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NATIONAL COOPERATIVE
HIGHWAY RESEARCH PROGRAM REPORT **312**

CONDITION SURVEYS OF CONCRETE BRIDGE COMPONENTS—USER'S MANUAL

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TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL

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CONDITION SURVEYS OF CONCRETE BRIDGE COMPONENTS

USER'S MANUAL

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AREAS OF INTEREST:

Structures Design and Performance
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TRANSPORTATION RESEARCH BOARD
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WASHINGTON, D.C. DECEMBER 1988

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

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FOREWORD

By Staff
Transportation Research Board

This manual is intended for engineers and technicians involved in the inspection of concrete bridges and concrete bridge components. The first part of the report supplements current guides for conducting federally mandated routine bridge inspections. The second part provides information on more elaborate procedures that may be needed for further in-depth investigations. The appendixes contain examples of load-carrying capacity computations.

Although concrete structures have generally demonstrated good resistance to loss of load capacity and have only rarely been removed from service for this reason, determination of bridge load capacities is often necessary to fully evaluate the effects of deterioration. Currently, work is progressing to develop methods to permit more accurate evaluation of structural capacity; however, inspection and reporting methods need to be enhanced or refined and then standardized to help support this work.

More than one level of inspection should be available for structures with severe damage. The current, federally mandated biennial inspections are expected to be adequate for the majority of structures; however, refinements and additional guidance would improve the uniformity of inspection and reporting. Structures, where the initial inspection and available data indicate a reduced load capacity, should be reinspected using procedures that provide a higher level and quality of information on the structure's properties.

Consequently, research was needed that would provide a framework for surveying and reporting the condition of reinforced and prestressed concrete structures. When conditions warrant, the framework would have to include more than one inspection level to improve the reliability of data.

Under NCHRP Project 12-28(5), "Standard Methodology for Conducting Condition Surveys of Concrete Bridge Components," New Mexico State University, Las Cruces, New Mexico, undertook the needed research. A User's Manual has resulted that is intended to supplement existing guidance, namely, the Federal Highway Administration's *Bridge Inspector's Training Manual 70,* and to a lesser extent, the American Association of State Highway and Transportation Officials' *Manual for Maintenance Inspection of Bridges.*

The first part (Chapters 1 and 2) of the report provides information on the types of concrete bridges, physical properties of concrete and steel, indicators and causes of deterioration of reinforced and prestressed concrete, and inspection techniques for helping to assess reductions in bridge capacity. The second part (Chapters 3, 4, and 5) describes the types of detailed inspections that are possible. The proper inspection techniques, including the types of expertise and equipment that is needed or available, and the procedures for estimating load-carrying capacities are presented. Appendixes A, B, and C contain example calculations to determine load-carrying capacities for rating bridges. Appendix D lists decision criteria that are keyed to sections of the Manual for the user's convenience.

The research effort that produced the manual is documented in the research agency's original manuscript which is available from the NCHRP on request.

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ACKNOWLEDGMENTS

The research performed herein was accomplished under NCHRP Project 12-28(5). The principal investigators were John Minor and Kenneth R. White, Professors of Civil Engineering at New Mexico State University. Raymond S. Busch, Structural Engineer, H. W. Lochner, Inc., was co-author of this report and made extensive contributions. Graduate Students Clifford Madrid, now with National Aeronautics and Space Administration, and Gary Kinchen, now with the Federal Highway Administration, did the capacity evaluations and comparisons for the manual.

The authors appreciate the valuable comments and suggestions from the project panel along with contributions from other interested persons from various transportation agencies.

CONDITION SURVEYS OF CONCRETE BRIDGE COMPONENTS—USER'S MANUAL

SUMMARY

NCHRP Project 12-28 (5) was initiated in order to meet the need for a comprehensive framework for surveying and reporting the condition of reinforced and prestressed concrete structures. The goal of the research conducted under Project 12-28 (5) was to provide a manual for conducting inspections of reinforced and prestressed concrete bridges to assess their condition and recognize the various types of distress and their significance on capacity. The manual that evolved from this research is published herewith. The research that led to the preparation of the manual is documented in the agency final report: "Standard Methodology For Conducting Condition Surveys of Concrete Bridge Components." That report is not published in the regular NCHRP report series, but a limited number of copies are available on a loan basis upon written request to the Cooperative Research Programs, Transportation Research Board, 2101 Constitution Avenue, NW, Washington, D.C. 20418.

The routine biennial inspection of concrete bridges or concrete bridge components requires the diligent attention of qualified bridge inspectors. Qualifications of bridge inspectors are given in the National Bridge Inspection Standards (NBIS) (*1*) and in the American Association of State Highway and Transportation Officials (AASHTO) publication, *Manual for Maintenance Inspection of Bridges* (*2*). The purpose of this manual (*NCHRP Report 312*) is to provide guidance to field inspectors that will allow them to recognize and evaluate the various types of distress on reinforced or prestressed concrete bridges. The manual is divided into two major parts comprised of five chapters. The first part (Chapters 1 and 2) describes techniques that are used in routine biennial inspection of concrete bridges. The techniques described include methods commonly used by various highway agencies and is intended to provide an enhancement to the Federal Administration (FHWA) publication, *Bridge Inspector's Training Manual 70* (*3*). The second part (Chapters 3, 4, and 5) describes methods that may be used for a higher level of inspection for concrete bridges with more severe damage. The techniques include refinements and additional guidance that are not generally used in the routine biennial inspections. This higher level of inspection should improve the reliability of the inspection data for both reinforced and prestressed concrete bridges.

The appendixes (A, B, and C) provide examples of computing the load-carrying capacity rating of concrete bridge components. These examples include both reinforced and prestressed concrete components. The examples are used as illustrations of the methods of analysis for typical types of concrete bridges. The final appendix (D) includes decision guidelines to assist in condition rating and evaluating concrete bridge components.

CHAPTER **1**

ENHANCEMENT OF ROUTINE BIENNIAL INSPECTIONS

1.1 GENERAL

The bridge inspector must be familiar with the various types of concrete bridges and bridge components that may be constructed of concrete to properly describe the bridge in an inspection report. The more common types of concrete bridges and concrete components of bridges are described in the following discussion.

1.2 TYPES OF CONCRETE BRIDGES

A bridge is classified by the primary load-carrying member or members. For example, for girder-deck systems, the inspector classifies the bridge according to the type of girders used (T-beams, I-beams, and so on). An accurate description of a bridge requires that the inspector include several modifying terms, such as if the primary load-carrying member is concrete, the bridge is classified as a concrete bridge; if the member is steel, the bridge is classified as a steel bridge. This classification applies even though other components, such as the deck or piers, are a different material. The type of span designed also enters into the description of the bridge. Each bridge is described in the FHWA Coding Guide (*4*), as simple spans, cantilever-suspended spans, or continuous spans.

1.2.1 Slab Bridges

A concrete slab bridge is nothing more than a wide shallow beam in which the beam itself acts as the deck. A concrete slab bridge is usually continuous, although some simple span slabs exist. Slabs can be made of either reinforced concrete or prestressed concrete. A typical reinforced concrete slab bridge is shown in Figure 1.

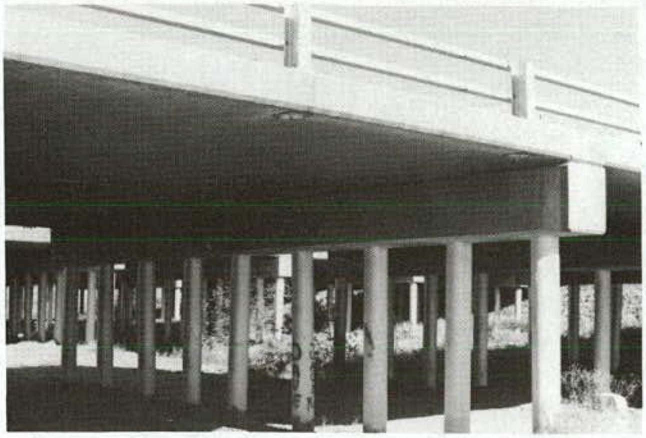


Figure 1. Typical concrete slab bridge.

Precast units are sometimes used to form a slab bridge. Several types of precast concrete units are used by various highway agencies in slab bridge construction. These precast units include the channel slab, solid slab, voided slab, and the pan slab (*5*). These special precast units may be constructed of either reinforced concrete or prestressed concrete.

1.2.2 Girder or Beam Bridges

A girder or beam bridge consists of a deck supported directly by longitudinal girders or beams. Concrete girder or beam-type bridges may be either reinforced concrete or prestressed concrete and are usually precast. Most concrete-beam-type bridges are also composite, that is, the beam and deck have a load-carrying connection between the beam and deck. This composite section allows the beam and deck to act together to carry the load. The T-beam and the I-beam are two common beam or girder-type concrete bridges. The T-beam is generally a cast-in-place monolithic deck-and-beam system. The T-beam is named such because of the "tee" shape used in a typical analysis of the section. A typical T-beam bridge is shown in Figure 2.

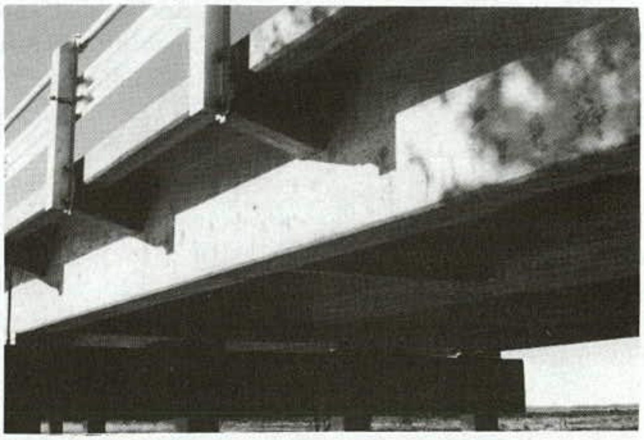


Figure 2. Typical T-beam bridge.

Several precast T-beam shapes are used by various highway agencies. These include the bulb tee, the double tee, the quad tee, the rib tee, and decked bulb tee shown in Ref. *5*.

The most common concrete I-beam shape is the AASHTO shapes used by most state highway agencies. These I-beams are normally precast and prestressed. Several highway agencies have developed variations of the AASHTO shapes to accommodate their particular needs. Examples of these shapes are also shown in Ref. *5.* A typical interstate prestressed girder bridge is shown in Figure 3. Older prestressed girder bridges generally are simple spans, whereas many of the newer bridges are simple span for dead load and continuous for live load. These bridges utilize cast-in-place continuous decks constructed on precast, prestressed girders.

1.2.3 Box Girders

Concrete box girders have become quite popular in recent years. As the name implies, the girders are constructed with a cross section that is rectangular or box-shaped such that the roof and floor act as flanges and the walls act as webs. The bridge may be a large box, as shown in Figure 4; or a multitude of smaller boxes, as shown in Figure 5. These structures may be simple span or continuous and either prestressed or reinforced concrete. The box units may be cast-in-place or precast, depending on the location or experience of the highway agency involved. A typical precast unit is shown in Ref. *5.*

Segmental box girders are frequently used for long span bridges. These units are very large box girder segments usually constructed by a cantilever method. Figure 6 shows a segmental box girder under construction; note, in this figure, the absence of falsework. The concrete segmental box girders are also used in cable-stayed bridges (*6*).

Figure 3. Prestressed girder bridge.

1.2.4 Concrete Box Culverts

A concrete box culvert (CBC) consists of a box-like concrete frame, generally normal to the roadway, which has a waterway or roadway passing through the culvert underneath the roadway. A culvert is defined as a bridge by the National Bridge Inspection Standards (NBIS) if the distance from backwall to backwall equals or exceeds 20 ft. Concrete box culverts may have individual openings or boxes of less than 20 ft but, grouped together, they meet the definition of a bridge and must be inspected as such. The CBC is usually analyzed as a continuous concrete

Figure 4. Large concrete box bridge.

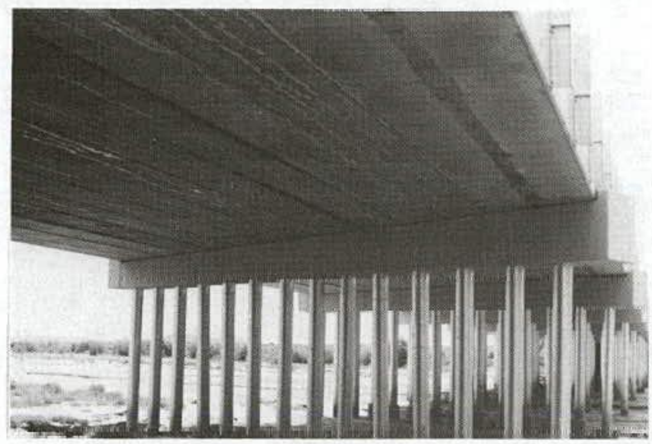


Figure 5. Precast multiple box bridge.

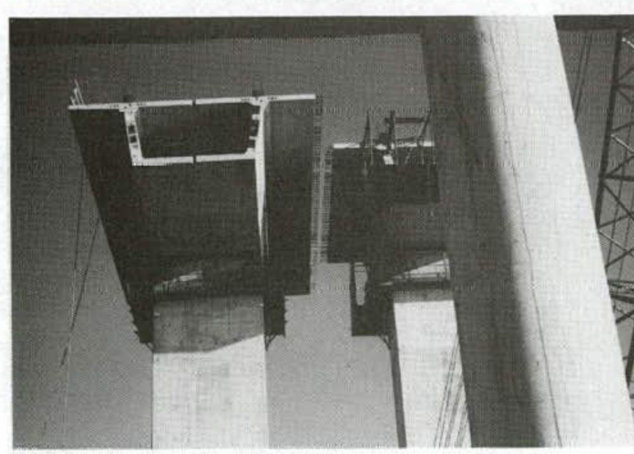


Figure 6. Segmental box girder under construction.

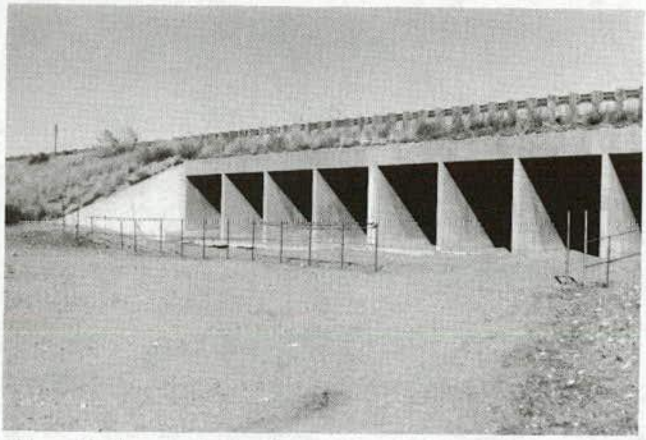


Figure 7. Concrete box culvert.

Figure 8. Spandrel arch bridge. (Photo Courtesy of Burgess & Niple, Limited)

frame and is frequently used over small or intermittent waterways. A typical CBC is shown in Figure 7.

1.2.5 Arch

A concrete arch is the natural extension from Roman stone arch. The true arch carries load by direct compression. The history and types defined by the shape of the arch are discussed in Ref. *3*.

The concrete arch bridge is generally of two types—the open or spandrel arch and the filled arch. The spandrel arch (see Fig. 8) consists of the deck girder system supported by columns or bents which rest on the arch proper. The filled arch has fill material contained by walls resting on the arch.

A third and older type, which resembles a truss, is the through tied arch. The main supporting member is the arch with hangers supporting a floor system and deck.

In all cases, the arch proper can be thought of as a long curved column.

1.2.6 Truss

A rare type of bridge is the reinforced concrete truss. A truss bridge is one in which the main supporting members are made up of a series of triangles the sides of which act in tension or compression.

1.2.7 Frame

A rigid frame reinforced concrete bridge is one in which the piers or abutments are cast monolithically with the main supporting member, either girders or slab, so that the abutment can assist in carrying the main supporting member loads. These rigid frame bridges can be single span or multispan as in a CBC. The bridge presents a pleasing esthetic shape primarily because of the relatively long span with a shallow depth.

1.2.8 Other

Any bridge in which the main supporting member is reinforced or prestressed concrete is classified as a concrete bridge. This classification includes cable-stayed bridges with concrete boxes as well as a series of concrete pipes, as long as the total length meets the requirements described in the NBIS (*1*).

The bridges previously discussed are those in which the main load-carrying members are reinforced or prestressed concrete. However, many bridges, even those that are classified as steel or timber, have major elements that are made of concrete. These components must be inspected with the particular properties of reinforced or prestressed concrete in mind. Several of the major concrete bridge components are discussed next.

1.3 BRIDGE COMPONENTS

All bridges can be considered as made up of various components. Many times a bridge that is considered to be a nonconcrete bridge will have numerous components that are made up of reinforced or prestressed concrete. For instance, a typical steel beam or girder bridge, which would be classified on the standard inventory and appraisal form as a steel bridge, would most likely have reinforced concrete abutments and piers as well as a reinforced concrete deck. These components, although on a nonconcrete bridge, would be evaluated as if they were part of a reinforced or prestressed concrete bridge. The following discussion concerns inspection of these components if made of concrete.

1.3.1 Concrete Decks

The deck is the load-carrying part of the superstructure that has direct contact with the wheel loads on a typical highway bridge. The most common construction material for decks is reinforced concrete. These decks are usually cast in place. Some concrete decks are precast, prestressed units if the designer wanted to take advantage of the compressive strength of the concrete or minimize cracking of the deck. The precast units are becoming popular as replacement decks where maintaining of traffic during replacement is a concern.

Concrete decks on girder bridges normally have the primary reinforcement in the transverse direction or perpendicular to the girders. The top layers of steel, in particular, can corrode as a result of deicing salts and cause spalling or delamination of the deck. Deicing salts can also cause deterioration of the concrete girders and the substructure. Such action will eventually cause carrying capacity problems and, therefore, is of concern to the inspector.

1.3.2 Abutments

Abutments are the part of the substructure that form the terminal ends of the bridge and support the end spans. Typical types of abutments are full heights, stub or semistub, as discussed in Ref. *3.* The abutment is normally composed of a footing, a breast wall, a bridge seat, a backwall, and wing walls. The most common construction material for abutments is reinforced concrete. Some special cases call for precast units or prestressed units, but the great majority are cast-in-place reinforced concrete. Because abutments are supports for end spans of bridges and must also retain the soil on the approaches, the inspection of the abutment is important for the inspector. The primary items of concern to the inspector are crack patterns that may exist and any movement of the abutment they may indicate. A reinforced concrete abutment supporting a steel bridge is shown in Figure 9.

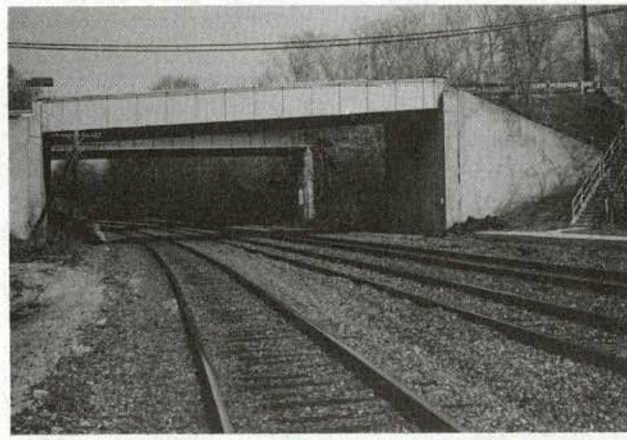


Figure 9. Concrete abutment supporting steel bridge.

1.3.3 Piers

Concrete piers are the substructure element between the abutments and are usually made up of footings, columns, and caps. The footings may be spread, pile, or drilled shafts. Each of these components of a pier is frequently constructed of reinforced concrete with precast or prestressed units used occasionally. Crack patterns that may exist or indications of movement are usually the primary concern of the bridge inspector. Another common name for a small pier consisting of a cap or two or more columns or piles is a bent. A reinforced concrete bent consisting of columns and a cap is shown in Figure 10.

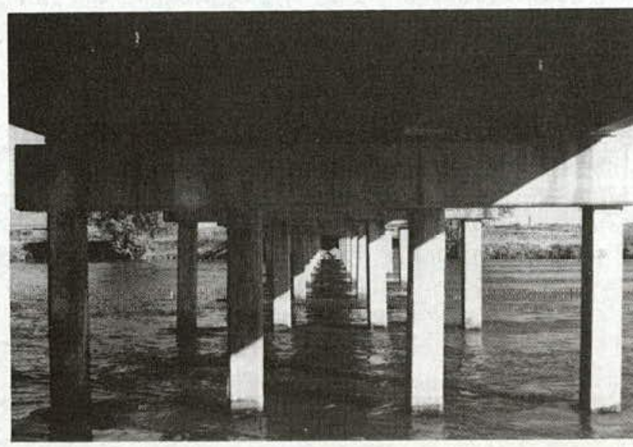


Figure 10. Reinforced concrete bent.

1.3.4 Miscellaneous

Other components of the bridge, such as bracing, diaphragms, railings, sidewalks, sidewalls, parapets and curbs, are often made of concrete even though they are not necessarily on a concrete bridge. These elements may be reinforced concrete or prestressed concrete and may be cast-in-place or precast units, depending on the requirements of the element and desires of the design engineer. To properly inspect these elements, the inspector must be aware of the types of forces and environmental factors that each of these elements must withstand.

1.4 PHYSICAL PROPERTIES OF CONCRETE AND STEEL

An inspection of a bridge requires some knowledge of the physical properties of the material of which it is made. Without some knowledge of the physical properties of the material the inspector would not be able to make a reasonable evaluation of the situation. Simple details, such as whether or not a material is strong in tension and compression, are important. In this section the general physical properties of the material that make up a reinforced or prestressed concrete bridge or component are discussed.

1.4.1 Concrete

Concrete is a man-made stone-like material made by proportioning a mixture of portland cement, sand and gravel, and water and allowing the mixture to harden. Concrete, when properly made and cured, provides a durable engineering material that has a multitude of uses. Concrete is the material of choice for a multitude of bridge components including, in many cases, the main supporting members of the superstructure.

The design and mixing of concrete will not be discussed here. Many guidelines for proportioning concrete can be found in publications of the Portland Cement Association, American Concrete Institute, and AASHTO (*7, 8, 9*).

The key engineering properties of concrete from a strength standpoint are its relatively high strength in compression and its lack of appreciable strength in tension. This important fact should always be remembered by bridge inspectors, in other words, concrete is strong in compression and weak in tension. Designers of concrete components of bridges are well aware of these properties and reinforce locations where tension is expected to occur with steel. Steel, as one may be aware, is strong in both tension and compression; however, in compression if it has no lateral support it tends to buckle or bow out of shape, thereby losing carrying capacity. The combination of concrete and steel is known as reinforced concrete and has wide applications in bridges and bridge components.

Concrete is a special material because it can be manufactured to a wide variation of strength and this strength varies with time. Figure 11 shows typical stress versus strain values that may be found in bridge structures using concrete strength at a standard 28-day test. Figure 12 shows the effect of age on compression strength. Normal design strengths for reinforced concrete construction are in the neighborhood of 3,000 psi to 4,000 psi, whereas other applications such as precast, prestressed-type construction use design strengths of more than 6,000 psi. What strength one finds in an existing structure can be higher than the design strength because of both the time effect and the fact that most design values are minimums and therefore exceeded in actual construction. However, in some cases concrete can deteriorate because of salt penetration or freeze-thaw action and the actual strength can be lower than the design strength.

Another notable property of concrete is its ability to creep. Creep is a nonelastic, time-dependent deformation created by a continuous, long term stress, that is, deformation or strain over a period of time without a change in stress. Most of this creep occurs within 2 years of construction. Evidence of creep in a structure may be observed as a sag or other deformation.

1.4.2 Steel

Steel, an iron with carbon chemically dissolved in it such that no carbon exists in the undissolved state, is strong both in tension and compression, as noted earlier. Reinforcing steel will have a specified minimum yield strength for design of somewhere around 40,000 psi to 60,000 psi and a tensile strength of around 80,000 psi. Actual in-place reinforcing steel normally will have a higher yield strength than the specified minimum for design. The yield strength is the stress at which the steel begins to strain or stretch without any gain in strength. Tensile strength is the maximum stress which can be obtained before fracture. An important property, which bears mentioning, is the ductility or ability to stretch without breaking beyond the stress required to cause the steel to yield. Most reinforcing steels will stretch

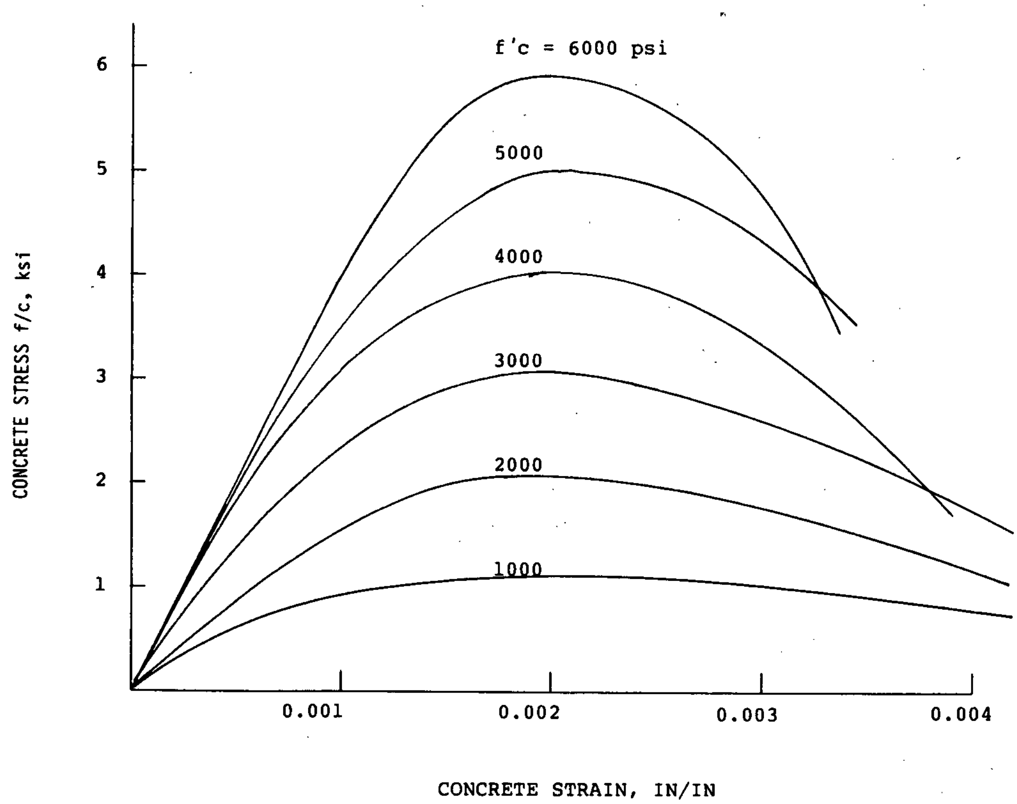


Figure 11. Stress versus strain for concrete.

well over 20 percent, some over 40 percent, before fracture. Contrasted with this amount, concrete will lose all of its strength at a compression deformation of about 0.3 percent and less in tension. A typical stress-strain curve for regular reinforcing steel is shown in Figure 13.

Steel used to prestress concrete has a much higher yield strength than regular reinforcing steel. The yield strength is in the neighborhood of 200,000 psi. The prestressing steel also has lower ductility than typical reinforcing steel. In addition, the ratio between the yield strength and the tensile strength for prestressing steel is much lower than that of common reinforcing steel. This lower ratio indicates that the yield strength can be quite close to the tensile strength in prestressing steel.

The high yield of the prestressing steel is necessary for the prestress to absorb losses due to, among other factors, elastic shortening and creep of the concrete and still have prestress remaining to create the proper internal stresses.

1.4.3 Summary

The physical properties of concrete and steel play an important role in how the materials are used in typical bridges. Concrete is weak in tension, but strong in compression. Steel is strong in tension and is used to carry the forces in locations where tension appears in reinforced (or prestressed) concrete structures. Steel also has the ability to stretch appreciably before fracture.

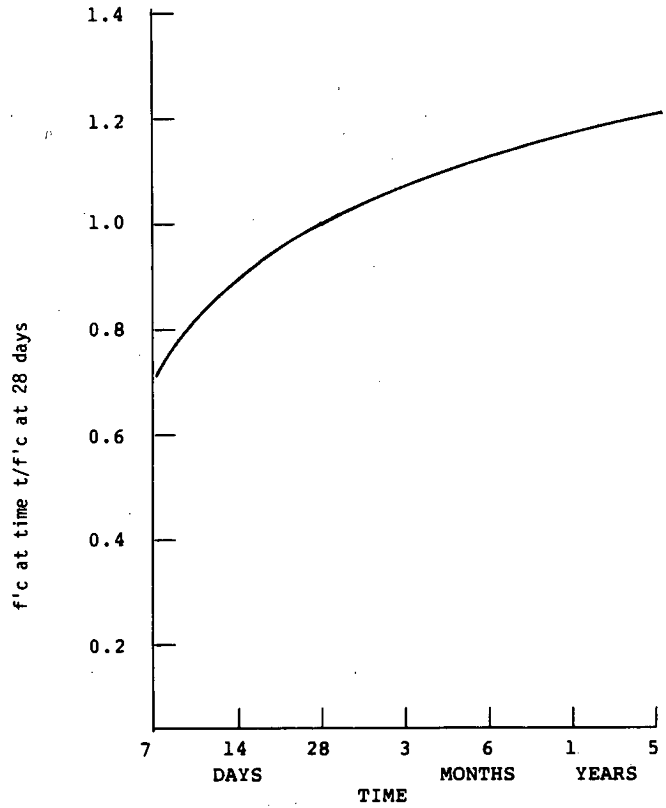


Figure 12. Effect of age on compressive strength.

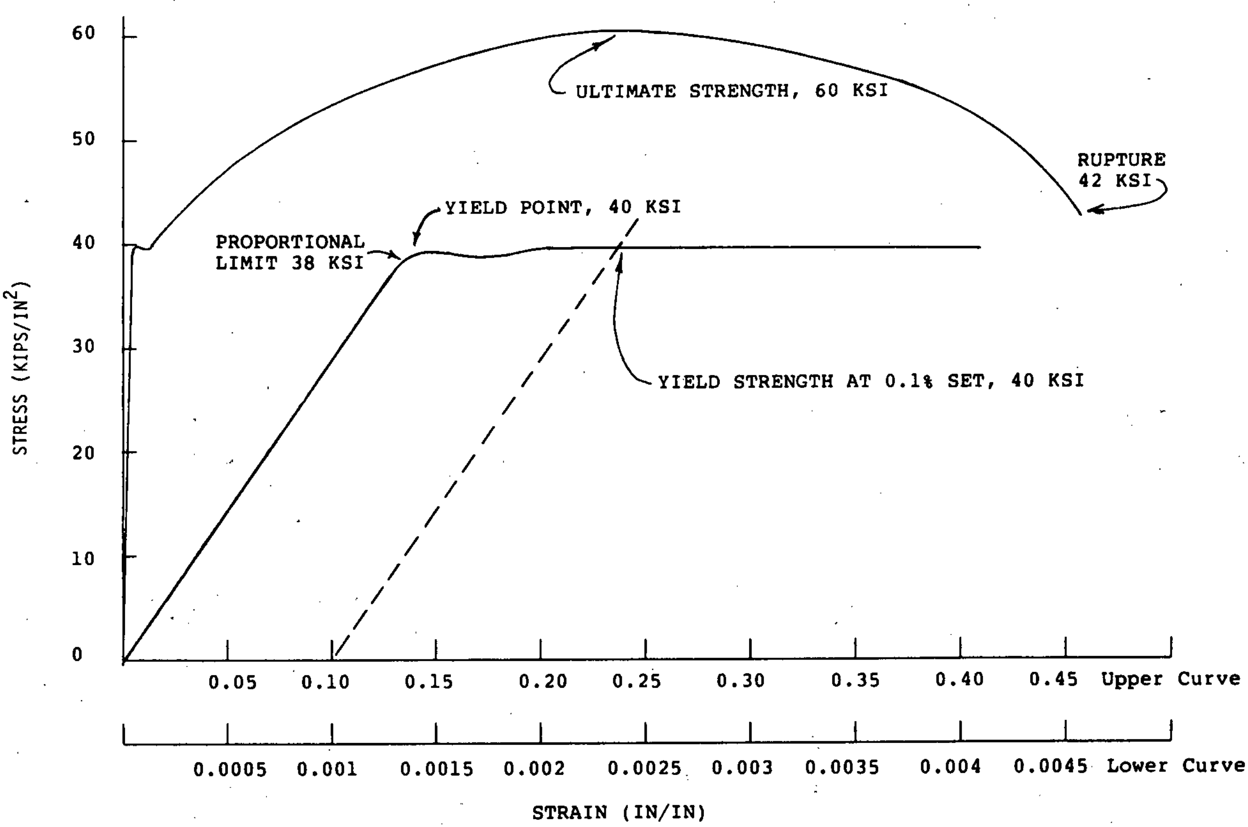


Figure 13. Typical stress versus strain for steel.

1.5 BRIDGE MECHANICS

The mechanics of a bridge structure can be defined as that physical science dealing with solid bodies at rest or forces in equilibrium. In particular, the bridge inspector is concerned with the manner in which forces acting on a bridge relate to the internal forces and stresses occurring in the structure. Both live load and dead load forces are described in the FHWA *Bridge Inspector's Training Manual 70* (*3*).

Although the mechanics of a bridge structure can be complex and detailed, the technician needs to be concerned with only a few of the basic mechanics of bridge structures. First the concept of compression and tension should be understood.

1.5.1 Stress

Compression is the concept of an object being forced to be shorter, whereas tension is the concept of an object being forced to be made longer. Hence, one can describe a force as being compressive, that is, the force that makes the object shorter; or tensile, that is, the force that tends to make the object longer.

In dealing with force per unit area, one deals with the concept of stress. Typical units are pounds per square inch (psi). A compressive stress makes that part of the object become shorter; conversely, a tension stress makes that part of an object become longer. This stretching and shortening can in many cases be observed in a bridge structure by deformation or movement.

Typically, if the part of the member is getting longer due to force, a tensile stress is involved; and if it is getting shorter, a compressive stress is involved. Keep in mind that sometimes the force or stress creates the deformation and sometimes the deformation creates the stress.

1.5.2 Bending

Typically with a bridge, the vehicle applies a transverse load to the girder, beam, or main supporting member. In this discussion, let one consider a simple beam-type bridge. The exact amount of the transverse load from the vehicle that gets to a particular beam is not important for this example, only that some or all of it gets to the beam. Transverse means that the load is applied perpendicular to the beam axis. A truck load or dead load would normally create a force in a downward direction. This downward force bends the beam downward. Close observation indicates that the top part of the beam is becoming shorter, and the bottom part longer, as the beam bends. This deformation makes compression stress on the top of this simple beam and tension stress along the bottom. The exact distribution of the stress from the top of the beam to the bottom is beyond the scope of this discussion; however, it can be found in any strength of materials textbook (a brief treatment is given in the *Bridge Inspector's Training Manual 70* (*3*)). This bending or deformation caused by a transverse force is called flexure and the stresses are known as flexure stresses. These flexure stresses can be multiplied by the distance to a location within the beam cross section and then summed over the cross-section area creating what is known as bending moment. This bending moment or flexural moment is internal to the beam.

Bending moment can be defined as the sum of the moments on a free body to the left, or right, of an imaginary cut section in the beam. Normally, it will vary along the length of the beam with the magnitude of bending moment directly related to the curvature. In the case of the simple, two support beam, the largest magnitude of bending moment due to dead load or a vehicle will occur near the middle of the span.

If one lengthens the beam and places a support in the middle, in addition to the supports at the ends, the result is what is known as a continuous beam rather than a simple beam. In particular, this beam would be described as a two-span continuous beam. One can expand this concept with more supports and get a three, or more, span continuous beam. Now consider the deformations of this continuous beam.

Imagine the deformations of a wet noodle over several supports. The noodle deforms downward between the supports because of the dead load—that is, the weight of the noodle itself—but can be observed to curve in an opposite direction over the supports. In particular, it deforms with an upward curvature between the supports, but curves downward across the support. This same type of curvature or deformation occurs, although with small magnitudes, with a continuous beam.

A two-span continuous beam will curve upward from the end supports, although deflecting downward, and curve downward near and over the middle support. Observing this deformation one can determine that compression occurs on the top near the ends of the beam and through the center of the span, but switches to tension on the top near and at the center support. With more than two spans, the same concept applies—compression stress, on the top near the middle of the spans and at the ends of the beam; and tension stress, on the top near the interior supports.

In summary, compression stress normally occurs on the top near the center of a span, but occurs on the bottom near and at an interior support. Tension occurs on the bottom near the middle of the span and on the top at and near the interior support of continuous members.

These regions of a beam of compression stress on the top and tension stress on the bottom are sometimes referred to as positive moment regions. If the tension stress is on the top and compression stress on the bottom, it can be referred to as the negative moment region. Therefore, positive moment occurs near the middle of a span and negative moment occurs at and near an interior support of continuous members.

These flexure stresses are important to the bridge inspector because concrete is weak in tension and strong in compression. In regions of tension, the designer will have placed steel to carry the tension. In order to carry the tension stress, the deformation of the steel will exceed that which the concrete can take in tension; hence, the inspector can expect flexure cracks in the tension regions—near the bottom along the middle of the span and on the top near interior supports. If the cracks are growing or moving, they should be of concern; however, normally they are just noted in the report.

1.5.3 Shear

Another type of stress that needs discussion is the shear stress. Shear forces and shear stress are covered in the *Bridge Inspector's Training Manual 70* (*3*); however, the subject bears further discussion with particular regard to concrete-type structures.

Shear stress occurs when normal or flexure stresses change in magnitude. Beams in flexure exhibit this phenomenon if there is a change in bending moment, that is, a difference in the magnitude of bending moment along the axis of the beam.

Shear stress is normally directly related to shear force. Shear force can be defined as the sum of the vertical forces to the left, or right, of an imaginary cut section of the beam. Using this definition one can determine that higher shear forces, hence shear stresses, occur near the supports of a ridge or beam. These locations are also the locations of the most rapidly changing bending moment. Shear forces are generally small near the middle of a span in both simple and continuous beams.

The interesting fact concerning shear stress is that it can be shown through stress analysis that a shear stress is numerically equal to a tension stress acting at an angle 45 deg to the plane of the shear stress. What does this tensile stress mean to the inspector? Concrete is weak in tension, shear forces are directly related to shear stress, and shear stress is directly related to tension stress. Therefore, at locations of large shear forces, such as near the supports, tension stress occurs. Normally this tension stress is in a direction about 45-deg from the axis of the beam. Because of the occurrence of tensile stress, the designer probably used steel to reinforce the member at these locations and, thus, the inspector is likely to find cracks at these locations.

In particular, the inspector is likely to find flexure cracks near the middle of the spans and flexure cracks over the supports. Shear stress cracks, sometimes referred to as diagonal tension cracks, would appear near the supports at an angle. A shear crack is shown in Figure 14. The locations of these potential cracks are shown in Figure 15.

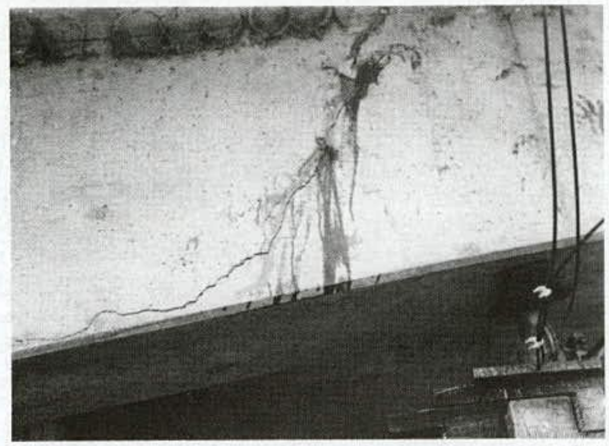


Figure 14. Shear crack in bridge; girder support is to left. (Photo Courtesy of Finley and Turnipseed)

1.5.4 Compression

An axially loaded beam is typically called a column. In most cases a column will be predominantly compression. Concrete columns in a bridge structure normally constitute piers or bents. An arch is a compression member and, hence, is a form of a column. Because concrete is strong in compression and columns are normally loaded in compression, no cracks should be expected due to stress. Most duress observed by an inspector in compression members is a result of deterioration. Earthquake or traffic damage or large settlement movements creating unplanned stresses are notable and obvious exceptions.

Most existing columns are designed so the long column effects, including buckling, are already considered.

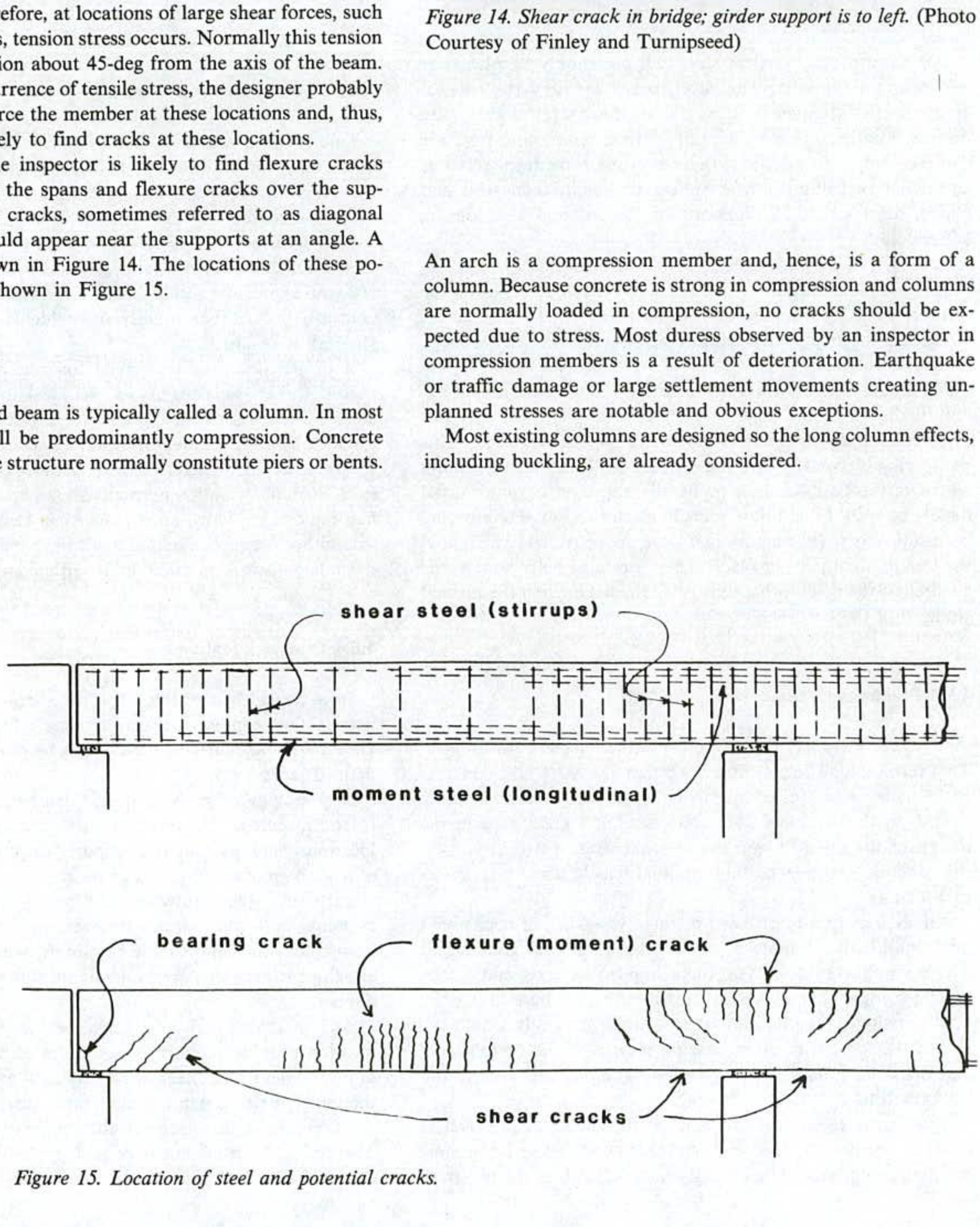


Figure 15. Location of steel and potential cracks.

1.5.5 Reinforced Concrete

In general, a designer will place some type of reinforcing steel wherever he determines the chance of a tension stress occurring in the structure. In addition, he may place steel in areas of large compressive stress to reinforce the concrete if the stresses are higher than the concrete can safely tolerate. Sometimes steel is placed in a reinforced concrete structure in the compressive stress locations, simply to reduce long time deflection due to creep of the concrete.

For flexure or bending, longitudinal steel will be placed near the surface along the top or bottom of a beam. This steel will be in all areas where tension is expected—near the center of a span on the bottom and on the top over interior supports. It is also common to place some longitudinal steel in compression zones to control long time deflections and unexpected tension forces.

For shear stress, vertical steel will commonly be placed at and near all supports. This steel serves to carry the vertical component of the tensile stress due to shear stresses that were discussed earlier. These places of vertical reinforcing steel are called stirrups. In addition, these stirrups sometimes serve as additional containment near splices in longitudinal steel and almost always serve as support for longitudinal steel during construction.

A sketch of a typical, reinforced concrete beam is shown in Figure 15, indicating the types and locations of steel within the member as well as the locations of potential cracks. The actual sizing and spacing of such reinforcing steel are beyond the scope of this discussion.

Although concrete is basically strong in compression, an axially loaded member, such as a column or pier, normally has some reinforcing to enhance the strength of the concrete. This reinforcing is typically near the surface on all sides of the column which is the best location to handle expected or unexpected flexure stresses. In addition, some loops or ties, small reinforcing perpendicular to the longitudinal steel, are provided to help hold the longitudinal steel in place. These ties also help contain the concrete if the column happens to be loaded beyond the normal strength of the concrete.

1.5.6 Prestressed Concrete

Concrete members of a structure sometimes are prestressed. This terminology simply indicates that the steel and concrete in the member is stressed internally before the loads are actually placed on the structure. The designer takes great care in the design of the member and the internal stress patterns so that the concrete experiences only minimal tensile stress under normal loading.

