoxy, and other types of crack sealers are now available to solve these problems.

These grout materials and advances in precasting technology have changed the way concrete decks are designed and constructed. In years past, the modules were so cumbersome and difficult to place that they were used at half-depth, with the top half cast in place. Now, full-depth precast modules are the norm, designed to specifications developed by the industry's producers.

A common design fills shear connector pockets with nonshrink grout to connect the panels and make the deck composite with the superstructure via welded studs. Bedding, whether a layer of concrete placed between the panels and girders, or a special bearing plate, such as one of the "poly" plastics, must be placed with extreme care. Leveling bolts are used for final adjustments to the deck geometry, prior to placing the bedding material, then cut below the surface of the deck and grouted.

SPECIAL DESIGNS

For many years, one of the popular deck designs combined precast and cast-in-place concrete. Prestressed concrete panels spanning between stringers to support the weight of the cast-in-place top half were—and still are—favored by contractors when they are given a choice between them and other types of forms. Such panels, they feel, offer convenient and safe working surface.

The primary consideration with these hybrid decks is to assure composite action between the two halves. Usually, only one layer of transverse and logitudinal steel is required, and the minimum cast-in-place depth should be 127 mm (5 in.). AASHTO specifications 10.38.4.3 and 8.24.3 govern these decks.

Prestressed concrete decks—sometimes unnecessarily identified as "precast prestressed"—share the same problems of adequate anchorage and bedding as precast modules. Prestressing a deck panel helps eliminate surface cracking and is often used to achieve a longer clear span between supports than would be possible otherwise. Long-range behavior of the prestressing strands is, however, somewhat problematic, as techniques for periodic nondestructive-testing inspection are only now becoming available. Some researchers expect to develop ways to embed fiber optic strands in the concrete for measurement of future stress conditions, but effectiveness of the method has yet to be demonstrated.

Post-tensioned decks are no longer considered novelties. In some decks, polymer concrete fills the joints between several panels that are then connected into a "monolithic" longitudinal section. Periodic nondestructive testing inspection of post-tensioning strands is just as important as that of prestressing wire or strand. Suitable tools for this are also in the development stage.

OVERLAYS

Overlays are used on concrete bridge decks for a variety of reasons. They are sometimes part of the original deck construction, often installed over a waterproofing membrane. The most common type is asphaltic concrete, which in many places has been installed later to provide a smooth riding surface over a seriously scaled or spalled surface. In snowbelt states, this practice is no longer justified because it accelerates destruction of the reinforced concrete deck by storing water and deicing salts. Asphaltic concrete overlays are also inappropriate where the profile grade of the deck exceeds 3% or so.

Properly applied, overlays can be used to extend the useful life of the deck (see below). Materials include latex modified concrete, high-density concrete, and thin mortars incorporating epoxies and other materials that replace portland cement. Thin overlays should contain highly abrasion resistant aggregate, which extends their own lives. Differences in the modulus of elasticity and the thermal coefficient of expansion and contraction must be considered in designing these thin overlays.

The decision whether or not to use an overlay, either in new deck construction or on a maintenance or rehabilitation project, will now be most affected by results of a life-cycle cost analysis. In the past, such a decision has been made on a first-cost basis, assuming 10–15 year service lives. Now, it is a matter of overlay cost vs. replacement over the life of the bridge.

PROTECTING THE REBAR

Although reinforced concrete is the "ideal" composite construction material, it is still not perfect. The concrete, high in compressive strength but weak in tension, is balanced by steel's high tensile strength. Steel has one drawback: it wants to return to the earth in the form of iron oxide. Encasing the steel in the highly alkali concrete generally protects it from corrosion unless—and until—moisture and oxygen reach it. Chloride ions in solution are the principal culprits. Sea water, spray, and vapor are excellent mediums, as are deicing salts (sodium or calcium chloride) used in winter climates. In reinforced concrete bridge decks, chloride ions migrate directly to the reinforcing steel through water-filled cracks. When the surface of a reinforcing bar begins to rust, its volume grows, and the physical expansion causes more cracking.

Of the several methods of preventing such chloride-steel interaction, the simplest is adequate coverage. A minimum of 50 mm (2 in.) of uncracked concrete over the steel will prevent corrosion. The key words are "*minimum*" and *"uncracked,"* although even uncracked concrete will in time become saturated with chlorides if exposed to them. Saturated concrete, by allowing the chloride ions to reach the rebar, can begin the rusting process even without cracks. And saturated concrete, with or without the presence of chlorides, is extremely susceptible to damage from freeze-thaw cycles.

Other methods of preventing corrosion of the reinforcing steel involve protecting the deck itself, either by sealers or overlays. Such additions must be durable yet breathable, allowing vapor to pass but barring water. Sealers come in both solvent- and water-based versions. According to research done for the Strategic Highway Research Program of the National Research Council, only penetrating sealers, silanes, and siloxanes (or combinations) are recommended. Other types failed to penetrate enough to resist traffic abrasion. When silanes and siloxanes penetrate the prepared concrete, they react with the pore walls to literally seal them against water entering the concrete.

Sealers should not be applied to chloride-contaminated concrete or where corrosion has begun. The critical level of contamination is when its level for 1% of the reinforcing steel is greater than 0.47 kg/m^3 (1 lb/cu yd). The sealers work best when applied to consolidated, well-cured concrete with a high water/cement ratio. They are not part of original construction, and should be applied under a maintenance contract during warm weather.

Precautions before applying a sealer include sandblasting or even shotblasting to remove oils and other surface contaminants. Any visible cracks should be filled, either with a separate epoxy application or by first sealing the cracks and then resealing the entire surface. The surface must be dry as well as clean, and protected against rain or traffic spray after application.

COATING STEEL REBAR

One of the newer ways to prevent corrosion of reinforcing steel is to coat the steel itself. For decades, the most common coating was zinc—the familiar galvanizing process. Research has shown that galvanized reinforcing steel in bridge decks can withstand exposure to chloride concentrations at least four to five times longer than black steel. Another advantage is that when the zinc does corrode, its oxides (rust) takes up far less volume than iron oxides, whose pressures are enough to cause further cracking.

Galvanized rebar should be inspected at the jobsite to see that fabrication has not damaged the coating, although small spots may be touched up with a zinc-rich paint. Precautions should be taken to keep it from direct contact with black steel reinforcing; otherwise, corrosive reactions will consume the sacrificial zinc.

Since the mid-1970s, epoxies have been widely used to coat reinforcing bars. Because it is essentially inert, epoxy will not corrode or "sacrifice" itself. While it remains intact, an epoxy coating will protect the steel indefinitely.

The only problems occur when the coating has been damaged, either in fabrication, transit, or installation. Careful handling and inspection at various stages are critical, although touch-up is possible for small damaged areas.

There have been several major research reviews of epoxy-coated steel reinforcment in the past few years, all of them emphasizing that good construction practices are necessary to the success of the product. The fact that the epoxy coating does prevent—or at least inhibit—corrosion does not excuse sloppy construction. It is necessary to use an adequate depth of concrete cover over the rebar, "adequate" being well over minimum standards, according to several researchers.

CATHODIC PROTECTION

Although cathodic protection has been installed in more than 550 bridge decks in North America since 1973, it is one of those good ideas that seem to be stuck in the experimental stage. The chief proponents of the method continue to be researchers supported in one way or another by the Federal Highway Administration, which has declared that cathodic protection is the only rehabilitation technique proven to stop corrosion in salt-contaminated bridge decks.

There are two basic types of cathodic protection: impressed current systems and sacrificial anode systems. The latter is based on the principle of electrical potential: connecting dissimilar metals in the right environment establishes a galvanic cell in which one of the metals will corrode, leaving the other intact. It's an advancement of the old galvanized iron idea, and indeed, zinc has been used as the sacrificial anode for many years. This type, however, can only be used where "the right environment" prevails—in warm and moist climates such as that of coastal Florida.

The impressed current system must be used in other areas, with low-voltage direct current supplied from a power source. Three variations of this system are (1) anodes embedded in rigid cementious overlays; (2) slotted systems; and (3) conductive coatings. The basic requirements for using one of these systems are that the reinforcing steel be continuous and that a continuous supply of direct current power be available. In remote areas, solar power has proven useful.

Other considerations for selecting cathodic protection for bridge decks are that the remaining service life of the bridge be more than 10 years, the area of delamination less than 40%, and concrete air entrainment less than 5% if there are freeze-thaw cycles.

The anode must be inert for long life, compatible with the concrete, able to withstand traffic loading, and easily installed. While zinc, copper, and even iron were used in the past, recent installations use an anode consisting of a precious-metal oxide catalyst applied to a titanium substrate. The anode may be in the form of wire-mesh panel or strips, or a flat ribbon. Because the electrochemical reaction is controlled by a catalyst, researchers expect the anode to last 50 years or more.

Other recent developments are sprayed zinc, used in both galvanic and impressed current systems; sprayed aluminum alloy wire anodes; and thermally applied titanium. The major developer is Corrpro Companies, Inc., West Chester, Pa., which often works under Federal Highway Administration research contracts. Tests of a new aluminum/zinc alloy that is thermally sprayed have been conducted by Florida DOT, but on bridge substructure components rather than decks. Results were positive, and the company is predicting that the alloy will perform better than zinc in less-than-tropical environments. This would make it suitable for protecting bridge decks in northern areas subject to attack from deicing chlorides.

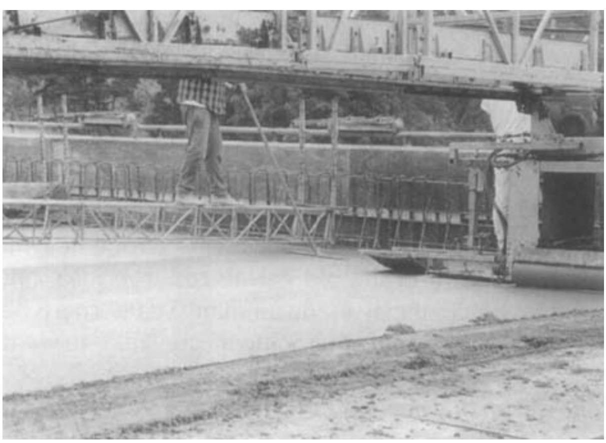

Fig. 5.1. Precast I-girders support cast-in-place bridge deck (Courtesy Portland Cement Assn.).

Fig. 5.2. Traditional reinforced concrete deck is finished by worker standing on bridge attached to rolling screed. Concrete containing calcium nitrate plasticizer is pumped onto removable plywood formwork (Photo courtesy W. R. Grace).

Fig. 5.3. Overlay of 3/8 in. polymer concrete installed in Grand Teton National Park helps protect bridge deck (Photo courtesy Transpo Industries).

Fig. 5.4. Reinforcing steel in this bridge deck is protected against corrosion by cathodic protection offered by titanium anode mesh (Courtesy Corrpro Companies Inc.).

CHAPTER 6

THE OTHER BRIDGE DECKS

STEEL GRID BRIDGE DECKS

The main reason that fewer steel grid decks have been installed over the decades than reinforced concrete is cost—first cost. The slightly lower first cost of reinforced concrete has been the determining factor in its selection, except where weight has been important in movable and long-span structures.

Weight was not only important but critical in the early part of the 20th century, when the growing use of automobiles and trucks prompted replacement of the wood plank flooring that served as decks for most bridges. The steel industry responded with open grids that were light enough to replace the planks without the need to reinforce existing superstructures.