An analogy of a prestressed member is a stack of books held horizontal with a compressive force applied at each end and no support in the middle. The force applied at each end is the prestress force that makes compression everywhere along the books, including the bottom where tension normally would occur. If the force is released, tension returns to the bottom part and there being nothing to carry tension along the bottom, the books will fall.

Two terms commonly heard with the subject of prestressed concrete are pretensioned and post-tensioned. These terms simply indicate at what time during construction that the prestress was actually applied to the structure. A pretensioned concrete beam has had stress applied to the prestress steel before the concrete was cast around the steel. In contrast, a post-tensioned concrete beam is typically cast with ducts or conduits left in the concrete for the addition of the prestress steel which is stressed after the concrete is hardened. From an inspector's standpoint, the terms indicate whether or not anchor plates, required for post-tensioned concrete, and the resulting potential problem of concrete failure at the anchor exist.

The locked-in internal stresses created by prestressing the member sometimes make a camber or upward deflection on the member. Because of creep, this camber can increase with time.

The inspector can determine if a member is prestressed by the general appearance, particularly the lack of tensile cracks at the locations described earlier in the reinforced concrete section.

1.5.7 Ductility Beyond Cracking

One of the essential features of proper design is the obtaining of ductility or slowness of failure beyond initial cracking or failure. This feature is designed into virtually all structures, including concrete bridge structures.

As noted, reinforcing steel and also, to a lesser extent, prestress steel will stretch a large amount between the yield stress (or strain) and the actual fracture of the material. Concrete, in contrast, fails rather rapidly or suddenly after its maximum strength is reached.

A proper design of reinforced concrete has the tension force exceed the elastic capacity of the steel to carry it before the concrete reaches its maximum carrying stress. This feature allows the bending moment to reach a known value and then hold the value as the member deforms because of the ductility of the steel. Such deformation normally shows up as cracks larger than hairline cracks. Thus, large cracks can be an indicator of overload either because of actual loads or, possibly, because of such secondary factors as differential settlement.

1.5.8 Critical Locations

In a typical bridge the inspector should look for duress or defects that can have a catastrophic effect on the structure. Defects or indicators of defects can be more critical in certain defined locations.

Any moving or growing cracks can be considered important to the inspector. However, if these cracks are at high stress locations, such as near the support (shear) or near the middle of a span (moment), they can be critical.

Parts of a structure where reinforcing steel terminates, such as joints or beam column connections, can be considered as critical locations and should be closely inspected. Bearing plate attachment locations are probably locations for cracks or other duress.

Any member, such as a cantilever or corbel, has a critical location near its support. These types of members have maximum moment and shear occurring at the same location. Furthermore, if the maximum moment is reached, that is the steel yields, there is no place for the moment to be redistributed. Therefore, the attachment end of a cantilever or corbel is a critical location.

In prestressed members that are post-tensioned, a critical area to look for duress is at the termination points of the prestressing steel. Cracks here, because of overstress near an anchor can be serious.

Settlement or movement needs to be evaluated as to cause, rate of movement, and potential effect on the structure. Sometimes watching is sufficient, but other times resetting of bearings, stabilization of soil or other corrective action is necessary.

1.5.9 Summary

It is hoped that this brief discussion of mechanics pertaining to bridges will help the inspector understand what he observes in his job. It is most important in reinforced and prestressed concrete structures to know where to expect cracks and when cracks occur that are not expected. Cracks are the most important indicator of duress in concrete; however, cracks do not necessarily mean duress. Knowing a little about bridge mechanics will help the inspector recognize this important difference.

1.6 INDICATORS OF DETERIORATION IN REINFORCED CONCRETE

There are a considerable number of forms in which deterioration of concrete can occur. Photographs of defects and deterioration are contained in the FHWA Manual-70 (*3*). The most common forms of deterioration are described in the following.

1.6.1 Cracking

A crack is usually defined as an incomplete separation of the concrete into two or more parts, with or without a space between them. Different types of cracking can occur in a concrete structure. The significance of cracks depends on the structure type, crack origin, and whether the width and length increase with time. Cracks are common in concrete because of its low tensile strength and relatively large volume changes that result from changes in humidity and temperature. The significance of cracks is dependent on their origin and whether the length and width are increasing with time.

Cracks can be categorized into several types including: (1) plastic shrinkage, (2) drying shrinkage, (3) settlement cracks, (4) structural cracks, (5) reactive aggregates, (6) corrosion of reinforcement, and (7) map freeze-thaw.

Plastic shrinkage cracks are caused by rapid drying of the concrete in its plastic state. The cracks are usually wide and shallow, are spaced at regular intervals, and may form a definite pattern. These shrinkage cracks are worth noting, but have little direct effect on the condition evaluation.

Drying shrinkage cracks result during drying of restrained concrete after it has hardened. They are usually finer and deeper than plastic shrinkage cracks and have a random orientation. These shrinkage cracks are worth noting in a report, but have little effect on the capacity analysis.

Settlement cracks may be of any orientation and width, ranging from hairline cracks above the reinforcement that result from subsidence of high slump concrete or from settlement of the formwork to wide cracks in supporting members caused by settlement of the foundation. The slump-caused cracks should be noted in the report because they can contribute to deterioration. The settlement cracks due to movement definitely should be recorded and the cause established. Such cracks can be critical and affect the carrying capacity of the bridge.

Structural cracks may occur from differences between assumed and actual stress intensity including fine cracks controlled by provision of reinforcement. The width of the cracks varies, but the orientation is normally well defined. Common noncritical examples are longitudinal cracks over internal voids in some slab decks and diagonal cracks in the acute corners of skewed decks. The cracks should be evaluated based on their location, size, and apparent cause.

Corrosion-induced cracks (resulting from the corrosion of steel reinforcement) are usually associated with shallow cover and are located directly above (or below) the reinforcement. The width of these cracks increases with time as the corrosion continues. The crack normally terminates at the reinforcement. Rust stains may be visible. Such cracks serve as an indicator of loss of bond, possibly anchorage, and loss of section. Corrosion-induced cracks can indicate a loss of capacity.

Map cracking is usually caused by chemical reactions between the mineral aggregates and the cement paste. The number and the width of the cracks usually increase with time as the concrete is subjected to moisture and freeze-thaw action. Various reactions are possible, although the most common reaction is between the alkalies from the cement, or from external sources, and constituents of some aggregates that produce alkali-silica or alkali-carbonate reactions. Both types of reaction result in serious damage to the concrete by causing abnormal expansion, cracking, and loss of strength. Note these cracks in the report and, if apparently serious, get expert help in the evaluation.

Freeze-thaw cracks are closely spaced cracks parallel to the concrete surface and, therefore, only visible in cores and are usually associated with scaling.

Structural cracks, corrosion-induced cracks, and map cracking if chemical reaction is severe, are considered the most significant in reinforced concrete bridges.

1.6.2 Scaling

Scaling is the flaking away of the surface mortar of concrete. As the process continues, the coarse aggregate particles are exposed and eventually become loose and are dislodged. Young concrete is particularly susceptible to scaling; however, weak surface layers resulting from improper finishing practices or concrete lacking adequate air entrainment may flake away to limited depth. Scaling is primarily caused by repeated freeze-thaw action on the concrete. Very fine, shallow surface cracking is usually evident. Gutter areas of bridge decks where water may stand or faces of curbs and barrier walls are particularly susceptible to scaling.

1.6.3 Spalling

Spalling of concrete is generally recognized to be a serious defect. Spalling can cause local weakening, expose reinforcement, impair riding quality of a deck, and grow to such extent as to cause structural failure. The spall is a depression caused by a separation, and removal, of the surface concrete. Spalling is related to the age of a structure because the major causes of spalling are corrosion of the reinforcing steel, overstress, and ice pressure. The amount of spalling increases with the age of a structure. Spalls are usually quite noticeable; however, in its early stages the spall may be a delamination in the concrete and not visible (see Sec. 1.6.4). Detection at this stage is accomplished by soundings or other inspection methods. Cracks associated with the spalling are usually wide, long, and deep enough to reach the reinforcing or prestressing steel.

1.6.4 Delamination

Delaminations are separations along a plane parallel to the surface of the concrete. Delaminations occur as the reinforcing steel corrodes and expands. As the corrosion process continues, increasing pressure is exerted on the concrete and eventually the delamination becomes detached from the main body of concrete resulting in a spall. Cracks may, or may not, be present depending on the degree of corrosion and amount of cover over the reinforcement. Bridge decks and corners of concrete beams, caps, and columns are particularly susceptible to delamination. Delaminations normally are indications of steel corrosion and ultimate spalling of the concrete.

1.6.5 Leaching

Leaching is the accumulation of salt or lime deposits, usually white in color, on the concrete surface. Water may carry lime from the cement to the concrete surface where the water evaporates leaving the white deposit on the concrete surface. In cracked areas, deicing salt can be carried through the cracks where it is left on the surface. Most common locations where leaching or efflorescence occurs are the underside of concrete decks and along cracks on vertical faces of abutment backwalls and wingwalls.

1.6.6 Stains

Many types of stains may be found on concrete; however, few have any significance. Oil, gasoline, and asphalt staining are common but seldom cause damage to the concrete. The most significant stain concerns rust stains that indicate corrosion of the reinforcing steel.

1.6.7 Hollow or "Dead" Sounds

Dragging a chain over the concrete or sounding the concrete by tapping on it with a hammer or rod will produce a hollow sound if the concrete is delaminated. If tapping with a hammer or rod produces a "dead" sound, it is usually indicative of low quality concrete. The concrete may have been fire damaged, may have suffered severe chemical damage, or may have been frozen during the curing period. Concrete containing too little cement, reactive materials, or excessive air entrainment may also produce a "dead" sound during the sounding test.

1.6.8 Deformations

Swelling or expansion of concrete is usually an indication of reactive materials. Localized swelling may be caused by compressive failure of the concrete. Twisting of substructure or superstructure units may be evidence of a settlement or foundation problem.

1.6.9 Patching or Other Repairs

Patching or other repairs are indicative of problems in concrete. The condition of the repair or patch will usually indicate whether the underlying problem has been solved or if it is a continuing and active problem. Cracking, delamination, rust stains, or spalling in or around the repair or patch can indicate that the problem still exists and further investigation and repairs are needed.

1.6.10 Ride Quality

The inspector must pay close attention to the riding quality of a bridge. Few, if any, structures are constructed with significant bumps or angular changes in the deck. Poor riding joints are not uncommon; however, the inspector must determine if the joint is functioning properly or has been damaged. Significant changes in the grade can result from settlement of foundation problems. Abutments on fills often rotate or settle causing pronounced changes in the riding quality of the bridge and may cause overstressing and cracking of the structure. Uneven settlement of substructure units can cause tilting or twisting of the bridge deck.

1.7 INDICATORS OF DETERIORATION IN PRESTRESSED CONCRETE

The indicators of deterioration in prestressed concrete are similar to those in reinforced concrete. The indicators common to both will not be repeated here. The following discussion centers on those indicators where their presence is associated with conditions unlike those for reinforced concrete.

1.7.1 Cracking

Cracking in prestressed concrete is an indication of a potentially serious problem. As prestressed concrete is under high compression, no cracks should be visible. Horizontal cracks near the ends of prestressed members may indicate a deficiency of reinforcing steel. These cracks are due to bursting stresses created at the transfer of tension stress. Vertical cracking, in the lower portion of the member, which is not near a support can indicate a very serious problem. These vertical cracks can indicate serious overstressing or loss of prestress. Vertical cracking in the bottom of the unit at the support may be a result of restricted movement in bearing assemblies or application of prestress force with beam end restrained during casting. Quite often, vertical cracks occur above the neutral axis of a precast-prestress member as a result of mishandling during transportation or erection; however, these cracks normally close when dead load of the deck is applied and are not easily detected. Various types of cracking that occur as a result of impact will vary according to the direction, location, and magnitude of the impact.

1.7.2 Leaching

Leaching in prestressed bridges is normally associated with box-girder-type construction where the deck is a part of the unit. The sources and types are the same as common with reinforced concrete.

1.7.3 Stains

The types and occurrences of staining in prestressed concrete are essentially the same as with reinforced concrete; however, rust stains may indicate corrosion of the prestressing steel, which is serious and could seriously affect the structural integrity of the unit.

1.7.4 Hollow or "Dead" Sounds

Hollow or "dead" sounds produced when testing prestressed concrete indicate essentially the same problems as they do in reinforced concrete. The "dead" sound indicates that the concrete may have been fire damaged, may have suffered severe chemical damage, may contain insufficient cement or reactive material, may have frozen during the curing period or may contain excessive air entrainment. Hollow sounds produced during sounding tests normally indicate delaminated concrete.

1.7.5 Deformations or Distortions

Deformation or distortion of prestressed concrete can occur if arrangement of prestress steel is improper or in the event some steel has been cut or damaged causing an imbalance of prestress force. A combination of prestress force and creep in the concrete can cause excessive camber or sweep. Deformation or crushing of the concrete around anchorages can occur.

1.7.6 Scaling

Scaling of prestressed concrete is the same process and occurs in the same manner as in reinforced concrete.

1.7.7 Spalling

Spalling of prestressed concrete can indicate a serious problem. Spalling can cause local weakening, expose reinforcing and prestressing steel, impair riding quality, and result in loss of prestress. The location and extent of the spalling are significant. The causes of spalling of prestressed concrete are, for the most part, the same as for reinforced concrete. Occasionally, the prestress force, combined with other force from corrosion or ice pressure, may cause more rapid or extensive spalling than would occur in nonprestressed concrete.

1.7.8 Delamination

Delaminations in prestressed concrete occur as the prestressing or reinforcing steel corrodes and expands. Pressure exerted within concrete by the products of corrosion causes separations to occur along a plane parallel to the concrete surface. The most common areas of delamination are deck sections, corners, and areas where steel is close to the concrete surface.

1.7.9 Patches and Other Repairs

Prestressed concrete structures can seldom, if ever, be completely repaired. Broken tendons cannot be replaced and lost prestress force can only be restored by external post-tensioning, which in most cases is difficult. Effective repairs and patching are usually limited to protection of exposed tendon and reinforcement. The inspector should be very skeptical of patches and repair work performed on prestressed concrete.

1.7.10 Ride Quality

Section 1.6.10 also applies to prestressed components, and the reader is referred to that discussion.

1.8 FACTORS CAUSING DETERIORATION IN REINFORCED CONCRETE

The factors that can cause deterioration in reinforced concrete are many and varied. They include poor design details, construction or maintenance deficiencies, reactive materials, environmental conditions, corrosion of reinforcing (or prestress) steel, wear, impact and overstress caused by overloading or foundation movement. In order to properly evaluate the severity of a deterioration problem, its probable cause should be determined. To do this, the bridge inspector should be familiar with both the factors causing deterioration and their symptoms.

1.8.1 Poor Design Details

Some design details that can cause concrete to deteriorate are insufficient expansion space or insufficient cover over reinforcing steel. Also the problem of drainage many times does not receive sufficient attention during design.

Insufficient expansion space provided between ends of slabs at expansion joints will cause spalling or compressive fracturing. Insufficient cover over reinforcement may cause corrosion of the reinforcing steel which, in turn, causes spalling or delamination of the concrete.

Lack of, or improperly placed, drains may cause ponding on the concrete surface that may result in scaling of concrete or early corrosion of reinforcement. Downspouts for drains or scuppers, if not provided or of inadequate length, can result in water being discharged directly on concrete surfaces below.

1.8.2 Construction Deficiencies

The construction deficiencies and construction operations that can result in deterioration of concrete are numerous. Some of the most common are noted in the following paragraphs.

1.8.2.1 Improper Concrete Mix. Addition of excessive water to concrete during mixing or applying water to the concrete during finishing operations lowers the quality of the concrete making it susceptible to damage from freeze-thaw and traffic.

1.8.2.2 Overworking of Concrete. Overworking of fresh concrete during finishing and the introduction of insufficient or excessive amounts of air entrainment admixture during mixing also lower the durability of the concrete and cause scaling.

1.8.2.3 Poor Curing Practice. Poor curing practices that allow rapid loss of moisture from the concrete will result in shrinkage cracking and impair the strength and durability of the surface concrete.

1.8.2.4 Improper Slump. Excessive concrete slump can result in excessive subsidence of the concrete that can cause surface cracking over the reinforcing bars.

1.8.2.5 Inadequate Support of Steel. Inadequate support of reinforcing mats can result in movement of the reinforcement as concrete is in the initial set stage, causing lack of bond or cracking of concrete above the reinforcing bars.

1.8.2.6 Improper Placement of Concrete. Allowing concrete to free fall during concrete placement in deep structures causes segregation and honeycombing in the concrete.

1.8.2.7 Improper Placement of Steel. Improper placement of reinforcing bars can result in inadequate cover and subsequent corrosion of reinforcement or serious misplacement can result in the component being below design strength. The weakened component may crack or fail under loading.

1.8.2.8 Lack of Foundation Investigation. Construction of piers or abutments on unsuitable materials may result in settlement which may, in turn, cause overstress in other concrete members. Such overstress may cause cracking, spalling, or both to occur in the concrete members.

1.8.3 Lack of Maintenance

There are many instances where a lack of proper maintenance may cause deterioration of concrete. Two more notable cases are given in the following paragraphs.

1.8.3.1 Bad Housekeeping. Failure to keep dirt and debris removed from concrete surfaces can cause moisture and chemicals to collect and penetrate into the concrete. Freeze-thaw damage or corrosion of reinforcement and subsequent spalling may occur.

1.8.3.2 Lack of Detail Maintenance. Failure to keep expansion joints clean can lead to a pressure buildup and subsequent slab displacement that can break bridge seat bearing areas or girder ends. The failure to provide and maintain adequate approach slab relief joints can have the same effect.

1.8.4 Corrosion

The use of deicer salts has greatly increased the problem of corrosion of reinforcing steel. Even if there are no cracks, water will permeate porous concrete. The water carries the deicing salt into the concrete and ultimately reaches the reinforcing steel. Salt in solution provides an electrolyte, and oxygen in the water provides the oxidizing agent. The resulting environment is ideal for the corrosion of the steel. As the products of corrosion occupy considerably more volume than the parent metal, tensile forces greater than the strength of the concrete are exerted within the concrete. Cracking and spalling result. There can also be a significant loss of section in the reinforcement that may reduce the load capacity of the structure.

1.8.5 Leaching Spalls

Leaching may or may not be indicative of a serious problem. Soft water may leach out the lime in the cement and leave a powdery residue. Leaching on the underside of a concrete deck in areas of map cracking is usually deicing salts and indicates a potential problem. Spalling is usually the sign of a more serious problem. A spall is a depression caused by a separation and removal of the surface concrete. The most common cause of spalling is the combined action of the corrosion of reinforcing steel, ice pressure, and shear stress linked with bending movements.

1.8.6 Overstress

The most common cause of overstress is a load greater than operating capacity. Concrete beams, girders, and decks are all subject to damage under overstress conditions. Overstressing of beams and girders normally causes vertical or diagonal cracking. Wide extensive vertical cracks extending upward from the bottom near the center of simple beams or copious diagonal cracks at the end of simple beams indicate possible overstressing. Beams that are continuous over a support may develop vertical cracks extending downward from the top of the beam over the support. Cracking in concrete decks parallel to and over supporting elements indicates possible overstress.

1.8.7 Foundation Movements

Generally, foundation movements generate sizable tensile stress in substructure units. This tensile stress can cause serious cracking in the concrete structure. Changes in the alignment or grade of the bridge superstructure and width of the joint openings are also symptoms of foundation movement.

1.8.8 Temperature

Freezing and thawing are common causes of concrete deterioration. Porous concrete absorbs water, and when the water freezes high expansive pressures are created because of the larger column created by ice formation. These pressures often produce cracking, spalling, or scaling. Aggregates such as chert, with lower coefficients of expansion than the paste, may also cause high tensile stresses, resulting in cracks and spalls. Concrete expands at an average of 5.5 millionths of an inch per inch for each degree Fahrenheit of temperature rise. Consequently, a 100-ft slab undergoing a 50-deg temperature rise will expand about one-third of an inch. Likewise, the same slab undergoing a 50-deg temperature drop will contract one-third of an inch. If the concrete is prevented from expanding or contracting because of friction or because it is being held in place, the slab will crack under tension.

1.8.9 Chemical Attack

The most common problem today is the application of deicing salts to concrete. These salts including chloride ions ultimately reach the reinforcing bars and cause corrosion of the steel. As the products of corrosion occupy considerably more volume than the parent metal, tremendous pressure is applied to the concrete from within, causing cracking and spalling of the concrete. Ammonium and magnesium ions react with the calcium in cement. Calcium and magnesium sulphates and sodium will react with the tricalcium aluminate in the cement paste. Acids will attack the cement paste by chemically transforming the paste composition. The common symptoms are: scaling, spalling of concrete surface, random cracking, parallel cracking and swelling in compression members, and exposed aggregate.

1.8.10 Traffic Impact

Few elements on bridges escape damage from impact or collision. Trucks, cards, ships, and barges often strike piers, overhead beams, and railings. Damage can range from scarring of the concrete to severe spalling and cracking, or complete destruction of the member. The impact may sever the tension reinforcement or prestressing strands in overhead beams, impairing the load capacity of the unit. Wear or abrasion by traffic can result in scaling in traffic lanes and raveling and cracking at joints. Snow removal equipment and sweepers damage or scar curbs and parapets.

1.8.11 Reactive Materials

Reactive aggregates, high alkali cement, and contaminated mixing water cause serious deterioration of concrete. The symptoms of such conditions are swelling and cracking, generally unsound concrete, and popouts. After a few years of being exposed to the weather, concrete that was made using reactive materials will begin to crack over its entire surface and appear to be expanding. As the deterioration progresses, the concrete will begin to crumble and disintegrate.

1.8.12 Fire

Generally, reinforced concrete bridges perform well from a structural standpoint after exposed to severe fire. Collapse of a concrete structure during or after a fire is rare (*10*).

Much information on high-temperature properties of concrete and reinforcement has become available in the past 25 years. Figure 16 shows the influence of heat on the strength of concrete and steel (*11*). The carbonate aggregate concrete retained about 80 percent of its original strength at temperatures to 1200°F. This property of concrete contributes to the excellent performance of this material in fires.

The yield strength of grade 40 steel begins an appreciable loss around 800°F and is reduced to about 40 percent of its original strength at 1200°F. However, as the steel cools, the steel will return to its original strength. Also, concrete acts as an insulation of heat and in many cases it can protect the reinforcing steel, depending on the intensity and duration of the fire. However, if the concrete structure is spalled and the reinforcing steel is exposed, there will be no protection.

Some of the problems that must be taken into account include the length of time the structure was exposed to the fire, how fast was the structure cooled down, and what changes in the geometry of the structure have occurred. A common occurrence is that a concrete structure is exposed to the fire for only a

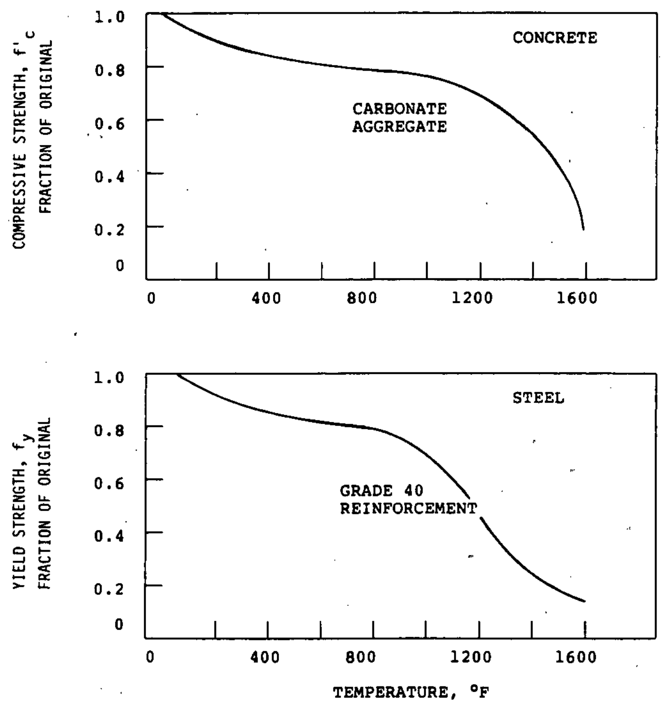


Figure 16. Steel and concrete strength variation with temperature.

relatively short period and the steel is insulated by the concrete; therefore, the yield strength is not appreciably reduced. At steel temperatures above 800°F, however, a marked reduction in the yield strength occurs.

If water is sprayed on the structure while hot, the outside is cooled rapidly, whereas the interior is still hot and in an expanded condition. The result is spalling and cracking of the outside layer of the concrete generally down to the steel level. If the structure is allowed to cool slowly, little spalling and cracking will occur. Damage may look bad, but structurally is not serious.

However, if the duration of the fire is such that the steel gets hot and loses enough of its strength so that dead load yields the steel, a more serious condition exists. Normally this condition is indicated by a permanent sag and large flexure cracks.

Because of the many unknowns in a burnt structure, it would be advisable to consult an expert in the field to acquire a more accurate assessment on the condition of the damaged structure.

1.9 FACTORS CAUSING DETERIORATION IN PRESTRESSED CONCRETE

The factors that can cause deterioration in prestressed concrete are common to reinforced concrete and are discussed in the previous section. In addition, this section discusses loss of prestress by corrosion, normal loss of prestress, and several factors particular to post-tensioned prestress components.

1.9.1 Loss of Prestress by Corrosion

Corrosion can cause failures in prestressed concrete members. If the prestressed strand or rod in the prestressed member begins to corrode, the combination of stress and corrosion will cause the strand to fail rapidly due to stress-corrosion. Failure of a sufficient number of prestressing strands or rods in this manner will cause the member to lose its tensile strength. The loss of tensile strength could lead to failure of the member.

1.9.2 Normal Losses of Prestress

There are several ways in which a prestressed member may lose some of its prestress.

Creep or relaxation of the prestressing steel due to the high sustained tensile stress can cause a gradual decrease in prestress force.

Shrinkage of the concrete causes a relaxation in the prestressing tendons, thereby lowering the effective prestress of the member. This shrinkage will also deform the member and, if the deformation is restricted, can create problems.

Creep of the concrete will cause a loss in prestress. A sustained, heavy, compressive load, will shorten the concrete length, a phenomenon known as creep. This shortening of the concrete mass causes a relaxation in the steel tendons, resulting in a loss in prestress of the member. These normal losses are considered in design.

Unlike cracks in high tension areas of reinforced concrete members, the appearance of cracks in a prestressed member may indicate serious problems with the structural integrity. It should be remembered that a prestressed member is normally designed to be under compressive stresses. Consequently, cracking should not be expected. If found, cracks should be accurately recorded, reported, and evaluated.

1.9.3 Factors Affecting Post-Tensioned Members

Post-tensioned slabs that have insufficient mild reinforcing steel near the ends at the prestress tendon anchor can develop a horizontal crack between the individual anchorages. A close inspection of the ends of such slabs should always be made.

Slippage at anchors can result in loss of prestress tension. Although this problem normally occurs during construction, it may occur later. Symptoms will be the normal unexpected cracks denoting loss of prestress. An expert should be consulted.

The concrete around the anchorages should be inspected for cracking and crushing because this is an area of very high compressive stresses. Although most post-tensioned tendons are grouted after stressing, thereby providing both corrosion protection and bond transfer away from the anchor, a crushing around the anchor can be very serious.

1.10 INSPECTION TECHNIQUES PARTICULAR TO CONCRETE

Inspection techniques particular to concrete bridges and concrete bridge components can be discussed in two ways—the components and the technique itself. The following discussion first covers the specific components together with the particular technique used in inspection and, then, focuses on the techniques themselves.

1.10.1 Decks

Decks require special attention both because of their susceptibility to deterioration and the possibility of rehabilitation of the deck without replacing the entire bridge. A detailed condition survey is necessary before a decision can be made for rehabilitation. This survey can be conducted with normal inspection crews, if the resources are available, or can be contracted out to qualified organizations if necessary. It is emphasized that a detailed deck condition survey is normally performed only after a deck has widespread deterioration and the question arises, "Should the deck be repaired or replaced?" (Techniques for a detailed deck condition survey are included in Chapters 4 and 5 of this manual.)

Routine deck inspection includes a visual survey to determine spall areas and other surface deterioration, sounding for delaminations in suspicious areas, and checking for tell-tale rust stains on the underside of the deck. Drains and scuppers should be checked to ensure that proper drainage and no ponding of deicing chemicals are occurring. The smoothness of ride on the deck should be considered as well as any patchwork on the wearing surface.

The deck should be considered for a detailed condition survey if the percentage of spall area exceeds 2 percent, because there is obvious and widespread deterioration of concrete; or if signs of delamination, such as hollow sounds, are widespread. Surface deterioration that requires frequent resurfacing or patching of the wearing surface is an indication for a detailed deck condition survey.

1.10.2 Horizontal Surfaces

Concrete horizontal surfaces other than the deck should receive careful scrutiny. These locations can include pier caps, bridge seats, or other places that water is likely to pond. These locations should receive a visual check as well as sounding with a hammer or other solid object. Cracking, spalling, or delamination that exposes steel should be a cause for alarm. In many cases the deicing salts drain directly onto these components and cause general deterioration of the concrete as well as corrosion of the reinforcing bar. A dull thud when tapped by a hammer indicates deterioration of the concrete, whereas a hollow sound is an indication that the reinforcing steel is corroding and has delaminated the outer layer of concrete.

1.10.3 Bearings

The concrete directly above a bearing or below a bearing seat should receive close inspection. Such locations frequently have the reinforcing steel too far from the stress concentration at the bearing and cracks and loss of sections result. Such a case is shown in Figure 17. Normally a visual check is all that is necessary, but this check should be at arms length from the bearing. Frequently, such defects are not visible from the ground until they become quite serious. A loss of section at this location is a direct loss of carrying capacity. The condition can be aggravated by improperly working expansion devices or construction defects.

1.10.4 Crack Inventory

The various types of cracking in concrete are discussed and categorized in Chapter V, Section 1 of the *Bridge Inspector's Training Manual-70* (*3*). The crack type, location, direction, size, and length are significant to the evaluation of the concrete member. The significance of cracks in concrete is dependent on the type of concrete structure, the crack origin, and whether the width and length increases with time. The inspector should determine and report these data and also make every effort to determine the cause of the cracking. The inspector should also keep in mind that some cracking, such as flexural cracking in tension zones near the middle of the span on the bottom, is normal and should be expected. However, if the cracks are larger than a tight hairline crack, the cause should be investigated further. Any cracks in a prestressed component should be reported and measured for future reference.

The best technique for the detection of cracking is the old standby, visual inspection. A concrete surface can be wetted and allowed to dry to help locate hairline cracks. Other techniques, such as ultrasonic pulse-velocity, infrared thermography, and the like, are discussed in Chapters 4 and 5 of this manual.

During the visual inspection, cracks can be described with respect to their location, orientation, and width. Crack depth is also important; however, unless the crack extends through the member to the opposite side, coring would be required to determine the depth. Normally, it is not essential that crack width be measured precisely; however, crack width should be determined with reasonable accuracy.

A small hand-held microscope which has a scale marked on the lens is available and is called a crack comparer. With this instrument a crack can be described as hairline, narrow, medium, and wide. Even without the instrument, an inspector should be able to identify hairline, narrow, or wide. A narrow crack can get a paper edge into it, whereas a wide crack will allow the paper to wiggle somewhat. Hairline cracks do not allow anything into it other than liquid.

It is often desirable to monitor and record crack movement. Most movement is perpendicular to the crack and extensometers or transducers can be used to measure movement over a time period. In most cases the field inspector can scratch on both sides of a wide crack and measure with a machinists scale.

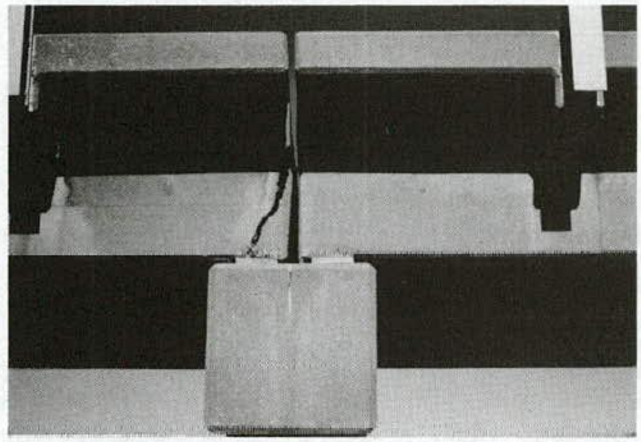


Figure 17. Bearing type crack.

1.10.5 Visual

As with all field structural evaluation, a visual examination is probably the easiest, yet the most important. The concrete bridge or bridge component is no exception. The inspector should always do a general walk-around-type inspection before getting concerned with details. He should mentally note the location of any wide, readily visible cracks, the location and amount of staining due to water seepage, any problem that may be evident concerning drainage, and anything that just looks out of the ordinary.

Most importantly, he should note, if apparent, the location and cause of unexpected cracks. There are some locations, such as the tension side of a beam in reinforced concrete, that are expected to have hairline cracks. These should be noted, but they are of no concern. The unexpected crack, however, should be observed closely as to its apparent cause, which may be traffic damage or unplanned settlement.

As the inspection progresses from superstructure, through bearings to the substructure, the visual examination from an arms length location remains the most important of the senses used in the inspection. Cracks, color, stains, debris, damage, deterioration, all can be seen.

1.10.6 Soundings

The sense of hearing can be used both with artificially created noise and with the common vehicular traffic noise. A hammer can be used to sound the concrete components to determine if a problem exists. A hollow sound is an indication of delamination occurring at that location. A chain drag on the deck is an alternative method of making soundings. With a hammer on the individual components such as piers, columns, or abutments, an inspector can readily get an indication if the material is sound. In addition to the hollow sound, a dull sound or thud is an indication that the concrete itself is of poor quality or in a state of deterioration. Excessive water or other careless construction practices can lead to such concrete. However, of larger concern is the concrete that was once sound and now comes up with the dull thud. Such concrete can be breaking down because of the freeze-thaw problem or, more common, by a chemical attack due to deicing salts. Such salts get into microscopic cracks and crystallize, thus breaking down the concrete at a microscopic level.

Common traffic noise can be evaluated as to whether or not it has any unusual slaps or rings to it. In cases where the noise does not sound normal, the cause should be investigated. A typical case is the situation where the expansion device in the deck is loose. Other causes can be a loose bearing or other unplanned movement in the superstructure.

Sound can be a very useful tool in the inspection of a bridge. It is the experience of the inspector that can best be used to evaluate the sound.

1.10.7 Stains

Stains on the surface of a concrete component can be useful in evaluating a bridge. Normally the stain itself is not harmful, but the indication that the stain represents is important. Most deck bottoms will have efflorescence or exudation of varying amounts. The efflorescence stain is a white powder that is caused by the leaching out of salts from inside the deck itself. Exudation is a gel or viscous liquid that surfaces through cracks and pores. This leaching is common, and can be used to indicate how pervious the deck actually is.

Stains, on the surface or coming from a crack, that are brown or rust colored can indicate that the reinforcing bar has begun to rust. This rusting will normally lead to the protective covering of the steel poping off. In extreme cases a significant loss of steel section can occur. Keep in mind that rust occupies about seven times the volume of regular steel. In a prestressed member, rust coming from a location indicating that the prestressing steel has begun to rust can be quite serious because highly stressed steel will corrode much faster than lower stressed steel. This corrosion of a highly stressed prestress strand can lead to early failure of the strand. Loss of section or a regular reinforcing bar will be much slower.

Stalactites, in severe cases, can result from the leaching of salts from both inside the concrete and through the deck from the deicing salts directly. In themselves, they are not important except for the fact that they indicate a large flow of water through the component with the resulting deterioration of the component.

Stains are an excellent indicator of what is going on inside the concrete component and should be judged accordingly.

1.10.8 Advanced Techniques

There exist many advanced techniques available to the inspector of concrete structures and components. Although many of the techniques work very well in special cases, all depend on the skill and expertise of the user. Some of the techniques include: sonic tests, ultrasonic tests, magnetic tests, chemical tests, nuclear tests, thermography, radiography, and air permeability. These tests come under the general heading of nondestructive tests and are discussed in some detail in later chapters of this manual. Destructive and laboratory tests that pertain to a better evaluation of the condition of a concrete structure include carbonation, water absorption, concrete strength, concrete permeability, cement content, modulus of elasticity, both static and dynamic, splitting tensile strength, petrographic examination, resistance of concrete to chloride ion penetration, and testing of total chloride ion content. These destructive tests are also discussed later in those chapters dealing with in-depth inspections and evaluations.

Actual load tests can also be considered as one of the advanced techniques to be used in the inspection and evaluation of a bridge. Such tests involve more than just driving a truck over the bridge and are discussed in the in-depth inspections and evaluations chapters.

Although a large amount of advanced techniques are available for evaluating the components and material of a bridge, very few are routinely used on the regular biennial examination of a bridge. Each test must be used based on its own merits and the results used in the manner intended. Such tests are part of an advanced evaluation that needs to be used only with the knowledge of what the test actually means with respect to the overall evaluation.

1.10.9 Tools

The routine biennial inspection of concrete bridges requires tools, some of which are particular to concrete. Most of the tools, however, are required for inspections of other types of bridges as well. A suggested list of the contents of a standard tool kit is given in Table 1.

1.11 INSPECTION CHECKLIST FOR CONCRETE COMPONENTS

Included in Table 2 is an inspection checklist for concrete components. This checklist is divided into four columns: what to look for, general location, inspection procedure, and tools required. The checklist is a guide for the inspection of concrete components, but it cannot cover every situation. The inspector himself must use his judgment for particular situations.

1.12 SCOUR OF STREAMBED

Although not unique to reinforced concrete or prestressed concrete bridges, the subject of streambed scour is so important that some mention is prudent. Additional discussion is contained in the *Bridge Inspector's Training Manual 70* (*3*).

Scour of the waterway is a major cause of bridge failures. Scour is defined as the removal and transportation of material from the bed and banks of rivers and streams as a result of the erosive action of running water. Such scour—whether local near a pier, a long-time degradation, or a lowering of the streambed—can be quite serious.

Table 1. Standard tool kit for inspection.

1. Field books, inspection forms, sketch pad, paper, pencil, clipboard and keel marker
2. 100 foot tape for measuring long cracks and large areas
3. 6 foot folding rule with 6 inch extender having 1/32 inch marking for measuring crack lengths and widths
4. Piano wire for measuring the depth of cracks and feeler gage or optical comparator for measuring the width.
5. Chipping hammer for sounding concrete and removing deteriorated concrete
6. Whisk broom for removing debris
7. Scraper for removing encrustations
8. Inspection mirror on a swivel head and extension arm for viewing difficult areas
9. Wire brush for cleaning exposed reinforcement
10. Calipers (inside and outside) or micrometer for measuring exposed reinforcing steel
11. Camera (35mm or Polaroid) for recording observed defects
12. Safety belt and hard hat for individual protection
13. Tool Belt
14. Flashlight for viewing darkened areas
15. Pocket knife
16. Tape recorder for recording narratives of deteriorated conditions
17. Binoculars

Table 2. Inspection checklist for concrete components.

WHAT TO LOOK FOR	GENERAL LOCATION	INSPECTION PROCEDURE	TOOLS REQUIRED
A. Cracking 1. Transverse	A. Approaches B. Bridge deck, sidewalks, curbs and parapets over transverse/ support members C. Pier caps D. Beams & Girders	A. Measure: 1. Length 2. Width 3. Depth B. Record and Report: 1. Length 2. Size Narrow (less than 1/32") Medium (1/32"-1/16") Wide (more than 1/16") 3. Depth 4. Location 5. Probable cause C. Take photograph & sketch crack	A. 100 foot tape B. Feeler gauge C. Piano wire D. Optical Comparator E. Folding rule 1/32" markings F. Camera & sketch pad G. Inspection documents, paper, pencil and clipboard
2. Longitudinal	A. Approaches B. Decks over longitudinal support members	A. Same as above	A. Same as above
3. Diagonal	A. Approaches B. Decks C. Vertical elements D. Beams and Girders	A. Same as above	A. Same as above
4. Vertical	A. Vertical elements B. Beams and Girders	A. Same as above	A. Same as above
5. Horizontal	A. Vertical elements	A. Same as above B. Take photograph & sketch area	A. Same as above
6. Pattern or Map	A. Approach slabs B. Decks C. Vertical elements	A. Same as D-cracking above	A. Same as above
7. Random	A. Same as above	A. Same as D-cracking above	A. Same as above
B. Longitudinal Splitting (serious defect)	A. Beams and girders, both reinforced and prestressed.	A. Same as longitudinal cracking	A. Same as above

Table 2. Continued

WHAT TO LOOK FOR	GENERAL LOCATION	INSPECTION PROCEDURE	TOOLS REQUIRED
C. Spalling	A. Approaches, decks and sidewalks over rebars B. Adjacent to expansion construction joints C. Underside and edges of beams, girders and caps D. Splash zone of members in water	A. Measure area and depth of spall B. Check for hollow zones around spalled areas C. Check for corrosion of rebars D. Compute loss of section in main structural members (piles, columns, etc.) E. Record and report: 1. Area of spall 2. Depth of spall 3. Area of hollow zones 4. Loss of section 5. Degree of rebar corrosion 6. Location 7. Probable cause F. Photograph & sketch affected	A. Same as above plus 1. Hammer to check for hollow zones 2. Wire brush to clean rebars 3. Direct reading calipers or micrometer to check diameters of rebars
D. Pop-outs	A. Can occur anywhere that concrete is used	A. Measure diameter and depth B. Record and report: 1. Number/unit area 2. Size 1/2" is small 1/2"-2 1/2" is medium Over 2 1/2" is large 3. Max depth 4. Location 5. Probable Cause C. Photograph & sketch the area	A. Same as for cracks
E. Mudballs (holes left in surface by dissolution of clay balls)	A. Can occur anywhere but particularly in slabs	A. Same as Pop-outs above	A. Same as for cracks

Table 2. Continued

WHAT TO LOOK FOR	GENERAL LOCATION	INSPECTION PROCEDURE	TOOLS REQUIRED
F. Scaling	A. Approach slabs, decks, curbs, sidewalks and water collection points subject to freeze/thaw cycle particularly where deicing salts or chemicals are present	A. Measure the depth and B. Record and report: 1. Degree of scaling Light-1/4" or less Medium-1/4"-1/2" deep Heavy-1/2"-1" deep 2. Area of scaling 3. Location 4. Probable cause C. Photograph & sketch affected area	A. Same as for cracks
G. Scouring or Erosion of bridge components (Inspect at low water)	A. Surf zone at water line and mud line of concrete piles and piers B. Check for cracks, spalls or exposed rebars	A. Measure: Depth of scour B. Compute loss of section C. Record and Report 1. Loss of section 2. Area of scour 3. Condition of rebars 4. Location 5. Probable cause D. Photograph & sketch area	A. Same as for spalling plus: 1. Scraper. 2. Special equipment a. Boat with motor oars life preservers b. Underwater inspection equipment & personnel, if available & required
H. Efflorescence White Salt surfacing through cracks and pores)	A. Underside of deck, abutments piers & retaining walls	A. Measure width and length of efflorescence B. Record and Report 1. Degree of efflorescence 2. Location 3. Probable cause C. Photograph & sketch area	A. Same as for cracking
I. Exudation (Light gel or viscous liquid surfacing through cracks and pores	A. Same as for efflorescence	A. Measure width and length of exudation	A. Same as for cracking

Table 2. Continued

WHAT TO LOOK FOR	GENERAL LOCATION	INSPECTION PROCEDURE	TOOLS REQUIRED
		B. Record and Report: 1. Degree of exudation 2. Location 3. Probable cause C. Photograph & sketch area	
J. Rust Stains	A. Cracks over reinforcement	A. Measure width and length of rust stains B. Record and Report: 1. Degree of rust stains 2. Location 3. Probable cause C. Photograph & sketch area	A. Same as for cracking
K. Internal Voids	A. Under bulged areas found in abutments, piers, bents, caps	A. Sound bulged area with hammer to detect hollow tone B. Record and Report: 1. Area of void 2. Location 3. Probable cause C. Photograph & sketch area	A. Same as for spalling
L. Collision Damage (Vehicle or vessel)	A. Curbs, railings, overhead beams, piles, piers	A. If a main structural member is involved, check remote elements as well as the immediate area B. Measure cracks, loss of section and/or displacement, if any C. Record and Report: 1. Broken or cracked 2. Loss of section 3. Displacement 4. Location 5. Probable cause D. Photograph & sketch area	A. Same as for cracking & scaling NOTE: The assistance of a structural engineer should be requested when there is evidence of severe damage to main structural members Underwater damage should be inspected by a special investigative team

The inspector should be aware of the problem. The type of foundation and its depth should be known. The relative location of the bed now compared to when the bridge was new is very important.

The problem stems from the fact that the streambed can be much lower during heavy flooding than with normal flow, particularly with localized scour; therefore, measurements during normal flow may misrepresent the actual conditions. Original bridge plans, which indicate the type and elevation of the foundation as well as the original location of the streambed can help. The current elevation measurement can be compared to the values in the plans and measurements from earlier inspections. Any appreciable degradation of the streambed should be cause for more detailed investigation by experts in the field. The present streambed may still be well above the bottom of the foundation because soil is redeposited after a flood and, yet, be a potential problem.

Any cracks due to movement of a pier or abutment, particularly after heavy flooding, is cause for additional investigation with regards to scour.

The routine biennial inspection should include reference to the location of the plans (what type of foundation), reference to the location, vertical and horizontal, of the streambed from prior reports and the plans (if old inspection reports do not include a stream profile, make sure your reports do), and, or course, a present stream profile. The profile should be referenced to a permanent benchmark and not the water surface.

The inspector should be aware that inspection relating to scour of waterways is not a secondary part of his inspection, but of equal importance with the superstructure.

1.13 MAINTENANCE INSPECTION REPORT

The bridge inspection report is as important as the inspection itself. Normally reports are written, although a critical situation may warrant a verbal report followed by a written report. The bridge inspector usually submits a written report for each bridge. The complete report is normally more than a check sheet or brief form. The AASHTO guidelines for inspection (*2*) indicate that the report should include as a minimum the following items: bridge number, date of inspection, bridge name and location, description of work done, structural inventory and appraisal (SI & A) sheet, stress analysis, photographic records, and a dated signature. This report should summarize exactly what the inspector sees or measures. The report concerns the condition of the bridge at the time of inspection and no speculation on what may happen in the future.