The open grids also found widespread use in movable bridges, which are widespread wherever waterway, railway, and highway traffic systems intersect. In both bascule and vertical lift bridges, deck weight must be balanced, pound for pound, by counterweights. Reducing the weight permitted reducing the design requirements for bearings and machinery. Furthermore, the wind load on an open grid deck is less than that on a solid deck.

A variation on the wind-load problem was pioneered by D.B. Steinman in his design of the Mackinac Straits Bridge in Michigan. He used open grid in half of the roadway so that, during periods of high wind, a vertical flow of air through the grid would break up the laminar flow of air across the deck. This reduced the likelihood that the deck would behave as an airfoil.

Another advantage of steel grids was that they could be prefabricated in modules, even mass produced. Shipped from Pittsburgh and other steel industry sites, the grids proved to be very suitable, if not ideal, for bridge repair and rehabilitation.

They were not ideal on two counts. First, a queasy public disliked driving over a surface with little visual protection between car and water. Second, engineering flaws surfaced, particularly fatigue. Industry specialists have offered several explanations for fatigue problems, from fabrication to vehicle speed. Grids that were welded, weighed less than 75 kg/m^2 (15 lb/sq ft), spanned more than 1.2 m (4 ft), and/or were fabricated with the main bars transverse to the direction of traffic generally exhibited more problems than their counterparts. Speed and impact of vehicle tires at the bridge approaches were implicated, as were stresses induced by opening and closing movable structures.

As early as the 1920s, those problems were attacked by the logical procedure of filling steel grids with concrete. (Another reason for filling the grids may well have been to protect the machinery in swing and bascule bridges from water and debris that would otherwise come thru the open grid.) In the 1950s, the half-filled steel grid was introduced as a compromise between weight and durability. Changes were also made in the design of the grids themselves.

Basic Design

The basic grid consists of a primary element, either a structural tee or specially rolled "I-beam" sections, and a number of smaller interlocking members. It also includes a light-gauge steel pan that serves as a concrete form. The pan is at the bottom for a full-depth concrete fill or rests on flanges at mid-depth. A variety of grid designs is available, although most have 108 mm (4-1/4 in.) or 132 mm (5-3/16 in.) deep main bars, and one or two supplementary bars. The grid panels are always oriented so that the main bars are perpendicular to the bridge's structural members supporting the deck.

Steel grid panels are shipped to the construction site in appropriate lengths up to about 15.24 m (50 ft), measured in the direction of the primary element. The standard 2.44 m (8 ft) maximum width is convenient for handling, coating operations, and transportation. The grids are filled either at the bridge site, at a nearby casting yard, or after placement on the bridge.

Structurally, the grid acts as reinforcing for the concrete, so much so that the fabricators have named their product "grid reinforced composite bridge decks" (GRCBDs), while pointing out that the deck weighs less than conventionally reinforced concrete. They also say that durability is one of the principal qualities of this deck type, attributing it to the confinement of the structural concrete within the cells of the steel grid.

Asphalt pavement overlays have contributed a great deal to the durability of concrete-filled steel grids over the years, although some nontopped installations have performed well for 30 and 40 years.

Design Methodology

Concrete-filled grid panels are designed as one-way slabs by the transformed area method, with allowable stresses governing maximum spans. Those procedures are covered by the AASHTO Service Load Design Method (Section 8.15 in the 15th Edition) and are illustrated by the design example at the end of this chapter, which is reproduced from information published by the industry association, the Bridge Grid Flooring Manufacturer's Association (BGFMA).

The AASHTO/LRFD (load and resistance factor) Design Code, published in June 1994 as the first edition contains provisions for a new "grid reinforced composite grid deck" design model. These new formulas take advantage of the "orthotropic plate behavior of a GRCBD," which is based on static testing (at the University of Pittsburgh) by one of the grid manufacturers. A comprehensive design package, developed in compliance with all applicable articles of that new code, is planned for distribution in 1997 by BGFMA.

GRCBD panels are typically attached by embedding headed shear studs in full-depth concrete over the top of bridge framing members. However, welding, the original means of attachment, is still used by some owners and agencies. Decks are designed compositely with bridge framing members, as described in AASHTO/LRFD Article 4.6.2, Approximate Methods of Analysis, with t equal to the overall deck thickness for full depth systems. Two separate field tests are planned to determine appropriate t-values for half-depth panels. That information will also be available in 1997 from BGFMA.

Load distribution factors have been established by field testing, and the method for calculating moments in bridge framing members is no different than that for a reinforced concrete slab, using the appropriate t-value for deck thickness. Those formulas appear in the AASHTO/LRFD specification.

Construction Practices

Constructing an overlay, either by separate placement or by integrally overfilling the grid, has several advantages. First, it improves the ride quality, compared to a GRCBD filled flush to the top of the steel grid. Second, the overlay provides corrosion protection to the steel grid.

Any overlay must be at least 38 mm ($1^1/_2$ in.) thick, whether it be constructed by the overfill method or a rigid one applied separately. The latter can be latex modified, silica fume, or dense (Iowa) concrete. Care should be taken that the top of the flush-filled grid is properly cleaned to ensure a good bond of the overlay to the deck.

Bituminous concrete overlays have also been used successfully, either with or without a membrane. Where dead-load requirements prohibit the use of one of the above overlays, a thin [6.35–12.7 mm (1/4–1/2 in.)] flexible epoxy or copolymer overlay provides both an improved ride quality and protection to the steel grid.

Early steel grid panels were installed with the bottom, exposed surface of the grid receiving the same coating system as the bridge structural steel. The portion of the steel grid in contact with the concrete was left uncoated. This practice continues, although a number of agencies now specify that some corrosion protection be applied to those areas of the grid. These include high-performance zinc paints, hot-dip galvanized coatings, and fusion-bonded epoxy coatings, and the process affords the steel grid two measures of corrosion protection.

EXODERMIC DECKS

An exodermic deck is a thin reinforced concrete slab made composite with an unfilled steel grid in a way that maximizes the use of compressive strength of concrete and the tensile strength of steel. Developed in the early 1980s, the design builds on the durability record of steel grids while using them in a new way to make more efficient use of the grid and concrete, and to deliver a standard reinforced concrete riding surface.

The units, fabricated by the same companies that manufacture other steel grid products, are supplied either completely prefabricated or ready for concrete placement on site. The reinforced concrete slab is cast on top of a conventional unfilled steel grid and made composite with it via additional grid bars called tertiary bars, which are attached to the grid, and extend up 25 mm (1 in.) into the concrete. This transfers horizontal shear between the concrete and the steel grid.

In turn, the exodermic deck is made composite with the bridge superstructure by headed studs welded to stringers, floor beams, and main girders. These studs are embedded in full-depth concrete haunches that are either poured at the same time as a cast-in-place exodermic deck or poured separately for a precast exodermic deck.

A cast-in-place exodermic deck provides a continuous concrete surface that can be maintained as any other reinforced concrete deck, with or without an overlay.

A precast exodermic deck permits deck replacement with minimal interruption of traffic. By removing several sections of an existing deck and replacing them immediately with exodermic units, the work can be completed while keeping the structure fully open during peak traffic hours.

A typical recent installation redecked a 326 m (1,070 ft) bridge over the Hudson River in Albany, N.Y. Each night, between 8 p.m. and 6 a.m., the contractor directed traffic to one lane of the four-lane bridge, removed a 12.8 by 6.7 m (42 by 22 ft) section of old reinforced concrete deck, welded shear studs to stringers and floor beams, dropped precast exodermic panels in place, and secured them, using a rapid-setting concrete. While this deck was not overlayed, other projects have used overlays of latex-modified concrete, or asphaltic concrete placed over a membrane.

Whether cast in place or precast, an exodermic deck will be substantially stiffer and stronger than comparable decks, while weighing less per square meter. Typical weights range from 215.6–411.6 kg/m^2 (44–84 lb/sq ft), depending on the grid components and spacing, and concrete type and thick-

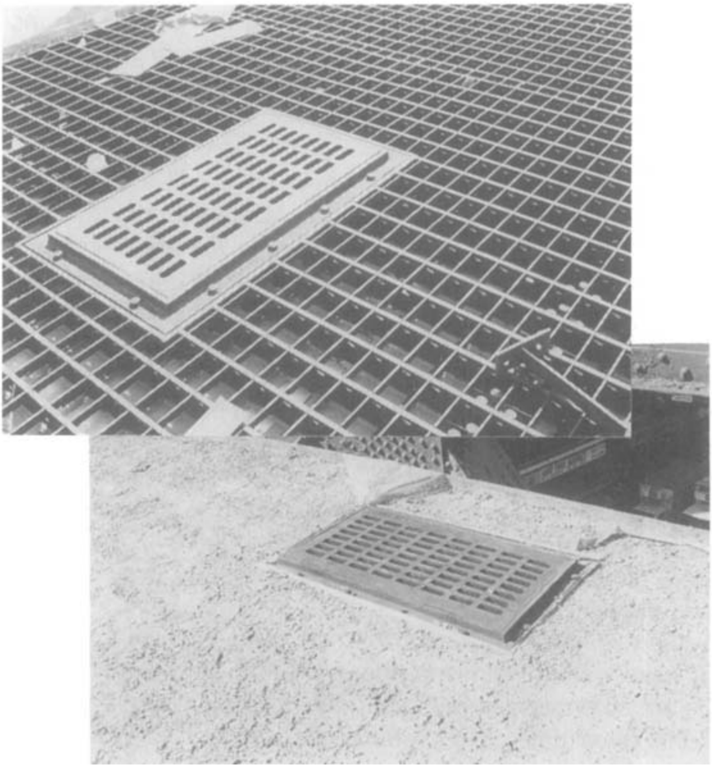

Fig. 6.1, a & b Concrete filled the steel grid deck on New York City's Manhattan Bridge and was later topped with an overlay.

Fig. 6.2. Precast exodermic modules form the deck of bridge over New York State Thruway near Albany.

ness. This is about 50–60% of the weight of a typical reinforced concrete slab. Overall thickness varies from 165–254 mm (6.5–10 in.) or more, depending on the grid main bearing bar selected and depth of concrete. Units have been designed to span up to 5.7 m (18 ft 8 in.), using standard components, although longer spans may be obtained using deeper main grid bars.

The typical steel grid is hot-dip galvanized after fabrication, and is composed of a two-way web of main bearing bars, I- or T-shaped, flat distribution bars at right angles, plus tertiary bars, parallel to the main bars, which project 25 mm (1 in.) above a galvanized pan that serves as the bottom form for the concrete. Vertical studs are welded to the tertiary bars and, together with the partial embedment of the tertiary bars, develop the horizontal shear transfer required to obtain composite behavior of grid and concrete.

Based on service-load (working stress) design in accordance with the AASHTO rules for filled grid decks and reinforced concrete slabs, design computation is straightforward and maximum stresses are conservative. The composite deck has an effective thickness at least equal to its overall depth, and the section modulus per unit of width is approximately 250% that of a grid filled with concrete of the same total weight. The deck provides extended fatigue life because the neutral axis is in the vicinity of the welds and stress-raisers in the steel grid.

Exodermic bridge deck is covered by name in ASTM specification D5484-94, Specification for Steel Grid Bridge Flooring. The 1994 AASHTO/LRFD specification includes exodermic decks as Unfilled, Grid Decks Composite with Reinforced Concrete Slabs in Section 9.8.2.4, combining the "advantages of a concrete deck and a steel grid deck."

History

Development of exodermic decks began in response to problems with concrete-filled steel grids that surfaced during the 1970s. These decks exhibited a "growth" phenomenon attributed to corrosion of the grid bars. At the same time, open grid decks were exhibiting poor skid resistance and early fatigue cracking. (Performance of both grid types has since been improved through research at West Virginia University and by the BGFMA.)

The first exodermic project was the 1984 roadway widening of the Driscoll Bridge on the Garden State Parkway in New Jersey. The exodermic lane filled the space between the separate northbound and southbound structures of the 4,400 ft long bridge. The new lane was cantilevered from the southbound side, and the work was accomplished by installing 500 precast deck modules in only six days.

The initial installation was preceded by a static and fatigue testing program carried out at Lehigh University. Later, exodermic modules were included in an extensive grid deck testing program at West Virginia University. Additional field tests, primarily on replacement projects, followed during the 1980s and 1990s.

Such tests showed that the exodermic design with tertiary bars develops full composite behavior, whereas other experimental designs that incorporated only headed studs welded to the relatively light grid members did not. Those studs, as well as those in other proposed deck types in which headed studs are welded to the top surface of thin flat steel plates, fail to develop the tensile and bending stresses at their bases that are required to produce the fixed-end condition of the studs, which is the basis for the original design and development of welded headed studs.

Two types of studs are used in the exodermic design. Short unheaded studs are welded to the tertiary bars at about 305 mm (12 in.) center to center. These studs prevent vertical separation of the two deck components. The second type of stud, with standard head, which creates the composite action between deck and superstructure, is installed in the field after the exodermic panels have been positioned. These headed studs are welded to stringers, floor beams, and main girders as appropriate. Their heads are embedded in the concrete haunch area, which is poured at the same time as the cast-in-place reinforced concrete deck or poured separately where precast panels are used.

The exodermic design might very well have been named "concrete orthotropic," as the "plate" is a thin element of reinforced concrete. The grid does not "support the plate" but instead, as in a steel orthotropic deck, the grid and the concrete are "welded" together into a composite unity.

The Exodermic Bridge Deck Institute (EBDI) (tel: 888/EXODERMIC) licenses steel grid fabricators to produce the deck panels that are considered generic in most jurisdictions because of availability to contractors from multiple, independent manufacturers. EBDI makes information about design and construction, including computer-aided-design files and analysis software, available to engineers at no charge.

Recent projects have included new decks on existing bridges in New York City and in Rockland County, N.Y., and emergency repairs to the New York Thruway's Tappan Zee Bridge. New construction includes a 520 m (1,700 ft) interim viaduct that will carry Interstate 93 southbound in Boston during construction of the new Central Artery in 1996.

STEEL ORTHOTROPIC DECKS

A singular type of deck made up of steel plates, the orthotropic deck, has been used on many long-span bridges, both for new construction and for redecking during renovations.

Roman Wolchuk, a Jersey City, N.J., consulting engineer, is the foremost proponent of orthotropic steel decks in the United States. He has contributed to the 1994 AASHTO/LRFD specification section that has been adopted for

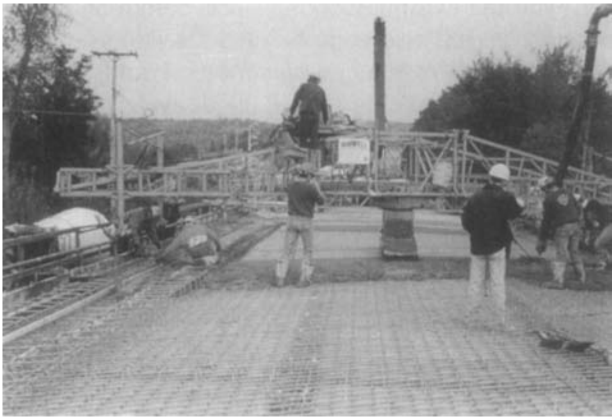

Fig. 6.3. Cast-in-place exodermic modules replaced decks of twin 600 ft bridges over Rt. 9W at Bear Mountain, NY.

Fig. 6.4. Steel othotropic deck was chosen for replacement of second level deck on the George Washington Bridge.

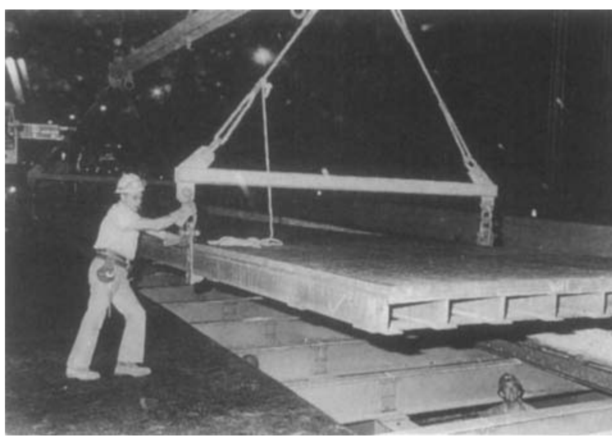

Fig. 6.5. Doweled glulam panels were field assembled to construct deck of all-timber bridge.

use as a parallel standard with the existing working stress design and load factor design AASHTO codes. He has also contribututed a section on steel plate deck bridges to the *Structural Engineering Handbook,* Fourth Edition (1995) published by McGraw-Hill. What follows is derived from the AASHTO/LRFD specification and Wolchuk's recent papers.

In its basic form, the orthotropic deck is an integral structure made up of stiffening ribs and transverse floor beams, with the deck plate serving as the structure's top flange. This deck structure acts as part of the main bridge structure, most often as the top flange of the main girders or trusses in box girder bridges.

For stiffening a suspension bridge, the steel deck may act as the upper flange of the stiffening girders, or may be used as the top part of a box section. In any bridge type, the steel orthotropic deck can add flexural and torsional rigidity through its closely spaced parallel ribs.

Orthotropic decks have also been used in rehabilitation of existing bridges. In these cases, the deck can either remain an independent element of the bridge or be made integral with the structure via shear connectors. Independent decks were used in redecking both the George Washington Bridge and the Golden Gate Bridge, and fully integrated orthotropic decks replaced the original on the Benjamin Franklin Bridge (Philadelphia to Camden).

There are two types of steel orthotropic decks: those with open ribs and those with closed ribs. They differ in lateral distribution of wheel loads, and the choice depends on their characteristic advantages and disadvantages.

The longitudinal ribs in open-rib decks are flat bars or bulb sections, or inverted tees and angles. Spans are 3 m (10 ft) or less. Although the last are strongest and most rigid, they accumulate dirt and offer perches for birds, causing serious maintenance problems. Other disadvantages include a large number of welds and large surface areas that are difficult to paint. Because open ribs are installed with simple fillet welds, the deck underside is completely accessible and there are no secondary local flexural stresses that sometimes cause fatigue problems in closed-rib decks.

To fabricate the ribs for closed-rib decks, trapezoids are bent from thin plates. Their rigidity increases the lateral load distribution capacity for the same amount of material, so the deck may be lighter than an open-rib deck. Spans can range up to 9.8 m (32 ft). The bevel groove welds are only half as long as required for open-rib construction, but are difficult to produce. Field splices are also difficult, and care must be taken to avoid secondary flexural stresses in the rib-deck junctions.

A prime reason for redecking an older bridge with a steel orthotropic structure is reduced dead weight. For a medium span of 90–180 m (300–600 ft), the reduction may be 15–30% of the original concrete deck. In new construction, such weight savings lead to design of thinner structural depths.

Construction time is also shortened because the steel deck can be prefabricated in large units. In turn, prefabrication makes a steel orthotropic deck suitable for redecking, as it may be installed in one lane while allowing traffic to continue on others.

Uniform reliability and safety of all bridge components is the guiding principle of the new specifications. In addition to the chosen safety index of 3.5, other important aims are redundancy, ductility, constructibility, maintainability, and long-term economy. The design is based on four limit states: strength, serviceability, fatigue, and extreme events. Each state is assigned appropriate load factors.

Sections of the 1994 AASHTO/LRFD specifications discuss general design principles, structural analysis, and specific design provisions. Section 9 includes provisions for orthotropic decks, although several other sections contain relevant provisions. Stipulated design details are based on studies of performance records and failure reports of orthotropic decks in the United States and other countries, including recent research on their fatigue strength.

The design provisions emphasize the importance of preventing fatigue cracks in the deck structure and surfacing failures on the decks. Specific provisions include the following:

1. The design load is a 325 kN truck with single axles of 35-145-145 kN superimposed on a uniform lane load of 9.3kN/m applied simultaneously to all traffic lanes on the bridge. This is heavier than the HS20-44 truck loading that governs design under current AASHTO specifications because it is a realistic view of actual current loads on U.S. highways and also takes future load increases into account.
2. Any suitable elastic analysis method—finite element, finite strip, or equivalent grillage—may be used for refined analysis. For approximate analysis, the Pelikan-Esslinger method as adapted in the American Institute of Steel Construction *Design Manual for Orthotropic Steel Plate Deck Bridges (1963)* may be used.
3. Detailing requirements include a minimum deck plate thickness 0.04 times the spacing of the rib webs, or at least 14 mm. This makes for a relatively stiff deck plate that helps reduce secondary stresses at the rib welds and improves performance of the wearing surface—an integral part of the deck. This requires determining the temperature-dependent mechanical properties of the surface—modulus of elasticity, tensile, shear, and bonding strength—over the service temperature range.
4. Cracking and bond failure can occur when tensile flexural stress exceeds the tensile strength of the surfacing, which de-

pends on the local curvature of the deck plate, material properties, temperature, and surfacing thickness. The AASHTO specification stipulates that the wearing surface be selected on the basis of appropriate mechanical properties, fatigue strength, and resistance to rutting, wearing, solar radiation, water penetration, and deicing salts.

TIMBER BRIDGE DECKS

For hundreds of years, sawn lumber plank decks were laid across all types of bridge structures to serve as the bearing surface for foot, hoofed, and wheeled traffic. These decks varied only in size, strength, and resistance to splintering and decay.

At present, however, timber bridge decks are generally used only where traffic volumes are extremely low, or where all-timber bridges are constructed to meet requirements of aesthetics and economy. Timber decks are not restricted to bridges with timber superstructures. In rural areas it is quite common to see bridges composed of two rolled steel beams topped by a one-lane timber deck.

The use of high-tech timber decks; glued-laminated panels and/or framing members, and prestressed/posttensioned assemblies of lumber, have been developed for the benefit of the timber industry and its customers. These decks of preservative-treated timber offer an option that should not be overlooked for construction of extremely low volume bridge decks.

Durable overlays might make timber decks suitable for other than extremely low volume service, but none have been developed to date. Asphaltic concrete overlays are frequently used on timber decks to improve skid resistance, particularly in wet pavement conditions, but they do not stand up to any substantial volume of traffic.

Two types of timber decks are included in this book: glulam decks and stress-laminated decks. Information prepared by the engineering staff of the United States Department of Agriculture (USDA) Forest Service, excerpts of which follow, is the most complete material available on timber deck design.

Glulam Decks

Nail lamination was the first main improvement over sawn plank decks. These were generally 50 mm (2 in.) thick and from 100–300 mm (4–12 in.) deep, placed side by side and nailed or spiked together to form a continuous surface. They performed well over closely spaced supporting beams and were popular from the 1920s to the 1960s. Development of glulam decks, however, have made them all but obsolete.

Glulam deck panels are normally 130–220 mm ($5^1/_8$–$8^3/_4$ in.) thick and 900–1,500 mm (3–5 ft) wide. They may be placed edge to edge without interconnections to form the deck or interconnected with steel dowels that improve load distribution and reduce differential displacements at the panel joints. The dowels also permit design of thinner decks and improve the performance of asphalt wearing surfaces.