1.13.1 Bridge Number

The bridge number is always the official designation for the bridge to be inspected. This number is unique and ensures that the bridge inspected correlates with other records associated with the structure.

1.13.2 Date of Inspection

Inspection date is the date on which the field investigation was made. This information is needed for official and legal references to the bridge in question.

1.13.3 Bridge Name and Location

The descriptive name of the bridge should be given, and any common names by which it is known may be placed in parentheses following the official name. The location of the bridge should be described such that it can be readily identified on a map or found in the field. Common descriptions will include the route number, county, city, river, interchange, and the log mile.

1.13.4 Description of Work Done

A brief summary of the overall condition plus an adjectival rating of the primary components are beneficial to anyone trying to determine the general condition of the bridge. Adjectival ratings summarize the condition of individual bridge components into four general categories of good, fair, poor, or critical. For instance, a deck is rated good, fair, poor, or critical based on the criteria of how well the deck is fulfilling the function for which it was designed. Most highway agency forms require a 0 to 9 rating corresponding to the ratings found in the federal coding guide (*4*). Adjectival ratings are usually correlated with the Federal ratings in the following manner:

Good—A rating of "good" correlates with a condition rating of 7, 8, or 9. The element or component is basically in new condition with no repairs necessary.

Fair—A rating of "fair" correlates with a condition rating of 5 or 6. The element or component is in need of minor repair to continue to function effectively.

Poor—A rating of "poor" correlates with a condition rating of 3 or 4. The element or component is in need of major repair and is deteriorated or damaged to the extent that the structural integrity is affected. Immediate repair is required for the member to perform its intended function in the future.

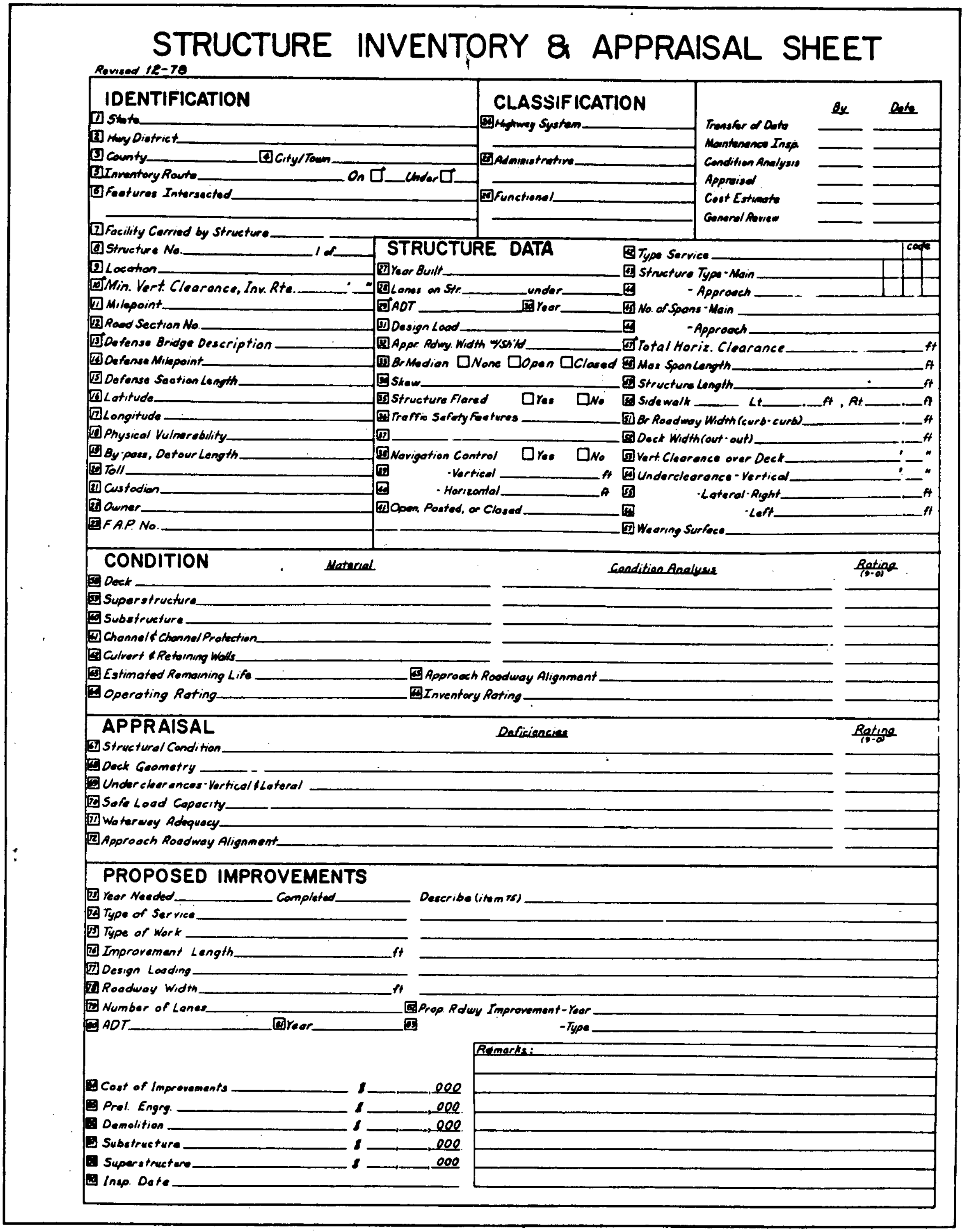
Critical—A rating of "critical" correlates with a condition rating of 0, 1 or 2. The element or component is not performing the function for which it was intended.

1.13.5 Structural Inventory and Appraisal Sheet

The Structural Inventory and Appraisal (SI & A) sheet is the official federal form indicating the data codes to record the inventory information, the condition ratings including the safe load-carrying capacity, and the appraisal ratings for the bridge under consideration. The minimum information required is shown on the "SI & A" sheet shown in Figure 18. Most highway agencies have developed expanded versions of this form to enhance the field inspection and expand their own data base. Appendix D provides guidelines that will aid the inspector in making uniform condition ratings for the structural inventory and appraisal report.

1.13.6 Stress Analysis

Part of the required information is a capacity rating for the bridge. This rating is obtained by a stress analysis. The stress analysis should include the calculations and loads used in determining the safe load capacity rating for each of the primary components of the bridge. The members governing the load capacity of the bridge should be clearly identified. The allowable stresses used in computing the load capacity should be identified and correlated with the bridge plans if possible. Any deviations from the standard AASHTO rating procedures should be noted in the summary. Stress analysis and capacity ratings are discussed in some detail in later chapters of this manual.



STRUCTURE INVENTORY & APPRAISAL SHEET

Revised 12-78

IDENTIFICATION
1 State
2 Hwy District
3 County 4 City/Town
5 Inventory Route On Under
6 Features Intersected
7 Facility Carried by Structure
8 Structure No. 1 of
9 Location
10 Min. Vert. Clearance, Inv. Rte.
11 Milepoint
12 Road Section No.
13 Defense Bridge Description
14 Defense Milepoint
15 Defense Section Length
16 Latitude
17 Longitude
18 Physical Vulnerability
19 By-pass, Detour Length
20 Toll
21 Custodian
22 Owner
23 F.A.P. No.

CLASSIFICATION
Highway System
Administrative
Functional

By Date
Transfer of Data
Maintenance Insp.
Condition Analysis
Appraisal
Cost Estimate
General Review

STRUCTURE DATA
Year Built
Lanes on Str. under
ADT Year
Design Load
Appr. Rdwy. Width w/Sh'ld
Br Median None Open Closed
Skew
Structure Flared Yes No
Traffic Safety Features
Navigation Control Yes No
- Vertical ft
- Horizontal ft
Open, Posted, or Closed

Type Service
Structure Type - Main
- Approach
No. of Spans - Main
- Approach
Total Horiz. Clearance ft
Max Span Length ft
Structure Length ft
Sidewalk Lt. ft, Rt. ft
Br Roadway Width (curb-curb) ft
Deck Width (out-out) ft
Vert. Clearance over Deck
Underclearance - Vertical
- Lateral - Right ft
- Left ft
Wearing Surface
code

CONDITION
Material Condition Analysis Rating (9-0)
Deck
Superstructure
Substructure
Channel & Channel Protection
Culvert & Retaining Walls
Estimated Remaining Life
Approach Roadway Alignment
Operating Rating
Inventory Rating

APPRAISAL
Deficiencies Rating (9-0)
Structural Condition
Deck Geometry
Underclearances - Vertical & Lateral
Safe Load Capacity
Waterway Adequacy
Approach Roadway Alignment

PROPOSED IMPROVEMENTS
Year Needed Completed Describe (item 75)
Type of Service
Type of Work
Improvement Length ft
Design Loading
Roadway Width ft
Number of Lanes Prop Rdwy Improvement - Year
ADT Year - Type
Remarks:
Cost of Improvements $,000
Prel. Engrg. $,000
Demolition $,000
Substructure $,000
Superstructure $,000
Insp. Date

Figure 18. Structural Inventory and Appraisal Sheet.

1.13.7 Photographs

At least two photographs should be included with each bridge inspection report. One photograph of the roadway and one photograph of an elevation view are considered the minimum criteria. Any significant deterioration or damage should be photographed for the permanent record. The use of a color polaroid-type camera is desirable because it provides the inspector with an on-the-spot assurance that he has photographed the observed condition. Normal 35-mm photographs are also considered desirable because more detail can usually be recorded with these cameras. The disadvantage of the nonpolaroid-type camera is the delay time in determining if the picture actually is what was desired.

All photographs should be marked with the bridge number, date, and picture number preferably on the front. A picture number makes it easy to reference the photograph in the narrative part of the report. A good idea, particularly if several photographs are taken, is to make a layout sheet indicating location and direction of each picture. Remember that the cost of a photograph is quite small compared to the rest of the inspection; yet, it provides an excellent permanent documentation.

1.13.8 Signature

The final report must be signed and dated by the individual responsible for the inspection of the bridge.

CHAPTER **2**

ASSESSMENT OF DETERIORATION ON BRIDGE CAPACITY

2.1 POTENTIAL MODES OF FAILURE OF REINFORCED CONCRETE

The AASHTO *Manual for Maintenance Inspection of Bridges—1983* (*2*) begins its discussion of capacity rating of bridges with several assumptions that the bridge inspector should keep in mind. The specification for computing the safe load capacity of bridges assumes that: (1) the materials are of good quality, (2) members are acting normally, and (3) reductions in size or area have been considered in deteriorated portions.

Therefore, it is imperative that the field inspector inform the engineer charged with evaluating the capacity of concrete components when any of these assumptions are not valid. This section of the manual provides information for the bridge inspector to help determine when the assumptions are not valid, where to look for problems, and how to provide critical information for the engineering evaluation of the bridge capacity.

Failure can be defined as the condition that exists when a structure no longer can perform its intended function. Here, the concern is with modes directly related to the carrying capacity of the reinforced concrete bridge. Each potential mode of failure related to carrying capacity is discussed separately.

2.1.1 Flexure

The safe load capacity of girder-type bridges is usually based on the flexural or bending forces produced in the girders (*3*). Bending of reinforced concrete girders will normally produce vertical hairline cracks on the tension side which is on the bottom for simple span bridges. As long as these cracks are small and evenly distributed, the girder is acting normally and the inspector should not be concerned. If the cracks are wide enough to allow water and other contaminants to enter easily, the inspector should note these conditions and monitor them more frequently. Efflorescence and rust stains normally indicate that water and chloride have entered the concrete girders and are causing corrosion of the reinforcement. Wide cracks indicating possible yielding are of concern.

The actual failure of a reinforced concrete in flexure is rare and is normally preceded by many warning signs. These warnings include: (1) large, moving cracks; (2) rust indicating loss of reinforcement section; or (3) severe deterioration and loss of concrete on the compression side.

2.1.2 Shear

Although the safe load capacity of a bridge is not normally governed by shear, diagonal cracks near the girder supports indicate that the shear force may have exceeded the values predicted by the designer. In design a portion of the shear is considered to be carried by the concrete and a portion by the steel. Even small cracks should be noted and reported because they could signify a reduction in the concrete portion of shear capacity. Cracks with efflorescence or rust stains indicate that significant deterioration may be taking place and should be reported immediately to a qualified structural engineer with bridge experience for further evaluation.

2.1.3 Bearing

Excessive bearing forces normally occur at the supports and usually cause crushing of the end of the reinforced concrete girders. Crushing and spalling of the concrete at or near the bearing devices should receive prompt attention. Crushing and spalling indicate that the concrete has been overstressed. The bridge under such conditions should be reevaluated for existing material and load conditions.

2.1.4 Bond

Bond failure in reinforced concrete girders is usually a result of inadequate cover or spacing of the reinforcing bars. Horizontal cracks parallel to the tension reinforcing bars usually indicate bond failure. Efflorescence and rust stains indicate that environmental factors, such as water and chlorides, are causing corrosion of the reinforcing bars. Bond failure is serious in some cases, yet benign in other cases. Cracks of this type should be promptly reported and evaluated by a qualified structural engineer with bridge experience.

2.1.5 Movement

Movement of reinforced concrete girders, except as designed at expansion devices, usually indicates that excessive forces are being transmitted to the girders. Vertical deflection of simple, supported, reinforced concrete girders is normally of little concern to the inspector. However, vertical deflection of continuous concrete girder systems should receive attention. Horizontal movement at bearing devices (except normal expansion-contraction) should be carefully evaluated. The inspector should look for cracking or crushing of the concrete to determine the degree of duress caused by horizontal movements.

Horizontal movement can pull or shove girders off the bearings. The effect of movement on the load capacity of a bridge requires detailed and sometimes complex analysis. The structural engineer that reviews the report needs to know the magnitude of any movement and the direction of movement relative to the normal position of the concrete girder.

2.1.6 Loss of Section

Reduction in the safe load capacity of reinforced concrete girders is most often caused by a reduction of the effective cross section of members or a reduction in the allowable stress that a member may carry. Reduction of the effective cross section is more common. The inspector must accurately measure the residual section of damaged or deteriorated components if an accurate capacity analysis is to be made. Loss of section of the concrete must be measured carefully and related to the tension and compression zones of the concrete member. Loss of concrete in the tension zone has little or no effect on the capacity of the girder unless the steel is rendered ineffective. Loss of concrete section in the compression zone does reduce the load capacity of a girder, but such reduction is usually insignificant because the concrete, which has relatively low stress in most designs, has the capacity for higher stresses. Loss of reinforcing steel section on the tension side will have a significant effect on the load-carrying capacity of concrete girders. Accurate measurement to determine the loss of section is often difficult because the steel may be hidden within the concrete. The loss may be estimated based on the amount of staining or it may be measured directly by exposing the steel at localized points for evaluation. Corrosion must be removed from the steel bars by scraping or cleaning to determine the amount of undamaged reinforcement remaining to carry the loads.

2.1.7 Reduction in Material Strength

The determination of the actual material strength of either the concrete or steel is not very accurate without destructive testing. Unless a core of the concrete or a coupon of steel is tested in the laboratory, the strength of either material is merely an estimate made by an engineer based on experience or literature references. Even with core or coupon testing, the strength of the material will be accurate only for that test of the material and probably would need to be extrapolated to other areas of the member. These limitations make the reduction in cross-sectional area the more commonly used method for estimating the safe load-carrying capacity of damaged or deteriorated reinforced concrete members. However, both reductions in cross section and estimates of loss in material strength should be considered.

2.1.8 Summary

When one considers modes of failure of concrete structures, initially it is the individual component which is of concern. In the final analysis of the situation, however, the concern is for the overall effect the defective component has on the structure. The inspector must consider the overall effect of the individual component on the bridge itself. Often, he must consult with structural engineers to reach a conclusion on the seriousness of the defect when related to the entire structure.

2.2 POTENTIAL MODES OF FAILURE OF PRESTRESSED CONCRETE

From an inspector's standpoint, the same conditions noted earlier apply to prestress concrete bridges as discussed under Section 2.1 for reinforced concrete with respect to bridge capacity. Computations for the operating capacity of prestressed concrete bridges are normally made using ultimate strength methods; therefore, the operating capacity computations are the same with or without prestressing.

2.2.1 Overload

Potential failure due to overload will appear as cracks that normally would not exist in a prestress concrete member. All cracks should be suspect of a potential problem with prestressed concrete. As in regular reinforced concrete, one can look for cracks at particular places for further evaluation. Three types of cracks are of a general nature and easily identifiable. These types include those due to bending moment, those due to shear, and those associated with bearings. Bearing cracks in prestressed concrete can occur in two places: in the normal bearing, such as one would find with the bearing on the supporting structure; and, in addition, one should be concerned with the potential for bearing cracks at or near the ends of the prestressing steel (see also Sec. 2.2.3).

In normal prestress design, one would not expect cracks to occur in the member unless there has been a moment overload that has stressed the steel beyond the yield point. Such cracks should be larger than hairline cracks and should be in the normal suspected areas for flexure cracks. Most design allows for slight tension in the concrete in flexure if the beam is loaded to maximum conditions. Normally, in practice, this tensile stress occurs only for a short time and any resulting cracks will close when the load is removed.

2.2.2 Shear

Cracks due to shear stress also should not appear in normal prestress construction. If the inspector encounters such cracks—cracks near the end of any span that are diagonal to the axis of the beam—he should arrange for an evaluation by qualified structural engineers in a most expedient manner. As in flexure, cracks should not normally occur in the shear location unless something unusual occurs.

2.2.3 Bearing

Bearing cracks can occur around the ends of prestress strands if the concrete adjacent to the tendon is overstressed. Normally, reinforcing steel of a mild nature is provided to contain the high stresses expected at these locations. Cracks, if they occur, will be parallel to the tendons and may be indicative of either a severe problem or nothing. The experts in prestress work need to evaluate such cracks. Points where the beam bears on the substructure are potential locations of trouble not unlike that of the regular reinforced concrete beam. Such areas usually are reinforced with mild steel, but there normally should not be any cracks if the beam was properly designed and constructed. Again a qualified structural engineer should be informed so he can make an evaluation.

2.2.4 Loss of Section

Loss of section in a prestressed concrete structure can be quite serious. It should be noted that the loss of section of steel is doubly important to the capacity calculations because not only is it directly related to the ultimate moment, but it will reduce the prestress force and hence the cracking load will be lower. The loss of section for concrete can be either serious or minor in nature. Traffic damage probably leads to most loss of section in a typical prestressed concrete bridge. Broken tendons are a loss of section and directly affect the ultimate moment and the prestress force. The loss of concrete due to such an accident is not always serious, but can be, depending on the amount and location of the loss. As in the cases of the cracks, if there is a loss of section, the higher level authority with the expertise of such structures should be contacted in an expedient manner.

2.2.5 Loss of Material Strength

The loss of material strength is not likely to occur if the material has been properly manufactured. Loss of concrete strength can occur if there has been an appreciable penetration of salt into the concrete section not related to the corrosion of the embedded steel. Penetration of the salt into the prestressed concrete is less likely than with the reinforced concrete because the concrete is prestressed in compression and hence less susceptible to the penetration of water and contaminants. Better quality concrete and quality control also contribute to this lesser penetration.

2.2.6 Loss of Prestress

Loss of prestress can be a detriment to a bridge. Total loss will be seen as excessive deflection and cracks. Small loss is not detectable by normal means. If large cracks do occur, in the flexure or shear zones, loss of prestress can be suspected. If one looks carefully at the calculations, prestress does not add to the ultimate strength of a beam in flexure—it only controls deflections.

2.2.7 Movement

Movement of a bridge can be either catastrophic or of minor consequence, depending on the amount and direction of the movement and the bridge configuration. Most minor differential settlement can be absorbed in the bridge structure without a major problem. This absorption of settlement can be accomplished even by redundant structures that, in theory, will have huge new stresses created by any new movement. Fortunately, the bridge in most cases redistributes the forces because of the built-in ductility of the structure. However, movement that pulls the superstructure off of the bearing devices or substructure can be serious and may even be catastrophic. For this reason, the inspector needs to be aware of all movements that the bridge is undergoing for any reason.

2.3 Critical Situations

Of the four general classifications for the condition of a bridge component or the bridge itself—good, fair, poor, or critical—critical is the only condition of a bridge which requires immediate action of some nature. Good, as the name implies, is the general classification that requires no action. Fair implies that there is some minor maintenance, such as cleaning debris, that is usually accomplished by routine maintenance on a routine basis. Poor implies that there is a possibility of major maintenance or rehabilitation in the near future. As previously mentioned, critical is the situation where the inspector needs to take some immediate action in the interest of public safety. Action can include a direct call to higher authority to obtain more expert advice, rerouting traffic, or even closing the bridge. As the name implies, it is a *critical* situation. Conditions which can be critical situations will be discussed next.

2.3.1 Movement

Movement by itself is not a critical situation, but the cause of the movement or the result of the movement may, indeed, be a critical situation. Although not designed to move, bridges sometimes do move. This movement can be caused by settlement or consolidation of the foundation, erosion, pressure from expanding pavement against the bridge, traffic impact, swelling of clays, or earthquakes. In all cases the inspector must determine the cause of the movement and the result of the movement, particularly if the movement is continuing. Settlement, consolidation, or swelling of the subfoundation material is a most common cause of movement. Again, this movement can be serious—particularly if it is continuous, rapid, large, or at an increasing rate. Most movements are relatively slow. If the movement is continuing, a foundation expert should be consulted. The settlement can be serious if it is creating large cracks in the bridge components or threatening to push one component off another. Another critical situation is the potential creation of an unstable condition, such as the tilt of a retaining or wing wall. These cases should be considered critical and an expert consulted.

Erosion, whether from runoff around an abutment or general degradation of a streambed, can be, or can become, critical. Situations have been recorded where a streambed has degraded over 10 ft in the period since the bridge was constructed. Because in this case the standard plan was to place the bottom of the footing 10 ft below the streambed, the obvious happened. The bridge failed. Routine recordkeeping of the streambed condition during bridge inspections would have prevented this surprise. Movement may not be detectable due to erosion until there is a failure. Therefore, any erosion should be recorded and repaired as a routine procedure before it becomes a critical situation.

In many cases, a portland cement concrete pavement increases in length with age by the mechanism of dirt and debris getting into joints and cracks during cool weather and expanding against this debris during warm weather. Over a period of time this growth, coupled with chemical reactions and thermal expansion, can put pressure against a bridge and cause movement of the bridge. Such movement can push the bridge off its bearings if allowed to continue. Symptoms include the cracks in the superstructure units around the bearing plates at the fixed end of a typical span. The obvious and simple solution is to have a space cut into the pavement just prior to the bridge to relieve the accumulated stress. Normally, such a situation is not critical unless the bridge is allowed to move off its bearing.

Swelling of clays can have an adverse effect on a bridge. The situation can become serious if such swelling leads to the movement that forces components off other components or causes large cracks indicating either yielding of steel or a lack of steel at that location. Again, the amount and the rate of movement play an important role in determining the seriousness of the problem. Because soil conditions are involved, a foundation expert should be consulted. If the movement is such that collapse is imminent, the situation is critical.

Traffic damage can create a critical situation. Fortunately, the mass of most concrete bridges is such that an impacting vehicle usually suffers the most extensive damage. However, after such an impact, the bridge should be checked to see if it has been shifted away from its normal location. Again, if the bridge has shifted off, or nearly off, its bearings, the situation could be serious. Each case of traffic damage has to be checked on its own merits. The actual point of impact may be on either the superstructure or substructure. In either case, a check for movement needs to be accomplished.

Earthquakes, or any type of motion or shock wave occurring at the structure, creates a need for investigation. Other shocks can include explosion or impact from large objects. In such cases, the bridge should be thoroughly checked to see if any movement causing damage has occurred. Such movement causing damage includes open cracks or relative movement between components.

In summary, movement can be of great concern or it can be of minor importance. It is important to consider (1) the result of the movement, and (2) if the movement has ceased. Large cracks can be an indicator of the movement causing damage. Another place to check is around the bearings to see if the potential exists for movement off of the bearing.

2.3.2 Wide Cracks at Critical Locations

Large, deep, wide cracks at critical locations can be a cause for alarm. Critical locations are places of high shear, high moment, and high bearing forces. These places include the locations in a girder near the support, a place of high shear force, as well as toward the middle of a span, a place of high moment. In continuous members, a similar place for high moment is across any support. The moment in this case has created tension on the top. Around the bearing plate is also a place of potential concern.

A crack designated as wide is more than 0.1 mm in width. A piece of paper could easily fit into the crack. Such a crack probably has yielded the reinforcing steel in the member. It becomes important to determine the cause of the crack. Large cracks can occur because of movement of the structure in a manner in which it was not intended or maybe because of a substantial overload. In all cases, the inspector should try to determine the cause of the crack and whether or not it is growing or "working." In cases where the crack is working, that is, the two sides of the crack are moving relative to each other because of traffic or other causes, it is a serious problem. Wide cracks in a prestressed girder or other component can also be a cause for alarm. Such cracks indicate that the steel has been overstressed or the prestressing steel has lost its prestress. In either case an expert should be consulted.

Diagonal cracks near the supports should be investigated because the concrete is designed to carry part of the shear. If a crack is at this location, the concrete cannot be carrying the forces it was designated to carry. Hairline shear or diagonal tension cracks in a reinforced concrete member are usually not serious; however, a wide crack can be. In a prestressed member any crack labeled a diagonal tension crack in reinforced concrete should be considered serious. The crack itself indicates that the prestress designed into the member is not there. An expert should be consulted.

All reinforced concrete members should have fine flexure cracks in locations where tension is to occur because of normal loading. Such cracks are of little concern. However, if a wide crack occurs in the same location, it can be an indication of yielding of the steel. This condition, although detrimental to the bridge, probably is not an indication of imminent collapse. If such a crack is working, it is much more serious. Consult an expert in this situation.

It cannot be overemphasized that in prestressed members or components, there should not be cracks of any magnitude. If there are cracks, the inspector should ascertain and document the cause of the cracks. If such cracks are working, it is a serious problem and should be looked at by a qualified structural engineer as soon as possible. Longitudinal cracks near an anchor of a prestressed beam should be investigated thoroughly. Such cracks can be an indication of an anchorage failure and, as such, should be cause for consulting an expert. Normally, neither type of crack is cause for a critical condition classification, unless the crack is working.

Cracks around a bearing are not at all uncommon. Such cracks, however, are serious, but rarely are cause for closure of the bridge. Such cracks, indicating some problem near the bearing due to movement or improper placement of reinforcing steel, are serious but rarely critical, unless allowed to progress and become so.

Concrete will crack when subjected to tension whether intended to be in tension or not. Therefore, the inspector should be cognizant of the crack, size of the crack, and orientation and location of the crack. He should determine if the crack is working. Most cracks even if serious are not critical, but if there is any doubt in the inspector's mind he should consult an expert.

2.3.3 Extensive Damage to Critical Members

A critical member can be defined as any nonredundant or fracture-critical member. One pier of a two-pier bent would be an example, as would be one girder of a two-girder bridge.

Again the critical designation occurs if there is an immediate threat to the safety of the public or stability of the structure.

Extensive damage to any critical member can be quite serious. The damage can be from an impact, fire, gradual deterioration, or other causes. Large loss of section, whether steel or concrete, can be categorized as extensive damage. The key to the situation is the determination of whether or not the member is a critical member.

Whether or not an appreciable loss of concrete section is critical depends on the location of the loss and whether the loss affects the load-carrying capacity. In a column, the critical concrete is the core of the column, that is, the concrete within the ties or spiral. Ties, similar to stirrups in a beam, are the small bars of steel perpendicular to the main reinforcing. Spiral steel, normally used in a circular column, is closely spaced small bar generally perpendicular to and containing the longitudinal steel. If the concrete within the core of a column, or any other compression-carrying member, has been damaged, it is likely that a critical situation exists for that component. If the component is critical to the integrity of the structure, a critical situation exists.

Broken steel or prestress strands can be serious if they represent a reasonable percentage of the total steel at the section. In general, the percentage loss of the tension steel is directly related to the moment-carrying capacity of a beam. Therefore, if in an essential, critical member a 50 percent loss of steel section occurs, there is a 50 percent loss of moment-carrying capacity and therefore a 50 percent reduction in the load capacity. A similar situation occurs with prestress steel.

The inspector must carefully investigate any significant loss of section, regardless of the cause. He must then be cognizant of the importance of the component with respect to the entire structure. If the loss of section is great, and the component is critical to the structure, a critical situation exists and the appropriate action must be taken.

2.4 SUMMARY

The critical situation of a bridge, in contrast to a component, normally requires immediate closing of the bridge. A critical condition of a component requires immediate action, but not necessarily closure of the bridge, depending on the relative importance of the component. In critical situations, make a knowledgeable judgment even if that judgment is to call someone more expert than the inspector.

CHAPTER 3

BACKGROUND—IN-DEPTH INSPECTIONS AND EVALUATIONS

3.1 INSPECTOR/EVALUATOR QUALIFICATIONS

Frequently, the individual charged with the responsibilities for bridge inspection is called on to conduct in-depth inspections or evaluations. If this supervisor or individual charged with the inspection responsibilities does not feel qualified to inspect or evaluate the particular defect, an inspection or evaluation specialist should be called.

The minimum qualifications of the individual charged with the responsibilities for bridge inspection are given in the AASHTO *Manual for Maintenance Inspection of Bridges—1983* (*2*). This individual must be a registered professional engineer, or have a minimum of 10 years of experience in bridge inspection assignments in a responsible capacity and have completed a comprehensive training course based on the *Bridge Inspector's Training Manual 70* (*3*). Further, this individual should be thoroughly familiar with the design and construction features of bridges that are being inspected under his supervision and should be able to compute the safe load-carrying capacity of the bridges.

It is unusual for one individual to have experience and expertise in all the fields of engineering associated with bridges. When the inspector and his supervisor do not have the proper background to inspect or evaluate a particular problem or defect with a concrete bridge or concrete bridge component, a specialist should be contacted. The specialist should be selected carefully to assure that he possesses the knowledge and skills necessary to properly evaluate the defect encountered. To properly inspect and evaluate concrete components, the specialist engineer should be thoroughly familiar with the structural mechanics associated with reinforced or prestressed concrete, with the basic properties of concrete, and with the safe load capacity procedures for the concrete component involved. The engineer should also be familiar with inspection techniques and the various test methods and equipment available for evaluating the damage or deterioration encountered in the concrete bridge component.

The engineering knowledge and skills necessary to properly evaluate the concrete bridge may vary widely depending on the complexity of the bridge involved. For instance, most engineers familiar with bridge design and construction could inspect and evaluate damage or deterioration associated with a reinforced concrete T-beam bridge. Much greater knowledge and skill are required for one to inspect and evaluate a cable-stayed structure with segmental box members of post-tensioned concrete.

3.2 PURPOSE OF IN-DEPTH INSPECTIONS OR EVALUATIONS

In-depth inspections and evaluations are used to verify and enhance the routine inspections conducted on concrete bridge components. The in-depth inspections and evaluations are usually conducted by individuals with greater knowledge and skills associated with concrete bridge components.

The in-depth inspections and evaluations are intended to determine the extent of any damage or deterioration to a concrete bridge component and the effect which such damage or deterioration has on the load capacity of the structure. The specialist may utilize equipment or tests that are not available for routine inspections. The in-depth inspection must also determine if any safety hazards exist as a result of the damage or deterioration. Finally, the in-depth inspection and evaluation are often used to determine whether the concrete bridge component may be repaired or rehabilitated, or whether the member or component needs to be replaced.

Numerous tests and procedures are available to the engineering specialist for inspecting and evaluating concrete bridge components. Most of the available test procedures and techniques used for evaluating concrete bridge components are discussed herein.

CHAPTER 4

DETAILED INVESTIGATION

4.1 INTRODUCTION

Detailed investigations are discussed in this chapter under four categories. The first category deals with the in-depth investigation of decks and step-by-step procedures for a condition survey of a concrete bridge deck. The second category concerns the field inspection conducted by a specialized engineer. Next is the general topic of testing. The testing is divided into field tests, destructive tests, and laboratory tests. The final category is on load testing.

4.2 DECK CONDITION SURVEY

For a deteriorated reinforced concrete deck, an important rehabilitation question must be answered, namely, Should the deck be repaired or replaced? Repairing the deck would consist of removing and replacing only the contaminated concrete, whereas replacing the deck would consist of removing and replacing all the concrete. The repair technique obviously requires less concrete but is very labor intensive. Therefore, it is important that the degree and extent of deck deterioration be determined as accurately as practicable so that a cost-effective decision can be made.

In order to restore the structural integrity of a deteriorated bridge deck, the removal of all delaminated, highly chloride contaminated, and deteriorated concrete is required. The placement of additional rebar, or replacement of severely deteriorated rebars, is often necessary. Determination of needed repairs requires a complete deck survey as to delaminations, corrosion potentials, and chloride contents except where visual and delamination surveys indicate complete deck replacement as the obvious economical alternative.

A detailed deck condition survey is usually performed only after a deck has been scheduled for rehabilitation. The condition survey consists of the following sequence of operations.

- Set up traffic control.
- Lay out grid pattern.
- Perform delamination survey.
- Perform half-cell potential (corrosion) survey.
- Perform full deck thickness crack survey.
- Perform chloride content survey.
- Inspect joints and bearings.
- Prepare condition survey report.

4.2.1 Traffic Control

The order and number of lane closures required to carry out the condition survey in the most expedient manner and with the least disruption to traffic must be determined. Traffic control procedures shall be in accordance with the *Manual of Uniform*

Traffic Control Devices (*12*) and shall be developed and implemented prior to beginning the survey.

4.2.2 Grid Layout

To accomplish the deck condition survey, a grid should be laid out on the surface of the deck to reference the test results. The grid layout is then drawn on 8½-in. by 11-in. sheets, and several copies are reproduced to be used in the field to record data collected during the condition survey.

In general, grid spacings between 5 and 10 ft on center are adequate. The 5-ft grid is most common. Longitudinal grid lines should run parallel to the curbs and should extend to within about 12 to 18 in. of the curbs. Transverse grid lines are normally located parallel to abutments and piers and, thus, may either be skewed or orthogonal to the longitudinal grid lines. Grid points should be marked on the deck surface. Three persons can conveniently lay out the grid by using one person to mark the grid points, while the other two persons handle the tape.

4.2.3 Required Tests

In addition to a visual survey to determine spall areas, cracking, and other surface deterioration, recommended tests to establish the condition of the deck are:

- Delamination detection—to be performed over the entire deck.
- Half-cell potential readings—to be taken at each grid point over the nondelaminated area.
- Samples for chloride content—one sample to be taken for each 750 sq ft of remaining deck with a 10 sample minimum.

4.2.4 Delamination Detection

Delaminations are the most significant form of deterioration not revealed by visual inspection. These delaminations normally consist of horizontal separations between the upper and lower portions of the concrete deck. They most commonly are located in the plane of the top reinforcing steel and develop as the steel corrodes.

Delamination detection can be performed by dragging a specially prepared chain across the deck and identifying those areas that emit a "hollow" sound as delaminated areas. Delaminated areas should be marked directly on the deck. These areas are then measured and recorded on the appropriate grid sheet.

4.2.5 Corrosion Potential Survey

A corrosion potential survey measures corrosion activity at the time of the test. Potential readings are normally taken using a copper-copper sulfate half-cell in accordance with ASTM C876 (*13*). Readings are taken at each grid point outside the delaminated area. For the results to be accurate, the deck must be surface dry at the time of the test, and the temperature should not be less than 40 deg. For the test, a positive ground connection is connected directly to the reinforcing steel or to a bridge component that is in direct contact with the reinforcing steel. A pachometer can be helpful in locating the reinforcing steel. A separate ground is required for each portion of the slab that is not continuous. If the half-cell potential readings at adjacent grid points differ by more than 0.15 volts, additional readings should be taken at grid midpoints. Data from the half-cell potential survey are recorded on the appropriate grid sheet at the time of the reading.

Potential readings more positive than −0.20 volts indicate that corrosion is unlikely. Between −0.20 and −0.35 volts corrosion may or may not be active. Readings more negative than −0.35 volts indicate with a high probability that the concrete is contaminated with chloride and that active corrosion of the reinforcing steel is taking place.

4.2.6 Sampling of Concrete

Samples are taken over the portion of the deck that does not have delaminations or potential readings in excess of 0.35 volts. The primary purpose of the samples is for laboratory analysis to determine the chloride content in the concrete.

The conventional method of performing the chloride analysis involves drilling into the deck with a rotary impact hammer, carefully collecting the pulverized concrete at various depths, and conducting a chemical analysis in the laboratory. See AASHTO T260, "Sampling and Testing for Total Chloride Ion in Concrete Raw Materials" (*33*). The laboratory analysis usually gives the quantity of chloride in the concrete either as a percentage or as parts per million based on weight. The conversion to pounds of chloride per cubic yard of normal weight concrete (145 pcf) is 1 pound chloride per cubic yard equals 0.0255 percent chloride or 255 parts/million chloride.

The threshold at which corrosion of the steel is likely to begin is when the concrete contains about 1.3 pounds of chloride per cubic yard. If the amount of chloride in the concrete is less than 1 pound per cubic yard of concrete, the concrete is probably sound. If the amount of chloride is between 1 and 2 pounds per cubic yard, the contamination may or may not be of concern. If the amount of chloride is greater than 2 pounds per cubic yard of concrete, the concrete is normally considered to be contaminated and of concern.

At least one core sample should be taken for compression testing of the concrete. The cores should be free from reinforcing steel and extend the full-depth of the deck. If necessary to obtain a representative assessment of the compressive strength of the deck concrete, a few additional cores should be taken for compression testing.

4.2.7 Visual Inspection

The underside of the deck needs to be inspected for areas of deterioration such as significant cracks, spalling, leaching, exposed reinforcing, and honeycombing. The location and extent of such defects are recorded on an appropriate grid sheet. Extensive cracking and leaching on the underside of the deck usually indicate areas of concrete which require full deck thickness removal and replacement.

The condition survey should include a visual inspection of the deck joints, drains, and possibly the bearings. The condition of these components should be recorded because their repair or replacement may be included in the rehabilitation contract. Photographs of the conditions of these components should be included in this visual inspection.

4.2.8 Condition Survey Report

A condition survey report documents the condition of the bridge deck and is used in selecting the rehabilitation method and in preparing the contract documents. The report need not be lengthy, but it must contain a written summary of significant findings. Sketches and photographs are an effective means of supplementing the text.

The single most important part of the condition survey report is a drawing that summarizes the data recorded on the grid sheets during the field survey. The drawing contains a plan view of the deck drawn to an appropriate scale so that the following information can be clearly shown.

- The overall pertinent dimensions of the bridge deck.
- The layout location of the reference grid.
- Outlined areas of delaminated concrete.
- Outlined areas of full deck thickness cracking and leaching.
- Potential readings at each grid location and outlined areas within which the potential readings exceed −0.35 volts.
- Core sampling locations if done.
- Chloride content sample locations.
- Other pertinent information, such as location of deck drains, surface spalls, exposed reinforcing.

A sample drawing is shown in Figure 19. In this figure, it can be determined from the information shown that 65 percent of the deck is failing. This determination is based on delaminations over a 19 percent area of the deck which overlaps the 5 percent area of full-depth cracking, active corrosion of the rebar over a 22 percent area of the deck, and contaminated concrete over a 24 percent area of the deck. It is noted that the three percentages are over nonduplicating areas of the bridge deck. Transparent overlays of individual data types can be helpful in making this drawing. In accordance with the FHWA Recording and Coding Guide (*4*) (see also Figure 20), this deck would have a condition rating of 3. If the condition rating of a deck is 4 or less, the deck should be replaced. Therefore, this deck would be a candidate for replacement rather than repair. Replacement requires removing and replacing the complete deck. A number of alternate schemes are available to the bridge engineer when replacing a concrete deck.

There is available a computer program entitled "Bridge Deck Survey Computer Program" from the FHWA Demonstration Projects Division which can reduce the repetitious effort required. The program presents graphically, in plan view, the corrosion reading contours, the outline of delaminated areas, and the core sample locations.

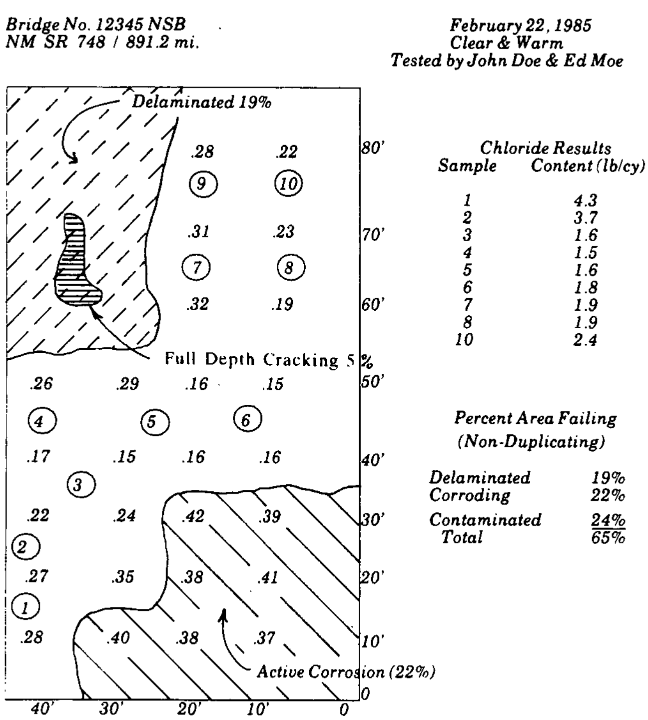


Figure 19. Sample deck survey for rehabilitation.

4.2.9 Concrete Deck Restoration

If the deck condition rating is 5 or 6, there is no clear cut answer whether protecting, restoring, or replacing the deck is the most cost effective. This requires an economical analysis that is beyond the scope of this guide. Many agencies have found that because of the labor-intensive nature of partial removal and replacement of concrete, it is more economical to replace a deck than it is to restore a deck. In addition, when a deck is restored there is always the question of whether or not all the contaminated concrete was removed and replaced.

However, if the decision to restore a deck is made, the engineer must predict at the beginning of construction that portion of the deck which will need only surface removal, the portion which will need partial depth removal, and the portion which will need full depth removal. During the course of construction, the engineer must be prepared to make on-site decisions concerning the extent to which additional removal must be made to obtain sound concrete. Because each type of removal requires different procedures and tools, each type of removal has a different associated unit cost. Therefore, it is necessary that accurate measurements be made on a continuing basis for payment purposes.

Because of the expensive handwork, the need for instant on-site decisions, and the uncertainty of the soundness of the restored deck, many agencies apply stop-gap measures with deteriorating decks to prolong their life and then replace rather than restore the deck.

4.3 FIELD INSPECTION WITH SPECIALIZED ENGINEER

One of the admonitions of the AASHTO *Manual for Maintenance Inspection of Bridges—1983* (*2*) is that field inspectors and supervisors should be aware of any limitations on the inspection of bridges due to a lack of experience or lack of knowledge in any area of work. Inspectors should not hesitate to request the aid of an engineer with specialized knowledge and skills. The type of concrete structure, the type of loading, the type of damage or deterioration, access to the bridge components and other factors may be criteria for seeking a specialized engineer to properly inspect and evaluate a particular bridge.

When seeking a specialized engineer to inspect and evaluate

		Condition Indicators (% deck area)			
Category Classification	Rating	Spalls	Delaminations	Electrical Potential	Chloride Content #/CY
Category #3	9	none	none	0	0
Light	8	none	none	none >0.35	none >1.0
Deterioration	7	none	<2%	45% <0.35	none >2.0
Category #2 Moderate Deterioration	6	<2% spalls <u>or</u> sum of all deteriorated and/or contaminated deck concrete <20%			
	5	<5% spalls <u>or</u> sum of all deteriorated and/or contaminated deck concrete 20 to 40%			
Category #1 Extensive Deterioration	4	<5% spalls <u>or</u> sum of all deteriorated and/or contaminated deck concrete 40 to 60%			
	3	>5% spalls <u>or</u> sum of all deteriorated and/or contaminated deck concrete >60%			
Structurally Inadequate Deck	2	Deck structural capacity grossly inadequate			
	1	Deck has failed completely Repairable by replacement only			
	0	Holes in deck - danger of other sections of deck failing			

Note: The specialized table can be used as a guide for evaluating deck conditions using different condition indicators

Figure 20. Concrete bridge deck evaluation—FHWA condition rating matrix. (From Ref. 4, p. 28)

a particular concrete bridge, the responsible agency should very carefully describe the type of bridge and bridge components involved, the type of loads, if known, that need to be considered, and the depth of evaluation expected. For instance, a short-span reinforced-concrete T-beam bridge with an H15 truck loading would probably require a relatively simple structural evaluation. A post-tensioned concrete segmental bridge with lane loadings plus wind would require much more sophisticated inspection techniques and equipment plus a detailed computer-aided structural analysis.

Other factors, such as specialized equipment for the inspection procedure or for gaining access to the bridge components, need also to be considered. The specialized engineer may identify some of the equipment needed for the inspection. The responsible agency should be sure that the specialized engineer is capable of operating and interpreting the results obtained from any sophisticated inspection equipment needed. The responsible agency should also be certain that qualified operators are available for special access equipment such as a snooper or cherry picker.

4.4 FIELD TESTING

Numerous nondestructive test procedures are available for use on concrete bridge components. The use of a number of these procedures requires specialists' firms or individuals and can be carried out only if suitable staff and equipment are available. Nondestructive testing can make valuable contributions to the investigation of concrete bridge components, but the nature of its contributions should be properly understood. Seldom will a single nondestructive testing technique be able to give the inspector or engineer all the information he wishes to obtain. Nondestructive tests will more likely provide additional evidence about a defect or problem which the inspector already suspects. Therefore, nondestructive testing is more likely to be used in a supporting role and should not be generally regarded as a diagnostic technique in itself.

Several nondestructive procedures are described briefly below in relation to the inspection of concrete bridge components. More detailed descriptions are available in references noted in the discussion. A more detailed summary of many of the tests is given in *NCHRP Synthesis of Highway Practice 118,* "Detecting Defects and Deterioration in Highway Structures" (*14*).

4.4.1 Physical Measurements

Visual inspection of bridges is the most common technique for identifying damage or deterioration to concrete bridge components. Cracks of various types are often indicative of duress in the components. Precise measurements of cracks is not usually warranted, but crack widths are often described according to the following scale: hairline (less than 0.004 in.), narrow (0.004 to 0.01 in.), medium (0.01 to 0.03 in.), and wide (greater than 0.03 in.). It should be noted that raveled edges or moisture associated with the crack may make the crack more visible and appear wider than it actually is.

4.4.2 Rebound and Penetration Methods

Rebound and penetration tests measure the hardness of concrete and are used to predict the strength of concrete. The Schmidt hammer is probably the most commonly used device of this type. It consists of a plunger and a spring-loaded mass that strikes the free end of a plunger that is in contact with the concrete and rebounds. The extent of rebound gives an indication of the strength of the concrete at the surface position tested. The measurement is influenced by the finish of the concrete, age, and other factors. As an inspection technique, the hammer may be used to compare the quality of the concrete in different parts of the concrete bridge components. It should be remembered that only the surface of the concrete is being checked and the strength values are relative. This test is covered in ASTM Test C805, "Test Method for Rebound Number for Hardened Concrete" (*15*). Actual strength must be determined by other means.

The relative compressive strength of concrete can also be determined by the "Windsor probe." The Windsor probe is a commercial test system that utilizes procedures outlined in ASTM C803, "Test Method for Penetration Resistance of Hardened Concrete" (*16*). This device drives a steel probe into the concrete using a constant amount of energy supplied by a precise powder charge. The length of the probes projecting from the concrete is measured. A normal result is based on the average of three measurements. This test and the Schmidt hammer are considered usable only with relatively new, less than one-year old, concrete.

4.4.3 Sonic Methods

Mechanical sonic pulse-velocity methods have been used for concrete for many years. Hammer blows create the impulse, and the time of travel of this sonic pulse between pickups placed on the concrete is measured. The time of travel is related to the modulus of elasticity and hence the strength. This technique can be effective, but is tedious and can be applied to small areas only. The procedure is capable of detecting differences between areas of sound and unsound concrete and is frequently used to detect delaminations or other fractures. The technique is impractical in evaluating large surface areas as concrete decks. However, on vertical surfaces there is currently no alternative that is practical and reliable.

Chain drags, sounding rods, or even hammers are frequently used for detecting delaminations on horizontal surfaces, such as decks or tops of piers. The chain drag can be used to quickly traverse a large area with reasonable accuracy in determining areas of delamination provided the inspector has experience in detecting hollow sounds. Chain-drag surveys of asphalt-covered decks are not totally accurate, but they are quick and inexpensive and may be used as an initial test to determine the need for more thorough investigations.

A portable electronic instrument known as a Delamtect has been developed for bridge decks. The instrument consists of three components: a tapping device, a sonic receiver, and a signal interpreter. The instrument is moved across a deck as acoustic signals are generated, propagated through the concrete, received, and interpreted electronically. The output is used to generate a plan of the deck indicating delaminated areas. The accuracy decreases when used on an asphalt-covered deck.

4.4.4 Ultrasonic Techniques

Ultrasonic devices are normally used by measuring the velocity in concrete of a pulse generated by a piezoelectric transducer. The pulse velocity depends on the composition and maturity of the concrete and its elastic properties. The relationship to strength depends on several other properties and is best determined experimentally.

The recommended procedure is the direct transmission method that has the transmission and receiving probes in line on opposite sides of a concrete thickness. Caution should be used in comparing results from indirect transmission tests with calibrations or tests from direct transmission techniques.

There appears to be reasonably good correlations between pulse velocity and compressive strength provided the system has been calibrated with cores of the particular concrete being evaluated. The concrete strength can be predicted within about 20 percent of the calibration curve established for the particular concrete being investigated. It is not possible to predict the strength of concrete without calibration with the particular concrete in question.

The presence of steel parallel to the line of transmission provides a path along which the pulse can travel more rapidly. Corrections can be made for this situation, but detailed information on the reinforcement is needed. It is generally desirable to choose path lengths that avoid the influence of reinforcing steel.

Open cracks or voids may also affect the ultrasonic pulse. The path of the pulse will thus travel around any cavity in the concrete and the time of transmission of the pulse is lengthened. Large cracks and voids may be detected by this means. Narrow cracks will transmit the pulse through points of contact, and small voids will increase the path length only a small amount and may not be distinguishable from the normal variability of the measurements.

Ultrasonic techniques can, with proper experience and training, provide excellent information regarding the condition of the concrete. However, the method is complex and requires some skill to obtain usable results. The technique is not normally used in routine bridge evaluation.

4.4.5 Magnetic Methods

The principal application of magnetic methods in testing of concrete bridge components is in determining the position of reinforcement. Magnetic methods are not techniques for detecting defects or deterioration directly, but the fact that inadequate cover is often associated with corrosion-induced deterioration indicates that a method for locating the reinforcing bars can be important in corrosion control.

Several portable, battery-operated magnetic devices known as cover meters or pachometers have been designed to detect the position of reinforcement and measure the depth of cover. The devices generate a magnetic field between the two poles of a probe, and the intensity of the magnetic field is proportional to the cube of the distance from the pole faces. When a reinforcing bar is present the magnetic field is distorted and the degree of distortion is a function of the bar diameter and its distance from the probe.

In general, the cover meters can measure cover within 0.25 in. in the range of 0 to 3 in. The instruments give satisfactory results in lightly reinforced members but, in heavily reinforced members or where large steel members are nearby, it is not possible to obtain reliable results. In addition, some reports indicate epoxy coatings distort readings.

4.4.6 Electrical Methods

Electrical methods for inspection of concrete bridge components include resistance and potential measurements. Electrical resistance has been used for measuring the permeability of bridge deck seal coats. The procedure has been published as a standard test in ASTM D 3633 (*18*) and involves measuring the resistance between the reinforcing steel and a wet sponge on the concrete surface.

Corrosion of reinforcement produces a corrosion cell caused by differences in electrical potential. This difference in electrical potential can be detected by placing a copper-copper sulfate half-cell on the surface of the concrete and measuring the potential differences between the half-cell and steel reinforcement (*17*). It is generally agreed that the half-cell potential measurements can be interpreted as follows:

- Less negative than −0.20 volts indicates a 90 percent probability of no corrosion.
- Between −0.20 and −0.35 volts, corrosion activity is uncertain; more negative than −0.35 volts is indicative of greater than 90 percent probability that corrosion is occurring.

If positive readings are obtained, it usually means that insufficient moisture is available in the concrete and the readings are not valid. These tests do not indicate the rate of corrosion, and the measurements only manifest the potential for corrosion at the time of measurement (*14*).

Although most commonly used with bridge decks, the half-cell has been used with other bridge components, such as bents, to determine active corrosion.

4.4.7 Nuclear Methods

The main use of nuclear methods is to measure the moisture content in concrete by neutron absorption and scattering techniques. These moisture measurements are then used to determine if corrosion of reinforcement is likely to occur. A more direct measurement of the rate of corrosion would be more useful to the bridge inspector and, hence, the nuclear methods are more research oriented than operational.

4.4.8 Thermography

Infrared thermography has been found to be a useful supplemental test in detecting delaminations in concrete bridge decks. The method could be used for other concrete bridge components exposed to direct sunlight. Thermography works on the principle that as the concrete heats and cools, there is substantial thermal gradient within the concrete because concrete is a poor conductor of heat. Delaminations and other discontinuities interrupt the heat transfer through the concrete, and these discontinuities cause a higher surface temperature during periods of heating than the surrounding concrete and the reverse situation during periods of cooling. The differences in surface temperature can be measured using sensitive infrared detection systems. The equipment can record and identify areas of delamination and correlations can indicate depth of delamination below the surface by the differences in surface temperature.

4.4.9 Radar

Ground-penetrating radar has been used to detect deterioration of bridge decks (*19*). These investigations are carried out by low-power, high-frequency pulsed radar. The radar picks up any discontinuity such as air to asphalt, asphalt to concrete, or cracks in concrete. The ability to measure the thickness of asphalt covering is an important benefit. The radar method also has an important potential for examining the condition of the top flange of box beams that are otherwise inaccessible (*17*). More than a little experience is necessary for proper interpretation of the data.

4.4.10 Radiography

Gamma radiation will penetrate concrete and therefore can be used to investigate concrete by exposing photograph film to radiation. A source of radiation is placed on one side of the concrete and a film is attached to the other side. Steel impedes the transmission and an image shows up on the developed film as lighter than the surrounding concrete. Void areas show up as darker images. The inspector then can get a reasonable idea of the concrete steel reinforcement pattern and the location and extent of defects in the concrete mass.

Radiography can be carried out only by licensed firms that can handle radioactive isotopes. Radiography of concrete is expensive and limited applications of the technique are likely to be used in bridge inspection.

4.4.11 Air Permeability

Air permeability is used primarily to assess the resistance of concrete to carbonation and to penetration of aggressive ions. The procedure has been investigated in other countries, such as Japan and Denmark, and is still considered experimental.

4.4.12 Comparison of Test Methods

NCHRP Synthesis of Highway Practice 118 (*14*) includes a table that summarizes the capabilities of the various nondestructive test methods in detecting defects in concrete bridge components. That table is reproduced here as Table 3. Additional discussion on the capabilities of the test methods can be found in the *NCHRP Synthesis.*

4.5 DESTRUCTIVE TESTS

Most of the destructive field testing begins with coring samples from the concrete component in question. Cores should have a diameter three times the maximum aggregate size if possible. The criteria for the cores is covered by ASTM C 42 (*20*). The coring should be carried out by a skilled operator to obtain useful samples. It should be noted that usable cores can normally be obtained only if the concrete is sound. If the concrete is poor and corrosion of the reinforcing steel likely, obtaining a core is difficult. Core drilling can weaken the structure and should only be authorized by a bridge engineer. All core holes should be filled with a nonshrink concrete grout mixture.

4.5.1 Concrete Strength

The actual strength of concrete can be obtained only by the destructive technique of removing a sample and taking it to a laboratory for strength tests. This topic is covered in some detail in the section on laboratory testing.

4.5.2 Reinforcing Steel Strength

The actual properties of reinforcing steel can only be obtained by removing a sample and having it tested in the laboratory. This removal of a portion of a reinforcing bar and subsequent testing can be detrimental to the capacity of the bridge and should not be done unless the resulting data are absolutely essential. Such removal of reinforcing steel should be only as authorized by the bridge engineer.

4.5.3 Weight Loss and Pit Depth

These two tests are performed on reinforced bars removed from the structure by coring operations. The loss of weight of the reinforcing bar or the depth of the pits caused by corrosion are compared to a standard uncorroded reinforcing bar. However, the extent of corrosion detected by these two methods is not very useful and can be misleading because the corrosion of steel in concrete is not uniform (*14*). In addition the removal of the bar is a complete loss of section for that bar; hence, such coring should be only as authorized by the bridge engineer.

4.5.4 Carbonation

Carbonation of concrete is the result of the reaction of carbon dioxide and other acidic gases in the air which form weak acids in solution. This results in a reduction of the alkalinity of the concrete and a loss of protection of the steel against corrosion. The depth of carbonation in a concrete bridge component can be measured by exposing fresh concrete surfaces to a 2 percent solution of phenolphthalein in ethanol (*17*). This solution is a pH indicator with a color change occurring about pH 10. Magenta areas of the exposed concrete represent uncarbonated concrete areas, and the colorless areas represent carbonated concrete. Fresh concrete surface areas can be exposed by breaking off pieces with a hammer or chisel.

4.5.5 Moisture Content

The moisture content in concrete at the level of the steel reinforcement can be considered an indicator for corrosion activity. The moisture content may be determined by use of the nuclear devices described under nondestructive testing, or it may be determined from concrete samples taken in the field and oven dried in the laboratory. Quick field tests for moisture could probably be used, but the test is an indirect indicator of corrosion and of questionable use to the bridge inspector or engineer evaluating the corrosion activity in the concrete component.

4.5.6 Concrete Permeability

Air and water permeability can be measured by a procedure that consists of drilling a small hole into the concrete, sealing the top with liquid rubber and, then, inserting a hypodermic needle to provide either a vacuum or water. Air permeability is determined by filling the hole with water and, then, measuring the flow into the concrete at a pressure similar to that created by rainfall (*17*). This procedure is not widely used in bridge inspection of concrete members.

4.5.7 Endoscopes

Endoscopes consist of rigid or flexible viewing tubes that can be inserted into holes drilled into concrete bridge components.

Light can be provided by glass fibers from an external source. In the rigid tubes viewing is provided through reflecting prisms, and in the flexible tubes a fiber optics system is used. These scopes allow close examination of parts of the structure which could not be otherwise viewed. The inside of a box girder or a hollow post-tensioning duct are two examples. Some equipment is available with attachments for a camera or television monitor (*17*). Although this is a viewing instrument, it is considered with destructive tests only because some destruction is necessary for its proper use with concrete.

4.5.8 Summary

The destructive tests should be used only when a particular piece of information is desired, and only when the results can be verified or compared to the results of other tests. These tests by themselves will not allow the bridge engineer to evaluate the capacity of a concrete bridge. When the tests are used, they should be conducted as standard ASTM or AASHTO procedures if available.

Table 3. Capability of investigating techniques for detecting defects in concrete structures and field use. (From Ref. 14, p. 21)

Technique	Cracking	Scaling	Corrosion	Wear & Abrasion	Chemical Attack	Voids in Grout
	Capability of Defect Detection[a]					
Visual	G	G	P	G	F	N
Hardness	N	N	P	N	P	N
Sonic	F	N	G	N	N	N
Ultrasonic	G	N	F	N	P	N
Magnetic	N	N	F	N	N	N
Electrical	N	N	G	N	N	N
Chemical	N	N	G	N	N	N
Nuclear	N	N	F	N	N	N
Thermo-graphy	N	G[b]	G	N	N	N
Radar	N	G[b]	G	N	N	N
Radio-graphy	F	N	F	N	N	F
Air Permea-bility	N	N	F	N	N	F

[a]G = good; F = Fair; P = Poor; N = Not suitable

[b]Beneath bituminous surfacings.

4.6 LABORATORY TESTING

There exist many tests that are conducted in the laboratory that supplement the field tests and observations. As with all testing, whether in the field or in the laboratory, the test should be run only if the result provides something useful in the overall evaluation of the bridge. The engineer must always be cautious of running tests without a firm grasp as to what the results actually contribute to the evaluation.

4.6.1 Compressive Strength of Concrete

One of the frequent questions asked during a bridge evaluation is, What is the strength of the concrete that currently exists in the bridge? Because concrete strength is dependent on its history as well as its original composition, sometimes the only way to determine the concrete strength is to core the structure and test the core. All nondestructive tests are good only for relative values, and any absolute strength values depend on a calibration with the existing concrete obtained by cores.

The detailed procedures for properly removing concrete samples by core drilling are given in ASTM C42 (AASHTO T24), "Standard Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete" (*20*). The following specific guidelines are of particular importance in core sampling:

1. *Equipment*—Cores should be taken using diamond-studded core bits when the cores will be tested for a strength property. A shot drill may be acceptable for other applications when the core is drilled vertically. Diamond-studded core bits are recommended for other than vertical orientation in all cases.
2. *Sample*—the number, size, and location of core samples should be carefully selected to permit all necessary laboratory tests. It is important to use virgin samples in order that there be no influence from prior tests.
3. *Core diameter*—Cores to be tested for a strength property should have a minimum diameter of 3 to 4 in., or 3 times the maximum normal size of the coarse aggregate, whichever is greater. Cores having a diameter less than 2 in. are generally not recommended for any application.
4. *Reinforcing steel*—Reinforcing steel should not be included in a core to be tested for a strength property.
5. *Depth*—Where possible, the core drilling should completely penetrate the concrete section to avoid having to break out the core during removal. Where through-drilling is not feasible, an extra 2 in. of depth or more should be allowed to take into account damage that may occur at the base of the core.
6. *Evaluation*—If the cores are taken for the purpose of testing to determine a strength property, at least three cores should be removed at each location in the structure where it is desired to determine the strength. The final accepted test value should be taken as the average test result of the three cores tested. No single core should be used to evaluate or diagnose a particular problem.

The ASTM C42 procedure also goes into some detail on the procedure for testing the cores or sawed beams. The tests discussed include compressive tests, splitting tensile strength determinations, and flexural strength determinations from beams sawed from the structure. Other ASTM procedures that may be of help in obtaining or testing cores for compressive strength include ASTM C174 "Method of Measuring Length of Drilled Concrete Cores" (*21*) and ASTM C823, "Recommended Practice for Examination and Sampling of Hardened Concrete in Construction" (*22*). Because coring can be quite destructive, one must be sure that the result of the test is actually needed, and not just "nice to have."

4.6.2 Cement Content

ASTM C85 (AASHTO T178), "Cement Content of Hardened Portland Cement Concrete" (*23*), is a method for finding the cement content from existing concrete. This test method for determining the cement content of concrete is applicable to hardened portland cement concretes except those containing certain aggregates or combinations of aggregates or admixtures that yield significant amounts of dissolved calcium oxide and dissolved silica under the conditions of the test. The use of this test in bridge inspection is limited unless there is some question as to the quality of the concrete that was actually used in the project.

4.6.3 Air Voids

Two tests exist for determining the air voids existing in hardened concrete. These are ASTM C457, "Recommended Practice for Microscopical Determination of Air-Void Content and Parameters of the Void System in Hardened Concrete" (*24*) and ASTM C642, "Test Method for Specific Gravity, Absorption, and Voids in Hardened Concrete" (*25*). Air voids in an existing structure are of little concern unless one is trying to determine reasons for apparent excessive deterioration beyond that which would normally be expected for a particular design of concrete.

4.6.4 Static Modulus of Elasticity

A method for determining the static modulus of elasticity is in ASTM C469, "Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression" (*26*). The modulus of elasticity can be helpful in relating loads to deflections in a structure. The modulus is also an indication of the strength of the concrete with stronger concrete having the higher modulus of elasticity.

4.6.5 Dynamic Modulus of Elasticity

ASTM C215, "Test Method for Fundamental Transverse, Longitudinal and Torsional Frequencies of Concrete" (*27*), provides a means for determining the dynamic modulus of elasticity of concrete. Dynamic modulus of elasticity is normally higher than the static modulus and may be helpful in pulse velocity tests of concrete. This property is not routinely used in evaluation of bridge structures, but may be helpful in high frequency dynamic tests.

4.6.6 Splitting Tensile Strength

The splitting tensile strength of concrete can be determined by ASTM C496 (AASHTO T198), "Splitting Tensile Strength of Cylindrical Concrete Specimens" (*28*). This method covers the determination of the splitting tensile strength of both molded cylinders and drilled cores. The splitting tensile strength is an indication of the tensile strength of concrete and is generally used with light-weight aggregate concrete. The tensile strength of reinforced concrete is normally ignored in flexural analyses and in direct capacity computations of concrete structure. However, diagonal tension or shear failures could be related to the tensile strength of the concrete.

4.6.7 Petrographic Examination

A standard practice for petrographic examination of hardened concrete is given in ASTM C856 (*29*). This test is used to examine the materials used in the manufacture of the concrete. It can be helpful if the concrete is showing evidence of chemical reactivity of the aggregate. Normally, the test is not used in regular evaluations of bridges, but it can be helpful in evaluating the original concrete. The following characteristics of the concrete in an existing structure are detectable by petrography:

1. Density of the cement paste, and color of the cement.
2. Homogeneity of the concrete.
3. Occurrence of settlement and bleeding in fresh concrete.
4. Weathering patterns of old concrete (from surface-to-bottom).
5. Occurrence and distribution of fractures.
6. Characteristics and distribution of voids.
7. Presence of contaminating substances, types, and conditions.
8. Proportion of unhydrated granules of concrete.
9. Presence of mineral admixtures.
10. Volumetric proportions of aggregates, cement paste, and air voids.
11. Air content and various parameters of the air void system (including entrained and entrapped air).
12. Presence of deterioration caused by exposure to freezing and thawing.
13. Presence of deterioration due to abrasion or fire exposure.

4.6.8 Chloride Ion Penetration

Several standard test procedures relate to chloride ion penetration. AASHTO T259, "Resistance of Concrete to Chloride Ion Penetration," (*30*), is a method that covers the determination of the resistance of concrete specimens to the penetration of chloride ion. It is intended for use in determining the effects of variations in the properties of concrete on the resistance of the concrete to chloride ion penetration. Variations in the concrete may include, but are not limited to, changes in the cement type and content, water-cement ratio, aggregate type and proportions, admixtures, treatment, curing, and consolidation. This test method is not intended to provide a quantitative measure of the length of service that may be expected from a specific type of concrete.

ASTM C672, "Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemical" (*31*), as the name implies, is a test method for testing combinations of concrete for resistance to deicing salts. The test is used for optimizing concrete design rather than for checking existing concrete.

AASHTO T277, "Rapid Determination of Chloride Permeability of Concrete" (*32*), covers the determination of the permeability of conventional portland cement concrete and specialized concrete, such as latex modified and polymer concrete to chloride ions. It consists of monitoring the amount of electrical current passed through 95-mm diameter by 51-mm long cores when one end of the core is immersed in a sodium chloride solution and a potential difference of 60 volts dc is maintained across the specimen for 6 hours. The total charge passed, in coulombs, is related to chloride permeability.

These tests have implications for design but little application in the capacity evaluation of an existing structure.

4.6.9 Total Chloride Ion

Sometimes in the determination of the condition of an existing deck or other component of concrete the total chloride ion in the concrete becomes important. It is particularly useful in the evaluation of whether or not a deck should be repaired or replaced. AASHTO T260, "Sampling and Testing for Total Chloride Ion in Concrete and Concrete Raw Materials" (*33*), covers such a procedure. The method is limited to materials that do not contain sulfides, but the extraction procedure described may be used for all such material.

4.6.10 Tension Testing of Reinforcing Steel

The tension testing of reinforcing steel is covered in ASTM Standard Method E8 (AASHTO T68), "Tension Testing of Metallic Materials" (*34*), pp. 156–177. The method covers the determination of yield strength, tensile strength, elongation, and reduction of areas. Because this test is destructive in nature, the engineer must be sure that the results are worth the destruction. Although the yield stress is used directly in capacity evaluations, an estimate, as provided for in the AASHTO *Manual for Maintenance Inspection of Bridges* (*2*), may be better, for all concerned, than removing a sample for a direct test.

4.6.11 Summary

The laboratory tests, as in all tests, should be used only when a particular piece of information is desired. When such is the case, standard procedures are usually available with either ASTM or AASHTO specifications and procedures.

4.7 LOAD TESTS

Load testing of bridges is used to help the engineer more effectively determine load distributions, measure stress in various concrete components under traffic loading, estimate the ultimate capacity, ascertain dynamic responses, and evaluate the effect of deterioration on the load capacity of the bridge (*35*). Often in the past, load tests were used as acceptance tests to fulfill established code requirements (*36*). Load tests as acceptance tests are discouraged. Evaluation of concrete bridges by load test is usually recommended only if one of the following conditions is encountered.

1. The complexity of the bridge design and lack of experience with structural elements make evaluation by analytical means impractical.
2. The loadings and concrete properties of the component are not readily determinate.
3. The degree of existing defects cannot be readily determined or the nature of existing distress introduces indeterminate parameters.
4. Where there is doubt concerning the adequacy of concrete components under present service load conditions.

Load tests are not cheap and require significant planning, equipment, and personnel. However, load tests can often be justified where the effect of deterioration or damage cannot be determined by analytical methods. Load tests in these cases may show the load capacity of a bridge higher than predicted and eliminate posting or replacement of the bridge (*5, 35*).

Load testing of bridges should generally be limited to evaluation of the structural strength to resist vertically applied gravity loads. Load testing is not recommended for evaluating the structural strength in relation to lateral loads such as wind or seismic loads.

4.7.1 Load Mechanisms

Static loading is the more common load evaluation of concrete bridges. These loads are usually in the form of loaded trucks placed at critical locations on the bridge. One or more trucks are used in the load test, depending on the type of structure and the size of the structure. Often, the truck is placed at successive positions to permit evaluation by an influence line for the structural component. Load distribution may be checked by placing trucks in different lanes and at the centerline of the bridge. Occasionally, static loads may be applied by dead weight or pulling anchored cables to induce maximum loading conditions.

Dynamic loading of concrete bridges is used occasionally to evaluate concrete bridge components. The objective of these tests is usually to vibrate the bridge in such a manner that defects are related to dynamic characteristics (*14*). The vibration may be induced by trucks or special equipment designed to seek the vibrational modes of the bridge. These dynamic procedures are generally limited to research applications and are not likely to be used routinely in load capacity evaluations at the present time.

4.7.2 Measurements

The response of the bridge to applied loads is usually assessed through strain measurements in the tension reinforcement, strain measurement of the concrete, or measurement of deflections. Vertical displacements are often measured in the main span at the quarter points and at midspan. In adjacent spans, the deflection is measured at least at the midspan. These deflections are usually measured using wire-support equipment, dial gage set-ups, liquid means, or through remote surveying instruments aimed at targets on the structure.

Electrical resistance strain gages are often bonded to tension reinforcement. This procedure requires that the bars be exposed to attach the gages. Generally, at least one bar in each girder is instrumented to measure load distribution and ensure uniformity of strain measurements (*37*).

4.7.3 Summary

Load testing should be conducted under strict criteria to ensure proper evaluation of concrete bridge components. A qualified bridge engineer should control the tests. The structure or portion of the structure to be loaded should be loaded in such a manner as to adequately test any suspected source of weakness. The results of the tests should be carefully evaluated. For instance, some field studies indicate that it is not feasible to evaluate the load-carrying capacity of deteriorated concrete bridge girders using service loads (*37*).

CHAPTER 5

CAPACITY EVALUATIONS

5.1 INTRODUCTION

Capacity evaluations of reinforced concrete and prestressed concrete bridges can be considered both straightforward and extremely difficult. The general guidelines are given in the AASHTO *Manual for Maintenance Inspection of Bridges* (*2*). NCHRP Project 10-15, "Structural Strength Evaluation of Existing Reinforced Concrete Bridges," deals with the subject in greater depth (*38*).

In this report, the general guidelines in Ref. *2* are used to examine various types of concrete bridges and bridge components. The relative effect of different types of deterioration or damage will be considered in the analysis of a bridge. The examination of a bridge and bridge components includes reinforced concrete and prestressed concrete.

The AASHTO *Manual for Maintenance Inspection of Bridges* (*2*) governs the safe load capacity of bridges. When the manual does not cover some specific aspect of the analysis, the AASHTO *Standard Specifications for Highway Bridges* (*39*) governs the analytical procedures. These publications will be referred to as the Manual and the Specifications, respectively.

5.2 REINFORCED CONCRETE EXAMPLES

Appendix A contains a series of sample calculations for determining bridge ratings. The results of various capacity analyses are discussed in this section.

5.2.1 T-Beam

A concrete deck girder bridge with a 50-ft simple span, center-to-center bearing was analyzed. The structure was built in 1973 and has the following properties:

Clear span	50 ft
Clear width	44 ft
Rating vehicle	HS-20
Concrete strength, f'_c	3 kips per sq in.
Reinforcing steel	Grade 40 steel
Wearing surface	3 in. asphalt

Table 4 gives values for moment capacity obtained by both the service load method and the load factor method. The load factor method yielded results that were a little larger. Using the load factor as the base, there was a 4.9 percent difference for operating rating and a 6.4 percent difference for inventory rating.

Table 4. Moment capacity ratings for T-beam.

	Load Factor Method	Service Load Method	Percent Difference (LF Base)
	Rating	Factors	
Inventory	1.72	1.61	-6.4
Operating	2.87	2.73	-4.9
	Capacity	Ratings	
Inventory	HS 34	HS 32	
Operating	HS 57	HS 55	

Table 5 shows values for shear capacity obtained by both methods. The service load method yielded the same results for the inventory rating but the load factor method yielded larger results for the operating rating.

Table 6 shows values for bearing capacity obtained for the T-beam. The load factor method yielded larger values. This substantial difference can be attributed to the conservative use of the allowable bearing concrete stress in Section 8.15.2.1.3 of Ref. *39*.

Table 7 summarizes the capacity ratings for moment, shear, and bearing for the T-beam as determined by the load factor method. The capacity ratings for shear were the smallest. Therefore, shear controls and the T-beam bridge is rated as an HS-20 for inventory and an HS-32 for operating. Items 64 and 66 of the Structure Inventory and Appraisal Sheet (SI&A) would be 236 and 257, respectively (*4*).

The moment capacity of a T-beam is influenced more by a change in steel strength than by a change in concrete strength. By using 50 ksi steel instead of 40 ksi steel, the moment capacity was increased by 24 percent, as indicated in Table 8. The moment capacity of the T-beam was increased only 0.8 percent by using 4,000 psi instead of 3,000 psi concrete, and 1.3 percent

Table 5. Shear capacity ratings for T-beam.

	Load Factor Method	Service Load Method	Percent Difference (LF Base)
	Rating	Factors	
Inventory	1.1	1.1	0
Operating	1.8	1.6	-11.1
	Capacity	Ratings	
Inventory	HS 21	HS 22	
Operating	HS 36	HS 31	

Table 6. Bearing capacity ratings for T-beam.

	Load Factor Method	Service Load Method	Percent Difference (LF Base)
	Rating	Factors	
Inventory	2.6	1.5	-42
Operating	4.3	2.5	-42
	Capacity	Ratings	
Inventory	HS 52	HS 30	
Operating	HS 86	HS 50	

by using 5,000 psi concrete instead of 3,000 psi as is shown in Table 9.

Table 10 shows that shear capacity increases about 7.0 percent as the concrete strength is stepped up from 3,000 to 4,000 psi. As is indicated in Table 11, the shear capacity is increased by 12 percent as the steel strength is stepped up from 40 ksi to 50 ksi and by 24 percent by using 60 ksi steel.

From the previous discussion, it can be concluded that steel strength has more influence on moment and shear capacity than does concrete strength.

Bearing capacity is almost directly proportional to concrete strength. As the concrete strength is stepped up from 3,000 to 4,000 psi, the bearing capacity of the T-beam is increased by one-third, as indicated in Table 12. Note that the moment and shear capacities are highly dependent on steel strength, which is more predictable than concrete strength, and they have a smaller factor of safety than bearing, which depends on concrete strength.

5.2.2 Slab Bridge

A slab bridge has the following properties:

Clear span	15 ft
Clear width	26 ft
Rating vehicle	HS-20
Concrete strength, f'_c	3,000 lb per sq in.
Reinforcing steel	Grade 40 steel
Wearing surface	3 in. asphalt

Table 13 gives the moment capacity rating for the slab bridge. The service load method yielded about a 2 percent lower inventory rating and about a 20 percent lower operating rating. The load factor rating for inventory was HS-20 and HS-34 for operating.

AASHTO specifications state that slabs designed for bending moment may be considered satisfactory in bond and shear. By checking the maximum shear stress for the slab in this example, one can confirm that the nominal shear stress value is below that which can be carried by the concrete alone. The maximum shear stress created at a distance d (10.5 in.) from the face of the support is 82.5 psi, which is less than that allowed in the concrete (110 psi).

Table 7. Capacity ratings for T-beam—load factor method.

	Moment	Shear	Bearing
	Rating	Factors	
Inventory	1.72	1.1	2.6
Operating	2.87	1.8	4.3
	Capacity	Ratings	
Inventory	HS 34	HS 21*	HS 52
Operating	HS 57	HS 36*	HS 86

*Controls

Table 8. Moment capacity of T-beam as a function of steel strength.

$f_{y(ksi)}$	M_U* (k-ft)	Change (%)
40	2765	---
50	3430	24
60	4082	48

*f'_c=3000 psi

5.2.3 Arch Bridge

An arch bridge is analyzed by dividing the arch into segments. Each segment is treated as a column subject to both axial load and bending moment.

A 132-ft span arch with a rise of 16 ft has been analyzed using the ultimate load method. The structure was built and designed around 1920 and has the following properties:

Table 9. Moment capacity of T-beam as a function of concrete strength.

f'_c (psi)	Mu* (k-FT)	(%) Change
3000	2765	---
4000	2787	+0.8
5000	2800	+1.3

*fy = 40 ksi

Table 11. Shear capacity of T-beam as a function of steel strength.

f_y (ksi)	V_s (kips)	V_u* (kips)	Change (%)
40	90	158	---
50	112.5	177	12.0
60	135	196	24.0

*f'_c=3000 psi

Table 12. Bearing capacity of T-beam as a function of concrete strength.

f'_c	R_b	Change (%)
3000	173	---
4000	230	33
5000	288	66

Table 13. Moment capacity rating for slab bridge.

	Load Factor Method	Allowable Stress Method	Percent Difference
	Rating	Factors	
Inventory	1.02	1.0	-2
Operating	1.70	1.64	-3
	Capacity	Ratings	
Inventory	HS 20	HS 20	
Operating	HS 34	HS 33	

Table 10. Shear capacity of T-beam as a function of concrete strength.

f'_c	V_c	V * (KIPS)	Change (%)
3000	96	158	---
4000	109	169	7.0
5000	120	179	13.3

*fy = 40 ksi

Depth at springing	5 ft 7.5 in.
Depth at crown	2 ft 6 in.
Concrete strength	3,000 lb per sq in.
Reinforcing steel, f_y	33 kips per sq in.
Area of tension steel	1.56 sq in.
Area of compression steel	1.56 sq in.

The first essential step is to obtain a method for analyzing the structure. Textbooks in structural analysis are available in assisting with the analysis of a typical arch structure (*40, 41, 42, 43*). The use of a computer, including a STRUDL analysis, can also be helpful (*44*). This arch structure was analyzed by using influence lines for maximum moment and thrust from Ref. *41*.

Table 14 gives the operating and the inventory ratings obtained. An inventory rating of HS-40 and an operating rating of HS-67 was obtained. High rating for concrete arch bridges based on the arch itself are not uncommon.

The effects of concrete strength on the axial load and moment capacity of a compression member are given in Table 15. With an increase of 33 percent in concrete strength, the ultimate axial load capacity was increased 32 percent whereas the ultimate moment was essentially unchanged. This shows that the axial capacity of a compression member is almost a linear function of concrete strength.

Steel strength had a different effect. Table 16 indicates that with a 21 percent change in steel strength, the moment capacity was increased 21 percent, while the ultimate axial capacity was virtually unchanged.

5.2.4 Box Culvert

A two-cell reinforced concrete box culvert was analyzed. To show the effects of live load as a function of depth, the box culvert was analyzed at depths of 2 ft, 6 ft, and 16 ft.

Table 17 gives capacity ratings for the structure. As the depth of fill increases, the effects of live load decrease and the capacity rating increases.

5.3 REINFORCED CONCRETE DETERIORATION EFFECTS

Many bridges across the country, including recently built ones, show some degree of deterioration. Deterioration is any significant change in physical, chemical, or mechanical property of a structure or of any of its components.

Many bridges in use today are constructed of reinforced concrete. Reinforced concrete is a low-cost, durable, multi-use building material. Unfortunately, the effects of inadequate design, defective materials, or unsatisfactory maintenance have resulted in the relatively rapid deterioration of reinforced concrete structures.

5.3.1 Deterioration of Concrete Bridge Decks

More than any other part of the bridge, the deck is subject to the adverse effects of traffic, weathering, and chemical action. The repeated use of deicing agents and abrasives on bridge decks is a major cause of deterioration.

Some of the indicators of deterioration in reinforced concrete decks include (*45, 46, 47*): cracking, scaling, spalling, delamination, leaching, deformations, stains, ride quality, and patching or other repairs.

Some of the factors causing deterioration in concrete include: deicing salts (moisture-related), corrosion, leaching (spalls), large loads, environment, traffic impact, reactive material, and foundation settlement.

5.3.2 Field Investigation

Rating a structure for its safe load-carrying capacity begins with a thorough field investigation. All physical features that have an effect on its structural capacity need to be examined for their condition. Any damage or deterioration needs to be noted and recorded. Cracking, scaling, spalling, and leaching are usually the first visual indications that some type of deterioration has occurred.

A crack by itself does not pose a significant structural problem, but it does provide an easy path for chloride-laden water and other contaminants to reach the reinforcing steel, which eventually leads to deterioration of the bridge structure.

Scaling is the loss of surface mortar and is usually an indication of improper construction techniques or the use of defective material. It may also be the result of inadequate maintenance or inadequate deck drainage.

A rough circular or oval depression caused by separation and removal of a portion of the surface concrete is called spalling. Spalls result from the large tensile forces within the concrete and internal stress concentrations. The expansive tensile force is caused by the corrosion of reinforcing bars or freezing of concrete deck slab.

Leaching is the formation of stains, efflorescence, and incrustation at a crack under the deck. In extreme cases, it could have a formation of stalactites. It is apparent that the corrosion of reinforcement is progressing or imminent.

Salt and other deicing chemical agents are the greatest cause of concrete deterioration. Chemical agents cause disintegration of concrete and separation of aggregate as well as promotion of corrosion of reinforcing steel.

5.3.3 Load Carrying Capacity

The load-carrying capacity of a bridge is generally dependent on moment, shear, and bearing. Which of these three is critical depends on the particular design and loading situation of the structure.

Deterioration in the superstructure does not necessarily reduce its load-carrying capacity. A spall or other loss of section, for example, near the support of a single span bridge does not necessarily affect the moment capacity because the deterioration has occurred in an area of low stresses away from the critical moment location. A spall at the midspan section of the same bridge could directly affect the load-carrying capacity because the midspan is the area of high moment. The actual amount of capacity reduction depends on both the design and location on the cross section.

Regions of maximum moment or shear have high stresses, while regions of low moment or shear have low stresses. Consequently, the location, both longitudinally and at the particular

Table 14. Capacity rating for arch bridge.

	Rating Factor	Capacity Rating
Inventory	2.0	HS40
Operating	3.34	HS67

Table 15. Capacity of compression members as a function of concrete strength.

f'c (psi)	Change (%)	Pu (kips)	Change (%)	Mu (k-ft)	Change (%)
3000	---	1210	---	237	---
4000	33	1594	32	237	0
5000	66	1978	64	237	0

Table 16. Capacity of compression members as a function of steel strength.

fy (ksi)	Change (%)	Pu (kips)	Change (%)	Mu (k-ft)	Change (%)
33	---	1210	---	237	---
40	21	1222	1.0	297	25
50	52	1240	2.5	370	56
60	82	1257	3.9	442	86

Table 17. Capacity analysis of box culvert.

Depth of Fill (ft.)	Rating Factor		Capacity Rating	
	Inv.	Opr.	Inv.	Opr.
2	1.0	1.6	HS20	HS32
6	2.40	4.00	HS48	HS80
16	Very Large		Very Large	

section, and the intensity of deterioration must be taken into account when determining the load-carrying capacity of the superstructure.

5.3.3.1 Moment Capacity. Spalling generally causes a reduction in cross section. Where spalling exposes reinforcing bars, the strength of the beam or girder may or may not be reduced. The severity of this condition should be considered along the entire length of the beam, and most particularly at the locations of maximum flexure, maximum shear, as well as at the end bearing point.

The moment-carrying capacity of a concrete structure is directly affected by a loss in reinforcing steel area. That is, the loss in steel area is proportional to the loss in moment capacity.

The moment-carrying capacity of a concrete structure is also affected by a change in the internal moment-arm. If concrete deterioration occurs on the tension side and the steel loses little or no cross section, the moment arm changes little if at all. However, if spalling occurs on the compressive side, the resultant compressive force moves toward the steel, thus shortening the moment-arm and causing an increase in both the concrete and steel stresses. Whether or not this increase is significant depends a great deal on the particular section. In many bridges with composite decks the actual concrete stresses are so low the increase in stress is insignificant. However, the shortening of the internal moment-arm can cause a small reduction in the ultimate moment capacity, because the controlling factor, the tensile force in the steel, does not change.

In determining the moment-carrying capacity of a reinforced concrete structure, the type and location of deterioration are important. Generally, if deterioration has occurred in a low-stress zone, the resulting situation is not considered critical in rating the structure. Good engineering judgment must be exercised in determining the severity of the deterioration and its effects on the moment.

The transfer of the force between the reinforcement and the surrounding concrete is achieved by the development of bond stresses along the reinforcing bar and bond anchorage at the ends of a bar. Of particular importance in this discussion is the concept of anchorage bond. If the bars are not anchored in the concrete sufficiently so that the applied tensile force can be developed by bond between steel and concrete, the bars will pull out. In such cases, the moment capacity becomes a function of the embedment lengths of the steel bars in both directions as well as the area of the steel.

Random experiments have been carried out to show that beams with poorly bonded reinforcement differ considerably from the normal situation where bond does exist. A study of beams with insufficient reinforcement bond has shown that bond strongly influences flexure stress and deflection (*48*). The ultimate moment is influenced to a considerably lesser extent where underreinforced beams are concerned. This lack of influence on ultimate moment has sometimes led designers to attach little importance to bond, because it can generally be shown that the ultimate moment is influenced only to a slight extent by the slip of the reinforcement in the concrete if the ends are properly anchored. In addition to the increased deflection caused by defective bond, a pronounced and serious increase in cracking can occur.

A crack in the tension zone of the concrete implies that there has been a certain slip between the reinforcing bars and the surrounding concrete. Wide cracks can occur in beams with large smooth reinforcing bars under heavy loads.

If there is total loss of both cover and bond of reinforcing steel over a significant length of the bar, it is the overall (three-dimensional) volume that can affect the load-carrying capacity (*38*). However, studies made by Minkarah and Ringo show that a concrete beam that had lost 32 percent of its cover and bond maintained the same ultimate strength as a beam with no loss of cover. Loss of cover occurring over 30 percent of the span rarely occurs (*49*). A reduction in cross-sectional area of reinforcing normally accompanies loss of cover and this does reduce the load-carrying capacity.

In order to properly relate the effects of deterioration to the load-carrying capacity of a reinforced concrete bridge, a satisfactory scheme needs to be employed. It is suggested in Ref. *38* that when a longitudinal crack greater than 0.1 in. in width occurs in the concrete along a line parallel to the bar, assume that the bond is reduced by 10 percent over the length of the crack. When a longitudinal crack less than 0.1 in. in width exists, a bond reduction shall be assumed to vary linearly with the width of the crack from 10 percent at a width of 0.1 in. to no reduction at a crack width of 0.01 in. or less. These guidelines are important in evaluating anchorage of the reinforcing steel.

If sufficient anchorage exists, full steel stress can be considered developed. If more than half of the perimeter of a steel bar is exposed, it can be assumed that no bond exists between the bar and the concrete over the length of exposure. If less than half of the perimeter is exposed, the bond needs to be reduced in proportion to the percentage of the perimeter exposed. This reduction in bond is particularly important in the development of anchorage.

In computing the moment capacity of a beam, one can assume that a percentage of the steel has been lost if rust stains are present. This loss is usually no greater than 5 percent on a typical corroded bar (*38*). If there is reason to believe that more corrosion has occurred in the steel, various test methods are available which can assist in determining the actual loss (*14*).

Figure 21 shows a cross section of a typical T-beam bridge with examples of some common types of deterioration. Tables 18, 19, and 20 show the effects that the loss of concrete compressive area and steel tensile area have on the ultimate moment capacity of a beam. Calculations are simplified by reducing the depth of the slab an amount equal to the loss of area in the concrete compression section.

Table 18 shows the percent reduction in moment capacity for each ½ in. reduction in slab depth. The deck can suffer a 38

Table 18. Loss of concrete compressive area.

Slab Lost Depth (in)	% Deterior-ation	Effective Depth (d)	Reduced * Moment Cap.	% Cap. Reduction
1/2	6.25	50.3	2737	1.01
1	12.5	49.8	2709	2.03
1-1/2	18.75	49.3	2681	3.04
2	25.0	48.8	2653	4.05
2-1/2	31.3	48.3	2625	5.06
3	37.5	47.8	2597	6.07

*Full Section Moment Capacity = 2765 k-FT

Table 19. Loss of steel tensile area.

Number of Bars Lost	% Deterior-ation	**Effective Depth (d)	Reduced * Moment Cap.	% Cap. Reduction
1/2	4.2	50.6	2640	4.52
1	8.3	50.3	2509	9.25
1-1/2	12.5	50.0	2380	13.9
2	16.7	49.6	2248	18.7
2-1/2	20.8	49.2	2118	23.4
3	25.0	48.7	1986	28.2

**Assumes Bar Loss on Bottom

Table 20. Loss of compressive area plus loss of steel tensile area.

Slab Lost Depth (in)	Number of Bars Lost	% Deter.	Effec-tive Depth	Reduced Moment Cap.	% Reduc-tion
1/2	1/2	10.5	50.1	2612	5.53
1	1	20.8	49.3	2457	11.1
1-1/2	1-1/2	31.3	48.5	2307	16.6
2	2	41.7	47.6	2154	22.1
2-1/2	2-1/2	52.1	46.7	2007	27.4
3	3	62.5	45.7	1859	32.8

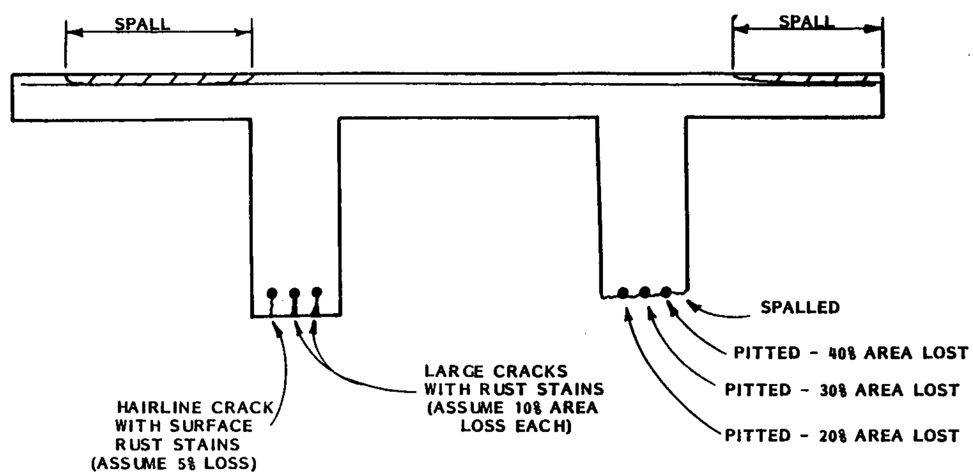


Figure 21. Deteriorated T-beam.

percent loss in cross-section area because of deterioration, but the girder experiences only a 6.07 percent reduction in moment capacity.