Noninterconnected glulam decks are more commonly used because they are easy to install with unskilled labor and without special equipment. Each panel acts individually to resist the stresses and deflection from applied loads. The deck is assumed to act as a simple span between beams and is designed for the stresses acting in the direction of the deck span as well as for deflection. Although deflection, rather than bending stress, controls most applications, the designer may establish different levels of acceptable deflection for different applications.

Design procedures are similar for noninterconnected and doweled deck panels, although the design loads differ. For noninterconnected panels, under special AASHTO provisions for timber decks, the HS 20-44 and H 20-44 design load is a maximum 53.4 kN (12,000 lb) wheel load. These provisions do not apply for doweled panels, which are designed for HS 20-44 and H 20-44 with 71 kN (16,000 lb) and for HS 15-44 and H 15-44 with 53.4 kN (12,000 lb) wheel loads. Specific design procedures and sample calculations are given in *Timber Bridges: Design, Construction, Inspection and Maintenance* (1992), published by the USDA's Forest Service.

Procedures and calculations are given separately for noninterconnected and doweled panels. The latter are more expensive because they require precise fabrication for proper installation and performance. The panels are designed for the primary moment, shear, and deflection requirements, with appropriate dowel size and placement preventing differential panel deflection under wheel loads.

The design procedures for doweled panels were developed by the USDA Forest Products Laboratory and adopted by AASHTO in 1975. They are based on analyses of the deck as an orthotropic plate acting as a simple span between two supports.

Construction

Glulam decks are attached to supporting beams with bolts, screws, and other mechanical fasteners. The attachments must hold the panels securely and transmit longitudinal and transverse forces from the deck to the beams. Because preservative treatment is best done after the modules are fully fabricated, the connections should not require holes or cuts to be made in the field.

Glulam decks are placed directly on glulam beams and attached with bolted brackets that connect to the beam sides or with lag screws placed through the deck into the beam tops. Lag screw attachments are not recommended because they require field boring and they are not accessible for future tightening if the deck is paved.

Panels may be placed directly on steel beams and secured with a bracket that bolts through the panel and under the top beam flange. Bolting through the flange is not recommended because it allows little or no tolerance for minor variations in panel moisture content or steel thermal expansion.

Glulam decks can be made watertight by sealing the joints with roofing cement or other sealer. Long bridges and those in warm, humid climates may require a 13 mm (1/2 in) transverse joint between every third or fourth panel. Galvanized steel nosing angles are placed on the edge of end panels to minimize damage from vehicle impact and abrasion.

Stress-Laminated Decks

In longitudinal stress-laminated deck superstructures, lumber is placed edgewise between supports and compressed transversely so that the deck acts as a continuous slab without transverse or longitudinal joints. Load transfer between laminations is developed by friction due to initial compression, without glue or nails. Compression is achieved by the same type of high-strength steel rods used for prestressing concrete, placed at regular intervals through prebored holes and stressed in tension by a hydraulic jack. Investigation of the possible use of composite rods instead of steel is underway.

For deck rehabilitation, the rods may be placed externally, over and under the lumber rather than through the laminations. In both methods, the rods are held in place by anchorages that distribute the tension force along the edge of the bridge deck. The steel rods and anchorage devices must be protected against corrosion.

Stress lamination was developed by the Ontario Ministry of Transport more than a decade ago, and by the USDA Forest Products Laboratory during the mid-1980s. FPL researchers worked with the University of Wisconsin, the University of West Virginia, and other state universities. The decks have proved successful in short-span bridges, but the need for longer spans has spurred research into using parallel-chord trusses in place of sawn lumber or glulam deck girders. Trusses will provide a stiffer system, using the same (or smaller) volume of lumber as would be required in a solid girder of similar carrying capacity.

Research into use of wood for bridges and other structures is continuous at the Forest Products Laboratory and at several universities. American Laminators, in Drain, Ore., owns the rights to a fairly recent development researched at Oregon State University. High-strength fibers such as aramids and carbon are extruded into a plastic matrix to form fiber-reinforced plastic panels, trademarked as *FiRP.*

Positioning the product in the high-stress portion of laminated timber beams increases bending strength and stiffness. Such beams have been widely tested in new timber bridges, and testing is underway to develop FiRP glulam bridge decks. Further information is available from Wood Science & Technology Institute, Inc., 2031 NW Monroe St., Corvallis, OR 97330.

CHAPTER 7

A GUIDE TO THE GUIDE SPECIFICATIONS

All persons involved with the selection, design, construction, and rehabilitation of bridge decks must be fully conversant with the specifications that apply in their jurisdiction. In addition, they may profit from the knowledge contained in other guides.

The American Association of State Highway and Transportation Officials AASHTO has promulgated design specifications for many decades. These specifications, adopted throughout the United States, have been updated periodically.

AASHTO's 15th Edition, *Standard Specifications for Highway Bridges,* published in 1992, is the current specification. Because of growing use of LRFD specifications (see below), the 15th Edition will probably not be revised in the forseeable future. It does, however, contain valuable information for those who deal with bridge decks. Some of the more pertinent sections are

- Section 3.24, Distribution of Loads and Design of Concrete Slabs, which carries a footnote stating, "The slab distribution set forth herein is based substantialy on the 'Westergaard' theory," citing publications in *Public Roads* and several University of Illinois bulletins (See chapter 2 of this book).
- Section 8.6, All About Concrete, which discusses protection of concrete from environmental conditions.
- Sections 8.17–8.32: Concrete reinforcement.
- Section 9: Prestressed concrete.
- Section 10.38: Composite girders, structures composed of steel girders with concrete slabs connected by shear connectors.
- Section 10.39: Composite box girders.
- Section 10.40: Hybrid girders, where lower strength steel is used in the web rather than in one or both flanges—composite and noncomposite plate girders, and composite box girders.
- Section 10.41: Orthotropic decks, steel.

- Section 10.42, which begins Part D, Strength Design Method (load factor design), described as "an alternate method for design of simple and continuous beam and girder structures of moderate length . . . a method of proportioning structural members for multiples of the design loads."
- Section 12: Steel grid flooring.
- Section 28: Wearing surfaces.

GUIDE TO LRFD

AASHTO'S *LRFD Bridge Design Specifications,* first published in 1994, is based on load resistance factors and employs the load and resistance factor design (LRFD) methodology. The factors have been developed from the theory of reliability based on current statistical knowledge of loads and structural performance. This is one step beyond the idea, central to load factor design (LFD), that the same factor of safety should not, logically, be applied to dead-load as well as live-load stresses.

In 1996, the concensus among bridge engineers appears to be that this is the specification that will carry bridge design into the 21st century. The current LRFD specification will probably be replaced by a second edition within the next five years. The format is particularly attractive: on each page, opposite each specification section, is a commentary on that section. There are also several internal tables of contents (one of which is reprinted below), apppendices that outline the specs step by design step, and well-organized tables. It is important for all who deal with bridge decks to read the introduction in full. Some pertinent sections include the following:

- Section 2.6.6: Roadway drainage.
- Section 3: Loads and load factors.
- Section 4: Structural analysis and evaluation.
- Section 5: Concrete structures, which includes an appendix that gives the "Basic Steps for Concrete Bridges," noting the section numbers pertintent to each step. Some individual items in Section 5 are 5.12.2, alkali-silica reactive aggregates; a table in 5.12.3 giving covers for unprotected main reinforcing steel; 5.13.1, deck slabs; 5.13.2, diaphragms and deep beams, brackets, corbels, beam ledges; 5.14.4, cast-in-place voided slab superstructures; and 5.14.4.3, precast deck bridges.

The table of contents to LRFD Section 9 defines its scope: as "An analysis of design of bridge decks of concrete, metal and wood, or combinations." The table of contents is as follows

9.8.5.2 Distribution of wheel loads
9.8.5.3 Composite action
9.9 WOOD DECKS AND DECK SYSTEMS
9.9.1 Scope
9.9.2 General
9.9.3 Design requirements
9.9.3.1 Load distribution
9.9.3.2 Shear design
9.9.3.3 Deformation
9.9.3.4 Thermal expansion
9.9.3.5 Wearing surfaces
9.9.3.6 Skewed decks
9.9.4 Glued laminated decks
9.9.4.1 General
9.9.4.2 Deck tie-downs
9.9.4.3 Interconnected decks
9.9.4.3.1 Panels parallel to traffic
9.9.4.3.2 Panels perpendicular to traffic
9.9.5 Stress laminated decks
9.9.5.1 General
9.9.5.2 Nailing
9.9.5.3 Staggered butt joints
9.9.5.4 Holes in laminations
9.9.5.5 Deck tie-downs
9.9.5.6 Stressing
9.9.5.6.1 Prestressing system
9.9.5.6.2 Prestressing materials
9.9.5.6.3 Design requirements
9.9.5.6.4 Corrosion protection
9.9.5.6.5 Railings
9.9.6 Spike-laminated decks
9.9.6.1 General
9.9.6.2 Deck tie-downs
9.9.6.3 Panel decks
9.9.7 Plank decks
9.9.7.1 General
9.9.7.2 Deck tie-downs
9.9.8 Wearing surfaces for wood decks
9.9.8.1 General
9.9.8.2 Plant mix asphalt
9.9.8.3 Chip seal

THE CANADIAN SPECS

The *Ontario Highway Bridge Design Code, 3rd Edition,* published in 1991 by the Ontario Ministry of Transportation, has greatly influenced bridge engineers in many parts of the world. This edition represents the culmination of more than a decade of experience in applying provisions of the previous two editions, published in 1979 and 1983. Length of bridges affected by the code has been increased to 150 m. Not included in its provisions are cable-supported bridges and some aspects of movable bridges. The section on rehabilitation of bridges and their decks is new to this edition.

The publication comprises two volumes, one for the Bridge Code and the other for Commentary. Both volumes are well illustrated. Of pertinent interest to those dealing with bridge decks are the following sections: parts of Section 1 dealing with drainage systems (1.8.4) and maintenance requirements (1.8.5), and in Section 3 on Analysis, 3.4.7, dealing with deck slabs. Also of interest are

- Section 4, Deck joints and bearings
- Section 8, Concrete structures
- Section 9, Wood structures
- Section 10, Steel structures
- Section 11, Evaluation
- Section 12, Rehabilitation

OTHER PUBLICATIONS

Two other excellent guides—and there are many that have been published just in the United States—are the New Jersey DOT standards for bridge projects and the New York State DOT *Bridge Deck Evaluation Manual.*

In the New Jersey DOT publication, Section 9, 'Reconstruction and Rehabilitation Projects,' covers concrete decks; field conditions and surveys; deck slab replacement; and special conditions that affect construction and traffic management. Section 9B covers bridge deck rehabilitation from plan submission requirements to classification of repair types. Deck evaluation surveys are covered in Section 9C, and Section 20 gives design guidelines for deck slabs.

New York's *Bridge Deck Evaluation Manual,* dated May 1992, states, "Bridge deck rehabilitation is a major component of the Department's bridge preservation program. An array of deck treatments are available that, when matched to deck condition and age, can provide cost-effective strategies for their preservation."

The text provides the deck designer with information on selecting evaluation methods, interpreting their findings, and specifying treatment. It includes a discussion of reporting requirements and an appendix of case studies.