Table 19 shows the percent reduction in moment capacity for each ½ bar lost. Each ½ bar is a loss of total steel section of 4.2 percent. If the web loses one-half of a bar, it will suffer a 4.5 percent loss in moment capacity. For each complete bar that it loses, the T-beam will lose about 9 percent in moment capacity. That is, the loss is directly proportional to the amount of steel lost or the capacity is proportional to the remaining steel.

Table 20 shows the percent reduction in moment capacity for a simultaneous loss of slab depth and reinforcing bars. For each ½ in. reduction in slab depth (i.e., 6.25 percent loss in deck section) and ½ bar loss (i.e., 4.2 percent loss in reinforcing steel), the T-beam loses about 5 percent in moment capacity. Again the most important factor is the amount of steel lost, or assumed lost.

The calculations are for individual beams and therefore do not take into account any redistribution of loads to adjacent girders. Reasonably, the total capacity loss of a bridge would be smaller unless the other girders were equally deteriorated.

5.3.3.2 Shear Capacity. Cracks per se do not create a significant structural problem, except for shear-type cracks in a high shear region of the girder and cracks along the line of reinforcing that can affect bond and anchorage.

Generally, deterioration of concrete does not adversely affect the shear capacity of a structure. It can be demonstrated that deterioration can reduce capacity, although the analysis (evaluation) of the remaining shear capacity of a girder is difficult.

The following symptoms indicate that the shear capacity has been affected:

1. Wide diagonal cracks.
2. Rust stains.
3. Concrete spalling along a crack.
4. Vertical displacement across a crack.

The following guidelines have been suggested by Ref. *38* for the evaluation of shear capacity:

1. *Diagonal cracks*—Reduce shear capacity of the concrete based on the width of the crack up to a maximum of 100 percent for crack widths of ¼ in. or more.

Table 21. Shear capacity of T-beam with loss of concrete capacity.

Crack Width (in)	Deteriora-tion (%)	Shear Force (kips)	Reduced Shear (Ultimate) * (kips)	Capacity Loss (%)
1/16	25	67	133	12.5
1/8	50	45	115	25
3/16	75	22	95	38
1/4	100	0	77	50

*Ultimate Capacity = 189 kips

Table 22. Shear capacity of T-beam with loss of steel.

Steel Loss (%)	Shear of Steel (kips)	Reduced Shear * (Ultimate) (kips)	Capacity Loss (%)
5	85	148	2.5
10	81	144	5.0
15	76	141	7.5
20	72	137	10.0

*Ultimate Capacity = 189 kips

2. *Rust stains*—Reduce area of shear steel by 5 percent.
3. *Vertical displacement across a crack*—Neglect shear capacity of concrete and reduce area of steel by 10 percent.
4. *Spalled concrete along diagonal cracks*—Neglect shear capacity of concrete.

In calculating the change in shear capacity as a function of deterioration, the T-beam in Figure 21 is again used.

Table 21 shows the change of shear capacity as affected by diagonal cracks or spalling along diagonal cracks. For each 1/16-in. crack, the shear capacity of the concrete is reduced by 25 percent. This reduction results in a 12.5 percent loss of ultimate shear capacity. If the concrete shear capacity is reduced by 100 percent, the ultimate shear capacity of the section is reduced by 50 percent.

Table 22 shows the change of shear capacity as affected by rust stains. For each 5 percent reduction in steel area, the ultimate shear capacity of the T-Beam is reduced by 2.5 percent.

5.3.3.3 Bearing Capacity. Deterioration and damage to bridge bearings frequently reduce the load-carrying capacity of bridges. This deterioration and damage usually result in a loss of contact area between the load-carrying member and the support.

To illustrate a method to determine the safe load capacity in bearings, refer to the previous concrete T-beam. The original bearing length of the beams was 12 in., but, because of damage, the effective bearing length has been reduced.

Table 23 shows the reduction in bearing capacity with a reduction in effective bearing length. The loss in bearing capacity is directly proportional to the loss of effective bearing length.

Table 24 shows the reduction in bearing capacity with a reduction in effective bearing length, assuming loaded area is subjected to high edge stresses. The bearing capacity is reduced substantially as the effective length decreases. The calculations are for individual bearings and do not include redistribution of loads to the other girders. The operating and inventory ratings are typical.

5.3.3.4 Compression Members. Defects in compression members are similar to other concrete components. The resulting cracks, collision damage, and scour can, however, become critical.

Table 23. Bearing capacity with loss of bearing length.

Effec-tive Depth	Effec-tive Area	Bearing Capacity	Inven-tory Rating	Operating Rating
12	192	173	HS 50	HS 86
11	176	158	46	77
10	160	144	42	70
9	144	130	37	62
8	128	115	33	55
7	112	101	29	48
6	96	86	25	42
5	80	72	20	33
4	64	58	17	28

Table 24. Bearing capacity with loss of bearing length edges subjected to high stress.

Effec-tive Depth	Effec-tive Area	Bearing Capacity	Inven-tory Rating	Operating Rating
12	192	130	42	70
11	176	119	36	60
10	160	108	30	51
9	144	98	25	42
8	128	86	19	32
7	112	76	15	25
6	96	65	8	13
5	80	54	2	4
4	64	44	0	0

Table 25 shows the result of capacity reduction in a rectangular column with the loss of concrete. The axial load capacity loss is almost as large as the loss of concrete if one assumes a small moment. The moment capacity, however, is virtually unchanged with a loss of concrete because the steel forms a couple.

Table 26 shows the result of capacity reduction in a rectangular column with a loss of steel. If the loss is on the tension side, the axial capacity is virtually unchanged, while the loss of moment capacity is directly proportional to the loss of steel. Complete loss of steel in the tension side yielded a 4.4 percent loss of axial capacity, but a 100 percent loss in moment capacity. If the loss is on the compression side, the loss of axial capacity is only slight and the moment capacity is unchanged. A 100 percent loss of steel in the compression side contributed to only a 4.4 percent loss in axial capacity. The location of the lost steel (i.e., compression side, tension side) as well as whether the column is a steel-controlled or concrete-controlled section is, therefore, very critical to the load-carrying capacity of the section.

An interaction diagram for the actual damaged column would be most helpful in evaluating a particular column for capacity dependent on both axial load and bending moment. Figure 22 shows the modified interaction curve if there was a 3-in. loss of concrete from the concrete arch structure discussed in Appendix A. This 3-in. loss of concrete represents a 4 percent loss in gross area. Figure 22 shows that the ultimate axial capacity for the column is 320 kips and the ultimate moment is 770 kip-ft. The axial capacity was reduced by 17 percent and the moment capacity was reduced by 32 percent. The operating rating factor was reduced by 37 percent as indicated in Table 27.

Figure 23 shows the modified interaction curve if there is a total loss of steel from the tension side of the arch structure cross section. Table 28 shows a reduced ultimate axial capacity of 312 kips and a reduced ultimate moment of 655 kip-ft. The revised inventory rating factor of 1.1 and operating rating factor of 1.9 have been reduced by 43 percent from the original rating factor.

5.4 SUMMARY

This section has outlined a procedure for evaluating existing reinforced concrete bridges. In addition, several reinforced structures in use today have been described. As a consequence, the inspector can make a better evaluation of the effect of particular defects on the capacity of the bridge. This study has also described two methods, the Service Load Method and the Load Factor Method, that can be used in obtaining a capacity rating for a reinforced concrete bridge. Concrete deterioration and its effects on the load-carrying capacity have been discussed.

With care, the capacity of reinforced concrete bridge structures can be determined by existing rational methods. The load factor method of rating a reinforced concrete bridge is fast and easy to perform and gives results much like those for the allowable stress method.

The moment and shear capacity of a T-beam is influenced more by a difference in steel strength than by a difference in concrete strength. The strength of concrete, if not deteriorated, has little effect on the capacity of typical reinforced concrete bridges.

If there is a loss of section, the moment capacity of a typical reinforced concrete beam is influenced more by a loss of tension

Table 25. Capacity of rectangular column with loss of concrete.

Loss (in.)	Loss (%)	Pu** (kips)	Capacity Loss (%)	Mu* (K-ft)	Capacity Loss (%)
1	1.5	1193	1.5	246	0
2	3.0	1180	2.5	246	0
3	4.5	1163	3.9	246	0

**Pu = 1210 kips *Mu = 246 k-ft

Table 26. Capacity of rectangular column with loss of steel.

Tension Side

Bar Loss	Loss (%)	Pu** (kips)	Capacity Loss (%)	Mu* (K-ft)	Capacity Loss (%)
1/4	25	1196	1.2	185	25
1/2	50	1183	2.2	124	50
3/4	75	1170	3.3	62	75
1	100	1157	4.4	0	100

Compression Side

Bar Loss	Loss (%)	Pu** (kips)	Capacity Loss (%)	Mu* (K-ft)	Capacity Loss (%)
1/4	25	1196	1.2	246	0
1/2	50	1183	2.2	246	0
3/4	75	1170	3.3	246	0
1	100	1157	4.4	246	0

**Pu = 1210 kips *Mu = 246 k-ft

Table 27. Capacity of arch structure with loss of concrete.

Loss of Concrete (in)	(%) Loss	Ultimate Axial Capacity (kips)	Ultimate Moment (k-ft)	Rating** Factor	Capacity Rating
3	4	320	700	1.2 (inv) 2.1 (opr)	HS25 HS41

**Original Rating Factor (Inv. = 2.0 and Opr. = 3.34)

Table 28. Capacity of arch structure with loss of steel.

Loss of Steel (sq. in.)	(%) Loss	Ultimate Axial Capacity (kips)	Ultimate Moment (k-ft)	Rating** Factor	Capacity Rating
1.56	100	312	655	1.1 (inv) 1.9 (opr)	HS23 HS38

**Original Rating Factor (Inv. = 2.0 and Opr. = 3.34)

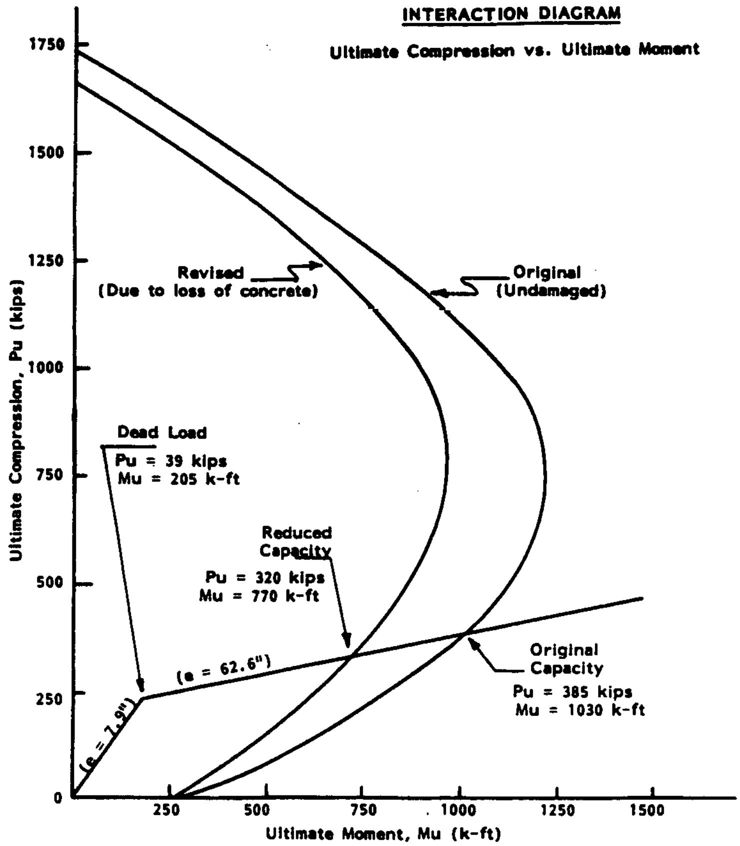


Figure 22. Interaction diagram with loss of concrete.

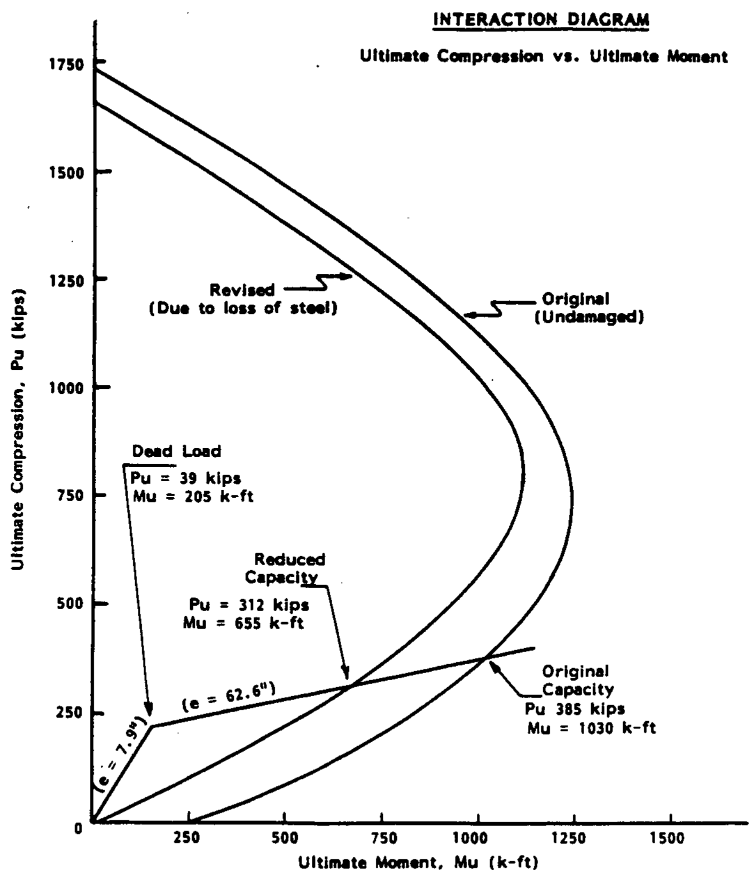


Figure 23. Interaction diagram with loss of steel.

steel than by a loss of concrete on the compressive side. The axial load capacity loss of a column is almost directly proportional to the loss of concrete.

The effects on axial load capacity, however, are influenced greatly by the compressive strength of concrete. The axial load capacity is almost a linear function of concrete strength.

A reinforced concrete beam can experience a substantial loss of cover and bond and yet experience little loss in bending strength. The ultimate moment of a reinforced concrete beam is influenced only to a slight extent by a loss of bond in the concrete, provided the ends are properly anchored.

The most reliable method in dealing with deterioration is to base the reduced strength on actual physical measurements of loss of cross-sectional area. Deterioration in the superstructure, however, does not necessarily reduce the load-carrying capacity. For instance, a deck can experience a considerable loss of concrete in the compression side and experience only a slight reduction in moment capacity. A loss of steel in the tension side of a girder has a greater effect in the reduction of moment capacity.

Shear cracks in a high shear area can be more serious than the flexure cracks that are produced in reinforced concrete under normal bending conditions.

The loss of bearing capacity is almost directly proportional to a loss in bearing length, hence directly affects capacity.

Deterioration leading to a loss of concrete is very critical to a compression member. Even though the moment capacity can be virtually unchanged, the axial load capacity of a compression member is almost directly proportional to the loss in concrete.

(The discussion and examples as well as Appendix A referred to in this section are from Ref. *50*.)

5.5 PRESTRESSED CONCRETE EXAMPLE RESULTS

Appendix B contains a series of sample calculations for determining bridge ratings of prestressed concrete bridges. The results of various capacity analyses are discussed in this section. The intent is to demonstrate the methods that can be used in the capacity analysis of various types of prestressed bridges. The bridges discussed include a normal simple span bridge with a composite deck and a simple span made continuous for live load. Modes of failure, such as shear, are not repeated for all examples because the analysis would be the same for all types.

5.5.1 Normal Prestressed Beam Bridge

Examples 1 and 2 in Appendix B are for a two-lane structure typical of many of the prestressed concrete bridges built at grade separations in the United States. The bridge superstructure consists of pretensioned AASHTO girders acting in a composite manner with a cast-in-place deck.

Example 1 uses the service load design method (allowable stress method) of rating the structure. Flexure is considered for both the inventory and operating rating in the evaluation. The inventory rating is controlled by the girder; however, the operating rating is controlled by the deck.

The bridge used in example 1 is again used in example 2. Example 2 is rated using the strength design method (load factor method). The deck is rated for flexure and an interior girder is rated for flexure, flexure cracking, and shear. Shear was found to be the controlling criterion for this analysis.

A comparison of examples 1 and 2 indicates that the two methods produced capacity ratings that are comparable in magnitude. The strength design method produced ratings slightly higher than the service load method which could possibly be predicted. The results are given in Table 29.

Table 29. Comparisons of bridge ratings for examples 1 and 2.

Failure Mode and Location	Allowable Stress		Load Factor	
	Inv	Opr	Inv	Opr
Deck - Flexure	HS21	HS31	HS19.6	HS32.5
Interior Girder - Flexure	HS21	HS45	HS28	HS47
Crack Control (SDM only)	-	-	HS22	-
Interior Girder - Shear				
h/2 from Support	-	-	HS19.5	HS33
$L_s/4$ from Support	-	-	HS23	HS38

5.5.2 Defective Bridge Members

All bridges, unless they are relatively new, will invariably have some degree of deterioration or possibly damage. Depending on the nature, degree, and location, defects may affect the load-carrying capacity of bridge members. This section contains information on the types of defects likely to be encountered on prestressed concrete bridge superstructures, effects on capacity, and recommendations on how to account for them in a load capacity rating. The recommendations are in general valid for both the service load design and the strength design methods. Appendix C contains several bridge rating examples of bridges with defective members.

Bridge inspection reports are usually the source of information regarding defects for a bridge which is to be rated. The report should contain all necessary information concerning defects, such as locations, size, and patterns of concrete damage; severity of cracks and spalls; damage to mild reinforcing steel and locations; and damage to prestress strands, number of severed or yielded strands, and locations (*45*).

Generally, it is the judgment of the engineer in charge that determines which defects are purely cosmetic, and which may significantly affect the structural response and load capacity of the bridge. Both the magnitude and location of defects must be considered carefully when determining capacity. In general, load-carrying capacity will be affected when defects result in any of the following (*38*):

1. Loss of material from a critical section.
2. Change in material properties which reduces ultimate stress levels.
3. Loss of continuity (composite action).
4. Stress concentrations (pitting of steel reinforcement).
5. Loss of stability and load redistribution due to a nonuniform loss of section.

Reinforced concrete decks on highway bridges have two functions: (1) the deck distributes load to the girders, and (2) the deck serves as the compressive flange on composite bridges (*38*).

It is on the basis of the first function, namely load distribution to the girders, that rating of the deck itself is concerned. It has been concluded from tests (*51,52*) that reinforced concrete decks are greatly overdesigned. Because of this overdesign, except in extreme cases, the deck rating need not consider normal deterioration or defects. Deck defects are rarely a capacity problem except when a loss of reinforcing steel is combined with a loss of appreciable concrete section. Deterioration in the deck will need to be considered when using the deck as part of a composite girder (*38*).

5.5.2.1 Types of Defects. Among older prestressed concrete bridges, defects due to inadequate design, fabrication, and construction procedures occasionally occur. Although design methods have been improved, defects due to construction methods still persist. Distresses attributable to these causes include horizontal and vertical cracking in the end blocks due to high tensile stresses and frozen bearings. Also included are cracking in or near girder flanges resulting from settlement of materials during casting, insufficient stirrups, or insufficient concrete cover (*47*). Although not as frequent now, some of these design or construction defects may show up on bridge members.

Currently the greatest single source of defects in prestressed concrete bridge members is traffic damage. In a survey conducted by Shanafelt and Horn (*53*), it was reported that approximately 80 percent of the prestressed concrete bridge girders reported damaged were damaged by overheight vehicles. Other causes included fabrication and storage defects, fire, and environmental factors.

Accidental damage to prestressed concrete girders usually occurs in the exterior or the first interior girders. Damage may be classified as minor, moderate, severe, or critical. These classifications are defined as follows (*53*):

Minor damage—Surface spalls and minor concrete cracking only, no exposed strand (damage to concrete only).
Moderate damage—Extensive spalling and fine-to-medium size cracks in flange, exposed strand or reinforcing, but no severed strands (still damage to concrete only).
Severe damage—Major loss of flange section, loss of strands and deformed strands, cracks from flange to web, loss of web but not at same location as loss of flange, horizontal and vertical misalignment.
Critical damage—Extensive loss of strands and prestress force that cannot be restored, wide flange cracks extending to web, abrupt lateral and vertical deflections, wide cracks indicating steel yielding.

Figure 24 shows a composite girder cross section and several typical types of defects.

5.5.2.2 Considering Defects in Capacity Ratings. The most important type of deck defect as far as load carrying capacity is concerned is reinforcement corrosion and resulting loss of tensile steel area. The losses in concrete cross-section area because of scaling are small and may be disregarded. Similarly, concrete cracking generally has little effect on flexural capacity.

If corrosion has produced a small loss in reinforcement cross-section area, or if rust stains are present, it is recommended (*38*) that a 5 percent loss in steel area be assumed. Mild reinforcement that has been exposed due to spalling of the concrete

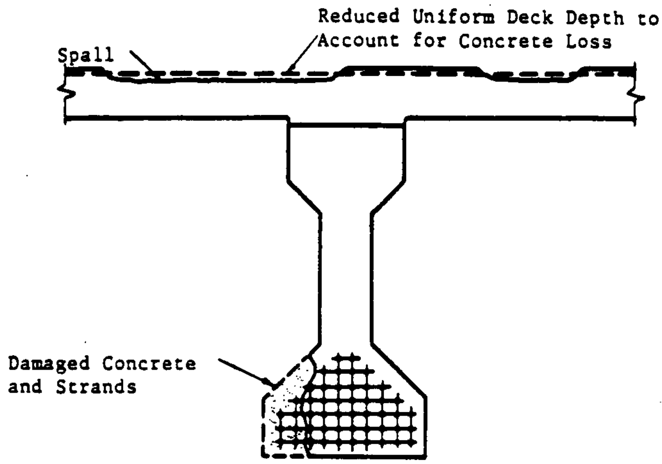


Figure 24. Typical cross section with defects.

cover may in some cases suffer a loss of bond between the steel bars and the surrounding concrete. Loss of appreciable bond may affect the ability of the reinforcement to achieve yield stress before pulling out of the concrete. This may be significant in regions of high stress. When this occurs, the steel area of exposed bars should be reduced in proportion to the amount of stress that cannot be developed (*38*).

Concrete spalling reduces the cross-section area of the deck. Recommendations are that the depth of the deck be reduced such that the area deleted is equal to the area lost due to spalling and undersurface fracture planes. The undersurface fracture plane is assumed to coincide with the top of the upper layer of steel reinforcement in the deck (*38*).

When damage to a girder occurs, the determination of flexural capacity should account for any significant losses of either concrete or prestress steel tendons. All concrete that has spalled, is loose, or has shattered must be considered to be nonexistent. Calculation of member cross-section properties should be based on the net remaining area. If damage extends to the prestress strands (severe or critical damage), all strands that have been damaged in any way must be considered to be ineffective. This damage includes strands that are cut, are pitted, have abrasions, or have permanent deformations (*53*).

Foundation settlement can result in additional stresses and deformations on continuous structures. Methods such as moment distribution or slope-deflection can be used to determine the settlement-induced forces. In the rating equations such forces are handled in much the same way as the secondary forces on a continuous member. That is, the magnitude of any settlement-induced forces is, with due regard for the signs of the quantities, subtracted from the member capacity.

The shear capacity of a section will not normally be greatly affected by defects unless any of the following coincide with high shear regions (*38*): wide diagonal cracking, rust stains (indicating corrosion of the stirrups), vertical displacement across a crack, or spalling along a crack. The following guidelines are recommended in Ref. *38* for the calculation of shear capacity of concrete members with such defects.

1. If diagonal cracks are present, reduce the concrete strength

V_c at that section based on the width of the crack up to a maximum of 100 percent of widths of ¼ in. or greater.

2. If there are rust stains, reduce the steel area A_v by 5 percent.
3. If there is vertical displacement across a crack, neglect the concrete strength V_c at the section and reduce the shear steel A_v by 10 percent.
4. For spalled concrete along a diagonal crack neglect the concrete strength V_c at that section.

If the field data or bridge plans indicate missing stirrups, a stirrup spacing greater than is currently allowed by AASHTO, or the amount of shear steel is below the recommended minimum value, the steel shear force V_s should be omitted from the calculations at that section. Furthermore, if spalling of the deck on a composite girder results in a decreased deck thickness used in capacity calculations, the reduced section height h should be used. The aforementioned applies when the effective depth d is taken equal to 0.8 times the section height h.

Bearing capacity is directly proportional to the bearing area. When deterioration or damage reduces this area, only the net remaining bearing area (after deducting all missing, loose, or shattered concrete) should be used in the calculation of capacity. Also, if additional bearing stresses are caused by excessive girder deflections, the capacity must be multiplied by a factor of 0.75. The factor A_2/A_1 is still applicable to deteriorated bearing areas.

If the live load distribution to the girders is altered by damaged or deteriorated members, the distribution factor should be altered accordingly. In this case it is expected that little or no live load distribution will occur should a line of wheel loads be placed on the defective girder (*45*). Therefore, when analyzing a beam on such a bridge, the distribution factor for live loads should be 1.0. On girder bridges the outside, or fascia girder, in some cases requires special considerations. According to Park (*45*) when a damaged fascia girder is being rated, it may be possible to assume no vehicle live load distribution to the girder. Other factors, such as damage to adjacent members, will need to be considered when deciding on the distribution factors.

5.6 EXAMPLE RATINGS OF BRIDGES WITH DISTRESSES

Appendix C gives examples of analysis of the prestressed bridge used in examples 1 and 2 of Appendix B after undergoing traffic damage. Example 1 uses the service design load method; example 2 uses the strength design method. Flexure only is checked in example 1, whereas both flexure and shear are checked in example 2. The examples are fictitious and used as examples demonstrating methods only.

The examples presented in the appendixes are for the demonstration of methods only. They are not intended to produce any new processes or techniques, but utilize the methods as can be interpreted from the AASHTO Manual. It is hoped that they can be of assistance in the analysis of such bridges and components.

(The discussion and examples as well as Appendix B and Appendix C referred to in this section are from Ref. *55.*)

REFERENCES

1. Federal Register, "Subpart C—National Bridge Inspection Standards." Source: 36 FR 7851, Apr. 27, 1971, unless otherwise noted. Redesignated at 39 FR 10430, Mar. 20, 1974.
2. AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS, *Manual for Maintenance Inspection of Bridges.* Washington, D.C. (1983) p. 50.
3. FEDERAL HIGHWAY ADMINISTRATION, U. S. Department of Transportation, *Bridge Inspector's Training Manual 70.* Washington D.C. (1970, corrected reprint 1979) p. 181.
4. FEDERAL HIGHWAY ADMINISTRATION, U. S. Department of Transportation, "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges." Washington, D.C. (1979) p. 38.
5. BEAL, D. B., and CHAMBERLIN, W. P., "Load Capacity of Concrete Bridge Decks." *Transportation Research Record 871,* Transportation Research Board, Washington, D.C. (1982) p. 70.
6. ROBINSON, R., "Cable Stays Catch On." *Civil Engineering Magazine,* American Society of Civil Engineering, Vol 56, No. 6, N. Y., N. Y. (June 1986).
7. PORTLAND CEMENT ASSOCIATION, "Design and Control of Concrete Mixes." Skokie, Ill. (1979) p. 139.
8. AMERICAN CONCRETE INSTITUTE, "Materials and General Properties of Concrete MCP-1." *Manual of Concrete Practice.* Detroit, Mich. (1986) p. 688.
9. AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS, "Standard Specification for Concrete Made by Volumetric Batching and Continuous Mixing." *Standard Specifications for Transportation Materials and Methods of Sampling and Testing,* 14th Edition, Part 1, Specifications. Washington, D.C. (1986) pp 687–697.
10. ABRAMS, M., "Performance of Concrete Structures." Portland Cement Association, Ninth SAMPE Technical Conference, Atlanta, Georgia (Oct. 4, 1977).
11. SALSE, E., and LIN, T., "Structural Fire Resistance of Concrete." Portland Cement Association, *J. Struct. Div.,* American Society of Civil Engineers (Jan. 1976) pp. 51–63.
12. FEDERAL HIGHWAY ADMINISTRATION, U. S. Department of Transportation, *Manual for Uniform Traffic Control Devices.* Washington, D.C. (1978).

13. AMERICAN SOCIETY FOR TESTING AND MATERIALS, "Test Method for Half Cell Potentials of Reinforcing Steel in Concrete." *Annual Book of ASTM Standards,* ASTM Designation: C 876, Section 4, Volume 04.02, Philadelphia, Pa. (1986).
14. MANNING, D. G., "Detecting Defects and Deterioration in Highway Structures." *NCHRP Synthesis of Highway Practice 118,* Transportation Research Board, National Research Council, Washington, D.C. (July 1985).
15. AMERICAN SOCIETY FOR TESTING AND MATERIALS, "Test Method for Rebound Number of Hardened Concrete." *Annual Book of ASTM Standards,* ASTM Designation: C 805, Section 4, Volume 04.02, Philadelphia, Pa. (1986).
16. AMERICAN SOCIETY FOR TESTING AND MATERIALS, "Test Method for Penetration Resistance of Hardened Concrete." *Annual Book of ASTM Standards,* ASTM Designation: C 803, Section 4, Volume 04.02, Philadelphia, Pa. (1986).
17. *Bridge Inspection Guide,* Department of Transportation, Scottish Development Department, Welsh Office, Northern Ireland, Her Majesty's Stationery Office, London (1983).
18. AMERICAN SOCIETY FOR TESTING AND MATERIALS, "Test Method for Electrical Resistivity of Membrane Pavement Systems." *Annual Book of ASTM Standards,* ASTM Designation: D 3633, Section 4, Volume 04.03, Philadelphia, Pa. (1986).
19. ALONGI, A. V., ET AL., "Concrete Evaluation by Radar, Theoretical Analysis." *Transportation Research Record 853, Concrete Analysis and Deterioration,* Transportation Research Board, National Research Council, Washington, D.C. (1982).
20. AMERICAN SOCIETY FOR TESTING AND MATERIALS, "Methods of Obtaining Drilled Cores and Sawed Beams of Concrete." *Annual Book of ASTM Standards,* ASTM Designation: C 42, Section 4, Volume 04.02, Philadelphia, Pa. (1986).
21. AMERICAN SOCIETY FOR TESTING AND MATERIALS, "Method for Measuring Length of Drilled Concrete Cores." *Annual Book of ASTM Standards,* ASTM Designation: C 174, Section 4, Volume 04.02, Philadelphia, PA (1986).
22. AMERICAN SOCIETY FOR TESTING AND MATERIALS, "Recommended Practice for Examination and Sampling of Hardened Concrete in Construction." *Annual Book of ASTM Standards,* ASTM Designation: C 823, Section 4, Volume 04.02, Philadelphia, Pa. (1986).
23. AMERICAN SOCIETY FOR TESTING AND MATERIALS, "Test Method for Cement Content of Hardened Portland Cement Concrete." *Annual Book of ASTM Standards,* ASTM Designation: C 85, Section 4, Volume 04.02, Philadelphia, Pa. (1986).
24. AMERICAN SOCIETY FOR TESTING AND MATERIALS, "Recommended Practice for Microscopical Determination of Air-Void Content and Parameters of the Air-Void System in Hardened Concrete." *Annual Book of ASTM Standards,* ASTM Designation: C 457, Section 4, Volume 04.02, Philadelphia, Pa. (1986).
25. AMERICAN SOCIETY FOR TESTING AND MATERIALS, "Test Method for Specific Gravity, Absorption, and Voids in Hardened Concrete." *Annual Book of ASTM Standards,* ASTM Designation: C 642, Section 4, Volume 04.02, Philadelphia, Pa. (1986).
26. AMERICAN SOCIETY FOR TESTING AND MATERIALS, "Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression." *Annual Book of ASTM Standards,* ASTM Designation: C 469, Section 4, Volume 04.02, Philadelphia, Pa. (1986).
27. AMERICAN SOCIETY FOR TESTING AND MATERIALS, "Test Method for Fundamental Transverse, Longitudinal and Torsional Frequencies of Concrete Specimens." *Annual Book of ASTM Standards,* ASTM Designation: C 215 Section 4, Volume 04.02, Philadelphia, Pa. (1986).
28. AMERICAN SOCIETY FOR TESTING AND MATERIALS, "Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens." *Annual Book of ASTM Standards,* ASTM Designation: C 496, Section 4, Volume 04.02, Philadelphia, Pa. (1986).
29. AMERICAN SOCIETY FOR TESTING AND MATERIALS, "Practice for Petrographic Examination of Hardened Concrete." *Annual Book of ASTM Standards,* ASTM Designation: C 856, Section 4, Volume 04.02, Philadelphia, Pa. (1986).
30. AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS, "Resistance of Concrete to Chloride Ion Penetration." *Standard Specifications for Transportation Materials and Methods of Sampling and Testing,* AASHTO Designation: T259, Part II, Washington, D.C. (1986).
31. AMERICAN SOCIETY FOR TESTING AND MATERIALS, "Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals." *Annual Book of ASTM Standards,* ASTM Designation: C 672, Section 4, Volume 04.02, Philadelphia, Pa. (1986).
32. AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS, "Rapid Determination of the Chloride Permeability of Concrete." *Standard Specifications for Transportation Materials and Methods of Sampling and Testing,* AASHTO Designation: T277, Part II, Washington, D.C. (1986).
33. AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS, "Sampling and Testing for Total Chloride Ion In Concrete and Concrete Raw Materials." *Standard Specifications for Transportation Materials and Methods of Sampling and Testing,* AASHTO Designation: T268, Part II, Washington, D.C. (1986).
34. AMERICAN SOCIETY FOR TESTING AND MATERIALS, "Methods of Tension Testing of Metallic Materials." *Annual Book of ASTM Standards,* ASTM Designation: E 8, Section 3, Volume 03.01, Philadelphia, Pa. (1986).
35. *A Guide for the Field Testing of Bridges,* American Society of Civil Engineers, N. Y., N.Y. (1980).
36. LADNER, M., "In Situ Load Testing of Concrete Bridges in Switzerland." *Strength Evaluation of Existing Concrete Bridges,* SP-88, American Concrete Institute, Detroit, Mich. (1985).
37. BAKHT, B., and CSAGOLY, P.F., "Diagnostic Testing of a Bridge." *J. Struct. Div.,* American Society of Civil Engineers, Vol 106, ST7, N. Y., N.Y. (1980).
38. IMBSEN, R. A., ET AL., "Strength Evaluation of Existing Reinforced Concrete Bridges." *NCHRP Report 292,* Transportation Research Board, National Research Council, Washington, D.C. (June 1987) 133 pp.
39. AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS, *Standard Specifications for Highway Bridges.* Washington, D.C. (1983).

40. PORTLAND CEMENT ASSOCIATION, "Analysis of Arches, Rigid Frame and Sewer Sections." Concrete Information ST 53, Skokie, Ill.
41. WILLIAMS, C. C., *The Design of Masonry Structures and Foundations.* McGraw-Hill Book Co., N. Y., N. Y. (1981).
42. URQUHART, L. C., and O'ROUKE, C. E., *Design of Concrete Structures.* McGraw-Hill Book Co., N. Y., N. Y. (1940).
43. LEONTOVICH, VALERIAN, *Frames and Arches.* McGraw-Hill Book Co., N. Y., N. Y. (1959).
44. STRUCTURES DIVISION AND CIVIL ENGINEERING SYSTEMS LABORATORY, Department of Civil Engineering, Massachusetts Institute of Technology, *ICES STRUDL, Frame Analysis,* Cambridge, Mass. (1968).
45. PARK, SUNG H., *Bridge Rehabilitation and Replacement.* S. H. Park, Trenton, N.J. (1984).
46. JOHNSON, S. M., *Deterioration, Maintenance, and Repair of Structures.* McGraw-Hill Book Co., N. Y., N. Y. (1965).
47. PARK, SUNG H., *Bridge Inspection and Structural Analysis.* S.H. Park, Trenton, N. J. (1980).
48. GRANHOLM, HJALMAR, *Reinforced Concrete.* John Wiley and Sons, N. Y., N. Y. (1965).
49. MINKARAH, C. F., ET AL., *Factors Affecting the Durability of Concrete Bridge Decks.* Miscellaneous Reports from Caltrans from 1971 to 1977, California Department of Transportation, Sacramento, Cal. (1977).
50. MADRID, C. D., *Capacity Evaluation of Reinforced Concrete Bridges.* Thesis submitted to the Graduate School, New Mexico State University in partial fulfillment of the requirements for the degree, Master of Science in Civil Engineering, Las Cruces, N. M. (Dec. 1986).
51. CSAGOLY, P., ET AL., "The True Behavior of Thin Concrete Bridge Slabs." *Proc., TRB Research Record 664,* Vol. 1, Transportation Research Board, Washington, D. C. (1978) pp. 171–179.
52. FULLARTON, D. H., AND EDMONDS, F. D., "Destructive Testing of the Mangateweka Stream Bridge." *Report No. 5-78/1,* New Zealand Ministry of Works and Development, Wellington, N. Z. (1978).
53. SHANAFELT, G. O., and HORN, W. B., "Damage Evaluation and Repair Methods for Prestressed Concrete Bridge Members." *NCHRP Report 226,* Transportation Research Board, Washington, D. C. (1980) 66 pp.
54. BEAL, D. B., and CHAMBERLIN, W. P., "Effects of Concrete Deterioration on Bridge Response." *Concrete Analysis and Deterioration, TRB Research Record 853,* Transportation Research Board, Washington, D.C. (1982) pp. 43–48.
55. KINCHEN, G. W., *Load Capacity Rating of Existing Prestressed Concrete Bridges.* Thesis submitted to the Graduate School, New Mexico State University in partial fulfillment of the requirements for the degree, Master of Science in Civil Engineering, Las Cruces, N. M. (May 1987).

APPENDIX A

CAPACITY CALCULATIONS

This appendix contains rating capacity calculations on four reinforced concrete structures: Example 1—Simple T-Beam (moment capacity, shear capacity, bearing capacity); Example 2—Simple Span Slab Bridge (moment capacity); Example 3—Reinforced Concrete Box Culvert (moment capacity); and Example 4—Reinforced Concrete Arch Bridge (moment capacity). Contained within each set of calculations is a dimensioned drawing of the reinforced concrete structure plus specified steel and concrete stresses.

EXAMPLE 1—SIMPLE T-BEAM

This example illustrates the process for finding the inventory and operating ratings for a selected simple T-beam reinforced concrete bridge.

The program is divided into three parts: moment capacity, shear capacity, and bearing capacity.

Each part (i.e., moment, shear, and bearing) is calculated using both the load factor method and the allowable stress method.

The dimensions and member properties of the T-beam are given in Figure A-1. The bridge was designed in accordance with AASHTO *Standard Specifications for Highway Bridges* in 1976.

The rating factor is calculated for an HS-20 vehicle. Normal traffic loadings are assumed. Problem variables include the following: $f_y = 40$ ksi, $f'_c = 3$ ksi, $b_w = 1$ ft 4-in., $d = 4$ ft 2.8 in., $t_s = 8$ in., $A_s = 18.72$ sq. in., and $n = 10$.

Moment Capacity—Load Factor Method

1. The total moment carrying capacity of the girder is given by: $a = T/0.85 f'_c b$; $a = (A_s \cdot f_y)/0.85 f'_c b$.

If a is equal to or less than the effective thickness of the slab, the slab and stem can be treated as a rectangular beam of width b and depth d, a very common occurrence.

The bending strength of the T-beam is controlled by the yield of the tensile steel: $a = [(18.72)(40)]/[(0.85)(3)(94)] = 3.12$ in.

Because the value of a is less than the slab thickness of 8 in., the treatment of the cross section as a rectangle is valid. Thus,

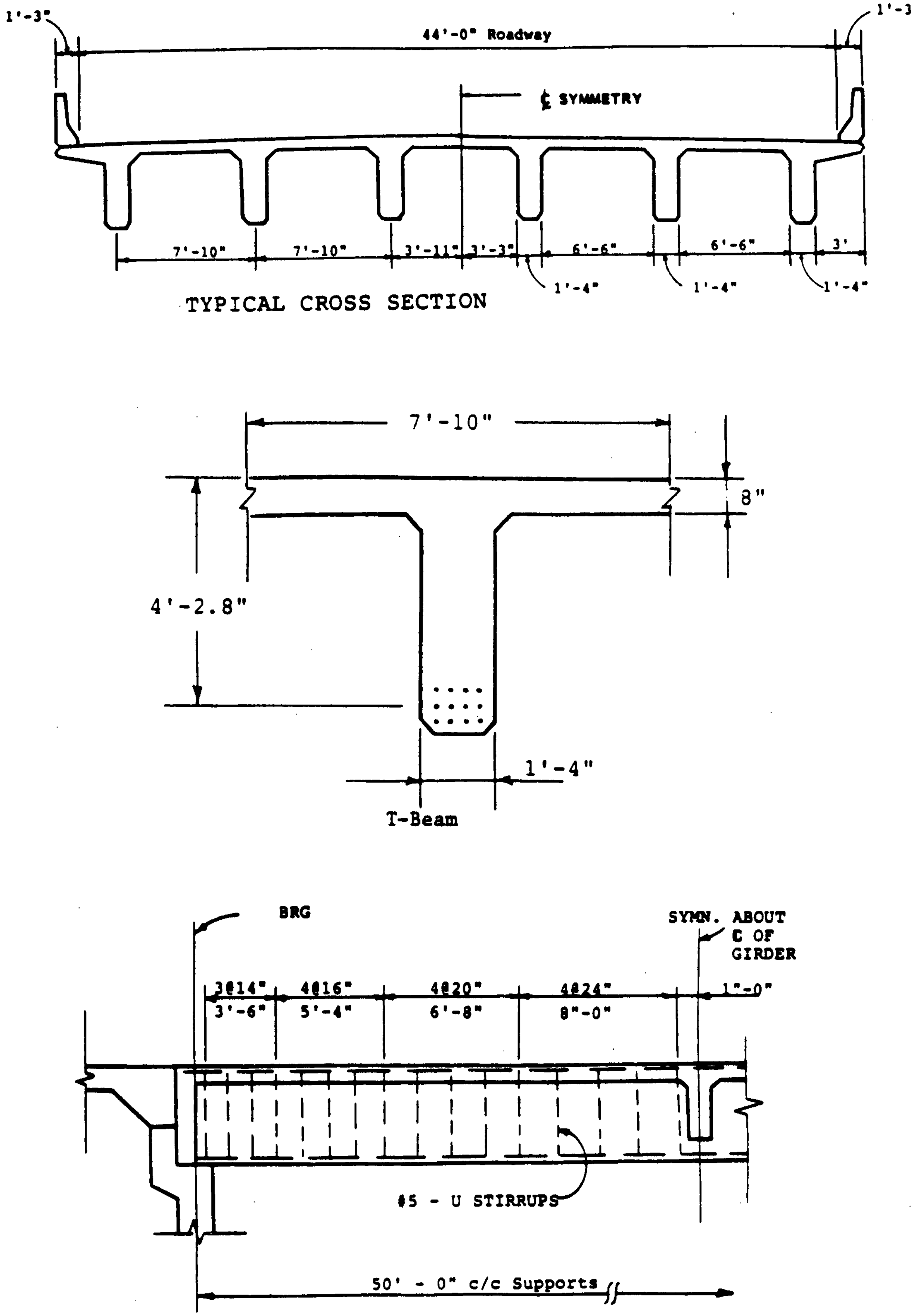


Figure A-1. T-beam.

$$M_u = \Phi(T)\left(d - \frac{a}{2}\right) = \Phi(A_s f_y)\left(d - \frac{a}{2}\right)$$

$$\Phi = 0.9 \text{ (bending in concrete)}$$

$$M_u = 0.9(18.72)(40)\left(50.8 - \frac{3.12}{2}\right)$$

M_u = 33,184 kip-in., say 33,200 kip-in. or

M_u = 2,765 kip-ft, say 2,770 kip-ft

M_u = 2,770 kip-ft

2. The dead load moment is given by:

$$\text{Weight of deck} = t \cdot b \cdot \gamma_c$$

$$W_1 = \left(\frac{8}{12}\right)(7.83)(0.150)$$

deck = 0.78 kip/ft of girder

$$\text{Weight of girder} = b_w \cdot d \cdot \gamma_c$$

$$W_2 = \left(\frac{16}{12}\right)(4.08)(0.150)$$

girder = 0.82 kip/ft of girder

$$\text{Weight of wearing surface} = t_w \cdot b \cdot \gamma_a$$

$$W_3 = \left(\frac{3}{12}\right)(7.83)(0.144)$$

W.S. = 0.28 kip/ft of girder

Add 0.10 kip/ft of girder of miscellaneous railings, sidewalks, curbs, and so on. Total W_D = 1.98 kip/ft of girder. Now: $M_D = W_D L^2/8 = [(1.98)(50)^2]/8$. M_D = 619 kip-ft per girder.

3. The live load moments can be taken from either the AASHTO *Manual for Maintenance Inspection of Bridges (A-1)* or from the AASHTO *Standard Specifications for Highway Bridges (A-2)*.

From Plate 9 of Ref. (*A-1*), M_{HS} = 310 kip-ft per wheel and $M_L = M_{HS}$ · (Wheel load distribution factor).

From Table 3.23.1 of Ref. (*A-2*), the distribution factor for a T-beam with two or more traffic lanes is $S/6.0$. Therefore, $M_L = (314)(7.83)/6.0$ = 410 kip-ft per girder.

The impact factor is given by: $I = (50)/(L + 125) \leq 0.30$ (Sec. 3.8.2.2 of Ref. A-1) · $I = (50)/(50 + 125) = 0.286$. $M_{L+I} = M_L(1 + I) = 410(1.286)$ = 527 kip-ft per girder.

4. Obtain the operating and inventory capacity rating factors (*A-2*):

$$RF = \frac{M_u - 1.3(M_D)}{1.3(M_{L+I})} \text{ (Operating)}$$

$$= \frac{2{,}770 - 1.3(619)}{1.3(527)}$$

$$RF \text{ (opr)} = 2.87$$

$$RF = \frac{M_u - 1.3M_D}{1.3(5/3)M_{L+I})} \text{ (Inventory)}$$

$$= \frac{3}{5}(RF \text{ (opr)})$$

$$= \frac{3}{5}(2.87)$$

$$RF \text{ (inv)} = 1.72$$

5. The capacity ratings based on an HS-20 vehicle are given by:

$$R \text{ (opr)} = RF \text{ (opr)} \times 20 = (2.87)(20) = 57.4$$

The operating rating is an HS-57 vehicle; thus:

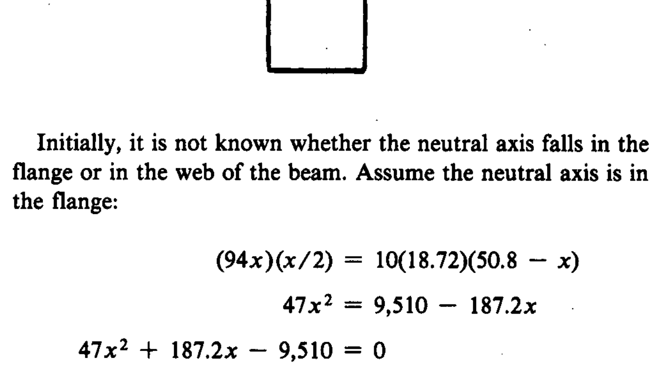
$$R \text{ (inv)} = RF \text{ (inv)} \times 20 = (1.72)(20) = 34.4$$

The inventory rating is an HS-34 vehicle. The loading on the Structure Inventory and Appraisal (SI & A) (*A-3*) would be expressed in terms of the total weight of the vehicle in tons. The first digit indicates type vehicle, and last two digits give weight in tons (e.g., the code for an HS-20 truck at 36 tons is 236). For example here, operating rating of 57.4 is converted using the ratio ((36/20) · 57.4 = 103), but the largest code available is 99. Likewise, the inventory rating of 34.4 is converted by ((36/20) · 34.4 = 61). Hence:

Item 64—Operating ratio 299
Item 66—Inventory ratio 261

Moment Capacity—Allowable Stress Method

1. The total moment carrying capacity of the girders by the allowable stress method is calculated as follows.



Initially, it is not known whether the neutral axis falls in the flange or in the web of the beam. Assume the neutral axis is in the flange:

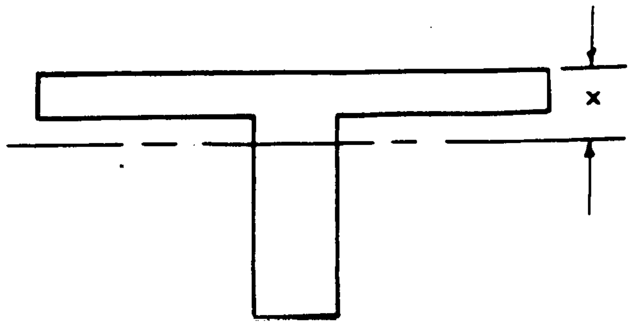
$$(94x)(x/2) = 10(18.72)(50.8 - x)$$

$$47x^2 = 9{,}510 - 187.2x$$

$$47x^2 + 187.2x - 9{,}510 = 0$$

$$x = 12.4 \text{ in.}$$

which is greater than the slab thickness of 8 in.

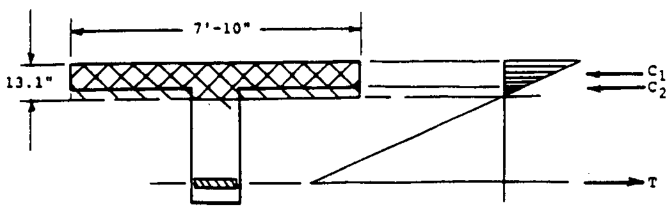


$$(94)(x)\left(\frac{x}{2}\right) - (78)(x - 8)\left(\frac{x-8}{2}\right) = (10)(18.72)(50.8 - x)$$

$$47x^2 - 39(x^2 - 16x + 64) = 9{,}518 - 187.2x$$

$$47x^2 - 39x^2 + 624x - 2{,}496 - 9{,}510 + 187.2x = 0$$
$$8x^2 + 811.2x - 12{,}006 = 0$$
$$x^2 + 101.4x - 1{,}500 = 0$$
$$x = 13.1$$

which is greater than the slab thickness. Therefore, the neutral axis is in the stem, as assumed.



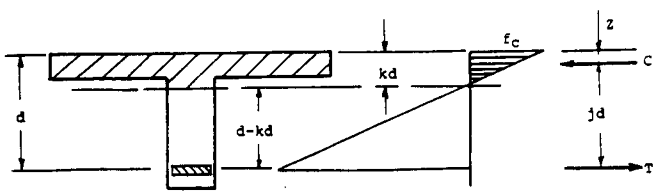
At bottom of flange:

$$f'_{2c} = \frac{x - t}{x} f'_c = \frac{13.1 - 8}{13.1}(f'_c) = 0.389 f'_c$$

$$C_1 = (94)(13.1)(f'_c/2) = 616\, f'_c$$

$$C_2 = (94 - 16)(13.1 - 8)\left(\frac{0.389 f'_c}{2}\right) = 77 f'_c$$

$$C = C_1 - C_2 = (616 - 77) f'_c = 538 f'_c$$



The distance from the compressive face to the compressive force C (i.e., Z) is found by summing moments about the compression face:

$$C \cdot Z = C_1 \cdot \frac{kd}{3} - C_2 \cdot \left(8 + \frac{5.1}{3}\right)$$

$$Z = \frac{(616 f_c \cdot 13.1/3) - (77 f'_c \cdot 9.7)}{538 f'_c}$$

$$Z = 3.61 \text{ in.}$$

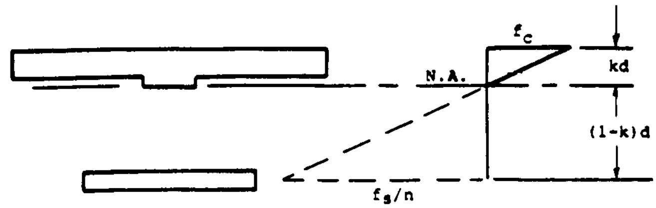
The moment of the couple $C \cdot jd$ or $T \cdot jd$ must equal the external moment M, i.e.,

$$M = C \cdot jd = T \cdot jd = A_s \cdot f_s \cdot jd$$

or

$$f_s = \frac{M}{A_s \cdot jd} = \frac{T}{A_s}$$

For the concrete



By similar triangles,

$$\frac{f'_c}{kd} = \frac{\frac{f_s}{n}}{(1-k)d} \rightarrow$$

$$f'_c = \frac{f_s}{n} \cdot \frac{k}{1-k} \qquad k = \frac{kd}{d} = \frac{13.1}{50.8} = 0.258$$

The following values of f_s and f'_c are obtained from the AASHTO *Standard Specifications for Highway Bridges* and *Manual for Maintenance Inspection of Bridges (A-2, A-1)* for an f_y of 40 ksi and f'_c of 3 ksi.

If designed correctly, the strength of the T-beam is controlled by the yielding of the tensile steel. This is the ideal situation, because a tensile failure that comes about by yielding of the steel is gradual, as opposed to a compressive failure in flexure, which comes about as a sudden crushing of the concrete.

INVENTORY	OPERATING
f_s (Reference 1, Sec. 5.4.4)	f_s (Reference 1, Sec. 5.4.4)
f_c(Reference 2, Sec. 8.15.2.1)	f_c(Reference 1, Sec. 5.4.5)
f_s = 20 ksi	f_s = 28 ksi
f_c = 1.2 ksi	f_c = 1.9 ksi

Check to see if steel or concrete controls:

- *Inventory:*

$$f'_c = \frac{20{,}000}{10} \cdot \frac{0.258}{0.742} = 695 \text{ psi} < 1{,}200 \text{ psi (allowable)}$$

Steel controls because f_s = 20,000 psi has been reached before f_c reaches its allowable value of 1,200 psi.

- *Operating:*

$$f'_c = \frac{28{,}000}{10} \cdot \frac{0.258}{0.742} = 974 < 1{,}900 \text{ psi (allowable)}$$

Steel controls.

- *Inventory Moment:*

$$M = AS \cdot f_s \cdot jd$$

$$= (18.72)(20)(47.2) = 17{,}671 \text{ kip-in.}$$

$$= 1{,}473 \text{ kip-ft, say } 1{,}470 \text{ kip-ft}$$

$$M(\text{inv}) = 1{,}470 \text{ kip-ft}$$

- *Operating Moment:*

$$M = (18.72)(28)(47.2) = 24{,}740 \text{ kip-in.}$$

$$= 2{,}062 \text{ kip-ft, say } 2{,}060 \text{ kip-ft}$$

$$M(\text{opr}) = 2{,}060 \text{ kip-ft}$$

2. The dead load moment is the same as that for the load factor method: $M_D = 619$ kip-ft.
3. The live load moment is the same as that for the load factor method: $M_{L+I} = 527$ kip-ft.
4. Obtain rating factors:

- *Inventory*:

$$RF = \frac{1{,}470 - 619}{527};\ RF\ (inv) = 1.61$$

- *Operating*:

$$RF = \frac{2{,}060 - 619}{527};\ RF\ (opr) = 2.73$$

5. The capacity ratings based on an HS-20 vehicle are given by: R (opr) $= 2.73 \times 20 = 55$.

The operating rating is an HS-55 vehicle; thus, R (inv) $= 1.61 \times 20 = 32.2$. The inventory rating is an HS-32 vehicle.

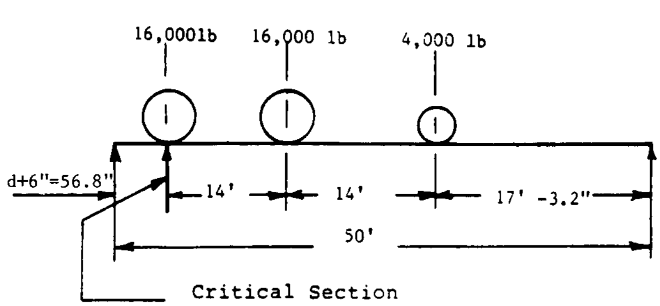
Shear Capacity—Load Factor Method

1. The total factored shear-capacity of the girders is $V_n = V_c + V_s$.

The critical section occurs at a distance d from the face of the support: $d = 50.8$ in. $= 4$ ft $- 2.8$ in. from face of support. The critical section is then one-half the bearing depth plus the distance d because the span is measured from the centerline of the bearing. The bearing depth is 12 inches in this case.

Since the shear and moment occurring at the critical section are necessary for a detailed shear capacity analysis, they will be computed here.

The absolute maximum shear occurs with a wheel of the truck at the reaction. However, since the critical section is at a distance $d + 6$ in. from the reaction, the maximum shear for capacity computations would be produced as shown.



This live load shear at the critical section can be obtained by summing moments about the right end of the simple span either by using Plate 7 of Ref. (*A-1*) or by using Appendix A of Ref. (*A-2*). Plate 7 of Ref. (*A-1*) yields:

$$V = \frac{36\,(x - 9.33)}{L}$$

where L = span = 50 ft; x = distance from right end of simple span $= 50 - (50.8 + 6)/12 = 45.27$ ft. Thus, $V = (36\,(45.27 - 9.33))/50 = 25.88$ kip.

The maximum moment at this critical section is calculated by the formula in Plate 9 of Ref. (*A-1*):

$$M_u = \frac{36\,(L - x)\,(x - 9.33)}{L} = \frac{36\,(50 - 45.27)\,(45.27 - 9.33)}{50} = 122.4 \text{ ft-kip}$$

The conservative shear strength of the concrete may be approximated by:

$$V_c = 2\sqrt{f_c'}\, b_w\, d = 2\sqrt{3{,}000}\,(16)(50.8) = 89 \text{ kip}$$

or it may be calculated by using the detailed AASHTO equation:

$$V_c = \left(1.9\sqrt{f_c'} + 2{,}500\,\rho_w \frac{V_u d}{M_u}\right) b_w d$$

where

$$\rho_w = \frac{A_s}{b_w d} = \frac{18.72}{16\,(50.8)} = 0.023$$

$$V_c = \left(1.9\sqrt{3{,}000} + 2{,}500\,(0.023)\,\frac{25.88\,(4.23)}{122.4}\right)(16)(50.8)$$

$$= 126 \text{ kip}$$

Regardless of the equation used, V_c cannot exceed the maximum allowable shear force of:

$$3.5\sqrt{f_c'}\, b_w d = 3.5\sqrt{3{,}000}\,(16)(50.8) = 156 \text{ kip}$$

The shear reinforcement yields:

$$V_s = \frac{A_v f_y d}{S} = \frac{2\,(0.31)(40)(50.8)}{14} = 90 \text{ kip}$$

The total shear force available is $V_n = 89 + 90 = 179$ kip; $V_u = \Phi\, V_n = (0.85)(179) = 152$ kip.

Note: Using the detailed AASHTO equation for the shear strength of concrete (V_c) will yield a total shear force (V_u) of $(0.85)(126 + 90) = 183.6$ kip. However, calculations based on the conservative approximation of V_c are more frequently used and, therefore, will be used in the following example.

2. For dead load shear, the total dead load weight of the T-

beam is 1.96 kip/ft. The maximum dead load shear at the end of the beam for a 50-ft span is 1.96 (50/2) = 49 kip.

At a distance of d from the face of the support or a distance of d + 6 in. from the centerline of the bearing:

$$V_D = \frac{49(25)(12) - (50.8 + 6)}{25\ (12)} = 40 \text{ kip}$$

3. For live load shear, the live load shear per wheel load was computed to be 25.88 kip. The distribution factor is taken from Section 3.23 of Ref. (*A-2*):

$$DF = S/6 = (7.83)/6 = 1.31$$

The impact factor is taken from Section 3.8.2.2 of Ref. (*A-2*):

$$I = \frac{50}{L + 125}$$

where $L = 50 - 4.73 = 45.27$; $I = (50)/45.27 + 125 = 0.29$; then,

$$V_{L+I} = V_L\ (1 + I)\ (DF) = 25.88\ (1.29)(1.31) = 43.7 \text{ kip}$$

4. Obtain the rating factors:

$$RF_{\text{opr}} = \frac{V_u - 1.3\ V_D}{1.3\ (V_{L+I})} = \frac{152 - 1.3(40)}{1.3(43.7)} = 1.8$$

$$RF_{\text{inv}} = 3/5\ (RF_{\text{opr}}) = 1.1$$

5. The capacity rating based on an HS-20 vehicle is:

$$R_{\text{opr}} = \text{HS-36};\ R_{\text{inv}} = \text{H-21}$$

Shear Capacity—Allowable Stress Method

1. The total shear-carrying force, V, of the girders is given by: $V = v \cdot b_w d$.

The total shear stress, v, is given by $v = v_c + v_s$.

From p. 57 the critical section shear and moment are V = 25.88 kip, and M = 122.4 ft-kip.

From Section 8.15.5.2 of Ref. (*A-2*), the allowable shear stress may be taken as:

$$v_c = 0.95\sqrt{f_c'} = 0.95\sqrt{3{,}000} = 52 \text{ kip}$$

The maximum allowable shear stress is

$$1.6\sqrt{f_c'} = 1.6\sqrt{3{,}000} = 88 \text{ psi}$$

Note: The longer AASHTO equation yields:

$$v_c = 0.9\sqrt{f_c'} + 1{,}100\ \rho_w\ (V \cdot d)/M$$

$$= 0.9\sqrt{3{,}000} + 1{,}100\ (.023)(25.88)(4.23)/122.4 = 72 \text{ psi}$$

Using the approximate concrete shear stress,

$$V_c = v_c b_w d = (52)(16)(50.8) = 42 \text{ kip}$$

The steel shear stress is:

$$v_s = \frac{A_v f_s}{b_w S} = \frac{2\ (0.31)(20{,}000)}{16\ (14)} = 55 \text{ psi}$$

The shear force available is

$$V_s = v_s b_w d = 55\ (16)(50.8) = 45 \text{ kip}$$

The total shear-carrying force is:

$$V = V_c + V_s = 42 + 45 = 87 \text{ kip}$$

2. The dead load shear as computed previously is: V_D = 40 kip.

3. The live load shear, as can be seen from previous computation, is: V_{L+I} = 43.7 kip.

4. For the rating factor, AASHTO does not give an inventory and operating stress level for concrete; hence, only the inventory rating factor is computed.

$$RF_{\text{inv}} = \frac{V - V_D}{V_{L+I}} = \frac{87 - 40}{43.7} = 1.1$$

5. The capacity rating based on an HS-20 vehicle is R_{inv} = HS-21.

Bearing Capacity—Load Factor Method

1. The total ultimate bearing capacity. R_{bu}, is:

$$\begin{aligned}
R_{bu} &= f_b \cdot a_b \\
f_b &= 0.85\ \Phi\ f_c' = (0.85)\ (0.70)(3{,}000) \\
f_b &= 1{,}785 \text{ psi} \\
A_b &= b \cdot L_b = (16)\ (12) \\
A_b &= 192 \text{ in.}^2 \\
R_b &= (1{,}785)\ (192) = 342{,}700 \text{ lb} \\
R_b &= 343 \text{ kip}
\end{aligned}$$

2. For dead load, as noted earlier:

$$\begin{aligned}
W_D &= 1.96 \text{ kip/ft} \\
R_D &= 1/2 \cdot \mathrm{W_D} \cdot \mathrm{L} = 1/2\ (1.96)(50) \\
R_D &= 49.0 \text{ kip}
\end{aligned}$$

3. The required live load for an HS-20 truck is determined using the formulas of Plate 7 of Ref. (*A-1*) or from Appendix A of Ref. (*A-2*). The formulas in Plate 7 yield: Reaction = 29.3 kip.

The distribution factor and the impact factor, from above, are:

$$DF = 1.31$$

$$\text{Impact} = 0.29$$

$$R_{L+I} = (29.3)(1.31)(1.29) = 49.5 \text{ kip}$$

4. Obtain rating factors:

$$RF_{opr} = \frac{R_b - 1.3\,R_D}{(1.3)\,R_{L+I}} = \frac{343 - 1.3\,(49.0)}{(1.3)(49.5)}$$

$$RF_{opr} = 4.3$$

$$RF_{inv} = 3/5\;RF_{opr}$$

$$RF_{inv} = 2.6$$

5. Obtain rating capacity:

$$R_{opr} = \text{HS-86};\; R_{inv} = \text{HS-52}$$

Bearing Capacity—Allowable Stress Method

1. The total bearing capacity, R_b, is:

$$R_b = f_b \cdot A_b$$
$$f_b = 0.30\,f_c' = 0.30\,(3{,}000) = 900 \text{ psi}$$
$$A_b = b \cdot L_b = (16)(12) = 192 \text{ in.}^2$$

$$R_b = (900)\,(192) = 172{,}800 \text{ lb}$$
$$R_b = 173 \text{ kip}$$

2. For dead load, from p. 58, $R_D = 49.0$ kip.
3. For live load, from p. 58, $R_{L+I} = 49.5$ kip.
4. Obtain the rating factor. Again, AASHTO only gives one stress level for concrete; hence:

$$RF_{inv} = \frac{R_b - R_D}{R_{L+I}} = \frac{173 - 49.0}{49.5} = 2.5$$

5. Obtain the rating capacity. R_{inv} = HS-50.

EXAMPLE 2—SIMPLE SPAN SLAB BRIDGE

This example illustrates the process for finding the inventory and operating ratings for a selected simply supported slab bridge.

The moment capacity is computed using both the load factor method and the allowable stress method.

The dimensions and member properties of the concrete slab are given in Figure A-2.

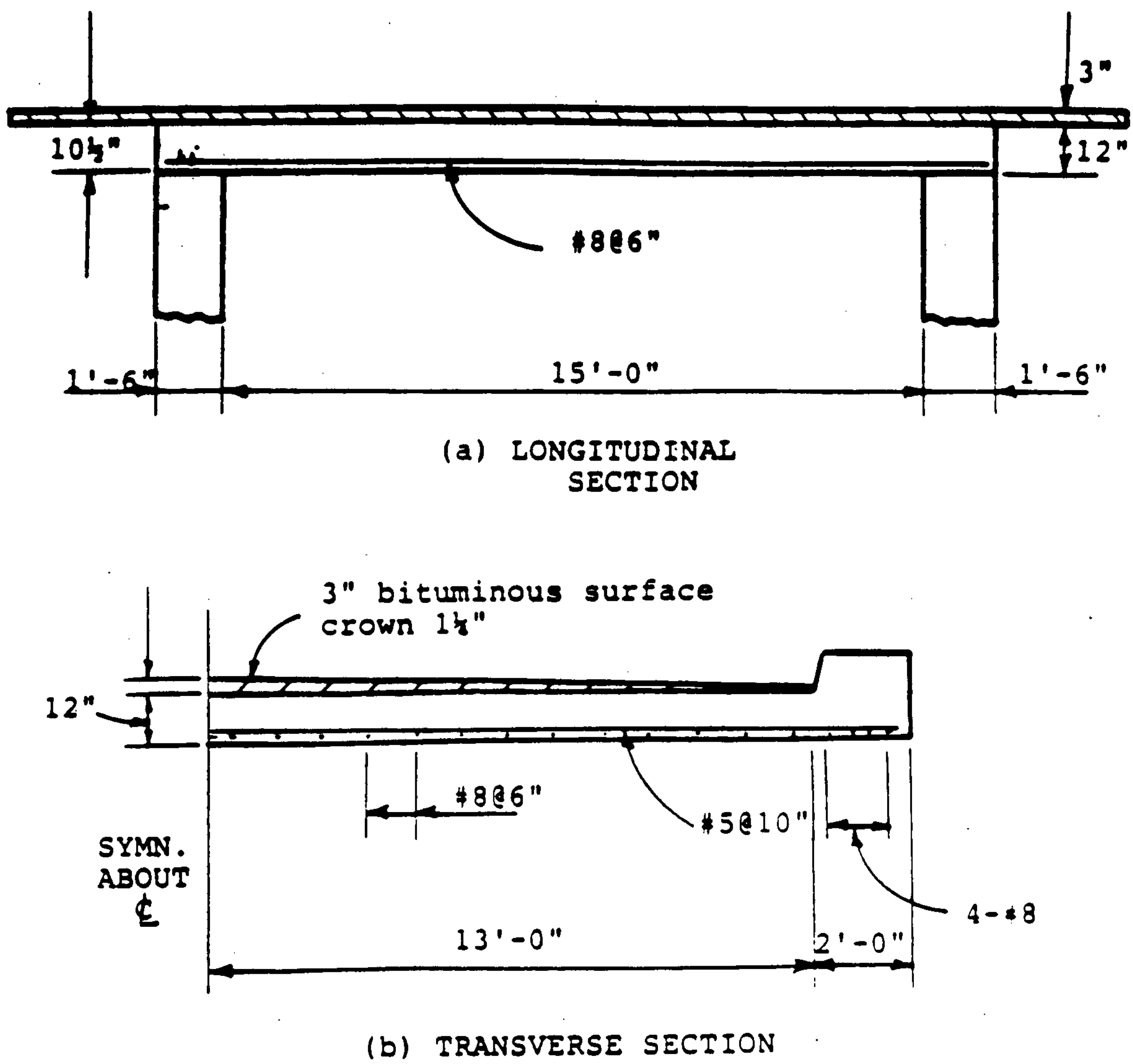


Figure A-2.
Slab bridge.

The rating factor will be calculated for an HS-20 vehicle. Normal traffic loadings are assumed. The problem variables are: $f_y = 40$ ksi, $f_c' = 3$ ksi, $d = 10.5$ in., $t_a = 3$ in., and $A_s = 1.58$ sq. in.

Load Factor Method

1. The total moment-carrying capacity per unit width of slab is:

$$M_u = T\left(d - \frac{a}{2}\right)$$

where

$$a = \frac{T}{0.85\, f_c'\, b} = \frac{f_y A_S}{0.85\, f_c'\, b}$$

Let unit width equal 1 ft. Therefore,

$$\begin{aligned}
A_S &= 2 \text{ (Area of No. 8 bar)}\\
a &= \frac{(1.58)(40)}{(0.85)(3)(12)} = 2.07 \text{ in.}\\
M_u &= \Phi(A_s f_y)\left(d - \frac{a}{2}\right)\\
&= 0.9\ (1.58)\ (40)\left(10.5 - \frac{2.07}{2}\right) = 538.4 \text{ kip-in.}\\
&= 44.9 \text{ kip-ft, say 45 kip-ft}\\
M_u &= 45 \text{ kip-ft}
\end{aligned}$$

2. Dead load moment:

$$\begin{aligned}
\text{Concrete} &= txY_c = \left(\frac{12}{12}\right)(0.150) = 0.150 \text{ lb/ft}\\
\text{Asphalt} &= t_a x Y_c = \left(\frac{3}{12}\right)(0.144) = 0.036 \text{ lb/ft}\\
\text{Total } w_D &= 0.186 \text{ lb/ft}\\
M_D &= \frac{wL^2}{8}\\
\text{use } L &= 16 \text{ ft}\\
M_D &= \frac{(0.186)(16)^2}{8} = 5.95 \text{ kip-ft}
\end{aligned}$$

3. The required live load moment is obtained by using Section 3.24.3 of Ref. (*A-2*)

$$E = 0.06S + 4 = 0.06(16) + 4 = 4.96$$

The load on a unit width of slab is:

$$P' = \frac{P}{E} = \frac{16{,}000}{4.96} = 3{,}230 \text{ lb}$$

The maximum moment for the concentrated load at the center of a simple beam is:

$$\begin{aligned}
M_L &= \frac{P' \cdot S}{4} = \frac{(3{,}230)(16)}{4}\\
W_L &= 12{,}900 \text{ lb-ft}
\end{aligned}$$

The impact factor is:

$$\begin{aligned}
I &= \frac{50}{L + 125} = \frac{50}{16 + 125} = 0.35\ (>0.30)\\
\text{Use } I &= 0.30\\
M_{L+I} &= (12{,}900)(1.30) = 16.77 \text{ kip-ft}
\end{aligned}$$

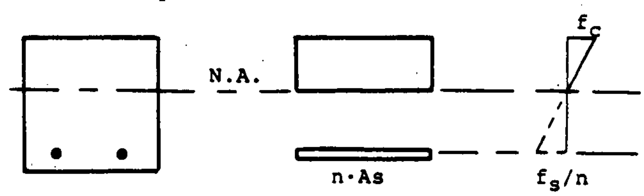
4. Obtain rating factors:

$$\begin{aligned}
RF_{\text{opr}} &= \frac{M_u - 1.3\, M_D}{1.3\, M_{L+I}} = \frac{45.0 - 1.3(5.95)}{1.3(16.77)}\\
RF_{\text{opr}} &= 1.71\\
RF_{\text{inv}} &= \frac{3}{5}(RF_{\text{opr}})\\
RF_{\text{inv}} &= 1.02
\end{aligned}$$

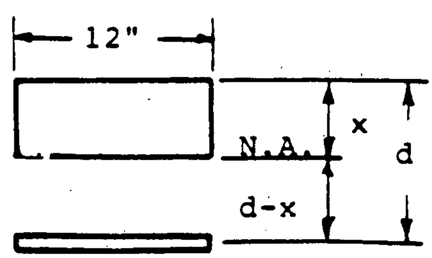
5. Capacity rating based on an HS-20 vehicle: R_{opr} = HS-34; R_{inv} = HS-20.

Allowable Stress Method

1. For the total moment-carrying capacity of the slab, assume a 1-ft wide strip:



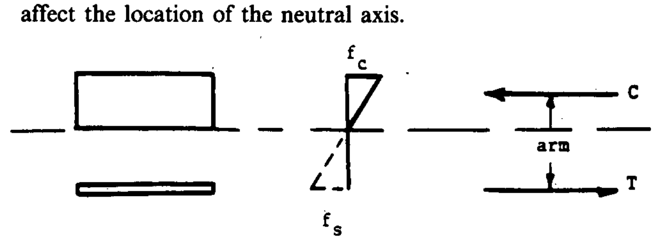
The steel bars are replaced with an equivalent area of fictitious concrete. Locate the neutral axis:



Taking moments about the neutral axis with $n = 10$ gives:

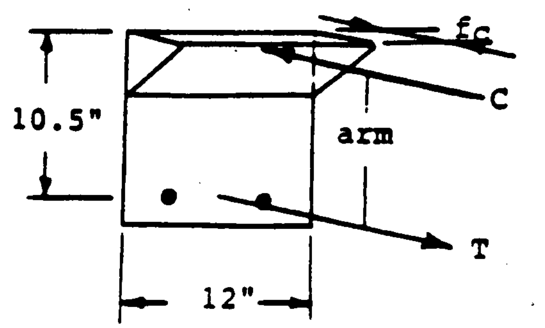
$$\begin{aligned}
(12x)\left(\frac{x}{2}\right) &= 10 \cdot (A_s)(d - x)\\
6x^2 &= 10 \cdot (1.58)(10.5 - x)\\
6x^2 &= 165.9 - 15.8\, x\\
x^2 + 2.63\, x - 27.7 &= 0\\
x &= 4.1 \text{ in.}
\end{aligned}$$

Only the properties of the section (depth, width, and steel area) affect the position of the neutral axis. The loading does not affect the location of the neutral axis.



Two equilibrium conditions apply to a section subjected to bending: (1) The internal resultant compressive force must be equivalent to the internal resultant tensile force; and (2) the moment of the internal couple is equivalent to the applied bending moment.

The resultant compressive force comes entirely from concrete stresses and is represented by a solid triangular wedge. The resultant tensile force comes entirely from tension reinforcement.



The moment arm of the internal couple is equal to the distance from the centroid of the compressive solid to the centroid of the tension steel. Thus,

$$\text{arm} = d - x/3 = 10.5 - (4.1/3) = 9.1 \text{ in.}$$

The bending moment is $M_w = (C \text{ or } T) \times \text{arm}$; $T = A_s f_s$. The allowable stresses for the concrete and steel are:

INVENTORY	OPERATING
F_S = 20,000 psi	F_S = 28,000 psi
f_c = 1200 psi	f_c = 1900 psi

- *Inventory Moment*
  $T = 1.58\ (20) = 31.6$ kip
  $C = 1/2(12)(1.2)(4.1) = 29.5$ kip (Controls)
  $M = C \cdot \text{arm} = 268$ kip-in.
  $M = 22.4 \cdot$ kip-ft

- *Operating Moment*
  $T = 1.58\ (28) = 44.2$ kip (Controls)
  $C = 1/2(1.90)(12)(4.1) = 46.7$ kip
  $M = 402.2$ kip-in.; $M = 33.5$ kip-ft

2. For dead load moment, refer to p. 60: $M_D = 5.95$ kip-ft.
3. For live load moment, refer to p. 60: $M_{L+I} = 16.77$ kip-ft.
4. Obtain rating factor:

$$RF_{\text{inv}} = \frac{M_w - M_D}{M_{L+I}} = \frac{22.4 - 5.95}{16.77} = 1.0$$

$$RF_{\text{opr}} = \frac{M_w - M_D}{M_{L+I}} + \frac{33.5 - 5.95}{16.77} = 1.64$$

5. The capacity rating based on an HS-20 vehicle is: R_{opr} = HS-33; R_{inv} = HS-20.

EXAMPLE 3—REINFORCED CONCRETE BOX CULVERT

This example illustrates the process of finding the inventory and operating ratings for a reinforced concrete box culvert.

The dimensions and member properties are given in Figure A-3. The problem variables include the following:

$$f_y = 60 \text{ ksi} \qquad \gamma_S = 0.12 \text{ kcf}$$
$$f_c' = 3.5 \text{ ksi} \qquad \gamma_c = 0.15 \text{ kcf}$$
$$\gamma_a = 0.144 \text{ kcf}$$

1. The total moment-carrying capacity per unit width of slab is:

$$T = A_s \cdot f_y = 0.61\ (60) = 36.6 \text{ kip}$$

$$a = \frac{T}{0.85 f_c' b} = \frac{36.6}{0.85(3.5)(12)} = 1.03 \text{ in.}$$

$$M_u = 0.9\ T\left(d - \frac{a}{2}\right)\left(\frac{1}{12}\right)$$
$$= 0.9\ (36.6)\left(8.5 - \frac{1.03}{2}\right)\left(\frac{1}{12}\right) = 21.9 \text{ kip-ft}$$

2. Dead load moment:

Two Ft Depth
24 in. Earth: $2 \times 0.12 = 0.240$ ksf
11 in. Roof: $11/12 \times 0.15 = 0.138$ ksf
3-in. Asphalt: $3/12 \times 0.144 = 0.036$ ksf
Total = 0.414 ksf

$$M_D = \frac{wL^2}{8}$$

$$M_D = 3.31 \text{ kip-ft}$$

Similarly:

Depth (ft)	w (ksf)	M(dead) (k-ft)
2	0.416	3.1
6	0.896	7.17
20	2.58	20.6

3. Live load:

a. *Depth of fill is 2 ft.* When the depth of fill is 2 ft and under, the wheel loads are distributed as if the loads were applied directly to the slab. From Section 3.24.3.2, AASHTO Ref. (*A-2*):

$$E = 0.06\ S + 4 = 0.06\ (8) + 4 = 4.48 \text{ ft}$$

The load on a unit width of slab is:

$$P' = P/E = 16{,}000/4.48 = 3{,}570 \text{ lb/ft width}$$

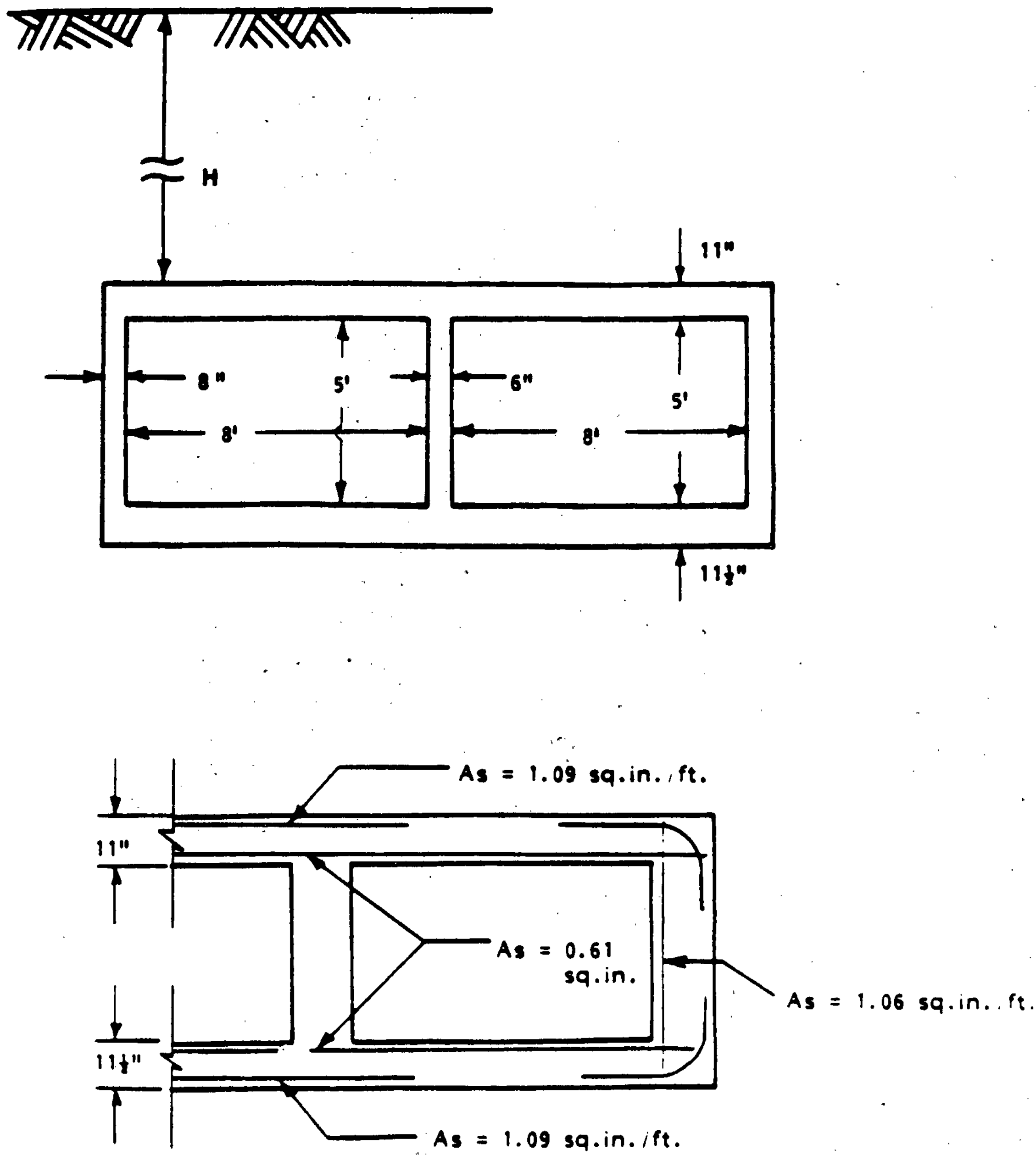
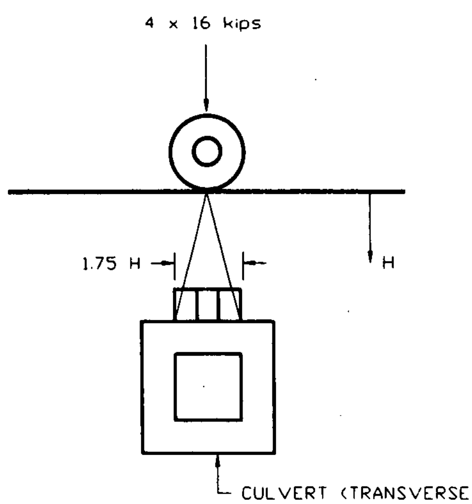


Figure A-3. Reinforced concrete box culvert.



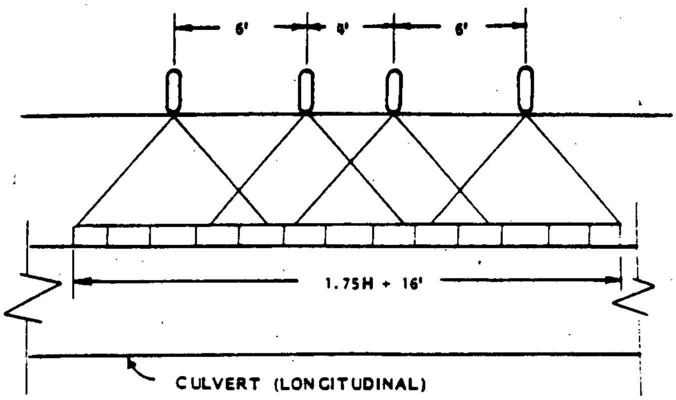
The maximum moment for a concentrated load at the center of the span (assuming simple span) is:

$$M \text{ (Live)} = \frac{P'S}{4} = \frac{(3{,}570)(8)}{4} = 7{,}140 \text{ kip-ft}$$

Impact = 20 percent (AASHTO Ref. (*A-2*), Sec. 3.8.2.2)
M_{L+I} = 8.57 kip-ft

b. *Depth of fill is 6 ft.* When the depth of fill is over 2 ft but less than 8 ft, the wheel loads are distributed over squares having sides equal to 1.75 times the depth of fill. If the squares overlap, the wheel loads are evenly spread over the gross area.

Side 1 $= 1.75\ H = 1.75\ (6) = 10.50$ ft (Span of culvert is 8 ft)



Side 2 $= 1.75\ H + 16 = 1.75\ (6) + 16 = 26.5$ ft

$$\text{Load intensity} = \frac{64 \text{ kip}}{(\text{Side 1})\ (\text{Side 2})}$$
$$= 64/(26.5)(8) = 0.302 \text{ kip/ft}$$

Impact is zero (AASHTO Ref. (*A-2*), Sec. 3.8.2.2)

$$M_{L+I} = \frac{wL^2}{8} = \frac{(0.302)(8)^2}{8} = 2.42 \text{ kip-ft}$$

4. Rating factor:
Depth = 2 ft:

$$RF_{(\text{opr})} = \frac{M_u - 1/3\ M_d}{1.3 M_{(L\ +\ I)}} = \frac{21.9 - 1.3\ (3.33)}{1.3\ (8.57)} = 1.6$$
$$RF_{(\text{inv})} = 3/5\ RF_{(\text{opr})}$$
$$RF_{\text{inv}} = 1.0$$

Depth = 6 ft:

$$RF_{(\text{opr})} = \frac{M_u - 1.3 M_d}{1.3 M_{(L+I)}} = \frac{1.9 - 1.3\ (7.17)}{1.3\ (2.42)}$$
$$RF_{\text{opr}} = 4.0$$
$$RF_{\text{inv}} = 3/5\ RF_{\text{opr}}$$
$$RF_{\text{inv}} = 2.4$$

5. Capacity rating:

Depth of Fill (feet)	Dead Load (k-ft)	Live Load (k-ft)	Rating Factor		Capacity Rating	
			Inv.	Opr.	Inv.	Opr.
2	3.33	8.57	1.0	1.6	HS20	HS32
6	7.17	2.42	2.40	4.00	HS48	HS80
16	16.8	0.0	Large			

EXAMPLE 4—REINFORCED CONCRETE ARCH BRIDGE

This example illustrates the process of finding the inventory and operating ratings for a selected reinforced concrete arch bridge.

The dimensions and member properties are given in Figure A-4. The bridge was designed around 1920.

The arch ring shown in Figure A-4 represents a section 1-ft long of an arch barrel loaded so that this 1-ft section is subjected to the loads shown. The arch has a span of 132 ft and a rise of 16 ft. The thickness at the crown is 2 ft 6 in. and at the springing 5 ft 7.5 in. The arch is reinforced with two square rods on centers, each with 1.25-in. sides. The loads are carried on spandrels spaced at 11 ft 1.5 in. apart.

The analysis of the arch bridge can be done by the use of influence lines. An influence line or diagram represents the variation of moment, shear, stress, or some other function at any particular point due to the placing of a vertical load of unity at every other point along the span. (For an explanation of the properties and derivation of influence lines, the reader is referred to various works on structural engineering.) The problem variables include: h (springing) = 5 ft $7\frac{1}{2}$ in., h (crown) = 2 ft 6 in., $f'_c = 3{,}000$ psi, $f_y = 33$ ksi, $A_s = 1.56$ sq. in., $A'_s = 1.56$ sq. in., $A_g = 360$ sq. in. (crown), and $A_g = 810$ sq. in. (springing).

If the moments at the crown and at the springing are plotted as ordinates from an axis, the resulting figure is an influence diagram which shows the variations of the moment as a unit load moves across the bridge. Figure A-5 shows the diagram for moments and thrust at the springing.

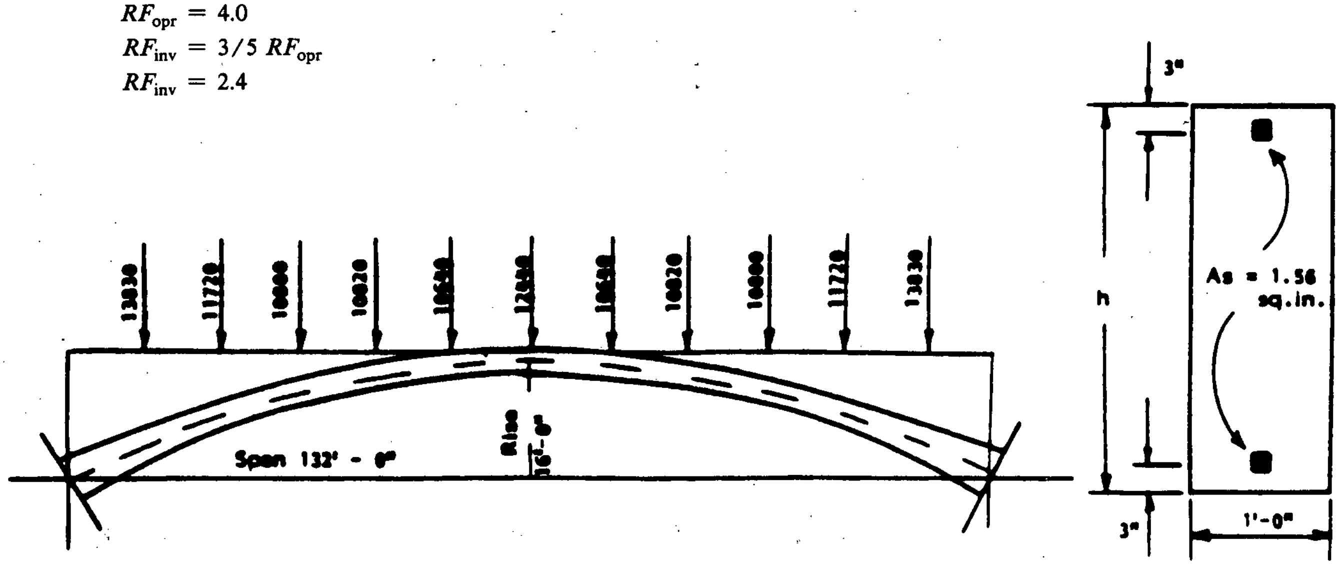


Figure A-4. Reinforced concrete arch bridge.

Load Factor Method

1. Dead load moment and axial load: Table A-1 shows the thrust and moment at the right springing obtained by using the influence points.

2. Live load: Figure A-6 shows how the HS-20 truck is positioned on the influence line to produce the maximum live load moment and thrust. If a load falls between two points on the influence line, the required value is found by linear interpolation.