Ordering addresses for the above publications are:

AASHTO
444 North Capitol St. N.W.
Suite 249
Washington, DC 20001

Ontario Government Bookstore
880 Bay Street
Toronto, Ontario M5S 1Z8
Canada
Toll-free phone: 800/668-9938

The New Jersey Department of Transportation
1035 Parkway Ave. CN-600
Trenton, NJ 08625

The New York State Department of Transportation
Structures Design and Construction Division
1220 Washington Ave.
State Campus, Bldg. 5
Albany, NY 12232

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Appendix A

BRIDGE DECK EVALUATION MANUAL

NEW YORK STATE

DEPARTMENT OF TRANSPORTATION

STRUCTURES DESIGN AND CONSTRUCTION DIVISION

MAY 1992

BRIDGE DECK EVALUATION MANUAL
CONTENTS

These Appendices have not been included as part of this publication but may be obtained from the New York State Department of Transportation

I. INTRODUCTION

Bridge deck rehabilitation is a major component of the Department's bridge preservation program. An array of deck treatments are available which, when matched to deck condition and age, can provide cost-effective strategies for their preservation. These include asphalt and concrete overlays, as well as complete deck replacement. Selection of appropriate treatments requires that the Design Engineer know the process of evaluating and interpreting deck condition, be familiar with the treatments available, and be able to integrate condition data, treatment type, and cost into selection of a cost-effective rehabilitation strategy.

The purpose of this Manual is to provide a single document that the Designer can use to select deck evaluation methods, interpret the findings, and select treatment strategies. Requirements for reporting deck evaluation results are included. It does not provide full details on how the various evaluations are performed. This information is contained in the Materials Bureau publication titled Field Survey Manual for Bridge Deck Overlay Projects (March 1989).

The contents of this Manual are applicable to monolithic decks, two-course decks, and asphalt overlaid decks. For the last two deck types, the primary difference in applying the Manual's techniques is in methods used to evaluate condition. An asphalt overlay, in particular, masks condition of the underlying concrete deck and prevents direct observation of spalls, as well as soundings to detect delaminations. Half-cell potential measurements to detect corrosion activities cannot be performed through the asphalt.

Chapter II describes currently available deck evaluation methods. Each method's purpose is described, along with details of the information obtained. Criteria are established for selecting evaluation methods. Finally, techniques for interpreting the data in describing deck condition are reviewed.

Chapter III lists various deck treatments in use in New York. Each method is explained and criteria for its use are established. Current information on service life and statewide average cost are included. A procedure for calculating present worth of each treatment is also described.

Chapter IV outlines criteria for treatment selection. Age and current condition of the deck, together with estimates of treatment service lives, are used to select treatments providing cost-effective rehabilitation. Total cost of each treatment, including cost of construction, maintenance and protection of traffic (M&PT), protection of workers from falls and other hazards, and environmental protection is used to estimate its present worth. The additional M&PT cost and worker protection makes the treatment selection process highly dependent on site conditions and traffic volume. For very high traffic sites, user costs resulting from construction delays may also be considered. These additional costs tend to shift treatment selection toward more-complete rehabilitations at such sites.

Chapter V discusses reporting requirements. Report format has been standardized to permit easier preparation and review.

An Appendix of case studies is also included. These case studies will provide the reader with specific applications of the procedures described.

II. EVALUATION METHODS

Thorough bridge deck evaluation is required to select the best method of rehabilitation. This Chapter describes evaluation techniques for both monolithic and two-course decks. The latter with either a concrete or asphalt wearing course are evaluated using essentially the same techniques. Evaluation methods currently available, along with a brief description of their purpose, limitations, and the information obtained, are describes here. The methods and their purposes are summarized in Table 1.

Each method's applicability is discussed as it relates to both monolithic and two-course bridge decks. For the latter, potential and sounding evaluation techniques are either restricted or limited in their use at the design stage. The wearing course must be removed if this technique is to provide meaningful and complete information, making it difficult for the Designer to estimate removal quantities accurately on two-course decks. The Designer should use visual, coring, chloride, and possibly limited sounding data, and past experience to estimate repair quantities and locations. A potential and sounding survey should be incorporated into the bridge deck rehabilitation contract, after wearing course removal, so that areas requiring reinforcing bar exposure can be accurately identified. Two-course decks programmed for total deck replacement or 100% reinforcement bar exposure do not require potential or sounding evaluation. Similarly, these evaluations are not needed if an asphalt overlay is chosen as a short-term repairs

Specific instructions for performing and interpreting a variety of monolithic deck survey procedures are given in the <u>Field Survey Manual For Bridge Deck Overlay Projects</u> prepared by the Materials Bureau. These details are not repeated here. For brevity, this document will be referred to as the <u>Field Survey Manual</u>.

A. <u>Visual Deck Examination</u>

The work should begin with a visual examination of the top and bottom deck surfaces. Safe access to the underside of the deck must be arranged for this examination. This examination identifies such forms of surface distress as cracks, spalling, scaling, efflorescence, rust on stay-in-place forms, and concrete discoloration. Each type of distress should be documented on a scaled map of the deck, as described in Chapter V ("Reporting Requirements"). Visual examination will identify the need for other evaluation methods. For example, if it reveals extremely severe deck distress or concrete deterioration, there is little need to continue with other evaluation procedures because the deck must be replaced. By contrast, if visual examination indicates relatively sound concrete with only isolated distress, such as spalling due to reinforcement corrosion, then additional evaluation procedures must be used to determine locations for reinforcing bar exposure. If visual examination reveals questionable deck bottom areas, then a more extensive evaluation will be needed to determine the extent of full-depth deck repairs.

Visual evaluation is an essential task that must be completed for both monolithic and two-course decks, using the same evaluation methods.

B. Sounding

This technique is described in the Field Survey Manual. It is used to locate areas of delaminated concrete by dragging a chain across the concrete surface or hitting it with a hammer and listening to the sound. If possible, both the top and bottom side of the deck should be sounded to identify delaminated concrete. Sounds from delaminated or hollow areas will be obvious. Sounding usually identifies delaminated concrete that results from expansive forces caused by reinforcing steel corrosion. These hollow or delaminated areas should be studied in conjunction with potential survey results. Both identify distress related to reinforcing steel corrosion. (See Chapter IV, Section A, "Deck Rehabilitation", for further information on identifying areas for concrete removal.

Sounding is used primarily on monolithic decks. On two-course decks (with both concrete and asphalt concrete wearing courses) it is difficult but possible to detect delaminations in the structural slab by pounding a hammer on the wearing course if background noise is low, but the chain drag is not sensitive enough for this application. Delaminations detected by a hammer in two-course decks should be confirmed by coring before performing an extensive survey. Experimental methods (radar and impact echo) for locating delaminations in two-course decks are described at the end of this Chapter. For two-course decks, sounding should be repeated with the wearing course removed. The contract documents should provide for this secondary sounding.

C. Potential Survey

This method locates areas of active reinforcing steel corrosion. Its use is limited to monolithic bridge decks and is detailed in the Field Survey Manual. Potential surveys cannot be used to evaluate two-course decks until the wearing course, including any protective membrane, is removed. By plotting electrical potential measurements on a grid map, areas of high and low potential can be located. As just stated regarding sounding, high potentials together with areas of spalls and delaminations are used to determine the extent of reinforcing bar exposure. Because concrete delamination is a progressive form of failure, there will generally be substantial increases in removal quantities from those identified in design. To account for this, the Designer may want to increase removal quantities by about 20 percent per year for each year lapsed prior to planed construction (i.e., 40% would become 48% for a one-year lapse between deck evaluation and construction).

On two-course decks, the contract documents should provide for completion of a potential survey after the Contractor removes the wearing course. This enables the Engineer to identify areas for reinforcing bar exposure more accurately.

D. Coring

1. Procedure

Coring is an important tool in determining structural condition of concrete and reinforcing steel, and the extent of repair. It is used in evaluating both monolithic and two-course decks. Its importance is greater in evaluating two-course decks because it uncovers distress that may otherwise go undetected.

The primary function of cores is to verify findings of other evaluation methods, determine extent of the distress, and determine its limits and depth. They are also used to evaluate concrete condition through laboratory testing. Available tests include compression, air content, freeze-thaw, and chloride determination. Each of these laboratory tests is described later.

Visual analysis is sufficient for most cores. Only a few representative ones should be selected for laboratory testing. Concrete that looks good generally is good. Cores exhibiting distress throughout their full depth (rubble) should not be selected for laboratory testing. One of the best tests available is in-service performance of the concrete deck. Cores having no visible signs of distress have met the test of time.

Before establishing a coring pattern, the deck should be closely inspected, on both its top and bottom sides, as described in the "Visual Deck Examination" section of this Chapter. Deck condition should be documented on scaled drawings and representative photographs taken, as described in Chapter V ("Reporting Requirements for a Bridge Deck Condition Report"). Deck condition is often repetitive from span-to-span and in specific locations, such as curb lines, transverse joints, etc.. Various typical conditions should be identified. Deck locations exhibiting each type and extent of distress should be selected for core analysis. It may not be necessary to core each span of a multiple-span structure unless differing conditions exist, but the Designer must be satisfied that all these conditions are evaluated. The coring pattern should be selected so that an estimate of repair can be made. If after the initial inspection it is obvious that complete deck replacement is warranted, then no cores are necessary.

When coring, a **qualified Engineer**, preferably the Designer responsible for the bridge rehabilitation, must be present. Modification of the original coring pattern by a qualified Engineer, based on results for cores as they are extracted, will result in the most efficient use of coring and eliminate the possibility of misinterpreting distress observed in the core. Cores can become damaged due to improper coring (excessive down pressure, worn bit, reinforcement wedging in the core bit, etc.). The interior of the core hole should always be inspected to confirm the condition of a broken core. By inspecting the interior of the core, the orientation of the reinforcing bars, cover on the transverse and longitudinal reinforcement, and the depth of the core and of delaminations can be determined. Comparison of the core to the core hole lining can determine if cracks in the core represent the deck condition or damage caused by the drilling operation.

Cores should be taken with a maneuverable pavement core drill for access to curb line or other restricted areas, using 4-inch diameter, thin-wall, diamond-bit core barrels. They should ideally be taken completely through the deck to permit full-depth concrete and lower-mat reinforcing steel evaluation. However, when core retrieval is not possible from the deck underside, the core may be broken off just below the level of the bottom steel mat. In any case, the core bit should progress well into the structural slab. When taking them on spans having corrugated-steel stay-in-place forms, coring should be discontinued when water is lost through the perforation made in the corrugations by the drill. Coring completely through a rubble structural deck can cause surface cave-in, which may be hazardous to traffic and require continuing maintenance. If this condition is encountered, subsequent cores may be broken off short to leave a base to hold the core hole patch. On multiple-course decks, it is generally best to core through individual courses and retrieve these before continuing into the structural slab. Coring through cold-patch material should be avoided as this may gum up the coring equipment and contaminate the structural concrete core.

When coring top slabs over prestressed concrete box-beams and prestressed concrete slabs, the bit should not be allowed to penetrate through the slab into the structural member.

2. Core Tests

The following core tests are equally applicable to evaluation of both monolithic and two-course bridge decks.

a. Visual Analysis Of Cores

Because the primary reason for taking concrete deck cores is to verify apparent surface condition, such as reinforcing bar corrosion, most cores should be taken for visual analysis only. When evaluating top reinforcement, the core need only be taken to the top reinforcing mat. When a core is taken to evaluate visible distress of the deck underside, a full-depth core should be obtained where possible.

Visual examination of deck cores is the primary means of determining the soundness of deck concrete. Cores should be physically tested only to resolve questions that cannot be answered by visual examination.

Visual examination must include written documentation to detail general condition of each core. The examination should be completed by a qualified Engineer and include depth of coring, rebars encountered and their position within the core, and field data and notes to help differentiate between coring damage and concrete deterioration.