- *Positive moment*
  8(5.70) + 32(10.64) + 32(9.16) + moment = 679.2 kip-ft
- *Negative moment*
  32(10.05) + 32(13.00) + 8(7.01) − moment = −793.7 kip-ft (Controls)
- *Thrust*
  thrust = 8(1.92) + 32(2.25) + 32(2.02) = 152 kip

The live load is assumed to be evenly distributed over the 10-ft width:

$$793.7 \div 10 = 79.4 \text{ kip-ft}; \; 152 \div 10 = 15.2 \text{ kip}$$

Impact:

$$I = \frac{50}{L + 125} = 0.19$$

$$M_{L+I} = 94.5 \text{ kip-ft}$$

$$M_{L+I} = 18.1 \text{ kip}$$

3. Moment capacity for reinforced arch bridge: For pure flexure, the maximum factored moment, M_u, is given by Section 8.16.2 of Ref. (*A-2*):

$$M_o = (d - d')A_s f_y = (61.5)(1.56)(33) \div 12$$
$$M_o = 264 \text{ kip-ft}$$
$$M_u = \Phi(264);\ \Phi = 0.9$$
$$M_u = 237 \text{ kip-ft}$$

For pure compression, the maximum axial force, P_u, is given by Section 8.16.4.2 of Ref. (*A-2*):

$$P_o = 0.85 f'_c (A_g - A_{st}) + A_{st} f_y$$
$$= 0.85(3)(810 - 3.12) + 3.12(33) = 2{,}161 \text{ kip}$$
$$P_n = 0.80\,(P_o)$$
$$P_n = 1{,}729$$
$$P_u = \Phi\, P_n;\ \Phi = 0.7$$
$$P_u = 1{,}210 \text{ kip}$$

For balanced strain conditions, the maximum moment, M_b, is given by Section 8.16.4.2.3 of Ref. (*A-2*).

$$M_b = 0.85 f'_c\, b\, a_b\, (d - d'' - ({}^{a_b}\!/2)) + A'_s f'_s\, (d - d' - d'') + A_s f_y d''$$

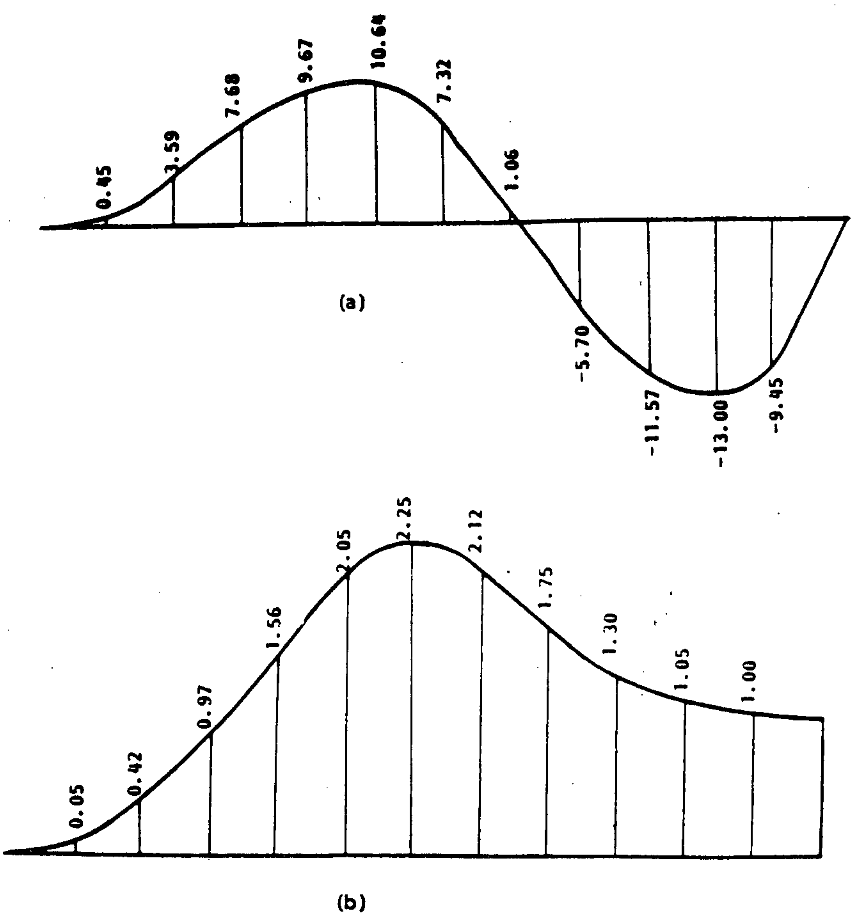


Figure A-5. Influence lines: (a) for maximum moment at springing, and (b) for maximum thrust at springing.

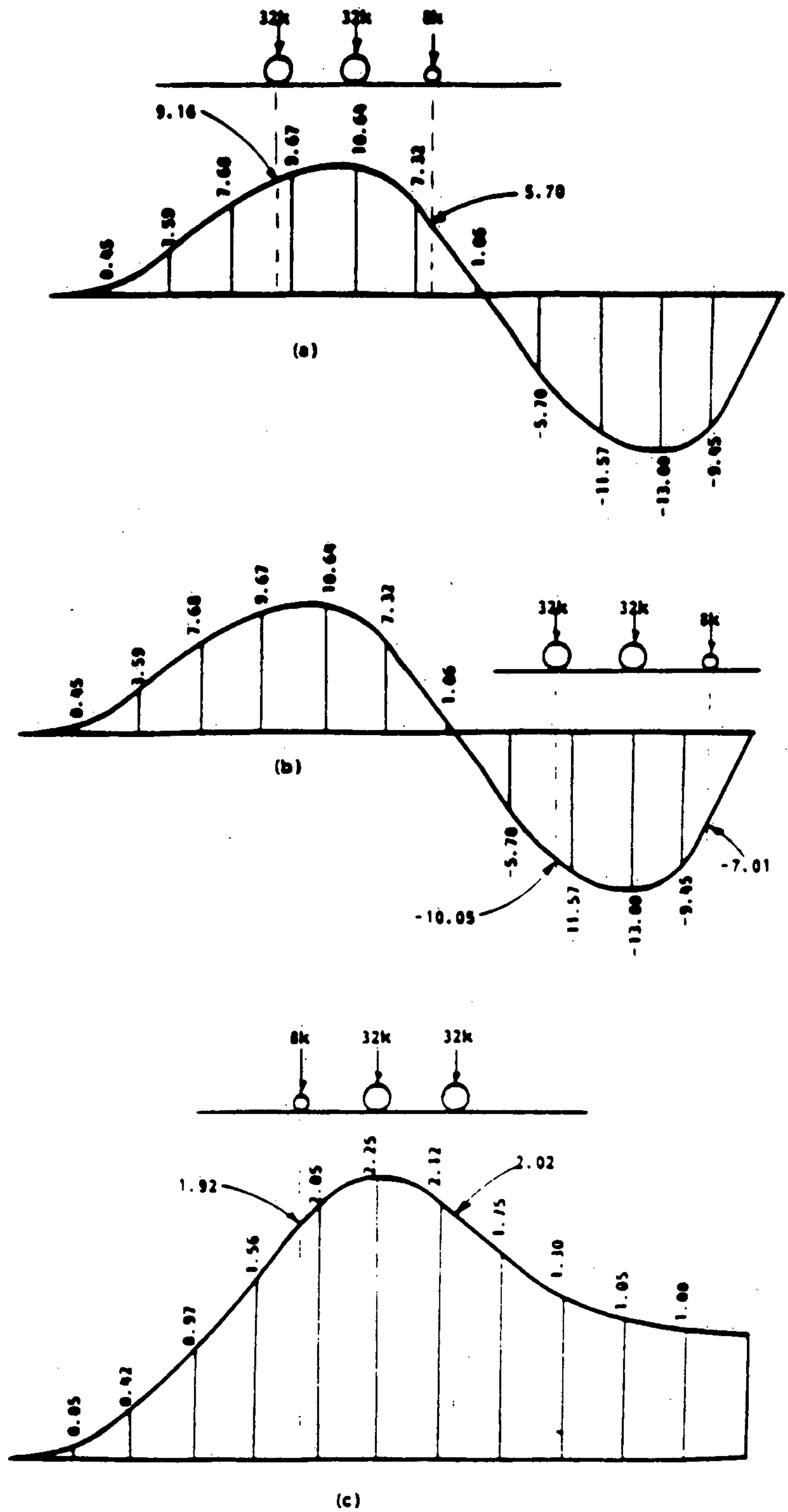


Figure A-6. Wheel loading: (a) for maximum positive moment, (b) for maximum negative moment, and (c) for maximum thrust.

where:

$$a_b = \left(\frac{87{,}000}{(87{,}000 + f_y)}\right) \beta \cdot d$$

$$= \left(\frac{87{,}000}{(87{,}000 + 33{,}000)}\right)(0.85)(64.5) = 39.8 \text{ in.}$$

$d' = 3.0$ in.

$$d'' = (h/2 - 3.0) = \frac{67.5}{2} - 3.0$$
$$= 30.8 \text{ in.}$$

$$M_b = 0.85(3)(12)(39.8)(64.5 - 30.8 - (39.8/2)) + 1.56(33)(64.5 - 3.0 - 30.8) + 1.56(33)(30.8)$$

Table A-1. Moments and thrusts at the right springing for dead load.

Point	Influence Ordinate Thrust	Influence Ordinate Moment	Loads (lb)	Thrust (lb)	Moment (lb-ft)
Left end A	0.05	0.45	13830	692	6224
B	0.42	3.59	11720	4922	42074
C	0.97	7.68	10800	10476	82944
D	1.56	9.67	10820	16897	104629
E	2.05	10.69	10640	21812	113742
F	2.25	7.32	12440	27990	91061
G	2.12	1.06	10640	22557	11278
H	1.75	-5.70	10820	18935	- 61674
I	1.30	-11.57	10800	14040	- 61674
Right J	1.05	-13.00	11720	12306	-152360
K	1.00	-9.45	13830	13830	-130693
Sum			164439	+ 451952	-561043

Thrust = 165 kips   Net Moment = -109 kip-ft

$= 20{,}034$ in.-kip

$= \quad 1{,}669$ ft-kip

$M_{ub} = \Phi \, M_b; \quad \Phi = 0.9$

$M_{ub} = \quad 1{,}502$

$P_b = 0.85 \, f'_c \, b \, a_b + A'_s f'_s - A_s f_y$

$= 0.85(3)(12)(39.8) + 1.56(33) - 1.56(33)$

$= \quad 1{,}218$ kip

$P_{ub} = \Phi P_b; \quad \Phi = 0.70$

$P_{ub} = 853$ kip

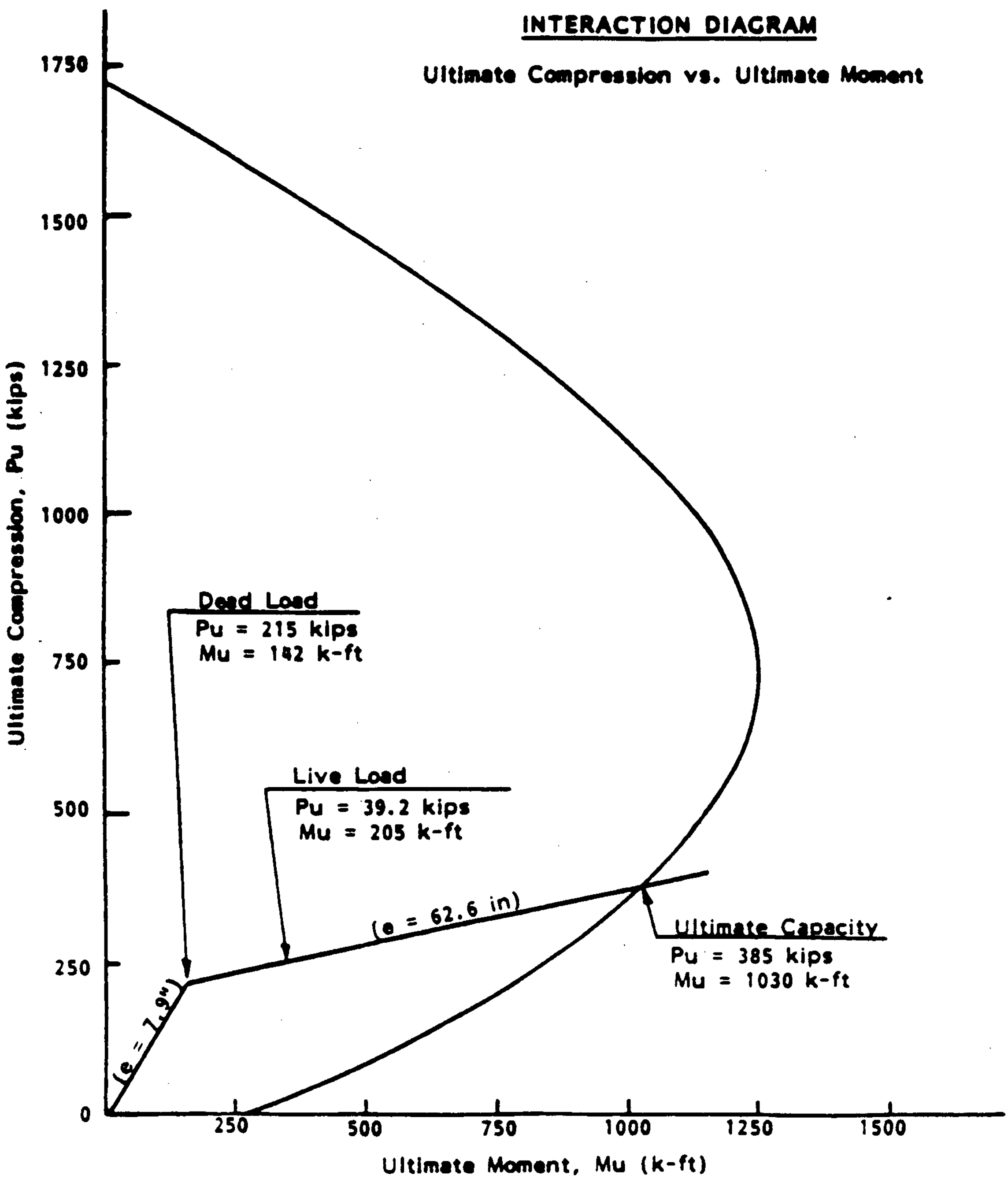


Figure A-7. Ultimate capacity interaction curve.

By using the foregoing values of ultimate moment and axial capacity and balanced moment and axial capacity, a column interaction diagram can be plotted. There are programs, however, available to determine the points of the interaction diagram for a given cross section (*A-4*). The diagram can be used to study the strength of the column (arch). Any combination of axial load and moment which falls inside the interaction diagram is satisfactory, while any combination that falls outside the curve represents failure.

4. Factored loads:

- *Dead Loads*
  $P_u = 1.3(1.65)$; $M_u = (1.3)(109)$; $e = M_u/P_u$
  $P_u = 215$ kip; $M_u = 142$ kip-ft; $e = 7.9$ in.
- *Live Loads*
  $P_u = (1.3)(1.67)(18.1)$; $M_u = (1.3)(1.67)(94.5)$
  $P_u = 39.2$ kip; $M_u = 204.7$ kip-ft
  $e = 62.7$ in.

5. Rating factors: Figure A-7 shows the interaction diagram that was developed from points obtained from a computer program (*A-4*). Point A, located at an eccentricity of 7.9 in. from the origin, represents the factored dead load moment and thrust that has been applied to the bridge. The factored live load moment and thrust have an eccentricity of 62.6 in. Continuing from point A with that eccentricity, one eventually intersects the interaction diagram at the point of maximum strength, i.e., P_u (maximum) is 385 kip and M_u (maximum) is 1,030 kip-ft.

$$RF_{\text{opr}} = \frac{M_u - M_D}{1.3\ M_{L+I}} = \frac{1{,}030 - 142}{1.3(204.7)} = 3.34 \qquad RF_{\text{opr}} = \frac{N_u - N_D}{1.3\ N_{L+I}} = \frac{385 - 215}{1.3(39.2)} = 3.34$$

$$RF_{\text{inv}} = 0.6\ RF_{\text{opr}} = 2.0$$

6. Capacity rating: R_{opr} = HS-67, and R_{inv} = HS-40.

REFERENCES

A-1. AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS, *Manual for Maintenance Inspection of Bridges.* Washington, D.C. (1983) p. 50.

A-2. AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS, *Standard Specifications for Highway Bridges.* Washington, D.C. (1983).

A-3. FEDERAL HIGHWAY ADMINISTRATION, U.S. Department of Transportation, "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges." Washington, D.C. (1979) p. 38.

A-4. MCPHERSON, R. B., AND UBILLA, G. B., "Civil Engineering Program Package for the IBM Personal Computer." Department of Civil Engineering, New Mexico State University, Las Cruces, New Mexico (1984).

APPENDIX B

EXAMPLE RATINGS OF NONDISTRESSED BRIDGES

This appendix contains capacity calculations on a prestressed concrete bridge. The bridge is divided into two examples. The bridge is rated by the service load design method (allowable stress method) in the first example, and it is rated by the strength design method (load factor) in the second example. The first example considers flexure in the deck and girder. The second example considers flexure and shear and compares flexure with the first example.

These calculations show the general procedures for computing the load rating for a prestressed concrete bridge using the procedures outlined in the 1983 AASHTO *Manual for Maintenance and Inspection of Bridges* henceforth referred to as the AASHTO Manual; and the AASHTO *Standard Specifications for Highway Bridges,* henceforth referred to as the AASHTO Bridge Specifications.

EXAMPLE 1

This bridge is a two-lane structure with three simply supported spans, typical of many prestressed concrete bridges built at grade separations for the Interstate system during the early sixties. The superstructure consists of pretensioned AASHTO girders acting with a composite cast-in-place deck. The bridge elevation and cross section are shown in Figures B-1 and B-2. The service load design method (allowable stress) will be used to rate the deck and girder of the center span for flexure. The rating vehicle is an HS-20 truck.

Construction and Design Details

- Prestressing steel: $\frac{7}{16}$ in., 7-wire strand, Grade 270, $A_s{}^*$ per strand = 0.115 in.^{2}
- Reinforcing steel: Grade 40; No. 5 bars, A_s = 0.31 in.^{2} per bar; No. 4 bars, A_s = 0.20 in.^{2} per bar (No. 5 used in deck, No. 4 used for stirrups).
- Concrete: Precast (girders), f_c' = 5,000 psi; cast-in-place (deck), f_c' = 3,000 psi.
- Girders were unshored during construction of the deck. A future wearing surface of 15 psf was assumed in design in addition to the $\frac{1}{4}$-in. wearing surface.

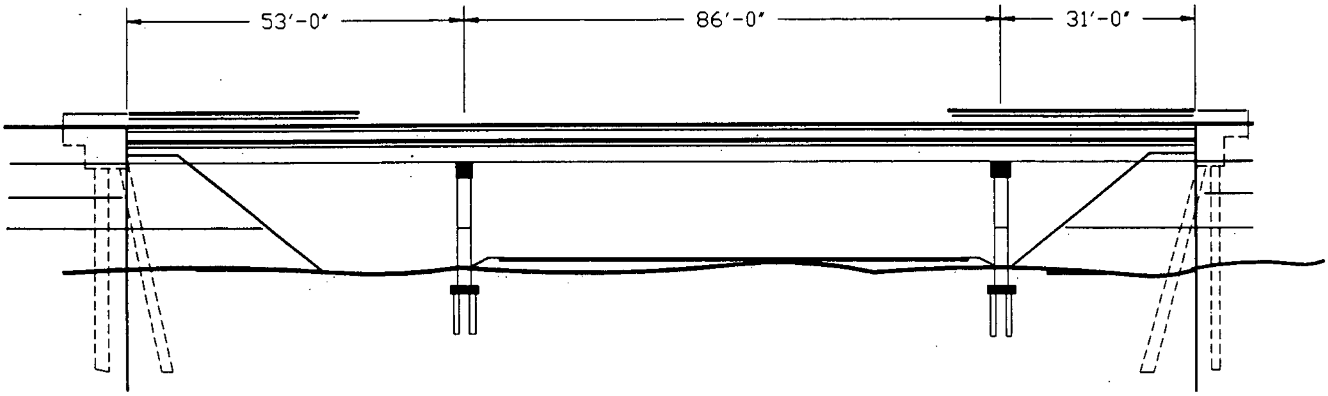


Figure B-1. Bridge elevation for Example 1.

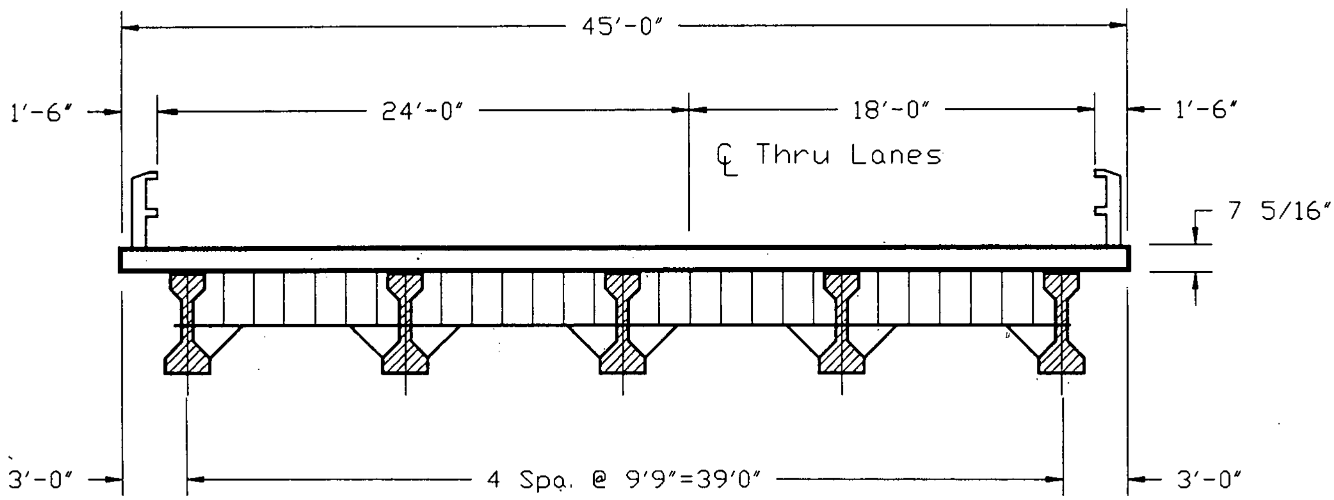


Figure B-2. Cross section for Example 1.

Deck Rating

1. Flexural capacity: Using a unit width section (12 in.) of the deck, as shown in Figure B-3, a ¼-in. wearing surface, the allowable stresses as listed in the AASHTO Manual, and the techniques for reinforced concrete, as discussed in Appendix A, the following moment capacities were determined: M_t (inv) = 6.24 kip-ft/ft, and M_t (opr) = 8.74 kip-ft/ft.

2. Dead load effect: Using the dead load of the deck, a future wearing surface of 15 lb/ft², a clear span of 8.08 ft between girders, and a continuity factor of 0.8 (because the deck is continuous), the following dead load moment was determined: M_{DL} = 0.70 kip-ft/ft.

3. Available live load capacity:

Inventory: $M_{LL} = M_t - M_{DL} = 6.24 - 0.70 = 5.54$ kip-ft

Operating: $M_{LL} = M_t - M_{DL} = 8.74 - 0.70 = 8.04$ kip-ft

4. Required live load capacity: Using an impact factor of 1.3 and the "Westergaard" formula of Section 3.24.3.1 of the AASHTO Bridge Specifications, the required live load capacity was determined for an HS-20 loading: M_{L+I} = 5.24 kip-ft/ft.

5. Deck capacity rating:

Inventory: $RF_{\text{inv}} = M_{LL}/M_{L+I} = 5.54/5.24 = 1.05$

Capacity rating = RF times HS designation
= (1.05) (HS-20)

Capacity rating = HS-21 (inventory)

Operating: $RF_{\text{opr}} = M_{LL}/M_{L+I} = 8.04/5.24 = 1.53$

Capacity rating = RF times HS designation = (1.53) (HS-20)
= HS-30.7 (operating)

Girder Rating—Flexure

1(a). Inventory flexural capacity: The cross section is shown

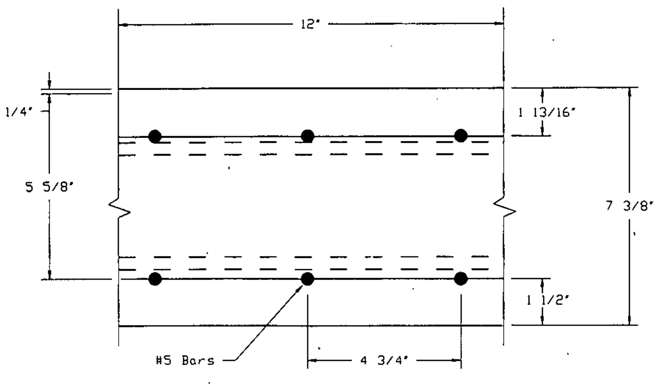


Figure B-3. Deck section for Example 1.

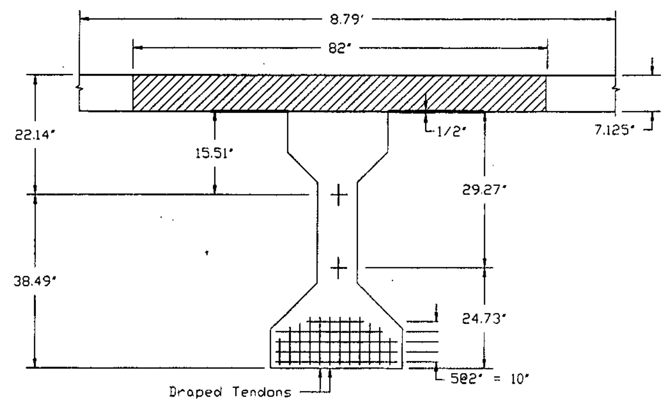


Figure B-4. Interior girder section at midspan for Example 1.

in Figure B-4. The cross-sectional properties of the AASHTO Type IV girder used are as follows:

Area, A_p = 789 in.^{2}
Moment of inertia, I_p = 260,741 in.^{4}
Centroid (to top), C_{1p} = 29.27 in.
Centroid (to bottom), C_{2p} = 24.73 in.
Radius of gyration, r_p^2 = 330 in.^{2}
Weight per foot, w = 822 plf
Section modulus (top), S_{1p} = 8,908 in.^{3}
Section modulus (bottom), S_{2p} = 10,544 in.^{3}

The composite cross-sectional properties are as shown. Effective flange width b_e is equal to the smallest of the following:

Spacing: S = 9.75 ft
One fourth of span: ¼ L_s = 21.5 ft
Or girder width plus 12 slab thicknesses: $b_v + 12t_f$ = [20 + 12(7.125)]/12 = 8.79 ft (Controls)

Thus, b_e = 8.79 ft.
The transformed flange width b_{tr} must be determined by rating the moduli:

$$E_c = 33w^{1.5}\sqrt{f_c'}$$
$$E_{c,girder} = 33(150)^{1.5}\sqrt{5{,}000} = 4.29 \times 10^6 \text{ psi}$$
$$E_{c,deck} = 33(150)^{1.5}\sqrt{3{,}000} = 3.32 \times 10^6 \text{ psi}$$
$$n_c = 3.32/4.29 = 0.774$$
$$b_{tr} = 0.774(8.79)$$
$$= 6.80 \text{ ft; or } 82 \text{ in.}$$

The composite neutral axis, y, from bottom of girder is:

$$y = \frac{7.125(82)(57.06) + 789(24.73)}{7.125(82) + 789} = 38.49 \text{ in.}$$

Other composite properties:

Area, A_c = 7.125(82) + 789 = 1,373 in.^{2}

Moment of inertia, $I_c = 260{,}741 + 789(38.49 - 24.73)^2 + 1/12(82)(7.125)^3 + 7.125(82)(57.06 - 38.49)^2 = 614{,}945$ in.^{4}

Centroid location:
c_{1c} = 54 − 38.49 = 15.51 in. (to top)
c_{2c} = 38.49 in. (to bottom)
c_{3c} = 22.14 in. (to interface)
Section moduli:
S_{1c} = 614,945/15.51 = 39,648 in.^{3} (to top)
S_{2c} = 614,945/38.49 = 15,977 in.^{3} (to bottom)
S_{3c} = 614,945/22.135 = 27,782 in.^{3} (to interface)

Prestressing steel eccentricity, e_p, at midspan is given by taking moments of the steel strands about the bottom of the girder (Figure B-4):

e_p = distance to centroid from bottom minus distance to steel centroid from bottom
e_p = 24.73 − [12(2 + 4 + 6) + 10(8) + 6(10)]/52
e_p = 24.73 − 5.46 = 19.27 in.

The effective depth of the composite section d_c is:

d_c = total depth minus distance to centroid of steel
d_c = 60.625 − 5.46 = 55.17 in.

Prestressing force p:

The total steel = No. strands times areas of each strand
A^*_s = 52(0.115) = 5.98 in.^{2}

A^*_s = 5.98 in.^{2}

The prestress force is 70 percent of the ultimate (grade) minus losses. Assume a loss of 45 ksi:

$$P = [0.70(270) - 45]5.98$$
$$P = 861 \text{ kip}$$

Dead load moments M_d and M_{dc}:

Noncomposite dead load w_d:
girders = 822 plf
deck = 7.375(9.75)(150)/12 = 900 plf
diaphragms = (Assume) 50 plf
Total = 1,772 plf; or 1.77 klf

Composite dead load w_{dc}:
wearing surface = 15(9.75) = 146 plf
curbs, rails, etc = (Assume) 100 plf
Total = 246 plf; 0.246 klf

Dead load moment = $w_d L^2/8$
$M_d = 1/8(1.77)(86)^2 = 1{,}636$ kip-ft
Composite dead load moment = $W_{dc}L^2/8$
$M_{dc} = 1/8(0.246)(86)^2 = 227$ kip-ft
Allowable concrete stresses:

	Girder	Deck
Compression ($0.4f'_c$)	−2,000 psi	−1,200 psi
Tension ($6\sqrt{f'_c}$)	424 psi	N/A

The allowable stress in the deck must be transformed in order to use the previous composite properties by dividing it by the modular ratio n_c: 1,200/0.774 = 1,551 psi.

The allowable compressive stresses are negative. The inventory live load capacity M_{LL} is the smaller of the moments controlled by the stress at the top of the girder, the bottom of the girder, or the deck.

Top of girder:

$$M_{LL} = \frac{S_{1c}}{S_{1p}}\left[-S_{1p}f_{c1} - P\left(\frac{r^2p}{c_{1p}} - e_p\right) - M_d\right] - M_{dc}$$

$$= \frac{39{,}648}{8{,}908}\left[\frac{-8{,}908(-2{,}000)}{12{,}000} - 861\left(\frac{330}{29.27} - 19.27\right)\right.$$

$$\left.\frac{1}{12} - 1{,}636\right] - 227$$

$M_{LL} = 1{,}653$ kip-ft

Bottom of girder:

$$M_{LL} = \frac{S_{2c}}{S_{2p}}\left[S_{2p}f_{c2} + P\left(\frac{r^2p}{c_{2p}} + e_p\right) - M_d\right] - M_{dc}$$

$$= \frac{15{,}977}{10{,}544}\left[\frac{10{,}544(424)}{12{,}000} + 861\left(\frac{330}{24.73} + 19.27\right)\frac{1}{12}\right.$$

$$\left.-1{,}636\right] - 227$$

= 1,404 kip-ft (Controls)

Deck:

$$M_{LL} = -S_{3c}\left(\frac{f_{c3}}{n_c}\right) - M_{dc}$$

$$= -27{,}782\left(\frac{-1{,}551}{12{,}000}\right) - 227$$

$M_{LL} = 3{,}364$ kip-ft

The inventory live load capacity is controlled by the stress (tension) at the bottom of the girder. Thus, $M_{LL} = 1{,}404$ kip-ft (Controls).

1(b). Operating flexural capacity: The transformed flange width (based on ratio of concrete strengths because computations are nonelastic) is, as before, $b_e = 8.79$ ft; b_{tr} = ratio of strengths times effective width; $b_{tr} = (3{,}000/5{,}000)8.79 = 5.27$ ft; or 63 in.

Steel failure stress f^*_{su}:

Reinforcement ratio $\rho^* = (A^*_s)/((b_{tr})(d_c)) = (5.98)/(63(55.17)) = 0.001721$
Effective steel stress $f_{se} = (P)/(A^*_s) = 861/5.98 = 144$ ksi
Steel ultimate stress $f'_s = 270$ ksi
Steel stress at failure, f^*_{su}, can be estimated by:

$$f^*_{su} = f'_s\left(1 - 0.5\frac{\rho^* f'_s}{f'_c}\right)$$

If $f_{se}/f'_s > 0.5$:
$f_{se}/f'_s = 144/270 = 0.53 > 0.50$

Therefore:

$$f^*_{su} = 270\left(1 - \frac{0.5(0.001721)(270)}{(5{,}000/1{,}000)}\right) = 257 \text{ ksi}$$

Check the distance to the bottom of the compressive stress block at failure to determine if the block is contained within the flange.

Compute compressive force, C, assuming entire flange is effective:

$$C = 0.85\,(f'_c)(b_{tr})(t_f)$$

$$C = 0.85\,(5{,}000)(63)(7.125)/1{,}000 = 1{,}908 \text{ kip}$$

Compute tensile force, T, at failure:

$$T = A^*_s(f^*_{su}) = 1{,}537 \text{ kip}$$

Because T of 1,537 kip is less than the C of 1,908 kip computed assuming the entire flange effective, the bottom of the compressive stress block is within the flange and the ultimate moment M_u can be computed as if a rectangular section of width b_{tr}.

The ultimate moment capacity M_u is:

$$M_u = \Phi A^*_s f^*_{su}\left(d = \frac{a}{2}\right)$$

where

$$a = \frac{T}{0.85(f'_c)(b_{tr})}$$

and

$$\Phi = 1.0 \text{ for prestressed girder}$$

$$a = \frac{1{,}537}{(0.85)(5)(63)}$$

$$M_u = (1.0)(5.98)(257)(55.17 - 5.72/2)/12$$

$$M_u = 6{,}700 \text{ kip-ft}$$

The operating capacity 0.75 M_u is:

$$0.75(6{,}700) = 5{,}030 \text{ kip-ft}$$
$$0.75\, M_u = 5{,}030 \text{ kip-ft}$$

2. Dead load effect: The moments M_d and M_{dc} were previously determined to be 1,636 kip-ft and 227 kip-ft, respectively. Dead load moment $M_{DL} = 1{,}636 + 227 = 1{,}863$ kip-ft; $M_{DL} = 1{,}863$ kip-ft.

3. Available live load capacity:

$$\text{Inventory: } M_{LL} = 1{,}404 \text{ kip-ft}$$
$$\text{Operating: } M_{LL} = 0.75\, M_u - M_{DL}$$
$$M_{LL} = 5{,}030 - 1{,}863 = 3{,}170 \text{ kip-ft}$$

4. Required live capacity: From Appendix A of the AASHTO Bridge Specifications, for a span of 86 ft, the maximum moment due to the HS-20 is 1,273 kip-ft or 636.5 kip-ft per wheel line.

Impact factor = 1.24
Distribution factor = S/5.5 = 9.75/5.5 = 1.77
Live load moment M_{L+I}:

$$M_{L+I}P_I = (DF)\,(IF)\,(M)$$
$$(1.77)(1.24)(636.5) = 1{,}397 \text{ kip-ft}$$
$$M_{L+I} = 1{,}397 \text{ kip-ft}$$

5. Capacity rating:

$$\text{Inventory:} \quad RF_{\text{inv}} = 1{,}404/1{,}397 = 1.01$$
$$1.01(\text{HS-20}) = \text{HS-20}$$
$$\text{Operating:} \quad RF_{\text{opr}} = 3{,}170/1{,}397 = 2.27$$
$$2.27(\text{HS-20}) = \text{HS-45}$$

Bridge Rating

The equivalent HS capacity ratings that were computed in this example are as follows:

	Inventory	Operating
Deck:	HS-21	HS-31
Girder (flexure):	HS-20	HS-45

It can be seen that the inventory rating is controlled by flexure in the girders. The operating rating, on the other hand, is controlled by the deck. The shear capacity ratings might also be a factor. Because the proceccdure for shear is identical in both the service load method and the strength design method, it is considered only in the strength design method example.

EXAMPLE 2

In this example, the bridge in Example 1 is rated using the strength design method (load factor). The deck is rated for flexure and an interior girder of the center span is rated for flexure, shear, and bearing. At the end of this example, the capacity ratings for the allowable stress and load factor methods are compared.

Deck Rating—Flexure

1. Flexural capacity: Effective depth $d = 5.625$ in., steel area $A_s = 0.783$ in.^{2}/ft, and depth of stress block $a = (A_s f_y)/(0.85\, ftb)$:

$$\frac{0.783\,(40)}{0.85(3{,}000/1{,}000)(12)} = 1.02 \text{ in.}$$

With a strength reduction factor of 0.9, ultimate moment capacity M_u, then, is:

$$M_u = \Phi A_s f_y(d; - a/2)$$
$$0.9(0.783)(40)(5.625 - 1.02/2)/12 = 12.04 \text{ kip-ft/ft}$$
$$M_u = 12.04 \text{ kip-ft/ft}$$

2. Dead load effect: From Example 1, $M_{DL} = 0.700$ kip-ft/ft.

3. Available live load capacity:

$$M_{LL} = M_u - 1.3\, M_{DL}$$
$$M_{LL} = 12.04 - 1.3\,(0.700) = 11.13 \text{ kip-ft/ft}$$

4. Required live load capacity: From Example 1, $M_{L+I} = 5.24$ kip-ft/ft.

5. Capacity rating:

Inventory:

$$RF_{inv} = \frac{M_{LL}}{1.3(5/3)(M_{L+I})} = \frac{11.1}{2.167(5.24)} = 0.98$$
$$0.98(\text{HS-20}) = \text{HS-19.6}$$

Operating:

$$RF_{\text{opr}} = \frac{M_{LL}}{1.3(M_{L+I})} = \frac{11.1}{1.3(5.24)} = 1.63$$
$$1.63\,(\text{HS-20}) = \text{HS-32.5}$$

Girder Rating—Flexure

1. Flexural capacity:

(a) Maximum strength criteria from Example 1, $M_u = 6{,}700$ kip-ft.
(b) Serviceability criterion (crack control).

Because fatigue strength is related to the strength of a prestressed concrete member to resist cracking, a logical serviceability criterion to check is the cracking moment, M_{cr}. An alternate RF for inventory is thus:

$$RF_{\text{inv}} = \frac{M_{cr} - M_{Dl}}{M_{L+I}}$$

$$M_{cr} = M_d + \Delta M_{cr}$$

$$M_d = 1{,}636 \text{ kip-ft from Example 1}$$

$$\Delta M_{cr} = \frac{S_{2c}}{S_{2p}}\left[S_{2p} f'_r + P\left(\frac{r^2 p}{c_{2p}} + e_p\right) - M_d\right]$$

$$f'_r = 7.5\sqrt{5{,}000} = 530 \text{ psi}$$

Other properties are from Example 1:

$$\Delta M_{cr} = \frac{15{,}977}{10{,}544}\left[\frac{10{,}544(530)}{12{,}000} + 861\left(\frac{330}{24.73} + 19.27\right)\frac{1}{12} - 1{,}636\right]$$

$$\Delta M_{cr} = 1{,}772$$

$$M_{cr} = 1{,}636 + 1{,}772 = 3{,}410 \text{ kip-ft}$$

2. Dead load effect: From Example 1, $M_{DL} = 1{,}863$ kip-ft.
3. Available live load capacity:

(a) Maximum strength criteria:

$$M_{LL} = M_u - 1.3(M_{DL})$$

$$M_{LL} = 6{,}700 - 1.3(1{,}863) = 4{,}280 \text{ kip-ft}$$

(b) Concrete crack control:

$$M_{LL} = M_{cr} - M_{DL}$$

$$M_{LL} = 3{,}410 - 1{,}863 = 1{,}547 \text{ kip-ft}$$

4. Required live load capacity: From Example 1, $M_{L+I} =$ 1,397 kip-ft.
5. Capacity rating:

(a) Maximum strength criteria:

Inventory:

$$\frac{M_{LL}}{1.3\left(\frac{5}{3}\right)(M_{L+I})}$$

$$RF_{\text{inv}} = \frac{4{,}280}{2.167(1{,}397)} = 1.41$$

$$1.41(\text{HS-20}) = \text{HS-28}$$

Operating:

$$RF_{\text{opr}} = \frac{M_{LL}}{1.3(M_{L+I})} = \frac{4{,}280}{1.3(1{,}397)} = 2.35$$

$$2.35(\text{HS-20}) = \text{HS-47}$$

(b) Concrete crack control:

$$RF_{\text{inv}} = \frac{M_{LL}}{M_{L+I}} = \frac{1{,}547}{1{,}387} = 1.11$$

$$1.11(\text{HS-20}) = \text{HS-22 (Controls)}$$

The inventory flexural capacity rating for the girders is controlled by strength rather than the concrete cracking. Therefore, the inventory and operating ratings for the girder are HS-19.6 and HS-32.5, respectively.

Girder Rating—Shear

Normally, the critical sections for shear capacity will be located at distances $h/2$ and $L_s/4$ from the support for a simply supported beam. Shear capacity will be rated at these two locations: first critical section, $h/2$ from support; and second critical section, L_s from support.

First Critical Section—h/2 From Support

1. Shear capacity: Section height h: $54 - 0.5 + 7.125 = 60.625$ in., or 5.05 ft; $h/2 = 2.52$ ft; say 2.5 ft (see Figure B-4).

Typically flexure-shear cracking is not a concern at locations close to the supports. It would therefore be expected that the flexure-shear cracking force V_{ci} would have a large value. Because of this fact, only the web-shear cracking force V_{cw} will be computed at this section. For the purpose of comparison, at this section, V_{ci} is equal to 1,250 kip.

(a) Prestressing steel eccentricity, e_p, is determined as follows. The ten innermost prestressing strands deflect upward as shown in Figure B-5, which shows the deflection of the top row of strands. By similar triangles the upward vertical displacement of the top row of strands at the critical cross section with respect to the midspan tendon arrangement (Figure B-2) is 37.95 in.

Each of the ten strands deflect upward by this same amount. By taking moments of the steel strands about the bottom of the girder and subtracting from the distance from the section centroid to the bottom,

$$\begin{aligned} e_p &= 24.73 - [10(2 + 4 + 6) + 8(8) + 4(10) \\ &\quad + 2(39.95) + 41.95 + 43.95 + 45.95 + 47.95)]/52 \\ &= 24.73 - 12.76 = 11.97 \text{ in.} \end{aligned}$$

(b) Noncomposite dead load moment, M_d, at the critical section is:

$$M_d = \frac{w}{2}(L_s X - X^2)$$

where L_s = span, X = distance from support, and w = unit dead load

$$(1.77/2)[86(2.5) - (2.5)^2] = 184.7 \text{ kip} = \text{ft}$$

(c) Centroidal stress f_{pc}, in a composite member, is the stress at the centroid of the composite section due to prestress and noncomposite dead load. Using properties from Example 1:

$$\begin{aligned} f_{pc} &= -P/\ P/A_p + Pe_p(c_{2c} - c_{2p})/I_p - M_d(c_{2c} \\ &\quad - c_{2p})/I_p \\ &= -861/789 + 861(11.97)(38.49 \\ &\quad - 24.73)/260{,}741 - 184.7(12)(38.49 \\ &\quad - 24.73)/260{,}741 \\ &= -0.664 \text{ ksi} \end{aligned}$$

Now, f_{pc} is taken as a positive quantity in the equation for V_{cw}.

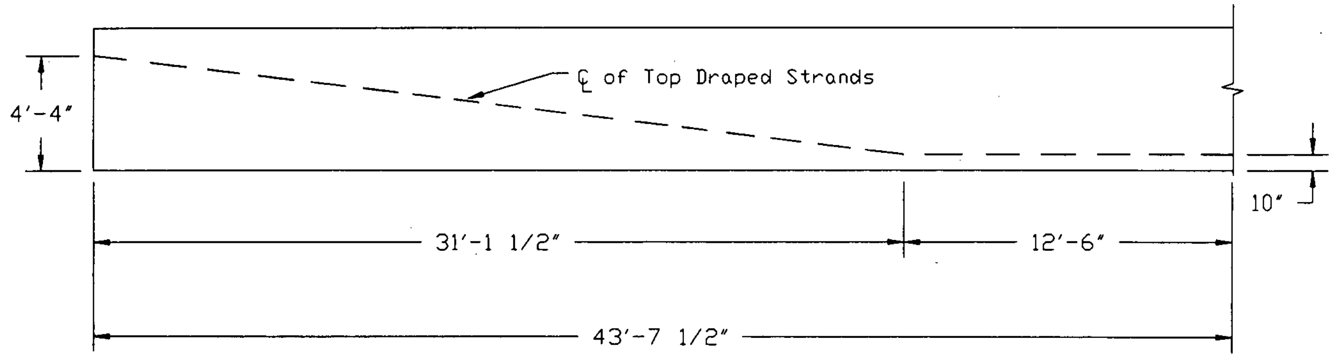


Figure B-5. Strand deflection for Example 2.

(d) Vertical prestressing component, V_p, is determined as follows. Only the ten deflecting strands contribute to the vertical prestressing component. The prestressing force being carried by the ten deflected strands is (10/52)(861) = 165.6 kip.

Theoretically, the vertical component is computed by multiplying the above force by the sine of the tendon angle. For simplicity, because the angle is small, the tangent may be used instead:

$$\tan \theta = 3.5/31.125 = 0.1125$$
$$V_p = 165.6\ (0.1125) = 18.63 \text{ kip}$$

(e) Dimensions: Web width $b' = 8$ in.; effective depth $d_c = 0.8(60.625) = 48.5$ in.

(f) Web-shear cracking load V_{cw}:

$$V_{cw} = (3.5\sqrt{f'_c} + 0.3 f_{pc})\, b'd + V_p$$
$$V_{cw} = [3.5\sqrt{5{,}000}/1{,}000 + 0.3(0.664)](8)(48.5) + 18.63$$
$$= 191.9 \text{ kip}$$

(g) Concrete shear strength, V_c, is determined by comparing the values of V_{ci} and V_{cw}; the smaller controls: thus, $V_c = 191.9$ kip

(h) Steel reinforcement shear force, V_s, for No. 4 stirrups: $A_v = 0.40$ in.^{2}; stirrup spacings (Figure B-6) = 9 in.

$$V_s = A_v f_y d/S$$
$$0.40(40)(48.5)/9 = 86.2 \text{ kip}$$
$$V_s = 86.2 \text{ kip}$$

Both S and V_s are within the limits specified in the AASHTO Bridge Specifications. Therefore, the shear reinforcement contributes to the ultimate shear capacity.