Visual examination should include using a magnifying glass. A clean broken face examined under magnification will show hidden details such as fine cracking, and/or the presence of entrained air.

Visual examination and documentation of each core should include the following:

Deck Condition. Deck surface and underside condition in the area of the core should be noted to record the purpose of the core.

Depth Of Coring. Note core depth and whether it is partial-or full-depth.

Layer Thickness. If all concrete layers are intact, this will only entail measurement of the core. If rubble or broken layers are encountered, thickness and original position in the deck must be determined by measuring inside the core hole. Thickness, type, and condition of all materials, including bituminous overlays or patches, should be noted. Where a membrane is present, its thickness, type, and condition should also be recorded.

Reinforcing. Size, location, and condition should be noted. The rebar may have to be broken out of the core to verify potential measurements by observing corrosion deposits on both the reinforcement and adjoining concrete surfaces.

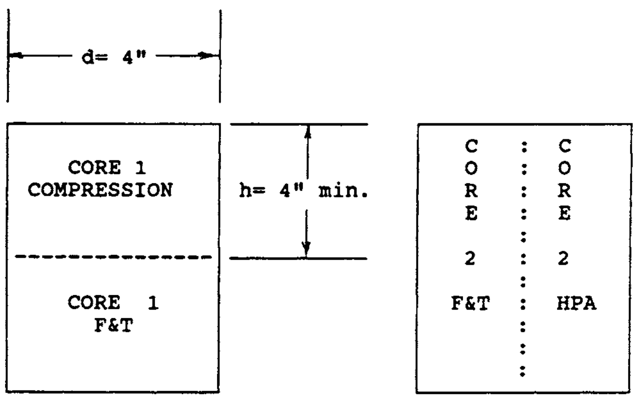
Concrete Condition. This could range from sound to rubble. Smooth and dense mortar in the core circumference indicates sound concrete. Rough, porous mortar indicates poor-quality concrete. Poor concrete consolidation during placement can result in excessive entrapped air (honeycombing, bugholes), resulting in poor concrete strength and durability.

Cracking description should include whether it is horizontal, layered (series of horizontal cracks), or vertical, and whether it goes through or around the coarse aggregate. Cracks through coarse aggregate indicate that they occurred after the concrete developed strength. Cracks going around coarse aggregate indicate shrinkage or a one-time overstressing very early in life of the deck, before concrete could develop strength needed to resist the loading condition.

Core Photographs. Closeup photographs of each core should be taken for permanent visual documentation. Cracks which are a result of the coring operation should be identified.

b. Laboratory Core Testing

Cores submitted to the Materials Bureau for testing must be properly marked with Core ID Number and tests to be performed (using a permanent marker). A memorandum describing test instructions, where the results should be sent, and who should be contacted if questions arise, must accompany the cores. Do not send wearing course segments or other portions of a core unless they are to be tested. No test can be performed on rubble. If there are segments, each should be marked with Core ID Number and test to be performed. Multiple testing on the same concrete specimen results in erroneous results, and thus must not be requested. Chloride testing, however, can be performed on a specimen before a freeze-thaw or air content test without affecting later results. If more than one test is desired on a whole core, it may be segmented to allow for multiple testing. The core should be marked to locate each cut; avoid cuts through reinforcing steel. Resultant segments should have Core ID number and test to be performed marked on them, as follows:



(1) <u>Compressive Strength Testing</u>

This quantifies the degree of concrete soundness. Concrete with no deterioration or visible cracking is proved strong and sound using this test. It is not necessary or desirable to test all cores for compression. Only a few carefully selected cores should be tested. Cores should be in good condition. Ideally, they should be at least 8 in. long, but lengths as short as 4 in. may be tested. They should not have reinforcing steel in their sides; steel through the core midpoint is okay.

Concrete strengths of 3,500 psi or less may be unable to withstand the rigors of repair with jack hammers and high-pressure water blasters. Additional cores and evaluation should be made before a decision on the appropriate deck treatment is made. There is no minimum concrete strength below which rehabilitation would be prohibited without additional study.

(2) Air Content of Concrete Cores

The high pressure air (HPA) test measures total air content of hardened concrete. About 1 to 2% of this total is entrapped air, and the rest entrained air. Air entrainment, with air bubbles of the proper size and dispersion, provides concrete with resistance to water freeze-thaw damage. As with compression testing, only a few select cores should be tested for air. This does not require a whole core; a 3 in. or larger piece of concrete can be used.

Usually, air testing is not even necessary. Concrete that has resisted freeze-thaw distress for years has withstood the test of time. Existing deck concrete still in repairable condition, whether air entrained or not, will be protected by an overlay system.

Total air contents ranging from 4 to 9% assure good durability. Concrete with less than 4% entrained air usually has poor resistance to freeze-thaw damage and must be protected if retained. Concrete with high total air (greater than 9%) may be investigated for strength, as compressive strength decreases with increased air content. However, if the deck is still in good condition then compressive strength should be okay.

Concrete from older structures (built before 1950) probably will not contain intentionally entrained air and testing should not be necessary. Between 1950 and 1960, natural cements were used with dry powdered air-agent admixtures, and special specifications calling for separate air entraining admixtures. All this provided some entrained air and varying protection; these structures may be tested for air content. Monolithic decks showing distress such as scaling should be tested for air content.

(3) Freeze-Thaw Testing Of Concrete

This complements the air content test. Adequate air yields little or no freeze-thaw loss of concrete but low air produces high loss. The test measures percent loss (by weight) of a concrete sample completely submerged in a salt (NaCl) solution and subjected to cyclic freezing and thawing. Freeze-thaw losses of less than 1% at 25 cycles usually indicate good durability. Those greater than 1% indicate poor durability. Older (before 1950) non-air-entrained concrete will generally show high or 100% loss, but properly air-entrained concrete will have low loss or none.

These results are an indicator of concrete durability. Concrete with no freeze-thaw loss will perform well when wet in winter, but that with freeze-thaw loss will slowly disintegrate. Test losses of less than 100% but more than 1% indicate concrete that will continue to deteriorate when exposed to additional cycles.

This test will not predict time to deterioration, because exposure conditions in the field are different and more variable than laboratory conditions. Concrete with high losses will perform well in service if it is protected and kept dry. As with air content testing, if freeze-thaw loss is to be evaluated, only a limited representative number of cores should be selected for testing. This cannot be run on cores tested for high pressure air or compression, unless the core is segmented as previously described.

E. Chloride Measurement

This technique consists of obtaining and testing powdered concrete samples, and is described in the Field Survey Manual. Samples can be obtained directly from the structural slab or cores taken from it. Chloride testing should be confined only to sound deck areas that are to remain after rehabilitation. High levels of chloride ($\geq$ 1.3 lb/c.y. of concrete), when moisture and air are present, cause accelerated rates of steel corrosion. Because overlay materials prevent or minimize moisture and air from reaching the underlying concrete, Department policy is to leave high chloride-contaminated concrete in place. This testing thus is not required for monolithic decks because it has no bearing on the type of repair.

Chloride testing on two-course decks may help a Designer estimate concrete removal quantities for the underlying structural slab. Structural slabs with low chlorides have been effectively protected over time, and thus will probably require little if any concrete removal resulting from corroding reinforcing steel. This testing is used only for estimating removal quantities. Final decisions on removal should be based on delamination and potential surveys made after the wearing course has been removed.

F. Thermography

This method is used to detect delaminations with an electronic thermometer device mounted in a moving vehicle. It is effective only on monolithic bridge decks and thus is inappropriate for two-course decks. Because data are collected from a moving vehicle, M&PT requirements are minimal. Its use should be limited to high-traffic locations where safety and cost are prime considerations. The Structures Division or the Field Engineering I Section of the Materials Bureau should be contacted before using thermography to assure its proper use. Thermography data plotted on a grid map identify areas for concrete removal.

G. Radar And Impact Echo

These are methods that have been used experimentally on several small bridge decks, and are not yet perfected for detecting delaminations. Although they can be used on monolithic decks, they are most advantageous on two-course decks. Results of the impact echo are very promising. The Materials Bureau is still developing these evaluation procedures. If interested in using these methods contact the Field Engineering I Section of the Materials Bureau (518-457-5956).

Table 1. Deck Evaluation Methods

Evaluation Method	Purpose	Output
Visual Examination	Locate cracks, spalls, patches, and other obvious signs of distress.	Distress Map
Sounding	Locate delaminated areas not visually evident.	Delamination Map
Potential Survey	Locate areas of actively corroding reinforcing steel.	Potential Map
Coring	Investigate areas where deck structural integrity is suspect or where depth of deterioration is unknown. Use in any questionable areas not adequately defined by other techniques, and to verify accuracy of sounding and potential surveys.	Core Data
Chloride Measurement	Determine quantity of chloride ion concentration at the rebars.	Chloride Data
Thermography	Locate delaminated areas through measurement of deck temperature differences.	Delamination Map
Radar and Impact Echo	Experimental methods of locating deck delaminations. May be especially useful on two-course decks.	Delamination Map

III. DECK TREATMENTS

This Chapter defines various deck treatment options and describes their advantages and limitations. Information is provided on rehabilitation tasks included in the treatments, expected service life and treatment cost, and a procedure for economic analysis of the alternatives.

A. Deck Treatments

1. Non-Protective Treatments

Maintenance of the existing surface or applying an asphalt overlay without a waterproof membrane are considered non-protective. Under these treatments, the deck continues to deteriorate, resulting in further structural damage. This damage in turn increases the cost of subsequent treatments whether those are protective or non-protective. Thus, although their initial cost is low, the long-term penalty must be recognized.

Because deterioration continues under non-protective treatments, their use should be limited to short-term applications. Asphalt overlays or maintenance treatments are effective for keeping a deck in service until it can be replaced. Maintenance filling of isolated potholes during the period between evaluation and design of a deck rehabilitation and its construction is also an appropriate use of asphalt concrete

2. Protective Treatments

Asphalt overlays with waterproof membranes and concrete overlays are all protective treatments, extending deck service life.

Asphalt overlays with protective membranes have shorter service lives than rigid concrete overlays. Given favorable roadway geometrics and traffic, the overlays average up to 11 years of service. The protective membrane has an estimated life of 22 years.

Asphalt concrete, especially in combination with a protective membrane, is very sensitive to plastic deformation (shoving/slippage) failure. The asphalt/membrane system thus should not be used on high-traffic roadways (> 5,000 AADT), steep grades (> 4%), sharp curves (i.e., ramps), and major interchanges with on/off ramps, which subject the pavement to severe acceleration and deceleration forces. In such situations, asphalt concrete service life will be significantly reduced and a rigid concrete overlay system should be selected. Asphalt concrete/membrane systems are best suited to rural, through-traffic structures where longer service lives are desired.

For both protective treatments, the extent of deep removal is the major factor in service life and cost. It is not possible to determine reliably the influence of variations in amount of deep removal on service life. Accordingly, selection of the appropriate amount of deep removal must be based on technical rather than economic factors. Delaminated areas and those experiencing active corrosion are the initial indicators in determining how much concrete to remove. Small islands and narrow peninsulas of concrete surrounded by areas of high half-cell potential readings should also be cleared. These areas may deteriorate rapidly after uncontaminated concrete is placed adjoining them. In general, islands and peninsulas should be removed when there area is less than 100 sq ft or the smallest dimension is less than 5 ft.

In addition, the percentage of removal should be considered. If only a small area requires repair the work should be confined to that area, but where the percentage is high it may be desirable to remove 100 percent. These issues are discussed further in Chapter IV.

3. Deck Replacement

Deck replacement is the treatment option with the highest first cost and should be considered to be a last resort. Deck maintenance and rehabilitation must be carefully managed to delay replacement for the longest possible time. A deck may have only localized areas of deterioration through its full thickness. Full-depth repair should be limited to those areas unless economic analysis shows complete replacement to be justified.

B. Deck Rehabilitation Tasks

Regardless of the deck treatment selected, several common construction tasks may be performed:

1. Wearing Course Removal

   Removal of asphalt or concrete overlay.

2. Structural Slab Scarification

   Deck concrete is removed by mechanical scarification. Unless a greater depth is indicated on the plans, the concrete is removed to a minimum of 1/4 in. and a maximum of 1/2 in. When 100% deep removal is specified, this pay item is not used.

3. Reinforcing Bar Exposure

In this operation, commonly referred to as deep removal, structural concrete is removed from the periphery of the uppermat reinforcing bars to provide a minimum 1-in. clearance between the reinforcing bar surface and remaining concrete surface. Deeper concrete removal may be needed to reach sound concrete.

4. Reinforcing Bar Cleaning

Blast cleaning to remove all grease, dirt, concrete, mortar, and injurious rust from reinforcing bars. Injurious rust includes all scale, loose rust deposits, or all rust not firmly bonded to the steel. Bar cleaning is paid under the payment item for concrete overlays.

5. Slab Reconstruction

Placement of concrete around exposed reinforcing bars to the level of the surrounding concrete or to 1/2 in. above the reinforcing steel. Bonding grout is placed on all surfaces receiving slab reconstruction concrete, which may be either Class D or one of the specialized concretes used for overlays.

6. Waterproof Membrane Application

Protective membranes applied to the concrete deck. An asphalt overlay is placed over the membrane.

7. Overlays

Asphalt and concrete overlays are used. Asphalt overlays are the same material and installed by the same procedures as highway pavement top courses. Specialized concrete materials are used for concrete overlays. Class E Concrete is used when the final overlay thickness will be greater than 3 in. One of the specialized concretes, at the Contractor's option, is used for overlays 3 in. and less in thickness. The specialized concretes include:

a. High Density Concrete

A portland cement concrete of very high density made from standard concreting materials, with a required slump between 1/2 and 1 in. The minimum thickness of overlay concrete is 2 in.

b. Latex Modified Concrete

A portland cement concrete with a styrene-butadiene latex admixture. The minimum thickness of overlay concrete is 1-1/2 in.

c. Microsilica Concrete

A portland cement concrete with a microsilica admixture. The minimum thickness of overlay concrete is 1-1/2 in.

8. Transverse Saw Cut Grinding - Required to achieve macro-texturing.

Application of these tasks to the various rehabilitation treatments is given in Table 2.

Table 2. Deck Rehabilitation Tasks

Task	Asphalt	Asphalt With Membrane	Concrete After Select Deep Removal	Concrete After 100% Deep Removal
Wearing Course Removal	As Needed	As Needed	As Needed	As Needed
Scarification	Not Required	Not Reqd.	Required	Not Reqd.
Rebar Exposure	As Needed	As Needed	As Needed	Required
Rebar Cleaning	As Needed	As Needed	As Needed	Required
Slab Reconstruction	As Needed	As Needed	As Needed	Required
Waterproof Membrane	Not Required	Required	Not Reqd.	Not Reqd.
Overlay	Required	Required	Required	Required

C. Service Estimates

Service life has been estimated for each treatment. "Service Life" means length of time that a particular treatment will last before additional deck work is needed. The formal definition is the age at which 50% of decks develop delaminations over 40% of their surface areas. The original estimates (1986) were based on interpretation and evaluation of deck deterioration data by the Technical Services Division. More recent studies have not improved those estimates, but suggest that amount of deep removal and quality of the removal and reconstruction specifications strongly influence the service life obtained. Thus, there is evidence that service life depends on the amount of deep removal, but insufficient data to reliably predict the magnitude of this effect. It is recognized that local conditions and experience may support use of different values for service life. Table 3 indicates service lives for each treatment.

Table 3 - Performance Estimates

Treatment	Service Life, Years
Maintenance Only	--
Asphalt Overlay	4
Asphalt With Membrane	22*
Concrete Overlay (Select Deep Removal)	25
Concrete Overlay (100% Deep Removal)	35
Replacement Deck	40

* The asphalt overlay with waterproof membrane treatment requires resurfacing after 11 years.

D. Treatment Costs

Determination of treatment costs is difficult because of the many variables involved. For specific projects costs should be estimated using regional values. The bridge rehabilitation module of the Preliminary Estimate Program (PEP) should be used for this purpose.

Analysis of 1989 weighted average bid prices produced the statewide average values shown in Table 4. It must be emphasized that these values and all cost values shown in this Manual are for illustrative purpose only, and should not be taken to represent actual cost experienced in any region. In addition, conclusions resulting from applying these costs are not intended to be absolute. Only costs specific to a project should be used in selecting deck treatments.

Table 4. Statewide 1989 average weighted bid prices.

Description	Cost
Asphalt W.C. Removal	$ 1.34/Sq.Ft.
Scarification	$ .99/Sq.Ft.
Rebar Exposure	$12.61/Sq.Ft.
Waterproof Membrane	$ 2.07/Sq.Ft.
Type 6F Top Course	$39.49/Ton
Type 3 Binder Course	$36.50/Ton
Concrete Overlay*	$ 4.89/Sq.Ft.
Concrete Removal	$12.30/Sq.Ft.
Concrete Placement	$22.34/Sq.Ft.
Steel Reinforcement	$ 0.78/Lb.
Transverse Saw Cut Grooving	$ 0.64/Sq.Ft.

* Composite of specialized concrete overlay materials.

These costs are combined as necessary to estimate expense of a treatment on a specific project. For example:

Treatment	Cost/Sq.Ft.
Asphalt With Membrane	$11.52
Concrete (Select Deep Removal)	$15.27
Concrete (100% Deep Removal)	$23.03
Replacement Deck	$39.60

Select deep concrete removal is assumed to involve 50% of the deck area. Cost of deep removal is large with respect to the other items comprising a deck rehabilitation. It is thus appropriate for the specific deck being investigated here. It is important to remember that amount of deep removal is a function of deck condition, not influenced by the type of overlay to be used. Using pay item data, cost of the treatment can be expressed as a function of percent deep removal.

E. Life-Cycle-Cost Analysis

Because of the different service lives demonstrated by the various treatments, a comparison based on initial cost is inappropriate. The present worth method accounts for service life differences and the time value of money.

Although deck rehabilitation treatment decisions are dictated primarily by technical considerations, life cycle cost of suitable treatments must be analyzed. Four fundamental decisions are required before such an analysis can be completed: 1) analysis method, 2) discount rate, 3) analysis life or planning horizon, and 4) sequence of treatments over the analysis life.

The first two are dictated by Department policy, which requires using the present worth method with a 4% discount rate. This method converts expenditures occurring at different times into equivalent amounts occurring at the present. For a particular expenditure, the present worth is computed as

$$PW = C \times SPPWF_y$$

where PW = present worth of the expenditure,

C = future cost of the expenditure.

and $SPPWF_y$ = single-payment present worth factor for an expenditure at year Y.

Total present worth for all treatments needed to extend deck life through the planning horizon equals the sum of the individual values.

Other economic analysis relationships needed are:

$USPWF_y$ = uniform series present worth factor for y payments,

and

CRF_y = capital recovery factor over y years.

These terms are defined mathematically in Appendix A.

The planning horizon should be selected on the basis of the unique characteristics of the bridge being rehabilitated. Generalized average data on bridge lives are irrelevant in a project-specific decision.

Except for non-protective treatments, bridge deck rehabilitation alternatives have service lives over 20 years. Because of these long lives, there has been no experience with retreatment. Lacking such experience, it may be assumed that the same sequence of actions will be repeated, except when the present treatment is non-protective. Thus, the possible treatment sequences are 1) a non-protective treatment followed at the end of its service life by repeated applications of a protective treatment, 2) deck replacement followed by repeated applications of one of the protective treatments, or 3) one of the protective treatments repeated as needed.

As an example of life-cycle-cost analysis, assume the following treatment costs and service lives:

Treatment	Cost	Service Life Years
Asphalt Overlay	\$ 1.92	4
Select Deep Removal	\$15.27	25
Replacement	\$39.60	40

Two treatment sequences have been determined to provide technically appropriate solutions for a deteriorated deck. Sequence A involves an immediate asphalt overlay with a deck replacement in 4 years. Sequence B involves select deep removal and overlaying at 25-year intervals. Compare the life cycle costs of these treatments using a discount rate of 4%.

Sequence A = $1.92 + 39.60 \times SPPWF_4 = \$35.77$/sq. ft.

For Sequence B, two applications provide 50 years of service compared to the 44 years provided by Sequence A. Accordingly, Sequence B costs must be adjusted to a planning horizon of 44 years:

Sequence B = $15.27 + 15.27 \times CRF_{25} \times USPWF_{19} \times SPPWF_{25} =$ \$20.09/sq. ft.

Sequence B clearly has the lower life-cycle cost.

IV. TREATMENT SELECTION

Various treatments and technical recommendations for selecting suitable applications have been described in Chapter III. Based on technical considerations, criteria can be established that eliminate particular treatments from further consideration. Having narrowed the field of possible selections, a final choice can be based on economic comparisons. Clearly, such comparisons should not be performed on treatments that do not satisfy the project's technical needs.

A. Deck Rehabilitation

A fundamental consideration in identifying suitable treatments is determining whether protective or non-protective action is appropriate. In general, non-protective treatments should not be used except where the deck must be kept in service relatively briefly until replacement. Use of non-protective treatments to provide service until a protective treatment can be applied should be discouraged. Deck deterioration develops gradually and only rarely can appropriate protective treatments not be normally programmed. Under a non-protective treatment deck deterioration may accelerate, resulting in increased cost for future protective treatments.

For protective treatments, two independent decisions are needed - area of concrete deep removal from around the top rebar mat and type of overlay material. Deep removal area is the more important and difficult decision. Both technical and economic considerations should be examined to resolve this issue. Appropriate area of deep removal does not depend on type of overlay material.

The following discussion is directly applicable to both two-course and monolithic bridge decks. For two-course decks, whether the top course is concrete or asphalt, deck condition evaluation methods differ as discussed in Chapter II. Except for condition evaluation, however, there should be no difference in the way rehabilitation treatments are selected. The objectives of a condition evaluation are to determine if the deck can be rehabilitated, and if so the minimum amount of rebar exposure required. With this minimum determined, the appropriate treatment is selected independent of existing deck type.

Three indicators of monolithic deck condition are used in determining the need for deep concrete removal and its extent -- spalls, delaminations, and half-cell potential measurements greater than 0.35 v. Spalls are the primary indicator because without this visible indication of deck failure further deck evaluation is unnecessary. Delamination and half-cell potential would not ordinarily be measured unless such evidence of deterioration were apparent.

Deck deterioration is conveniently reported in two ways -- as area of spalls, or as total damaged area, both expressed as percent of deck area. Total damaged area is taken as the sum of non-overlapping area of spalls, delaminations, and half-cell potentials greater than 0.35 v.

These indicators provide a quantitative measure of deck condition and identify deck areas that are currently damaged or actively corroding. The total damaged concrete area, as a minimum, must be removed to a level at least 1 in. below the top mat of reinforcement. There are additional, less objective reasons for increasing the area of deep concrete removal. Half-cell potential measurements on recently repaired decks indicate that corrosion activity often increases dramatically in concrete that is left in place. Thus, concrete that did not warrant removal because of half-cell potential readings before repair may show values that would justify removal after repair. This concrete generally has medium potential (0.15 to 0.34) before removal and represents a deck area less than the total damaged area. To eliminate the possibility of premature failure of the rehabilitated deck, these areas should be removed according to the following criteria:

1. If the sum of all medium-potential areas equals or is less than total damaged area, then they should be removed. This comparison should be on a span basis.

2. If any medium-potential area is less than 100 sq. ft. or has a minimum dimension equaling or less than 5 ft., it should be removed.

The total area to be removed is defined as the sum of total damaged area and the areas just identified by Criteria 1 and 2.

The total removal area is based solely on technical considerations and represents that necessary to assure that at least half of the repaired deck achieve the service lives given in Chapter III. At some locations, conditions may exist requiring greater confidence in longevity of the rehabilitation. Specifically, 100% deep concrete removal may be justified on bridges in urban areas with high-traffic density whenever one or more of the following conditions are met:

1. Area of spalls exceeds 2%,

2. Area of delamination exceeds 30%,

3. Area of half-cell potential greater than 0.35 exceeds 40%, or

4. Total damaged area exceeds 50%.

These conditions are exclusive of distress within 2 ft. of a bridge joint.

For two-course bridge decks, electrical potential cannot be tested before construction because of presence of either a non-conductive asphalt overlay or mesh-reinforced concrete overlay. Because of absence of this information, amount of reinforcing bar exposure required must be estimated. Cores, sounding (chain drag or hammer), visual examination of the underside of the deck surface, and the experience of the Designer should all be used in estimating quantity to be removed. After the wearing surface is removed, the monolithic deck evaluation criteria can be used to establish the final removal limits.

Absolute traffic levels cannot be established at which 100% deep-removal is justified, but two additional considerations can be included in a rehabilitation plan to permit comprehensive comparison of deck treatments -- 1) M&PT cost and 2) user cost associated with construction delays. M&PT costs cannot be generalized, but they can be estimated accurately on a project basis. User costs associated with construction delays are estimated in terms of vehicle-hours. It is generally assumed that there must be a minimum delay (5 minutes) before any cost is assessed. Estimating duration of construction delays is considerably more complex than estimating costs of maintaining and protecting traffic.

Because M&PT and user costs are strongly influenced by site characteristics and only secondarily by type of deck treatment, including these costs increases treatment costs by a constant dollar amount on a specific project. Thus, proportionate change in cost is less for longer-lived treatments having higher initial cost. The net effect is to tilt the present worth comparison in the direction of treatments with longer lives.

As previously noted, the overlays appropriate for protective treatments are asphalt with waterproof membrane, Class E concrete, and the three specialized concretes - (high density, latex modified, and microsilica). Limitations on use of asphalt with waterproof membrane are described in Chapter III (Section A2).

The Class E concrete is restricted to overlay thicknesses greater than 3 in. because of permeability and aggregate size. If specialized concrete overlays are specified, selection of one of the three possibilities is the Contractor's option. Current service life data are inadequate to demonstrate a difference in durability of any of the rigid overlay materials.

B. Deck Replacement

Although cost of a deck replacement is substantially greater than that of a deck rehabilitation, the decision to replace is primarily technical. Indications of deck underside dampness or efflorescence strongly suggest need for replacement. Presence of delaminations or spalls on the underside of the deck necessitate replacement. These conditions may be local, and partial full-depth repair may be all that is necessary to restore the deck. Nevertheless, M & PT cost and user delays may justify complete replacement.

C. Treatment Selection Considerations

Using the methods explained in Chapter III, total present worth of each deck treatment that will remedy the deck's technical deficiencies can be estimated. Although this estimate is a representative value for true cost of deck rehabilitation, it is not appropriate simply to select the treatment with lowest present worth. Cost estimating is not an exact science and even bid prices will not necessarily reflect actual cost to perform the work.

Treatment selection should favor the treatment with longest expected service life. Thus, small premiums for additional service life are warranted. The size of an acceptable premium is sensitive to local conditions and concerns and is thus left to the discretion of the individual regions.

D. Examples

Two examples of cost estimating and project selection are given in Appendix B. In the first, a 6,400 sq ft deck, the difference in present worth between 100% deep removal (35-year life) and 50% deep removal (25-year life) is only $5,850 (less than 3%). Total initial cost of the treatment with longer life is 23% larger ($51,600) than the 50% removal option. Deck treatment costs are 56% of total project cost.

In the second example, two interstate bridges with a total area of 12,960 sq ft are being rehabilitated. Present worth of Alternative 2 with the longer life exceeds the 50% removal option by $20,426 (5%). The difference in initial cost is $104,593 (25%) greater than the 50% option. Deck treatment costs are 58% of the total cost.

Cost estimating generally involves comparison between cost of the minimum amount of deck removal that will satisfy technical requirements of the job and that of 100% deep removal. In determining the technically acceptable removal quantity, care should be taken to follow all provisions of this Manual for deck evaluation. Questionable deck areas should always be removed.

V. REPORTING REQUIRFMENTS

This Chapter explains how to document findings of the deck evaluation and present them in a manner consistent with the evaluation methods outlined in Chapter II.

It is important that all sections of this Chapter be completely satisfied to document deck condition properly, and to support the recommendation to repair or replace the deck. Content of the sections may vary because of the severity of deck deterioration or extensiveness of the proposed rehabilitation. Appendix C shows a sample Bridge Deck Evaluation Report.

A. TITLE: BRIDGE DECK EVALUATION REPORT

Identify Structure
- BIN Number
- County
- Town, City, Village
- Region
- Feature Carried
- Feature Crossed

B. INTRODUCTION

Bridge History
- Year Built
- Bridge Type
- Structure Length and Out-To-Out Width
- Previous Work Done, particularly on the deck
- Planned Future Work

Highway Classification
- Traffic Volumes
- Plan for Maintenance and Protection of Traffic

C. DECK INSPECTION FINDINGS

This includes data collected and developed during deck evaluation field work. All survey work must be recorded by span for both the top and bottom of the deck. The following should be provided for review:

1. SKETCH OF DECK UNDERSIDE:

a) Framing system
b) Cracks
c) Damp areas
d) Areas of efflorescence
e) Rusted stay-in-place Forms
f) Spalls and exposed rebars
g) Other indications of deterioration

All deterioration should be quantified based on percentage of the deck exhibiting the respective type of deterioration.

2. COLOR PHOTO OF UNDERDECK:

   a) Typical good areas
   b) Areas of deterioration in each span, showing any of the seven types of deterioration just listed.

3. SKETCH OF DECK SURFACE:

   a) Spalls
   b) Cracks
   c) Joint problems
   d) Patches
   e) Other indications of deterioration
   f) Core locations
   g) Areas of high potential (0.35v), as appropriate
   h) Areas of delamination, as appropriate

All deterioration should be quantified on percentage of deck exhibiting the particular type of deterioration.

4. COLOR PHOTOS OF DECK SURFACE:

   a) Typical good areas
   b) Areas of deterioration in each span, showing any of the seven types of deterioration.

5. Color photos of the bridge in elevation, approaches, substructures, and any problem areas.

6. Photo layout sheets indicating location of photographer and camera orientation.

7. Inspection Forms: Copies of Forms TP349 and TP350 from the most-recent biennial inspection should be reviewed for comments prior to the start of the deck inspection. These forms should be attached to the deck report with additional comments added, as appropriate.

D. <u>DECK CORE EVALUATION AND TEST RESULTS</u>

1. DETAILED VISUAL EXAMINATION:

   General description of core(s) and any defects. Examination complemented by field data and notes to help differentiate between any coring damage and concrete deterioration. Examination should also determine depth and location of materials encountered in the core.

For each core or series of cores, report the following:

a) Explain why this core location was selected.

b) Depth of Coring: Note whether core is full-depth or partial-depth and if appropriate the reason for partial-depth coring. Also note deck surface and underside condition in the core vicinity.

c) Note the thickness of layers making up the core. If all concrete layers are intact, this will only entail measurements. If rubble or broken layers are encountered, their thickness and original position in the deck should be determined during coring by measuring inside the core hole. Thicknesses, type, and condition of bituminous overlays or patches should be noted.

d) Presence of a membrane, and its thickness, type, and condition.

e) Reinforcing steel location, size, and condition, e.g. 1-1/2 in. cover, No. 5 bar, no rust. A rebar will often have to be broken out of the core after completing its examination, to check for corrosion.

f) Concrete: Condition of the concrete may range from sound to rubble. (This discussion should include all observations resulting from the evaluation techniques suggested in Chapter II.)

   1) Concrete Mortar Quality: type, depth, and amounts of deterioration should be noted.

      a. Concrete mortar scaled away due to moisture freezing and thawing.

      b. Concrete spalling caused by internal pressures such as expansive corrosion.

      c. A smooth, dense mortar on the core circumference indicates sound concrete.

      d. A rough, porous core circumference indicates possible deterioration. Coring may wash away poor-quality mortar, leaving a rough irregular surface.

   2) Any voids and honeycombing due to lack of consolidation, or excess entrapped air voids should be noted.

3) Cracking (whether horizontal, layered, or vertical) should be described.

This information can be summarized in the Bridge Deck Core Record (Appendix D).

Close-up photos of each core with proper identification (including BIN Number) on a card in the photos. They should be taken straight-on, with a scale used as a reference in each shot. In addition to the photos, each core should be documented as follows:

SAMPLE BRIDGE DECK CORE RECORD

Core No.:	1
Depth:	18" full depth, depression in asphalt overlay, underside normal
Overlay:	5" total, two 1" layers of top course over a 3" binder
Wearing Course :	4", total deterioration, steel mesh 1/2" from bottom
Membrane:	None
Structural Slab:	9" total, slight 1/8" scaling at top, layered layered cracking through mortar around crushed stone, coarse aggregate in top 3", No. 5 bar top rebar 1-1/2" down shows heavy corrosion; remaining 6" of concrete appears sound, no excess voids, good consolidation, no corrosion on bottom steel, no staining on bottom of core.
Tests:	(As Appropriate) Compression: 5000 psi, structural slab. (It not necessary to test each core. This will be determined by the Engineer.)
Freeze-Thaw:	NaCl Solution: 3" section of structural slab, 100% loss in 20 cycles.
Air Content:	Structural slab 1.2% entrapped, 0.17% entrained.
NOTE:	See Appendix E for form to mount core photo along with appropriate documentation description.

E. RECOMMENDATIONS

A recommendation for scope of work should be included based on the engineering evaluation. All recommended repairs should be described with sketches provided for other than routine recommendations.

INDEX

Author's Note About the Index: The three columns on the right hand side of the pages which follow refer to:

AASHTO Standard Specification for Highway Bridges, Fifteenth Edition - 1992

AASHTO LRFD Bridge Design Specifications, First Edition - 1994

Ontario Highway Bridge Design Code, Third Edition - 1991, and a separate volume entitled "Commentary" - for the above Code.

The first column references are all to page numbers, as the 15th edition pages are numbered sequentially.

The second column references, for the LRFD (Load and Resistance Factor) specification, such as 9-37 refers to a single page, as all of the pages in this specification are numbered sequentially ***by Chapter***, with the first number being the chapter. When I wanted to refer to a sequential group of pages, I used a notation such as 6-94—6-97, meaning pages 6-94, 6-95, 6-96, and 6-97.

The third column references, the Ontario specification, refer to page numbers ***by Section,*** so that 4-5 means the 5th page in section 4, and C4-5 refers to a page in the Ontario Commentary volume.