(i) Ultimate shear strength V_u, with a strength reduction factor of 0.90, is:

$$V_u = \Phi(V_{cw} + V_s)$$
$$0.90(191.9 + 86.2) = 250 \text{ kip}$$
$$V_u = 250 \text{ kip}$$

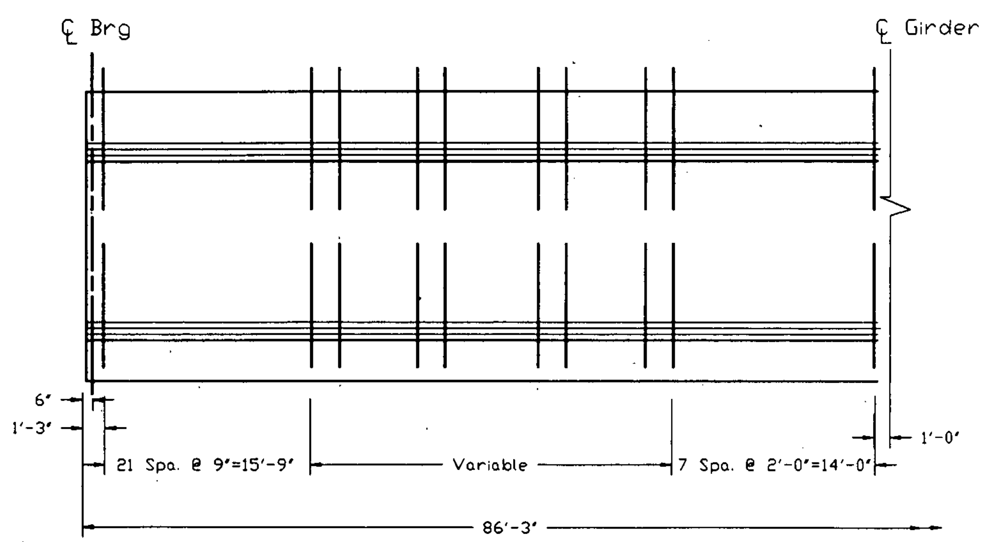


Figure B-6. Location of stirrups for Example 2.

2. Dead load effect: As previously determined in Example 1, $w_{DL} = 2.02$ klf. Dead load shear V_{DL} is:

$$V_{DL} = \frac{w}{2}(L_s - 2x)$$

$$(2.02/2)[86 - 2(2.5)] = 81.8 \text{ kip}$$

$$V_{DL} = 81.8 \text{ kip}$$

3. Available live load capacity:

$$V_{LL} = V_u - 1.3\,V_{DL}$$

$$V_{LL} = 250 - 1.3(81.8) = 143.7 \text{ kip}$$

4. Required live load capacity: Impact $I = (50)/(83.5 + 125) = 0.24$.

Plate 7 on page 46 of the AASHTO Manual will be used to determine the shear due to one wheel line of HS-20 loading. For a span length greater than 42 ft, the quantity X is $86 - 2.5 = 83.5$ ft; then:

$$V = 36(83.5 - 9.33)/86 = 31.0 \text{ kip}$$

Using the distribution factor of 1.77, the live load shear V_{L+I} is:

$$1.77(1.24)(31.0) = 68.0 \text{ kip}$$

$$V_{L+I} = 68.0 \text{ kip}$$

5. Capacity rating:

Inventory:

$$\frac{V_{LL}}{1.3\left(\frac{5}{3}\right)V_{L+I}}$$

$$RF_{\text{inv}} = \frac{143.7}{1.3\left(\frac{5}{3}\right)(68.0)} = 0.975$$

$$0.975\text{ (HS-20)} = \text{HS-19.5}$$

Operating:

$$RF_{\text{opr}} = \frac{V_{LL}}{1.3\,V_{L+I}} = \frac{143.7}{1.3(68)} = 1.63$$

$$1.63\text{ (HS-20)} = \text{HS-33}$$

Second Critical Section—$L_s/4$ From Support

1. Shear capacity: $L_s/4 = 21.5$ ft.

In contrast to the previous location, at this section web shear cracking is highly unlikely. Instead, flexure-shear cracking is more likely to control the concrete strength. Therefore, only V_{ci} will be calculated. For comparison, the magnitude of V_{cw} at this location is 243 kip.

(a) Prestressing steel eccentricity e_p: At this section, the vertical displacement of the top row of strands at this section with respect to the midspan cross section is by similar triangles $42[(31.125 - 22.0)/31.125] = 12.31$ in.

Each of the ten strands deflects upward by this same amount. As before, by taking moments of the steel strands about the bottom of the girder and subtracting from centroid distance:

$$\begin{aligned} e_p &= 24.73 - [10(2 + 4 + 6) + 8(8) + 4(10) \\ &\quad + 2(14.31 + 16.31 + 18.31 + 20.31 + 22.31)]/52 \\ &= 24.73 - 7.83 \\ &= 16.90 \text{ in.} \end{aligned}$$

(b) Dimensions: Web width $b' = 8$ in.; effective depth $d_c = 60.625 - 7.83 = 52.80$ in., which is greater than $0.8h = 48.5$ in. Use the larger value, $d_c = 52.80$ in.

(c) Noncomposite dead load shear V_d:

$$V_d = \frac{w}{2}(L_s - 2x) = (1.77/2)[86 - 2\,(21.5)]$$

$$V_d = 38.1 \text{ kip}$$

(d) Calculation of M_{max} and V_i: Both of these quantities are computed using factored superimposed dead and live loads. Hence,

$$\begin{aligned} M_{\text{max}} &= 1.3[M_{dc} + (5/3)M_{L+I}] \\ V_i &= 1.3[V_{dc} + (5/3)V_{L+I}] \\ M_{dc} &= (0.246/2)[86(21.5) - (21.5)^2] = 171 \text{ kip-ft} \\ V_{dc} &= (0.246/2)[86 - 2(21.5)] = 5.29 \text{ kip} \end{aligned}$$

Again plates from the AASHTO Manual will be used to determine the force due to the HS-20 truck. Plate 9 is applicable for the moment and Plate 7 is applicable for the shear. For both, the quantity X is taken as 64.5 ft and $(L - X)/L$ is 0.25.

$$M = 36(0.25)(64.5 - 9.33) = 497 \text{ kip-ft}$$

$$V = 36(64.5 - 9.33)/86 = 23.1 \text{ kip}$$

With impact allowance and distribution ($I = 0.26$ for shear at this location along the beam, $I = 0.24$ for moment $DF = 1.77$ as before),

$$M_{L+I} = 1.77(1.24)(497) = 1{,}091 \text{ kip-ft}$$

$$V_{L+I} = 1.77(1.26)(23.1) = 51.5 \text{ kip}$$

Thus,

$$\begin{aligned} M_{\text{max}} &= 1.3[M_{dc} + (5/3)M_{L+I}] \\ &= 1.3[170.6 + (5/3)1{,}091] = 2{,}590 \text{ kip-ft} \\ V_i &= 1.3\,[V_{dc} + (5/3)V_{L+I}] \\ &= 1.3\,[5.29 + (5/3)51.5] = 118.5 \text{ kip} \end{aligned}$$

(a) Cracking moment ΔM_{cr}:

Modulus of rupture f'_r for shear calculations:

$$f'_r = 6\sqrt{f'_c} = 6\sqrt{5{,}000}$$

$$f'_r = 424 \text{ psi}$$

(b) Noncomposite dead load moment M_d:

$$M_d = \frac{w}{2}(L_s X - X^2)$$
$$= (1.77/2)[86(21.5) - (21.5)^2]$$
$$M_d = 1{,}227 \text{ kip-ft}$$
$$\Delta M_{cr} = \frac{S_{2c}}{S_{2p}}\left[S_{2p} f'_r + P\left(\frac{r^2 p}{c_{2p}} + e_p\right) - M_d\right]$$
$$\Delta M_{cr} = 15{,}977/10{,}544[10{,}544(424)/12{,}000 + 861(330/24.73 + 16.90)/12 - 1{,}227]$$
$$= 1{,}993 \text{ kip-ft}$$

(c) Flexure-shear cracking load V_{ci}:

$$V_{ci} = 0.6\sqrt{f'_c}\, b'd + V_d + \frac{V_i \Delta M_{cr}}{M_{max}}$$
$$V_{ci} = 17.92 + 38.1 + 118.5(1{,}993)/2{,}590$$
$$= 147.2 \text{ kip}$$

(d) Concrete shear strength V_c: At this section, as expected, flexure-shear cracking controls. $V_c = 147.2$ kip.

(e) Steel reinforcement shear force V_s:

For No. 4 stirrups: $A_v = 0.40$ in.^{2}

Stirrup spacing S: 15 in.

$$V_x = A_v f_y d/S$$
$$0.40(40)(52.80)/15 = 56.3 \text{ kip}$$
$$V_s = 56.3 \text{ kip}$$

Again, both S and V_s are within the limits specified and, therefore, the shear reinforcement is effective.

(f) Ultimate shear strength V_u: With a strength reduction factor of 0.90,

$$V_u = 0.9(V_c + V_s) = 0.90(147.2 + 56.3)$$
$$V_u = 183.2 \text{ kip}$$

2. Dead load effect:

Dead load shear V_{DL}:

$$V_{DL} = \frac{w}{2}(L_s - 2x)$$
$$(2.02/2)[86 - 2(21.5)] = 43.4 \text{ kip}$$
$$V_{DL} = 43.4 \text{ kip}$$

3. Available live load capacity:

$$V_{LL} = V_u - 1.3\, V_{DL}$$
$$V_{LL} = 183.2 - 1.3(43.4) = 126.8 \text{ kip}$$

Table B-1. Comparison of bridge ratings for Examples 1 and 2.

Failure Mode and Location	Service Load (Allowable Stress)		Strength (Load Factor)	
	Inv	Opr	Inv	Opr
Deck - Flexure	HS21	HS31	HS20	HS33
Interior Girder - Flexure Crack Control (LFM only)	HS20 -	HS45 -	HS28 HS22	HS47 -
Interior Girder - Shear h/2 from Support $L_s/4$ from Support	 - -	 - -	 HS20 HS23	 HS33 HS38

4. Required live load capacity: In the calculation of V_i above, V_{L+I} was determined to be 51.5 kip.

5. Capacity rating:

Inventory:

$$\frac{V_{LL}}{1.3(5/3)(V_{L+I})}$$
$$RF_{inv} = \frac{126.8}{2.167(51.5)} = 1.14$$
$$1.14(\text{HS-20}) = \text{HS-23}$$

Operating:

$$RF_{opr} = \frac{V_{LL}}{1.3(V_{L+I})} = 1.90$$
$$1.90(\text{HS-20}) = \text{HS-38}$$

The shear capacity ratings are lower at the first critical section where web-shear cracking controls. Hence, the shear capacity ratings for this bridge are: Inventory = HS-20; and Operating = HS-33.

Comparison of Capacity Ratings Using the Allowable Stress and Load Factor Methods

Capacity ratings for this bridge, computed using the two methods, are given in Table B-1. As can be seen, the two methods produced capacity ratings that are comparable in magnitude. The load factor method ratings are slightly higher for girder flexure. The final bridge ratings are: Inventory HS-20, and Operating HS-33.

The deck and shear both indicated an HS-20 for an inventory capacity as they did for the operating capacity.

The serviceability criteria for LFM in flexure indicates a value much closer to the service load than the strength criteria alone. This closeness can be expected because the service load method has serviceability criteria built into its procedure and allowable stresses.

APPENDIX C

EXAMPLE RATINGS OF BRIDGES WITH DISTRESS

This appendix takes the bridge that was rated without distresses in Examples 1 and 2 of Appendix B, assumes traffic damage, and rerates the bridge. The damage is assumed to be from an overheight truck that has damaged the fascia and first interior girder. Example 1 of this appendix does the load rating using the service design load method (allowable stress) assuming that flexure controls.

Example 2 uses the strength design method to rate the bridge, checking both the shear and flexure modes of failure. It will be shown that in this case the shear mode of failure controls.

EXAMPLE 1

In this example the bridge that was rated in the two examples of Appendix B is now rated assuming that the bridge has been damaged. Truck damage to the fascia and first interior girder at $0.3L_s$ in the 86-ft span has occurred. The damaged section is as shown in Figure C-1. As can be seen from the figure, there is damage to both concrete in the lower flange and prestressing tendons. In this example the interior girder will be rated for flexure at that location using the allowable stress method.

1(a). Inventory flexural capacity: The cross-sectional properties of the damaged AASHTO Type IV girder are shown below, as computed using normal mechanics of materials. A typical division into simple shapes is shown.

Area	$A_p = 655.5$ in.^{2}
Moment of inertia	$I_p = 188{,}600$ in.^{4}
Centroid, top	$c_{1p} = 24.97$ in.
Centroid, bottom	$c_{2p} = 29.03$ in.
Square of radius of gyration	$r^2_p = 287.7$ in.^{2}
Unit weight	$w = 683$ plf
Section modulus	$S_{1p} = 7{,}553$ in.^{3}
Section modulus	$S_{2p} = 6{,}497$ in.^{3}

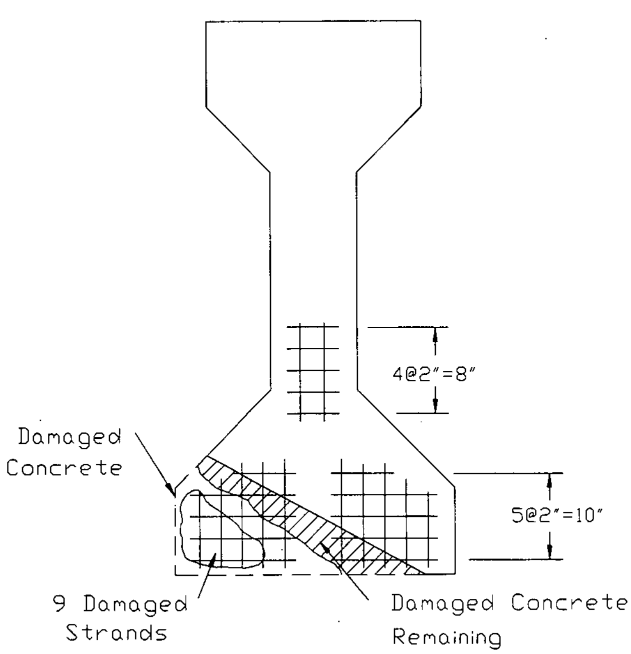
The composite cross-sectional properties from Appendix B are $b_e = 8.79$ ft and $b_{tr} = 82$ in.

Locate the composite neutral axis from the bottom of girder:

$$Y = \frac{7.125(82)(57.06) + 655.5(29.03)}{7.125(82) + 655.5} = 42.24 \text{ in.}$$

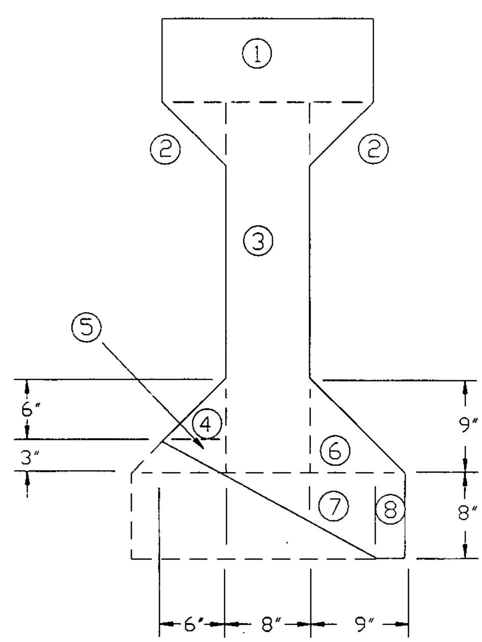
Properties include:

Area	$A_c = 1{,}240$ in.^{2}
Moment of inertia	$I_c = 433{,}800$ in.^{4}



a) Damage to Girder

Figure C-1. Damaged girder section at $0.3L_s$.



b) Sections for Calculating Section Properties

Figure C-1. Continued

Centroid, interface	$c_{1c} = 11.76$ in.
Centroid, bottom	$c_{2c} = 42.24$ in.
Centroid, top	$c_{3c} = 18.38$ in.
Section modulus, interface	$S_{1c} = 36{,}890$ in.^{3}
Section modulus, bottom	$S_{2c} = 10{,}270$ in.^{3}
Section modulus, top	$S_{3c} = 23{,}600$ in.^{3}

Prestressing steel eccentricity e_p: Figure C-1 shows that nine strands are either damaged or broken and all nine are considered to be ineffective. The number of remaining strands is 52 − 9 = 43 strands. Using Figure B-5 and by similar triangles, at $0.3L_s$ each of the ten draped tendons rises by the following amount: 42[(31.125 − 26.3)/31.125] = 6.51 in. By taking moments of the strands about the bottom of the girder, the centroid of the steel is established. Subtracting this distance from the centroid distance for the girder yields e_p, the prestressing steel eccentricity.

$$e_p = 29.03 - [6(2) + 7(4) + 8(6) + 8(8) + 4(10) + 2(8.51 + 10.51 + 12.51 + 14.51 + 16.51)]/43$$
$$e_p = 29.03 - 7.37 = 21.66 \text{ in.}$$

Effective depth of the composite section d_c:

d_c = depth minus distance from bottom to centroid of steel
$$d_c = 60.625 - 7.37 = 53.26 \text{ in.}$$

Prestressing force P:

P = (70 percent f'_s − loss) A^*_s
Steel area $A^*_s = 43(0.115) = 4.95$ in.^{2}
Prestress losses are assumed to be 45 ksi
P = [0.70(270) − 45]4.95 = 713 kip

Dead load moments M_d and M_{dc}:

Noncomposite dead load w_d:
girders = 683 plf
deck = 7.375(9.75)(150)/12 = 900 plf
diaphragms = (Assume) 50 plf
Total = 1,630 plf, or 1.6 klf

Composite dead load w_{dc}: From Appendix B the composite dead load remains the same.

$$w_{dc} = 0.246 \text{ klf}$$
$$M = W/2\,(L_s x - x^2)$$
$$M_d = 1.63/2[(86(25.8) - (25.8)^2] = 1{,}266 \text{ kip-ft}$$
$$M_{dc} = 0.246/2[(86(25.8) - (25.8)^2] = 191.0 \text{ kip-ft}$$

Allowable concrete stresses:

	Girder	Deck
Compression ($0.4f'_c$)	−2,000 psi	−1,200 psi
Tension ($6/f'_c$)	424 psi	—

The allowable stress in the deck is transformed by dividing it by the modular ratio, between the concretes, n_c:

$n_c = 0.774$ from Appendix B

$$f_c = f_{cd}/n_c$$
$$1{,}200/0.774 = 1{,}551 \text{ psi}$$

It is noted that the allowable compressive stresses are taken to be negative

Inventory live load capacity M_{LL} is the smallest based on the stress at the top of the girder, the bottom of the girder, and the top of the deck:

Top of girder:

$$M_{LL} = \frac{S_{1c}}{S_{1p}}\left[-S_{1p}f_{c1}P\left(\frac{r_p^2}{C_{1p}} - e_p\right) - M_d\right] - M_{dc}$$
$$36{,}890/7{,}553[-7{,}553(-2{,}000)/12{,}000$$
$$-713(287.7/24.97 - 21.66)/12 - 1{,}266] - 191.0$$
$$= 2{,}720 \text{ kip-ft}$$

Bottom of girder;

$$M_{LL} = \frac{S_{2c}}{S_{2p}}\left[-S_{2p}f_{c2} + P\left(\frac{r_p^2}{C_{2p}} + e_p\right) - M_d\right] - M_{dc}$$
$$10{,}270/6{,}497[6{,}497(424)/12{,}000$$
$$+ 713(287.7/29.03 + 21.66)/12 - 1{,}266] - 191.0$$
$$= 1{,}136 \text{ kip-ft (Controls)}$$

Deck:

$$M_{LL} = S_{3c}\left(\frac{f_{c3}}{n_c}\right) - M_{dc}$$
$$-23{,}600(-1{,}551)/12{,}000 - 191.0 = 2{,}860 \text{ kip-ft}$$
$$M_{LL} = 1{,}136 \text{ kip-ft}$$

The effects of the smaller cross-sectional properties are clearly illustrated here. The missing concrete at the bottom of the section has caused an upward shift in the neutral axis. The result is that the capacity of the section as limited by the bottom fibers is significantly lower than the capacity as limited by the upper fibers.

1(b). Operating flexural capacity: The transformed flange width, as before, is $b_e = 8.79$ ft; and $b_{tr} = (3{,}000/5{,}000)8.79 = 5.27$ ft, or 63 in.

Steel failure stress f^*_{su}: Reinforcement ratio $p^* = 4.9/(63(53.26)) = 0.001475$. From Appendix B, the ratio of effective prestress f_{se} to the ultimate stress of the steel tendons f'_s is 0.53, which is greater than 0.50. Therefore, the steel stress at failure can be estimated by:

$$f^*_{su} = f'_s\left(1 - 0.5\,\frac{p^*f'_s}{f'_c}\right)$$
$$f^*_{su} = 270[1 - 0.5(0.001475)(270)/(5{,}000/1{,}000)]$$
$$= 259 \text{ ksi}$$

The section has all of the compressive block at failure in the flange as shown in Appendix B. T is less here; C remains the same.

Ultimate moment capacity, M_u:

$$M_u = \Phi\, A^*_s f^*_{su}\left(dc - a/2\right)$$
$$a = \frac{f^*_{su}A^*_s}{0.85\,(f'_c)(b_{tr})}$$
$$a = \frac{(259)(4.95)}{0.85(5)(63)} = 4.79$$
$$M_u = (1)(4.95)(259)\left(53.26 - \frac{4.79}{2}\right)\frac{1}{12}$$
$$M_u = 5{,}430 \text{ kip-ft}$$

Operating moment, 0.75 M_u:

$$0.75\ M_u = 0.75\ (5{,}430) = 4{,}070 \text{ kip-ft}$$

2. Dead load effect:

$$\begin{aligned} M_{DL} &= M_d + M_{dc} \\ M_d &= 1{,}266 \text{ kip-ft} \\ M_{dc} &= 191.0 \text{ kip-ft} \\ M_{DL} &= 1{,}266 + 191.0 = 1{,}457 \text{ kip-ft} \end{aligned}$$

3. Available live load moment:

$$\begin{aligned} \text{Inventory:}\quad M_{LL} &= 1{,}136 \text{ kip-ft} \\ \text{Operating:}\quad M_{LL} &= 0.75\ M_u - M_{DL} \\ M_{LL} &= 4{,}070 - 1{,}457 = 2{,}610 \text{ kip-ft} \end{aligned}$$

4. Required live capacity: Use Plate 9 on page 48 of the AASHTO Manual. For the HS-20, the formulas are not applicable for X equal to $0.3L$. Therefore, take $L - X$ equal to $0.3L$. Then,

$$\begin{aligned} 0.3L &= 25.8 \text{ ft} \\ (L - X)/L &= 0.3 \\ M &= 36\ (L-x/L)(X - 9.33) \\ M &= 36(0.3)(25.8 - 9.33) \\ M &= 177.9 \text{ kip-ft} \end{aligned}$$

Note that this moment is already for a wheel line, and hence, need not be divided by two.

Impact I: For live load moment computations on simple spans, the impact allowance does not change at different locations along the beam. Therefore, from Appendix B, I is 0.24 and $1 + I = 1.24$.

Distribution factor: The distribution factor remains 1.77 from Appendix B.

Live load moment M_{L+I}: $M_{L+I} = (DF)(IF)(M)$

$$\begin{aligned} &= (1.77)(1.24)(177.9) \\ M_{L+I} &= 390 \text{ kip-ft} \end{aligned}$$

5. Capacity rating:

$$\begin{aligned} \text{Inventory:}\quad RF_{\text{inv}} &= M_{LL}/M_{L+I} \\ &= 1{,}136/390 = 2.91 \\ 2.91(\text{HS-20}) &= \text{HS-58} \end{aligned}$$

$$\begin{aligned} \text{Operating:}\quad RF_{\text{opr}} &= M_{LL}/M_{L+I} \\ &= 2{,}610/390 = 6.69 \\ 6.69(\text{HS-20}) &= \text{HS-134} \end{aligned}$$

In comparing the foregoing computed ratings with those from Appendix B, it is evident that flexure capacity at the damaged section does not control. This capacity was not unexpected because distresses tend to cause significant reductions in load-carrying capacity only when they occur in critical regions. This particular location along the beam is not in a region of high flexural stresses. However, it is in a region of high shear stresses, which potentially can cause a reduction in carrying capacity.

EXAMPLE 2

The bridge in Example 1 is rated at the damaged section using the load factor method. Both the flexure and shear failure modes are considered. The serviceability criteria will not be checked in this example; some question exists as to its applicability.

Flexure Rating

1. Flexural capacity (maximum strength criteria): From Example 1, at $0.3L_s$, $M_u = 5{,}430$ kip-ft.
2. Dead load effect: $M_{DL} = 1{,}457$ kip-ft.
3. Available live load capacity: $M_{LL} = 5{,}430 - 1.3(1{,}457) = 3{,}540$ kip-ft.
4. Required live load capacity: $M_{L+I} = 390$ kip-ft.

Note: The above four items are identical to Example 1

5. Capacity rating:

$$\begin{aligned} \text{Inventory:}\quad RF_{\text{inv}} &= \frac{M_{LL}}{1.3\left(\frac{5}{3}\right) M_{L+I}} \\ &= \frac{3{,}540}{1.3\left(\frac{5}{3}\right)(390)} = 4.19 \\ 4.19(\text{HS-20}) &= \text{HS-84} \\ \text{Operating:}\quad RF_{\text{opr}} &= \frac{M_{LL}}{1.3 M_{L+I}} \\ &= \frac{3{,}540}{1.3(390)} = 6.98 \\ 6.98(\text{HS-20}) &= \text{HS-140} \end{aligned}$$

As in the allowable stress method, flexure is not a concern at the damaged section.

Shear Rating

As far as shear capacity is concerned, the most important result of the damage is the reduced section properties and web height. Because this location is near a region of high shear, it is possible that the shear rating may be lowered. Flexure-shear cracking, rather than web-shear cracking, dominates at this location and, hence, only the flexure-shear cracking load will be calculated.

1. Shear capacity:

(a) Prestressing steel eccentricity e_p: The prestressing steel eccentricity is 21.66 in. as determined in Example 1.

(b) Dimensions: Web width b' is 8 in.; section height h, at the midpoint of the web, is approximately 48 in.; effective depth d_c is $0.8h = 38.4$ in. The effective depth as computed in Example 1 is 53.26 in. However, because this value extends below the section, it should not be used. Therefore, take d_c equal to 38.4 in.

(c) Noncomposite dead load shear V_d:

$$w_d = 1.63 \text{ klf}$$
$$V_d = w/2\ (L_s - 2x)$$
$$V_d = (1.63/2)[86 - 2(25.8)] = 28.0 \text{ kip}$$

(d) Calculation of M_{max} and V_i: As in Example 1 of Appendix B, both of these quantities are computed using factored superimposed dead and live loads. Hence,

$$M_{max} = 1.3[M_{dc} + (5/3)M_{L+I}]$$
$$V_i = 1.3[V_{dc} + (5/3)V_{L+I}]$$

From Example 1:

$$M_{dc} = 191.0 \text{ kip-ft}$$
$$V_{dc} = w_{dc}/2\ (L_s - 2x)$$
$$V_{dc} = (0.246/2)[86 - 2(25.8)] = 4.23 \text{ kip}$$

Plates from the AASHTO Manual will be used to determine the force due to the HS-20 truck. Plate 9 is applicable for moment and Plate 7 is applicable for shear. Take $(L - X)/L$ as 0.3 and X as $0.7L$, or 60.2 ft.

$$M = 36\ (L-X)\ (X - 9.33)/L$$
$$M = 36(0.3)(60.2 - 9.33) = 549 \text{ kip-ft}$$
$$V = 36(X - 9.33)/L$$
$$V = 36(60.2 - 9.33)/86 = 21.3 \text{ kip}$$

For moment the impact allowance is 0.24. For shear, however, L_s in the impact formula is taken as 60.2 ft and the resulting impact is 0.27. The distribution factor remains equal to 1.77.

$$M_{L+I} = (DF)\ (IF)\ (M)$$
$$M_{L+I} = 1.77(1.24)(549) = 1{,}205 \text{ kip-ft}$$
$$V_{L+I} = (DF)\ (IF)\ (V)$$
$$V_{L+I} = 1.77(1.27)(21.3) = 47.9 \text{ kip}$$
$$M_{max} = 1.3[191.0 + (5/3)1{,}205] = 2{,}860 \text{ kip-ft}$$
$$V_i = 1.3[4.23 + (5/3)47.9] = 109.3 \text{ kip}$$

(e) Cracking moment ΔM_{cr}: Since fatigue strength is related to member's resistance to cracking, use the modulus of rupture f'_r of 424 psi and the noncomposite dead load moment M_d of 1,266 kip-ft.

$$\Delta M_{cr} = \frac{S_{2c}}{S_{2p}}\left[S_{2p}f'_r + P\left(\frac{r_p^2}{C_{2p}} + e_p\right) - M_d\right]$$
$$\Delta M_{cr} = 10{,}270/6{,}497[6{,}497(424)/12{,}000 + 713(287.7/29.03 + 21.66)/12 - 1{,}266]$$
$$= 1{,}327 \text{ kip-ft}$$

(f) Flexure-shear cracking load V_{ci}:

$$V_{ci} = 0.67\sqrt{f'_c}\,b'd + V_d + \frac{V_1 \Delta M_{cr}}{M_{max}}$$
$$0.67\sqrt{f'_c}\,b'd = 0.6/5{,}000\ (8)(38.4)/1{,}000 = 13.03 \text{ kip}$$
$$V_{ci} = 13.03 + 28.0 + 109.3(1{,}327)/2{,}860$$
$$= 91.7 \text{ kip}$$

(g) Concrete shear strength V_c: $V_c = 91.7$ kip.

(h) Steel reinforcement shear force V_s: For No. 4 stirrups, $A_s = 0.40$ in.^{2}. Stirrup spacing S (Figure B-6) is 21 in.

$$V_s = A_v f_{sv} d/S$$
$$0.40(40)(44.1)/21 = 33.6 \text{ kip}$$
$$V_s = 33.6 \text{ kip}$$

(i) Ultimate shear strength V_u: The strength reduction factor is 0.90.

$$V_u = \Phi\ (V_c + V_s)$$
$$0.90(91.7 + 33.6) = 112.8 \text{ kip}$$
$$V_u = 112.8 \text{ kip}$$

2. Dead load effect:

$$V_d = 28.0 \text{ kip}$$
$$V_{dc} = 4.23 \text{ kip}$$
$$V_{DL} = 28.0 + 4.23$$
$$= 32.2 \text{ kip}$$

3. Availability live load capacity:

$$V_{LL} = V_u - 1.3\ V_{DL}$$
$$V_{LL} = 112.8 - 1.3(32.2)$$
$$= 70.9 \text{ kip}$$

4. Required live load capacity: In the calculation of V_i, above, V_{L+I} was determined to be 47.9 kip.

5. Capacity rating:

$$RF_{inv} = \frac{V_{LL}}{1.3(5/3)(V_{L+I})}$$

$$\text{Inventory: } RF_{inv} = \frac{70.9}{1.3(5/3)(47.9)} = 0.683$$

$$0.683(\text{HS-20}) = \text{HS-14}$$

$$\text{Operating: } RF_{opr} = V_{LL}/1.3\ (V_{L+I})$$
$$= 70.9/1.3(47.9) = 1.138$$
$$1.138(\text{HS-20}) = \text{HS-23}$$

The foregoing computed shear ratings are lower than any of the previously computed ratings. Shear at the damaged section controls, and the overall capacity ratings for the entire bridge are taken to be HS-14 for inventory and HS-23 for operating. Previously, the capacity ratings were HS-20 for inventory and HS-33 for operating, both based on shear capacity at $h/2$ from support.

APPENDIX D

DECISION GUIDELINES FOR CONDITION RATING AND EVALUATION OF CONCRETE BRIDGE COMPONENTS

Included in this appendix are guidelines to assist the inspector in coding the condition ratings for the concrete components of a bridge. Four sets of guidelines are furnished—deck, reinforced concrete superstructure, prestressed concrete superstructure, and substructure. These guidelines will assist the inspector in making uniform condition ratings for the structural inventory and appraisal report.

In addition, guidelines are furnished for evaluating a component that may require particular action by the inspector. These guidelines indicate the problem, probable cause, and conceivable action.

References cited in the guidelines refer to applicable sections in the main text of the manual.

CONDITION DECISION TREE

Deck

Code and Description

9 **As Built Condition:** No noteworthy deficiencies.
Reference: Chapter 1, Art. 1.6

8 **Very Good Condition:** No problems noted. Minor cracks with no spalling, scaling, delamination, or leaching. No electrical potential greater than 0.35 V. Chloride content less than 1 lb per cu yd.
Reference: Chapter 1, Art. 1.6, 1.10; Chapter 4, Art. 4.2

7 **Good Condition:** Some minor problems. Sealable deck cracks, light scaling, less than 10 percent of deck area is deteriorated including any repaired areas and/or areas in need of corrective action. No spalling but visible tire wear acceptable. Electrical potential greater than 0.35 V on less than 45 percent of deck area. Chloride content less than 2 lb per cu yd.
Reference: Chapter 4, Art. 4.2

6 **Satisfactory Condition:** Some minor deterioration. Open cracks at intervals of 5 ft or less with or without efflorescence. Medium scaling, 2 percent or less of deck is spalled, or less than 20 percent of deck is water-saturated, contaminated, or deteriorated including repaired areas and areas in need of corrective action. No full depth failures.
Reference: Chapter 4, Art. 4.2

5 **Fair Condition:** Excessive cracking resulting in 2 percent to 5 percent of the deck spalled. Heavy scaling or 20 percent to 40 percent of the deck is deteriorated or contaminated including any repaired areas and areas in need of corrective action. Some full depth failures. Considerable leaching.
Reference: Chapter 4, Art. 4.2

4 **Poor Condition:** Advanced section loss, deterioration, or spalling. More than 5 percent of the deck is spalled or 40 percent to 60 percent of the deck is deteriorated or contaminated including any repaired areas and areas in need of corrective action. Extensive full depth cracks present. Leaching throughout deck.
Reference: Chapter 4, Art. 4.2, 4.4, 4.5, 4.6

3 **Serious Condition:** More than 60 percent of the deck is water-saturated and/or deteriorated or contaminated. This total includes the sum of all nonduplicating areas: full depth cracking, delamination, active corrosion, and chloride contamination, including any repaired areas and areas in need of corrective action.
Reference: Chapter 4, Art. 4.2, 4.4, 4.5, 4.6

2 **Critical Condition:** Full deck failures over much of deck.
Reference: Chapter 4, Art. 4.2, 4.4, 4.5, 4.6; Chapter 5, Art. 5.3

1 **Failure Condition:** Bridge closed. Corrective action may put back into light service.

0 **Failure Condition:** Bridge closed. Replacement necessary.

Superstructure—Reinforced Concrete

Code and Description

9 **As Built Condition:** No noteworthy deficiencies.
Reference: Chapter 1, Art. 1.6

8 **Very Good Condition:** No problems noted. No repairs needed. Possible minor collision damage without misalignment or corrective action required. If composite, box, or T-beam, consider deck with superstructure evaluation. Stains OK. Hairline flexure cracks OK. Comprehensive rehabilitation restores to 8.
Reference: Chapter 1, Art. 1.6, 1.8, 1.10, 1.11, 1.13.4

7 **Good Condition:** Some minor problems correctable by routine maintenance. Minor longitudinal or transverse move-

ment of the superstructure. Hairline cracks without disintegration in concrete girders, precast panels, etc. If integral deck, consider deck with superstructure evaluation. Stains OK.
Reference: Chapter 1, Art. 1.6, 1.8, 1.10, 1.11, 1.13.4

6 **Satisfactory Condition:** Some minor deterioration. Minor collision damage to nonstructural support elements. Major maintenance needed. Generally highest rating if repaired without comprehensive rehabilitation. Minor deterioration of slab ends, deck girder ends, precast stems, etc. If integral deck, consider deck with superstructure evaluation. Light leaching, no delamination.
Reference: Chapter 1, Art. 1.6, 1.8, 1.10, 1.11, 1.13.4

5 **Fair Condition:** All primary structural elements are sound but may have minor cracking or spalling. Secondary elements may have significant deterioration. Minor rehabilitation needed. No primary steel loss of section. Bearing devices need attention, not functioning. If integral deck, consider deck with superstructure evaluation.
Reference: Chapter 1, Art. 1.6, 1.8, 1.10, 1.11, 1.13.4

4 **Poor Condition:** Advanced section loss, deterioration, or spalling. Primary steel remains anchored although exposed. Little or no primary steel section loss. Critical collision damage to structural support elements and precautionary measures needed such as temporary shoring. Nonfunctional bearings causing problems to superstructure and/or substructure. No core cracking or loss of concrete inside steel cage of compression member.
Reference: Chapter 1, Art. 1.6, 1.8, 1.10, 1.11, 1.13.4; Chapter 2, Art. 2.1, 2.4; Chapter 3, Art. 3.2; Chapter 4, Art. 4.3

3 **Serious Condition:** Loss of steel section, deterioration, or spalling has seriously affected primary structural components. Repair or rehabilitation required as soon as possible. Damage or disintegration of a structural support element which requires shoring, auxiliary splices, or substitute members. Severe disintegration of concrete. Diagonal shear cracks. Wide flexure cracks. Discoloration along primary steel lines, delamination from primary steel. Core of column concrete affected. Consider condition of deck if integral with girder. Reevaluation of capacity needed. Consultation with cognizant engineer may be prudent.
Reference: Chapter 1, Art. 1.6, 1.8, 1.10, 1.11; Chapter 2, Art. 2.1, 2.3; Chapter 4, Art. 4.3, 4.4, 4.5, 4.6, 4.7; Chapter 5, Art. 5.1

2 **Critical Condition:** Advanced deterioration of primary structural elements. Repair or rehabilitation urgent. Wide shear or flexure cracks may be present. Main support member may show permanent deformation. Concrete disintegrated around reinforcing steel with loss of end anchorage. Reevaluation of capacity necessary. Notification of proper authority necessary. Consultation with cognizant engineer prudent. Bridge should be closed until corrective action is taken.
Reference: Chapter 1, Art. 1.6, 1.8, 1.10, 1.11; Chapter 2, Art. 2.1, 2.3; Chapter 4, Art. 4.3, 4.4, 4.5, 4.6, 4.7; Chapter 5, Art. 5.1

1 **"Imminent" Failure Condition:** Bridge closed. Major deterioration or section loss present in critical structural components. Corrective action may put back into light service.
Reference: Chapter 1, Art. 1.6, 1.8, 1.10, 1.11; Chapter 2, Art. 2.1, 2.3; Chapter 4, Art. 4.3, 4.4, 4.5, 4.6, 4.7; Chapter 5, Art. 5.1

0 **Failed Condition:** Out of service. Beyond corrective action.

Superstructure—Prestressed Concrete

Code and Description

9 **As Built Condition:** No noteworthy deficiencies.
Reference: Chapter 1, Art. 1.7

8 **Very Good Condition:** No problems noted. Possible minor collision damage without misalignment or corrective action required. Such damage should be documented. If composite, box, or T-beam, consider deck with superstructure evaluation. Stains OK. Hairline flexure cracks should be noted. Comprehensive rehabilitation restores to this condition rating.
Reference: Chapter 1, Art. 1.7, 1.8, 1.9, 1.10, 1.11, 1.13.4

7 **Good Condition:** Some minor problems. Minor longitudinal or transverse movement of the superstructure. Hairline cracks without disintegration in concrete girders, precast panels, etc. If integral deck, consider deck with superstructure evaluation. Stains OK.
Reference: Chapter 1, Art. 1.7, 1.8, 1.9, 1.10, 1.11, 1.13.4

6 **Satisfactory Condition:** Some minor deterioration. Minor collision damage to nonstructural support elements. Generally highest rating if repaired without comprehensive rehabilitation. Minor deterioration of slab ends, deck girder ends, precast stems, etc. If integral deck, consider deck with superstructure evaluation. Light leaching, no delamination, no cracks other than hairline.
Reference: Chapter 1, Art. 1.7, 1.8, 1.9, 1.10, 1.11, 1.13.4

5 **Fair Condition:** All primary structural elements are sound but may have minor cracking or spalling. Secondary elements may have significant deterioration. No prestress steel loss of section. Bearing devices need attention, not functioning. Consider deck with superstructure evaluation if integral with girder.
Reference: Chapter 1, Art. 1.7, 1.8, 1.9, 1.10, 1.11

4 **Poor Condition:** Advanced section loss, deterioration, or spalling. Primary steel remains anchored but may be exposed. Critical collision damage to structural elements and precautionary measures such as temporary shoring needed. Nonfunctional bearings causing problems to superstructure and/or substructure. Bearing type cracks.
Reference: Chapter 1, Art. 1.7, 1.8, 1.9, 1.10, 1.11, 1.13.4; Chapter 2, Art. 2.1, 2.2, 2.3; Chapter 3, Art. 3.2

3 **Serious Condition:** Loss of steel section, broken prestress strands, deterioration, or spalling has seriously affected primary structural components. Damage or disintegration of a structural support element which requires shoring, auxiliary splices, or substitute members. Severe disintegration of concrete. Diagonal shear cracks. Flexure cracks. Discoloration along primary steel lines, delamination from primary steel. Consider condition of deck with superstructure evaluation if integral with girder. Reevaluation of capacity needed. Consultation with cognizant engineer may be prudent.
Reference: Chapter 1, Art. 1.7, 1.8, 1.9, 1.10, 1.11; Chapter 2, Art. 2.1, 2.2, 2.3; Chapter 3, Art. 3.2; Chapter 4, Art. 4.3, 4.4, 4.7; Chapter 5, Art. 5.5, 5.6

2 **Critical Condition:** Advanced deterioration of primary structural elements. Shear cracks may be present. Main support member shows unplanned deformation. Concrete disintegrated around reinforcing steel with loss of end anchorage. Reevaluation of capacity necessary. Notification of proper authority necessary. Consultation with cognizant engineer prudent. Bridge should be closed until corrective action is taken.
Reference: Chapter 1, Art. 1.7, 1.8, 1.9, 1.10, 1.11; Chapter 2, Art. 2.1, 2.2, 2.3; Chapter 3, Art. 3.2; Chapter 4, Art. 4.3, 4.4, 4.7; Chapter 5, Art. 5.5, 5.6

1 **"Imminent" Failure Condition:** Bridge closed. Major deterioration or section loss present in critical structural components. Corrective action may put back into light service.
Reference: Chapter 1, Art. 1.7, 1.8, 1.9, 1.10, 1.11; Chapter 2, Art. 2.1, 2.2, 2.3; Chapter 3, Art. 3.2; Chapter 4, Art. 4.3, 4.4, 4.7; Chapter 5, Art. 5.5, 5.6

0 **Failed Condition:** Out of service. Beyond corrective action.

Substructure

Code and Description

9 **As Built Condition:** No noteworthy deficiencies. Possible scrape marks caused by drift or collision.
Reference: Chapter 1, Art. 1.3, 1.6

8 **Very Good Condition:** No problems noted. Shrinkage cracks, light scaling, or insignificant spalling which does not expose reinforcing steel. Insignificant damage caused by drift or collision with no misalignment. No corrective action required.
Reference: Chapter 1, Art. 1.3, 1.6

7 **Good Condition:** Some minor problems including light deterioration or initial disintegration, minor water saturation, cracking with some leaching, or spalls on concrete unit with no effect on bearing area. Leakage from deck expansion devices has initiated minor cracking. Local waterway scour near footing without misalignment, top of footing not exposed.
Reference: Chapter 1, Art. 1.3, 1.6, 1.10, 1.11, 1.12

6 **Satisfactory Condition:** Moderate to major deterioration or disintegration, spalls, extensive cracking, and leaching on concrete units with little or no loss of bearing area. No loss of concrete within core (cage) of column or pile. Scour more prominent with exposed top of footing but no misalignment by settlement. Maximum rating with structure that has received corrective action unles subjected to comprehensive rehabilitation.
Reference: Chapter 1, Art. 1.3, 1.6, 1.10, 1.11, 1.12

5 **Fair Condition:** Units show substantial section loss with exposed reinforcing steel. Core intact. Some minor loss of reinforcing steel section, no broken steel. Extensive scouring or undermining of footing potentially affecting the stability of the unit and requiring corrective action.
Reference: Chapter 1, Art. 1.3, 1.6, 1.10, 1.11, 1.12; Chapter 2, Art. 2.1, 2.4; Chapter 3, Art. 3.2; Chapter 4, Art. 4.3

4 **Poor Condition:** Advanced section loss, deterioration, or spalling. Structural cracks in concrete units, core intact. Extensive loss of reinforcing steel section. Severe scouring or undermining of footings affecting the stability of the unit which requires corrective action. Minor settlement of the substructure may have occurred. Consultation with cognizant engineer may be prudent.
Reference: Chapter 1, Art 1.3, 1.6, 1.10, 1.11, 1.12; Chapter 2, Art. 2.1, 2.3; Chapter 3, 3.2; Chapter 4, 4.3

3 **Serious Condition:** Loss of section, deterioration, or spalling has seriously affected primary structural component. Concrete core deteriorated or cracked, extensive loss of steel section. Bearing areas seriously deteriorated with considerable loss of bearing area. Blocking and shoring considered necessary to maintain the safety and alignment of the structure. Consultation with cognizant engineer prudent.
Reference: Chapter 1, Art. 1.3, 1.6, 1.10, 1.11, 1.12; Chapter 2, Art. 2.1, 2.2, 2.3; Chapter 3, Art. 3.2; Chapter 4, Art. 4.3; Chapter 5, Art. 5.3

2 **Critical Condition:** Advanced deterioration of structural elements. Concrete cap is soft and spalled. Tension reinforcing steel exposed with no anchorage at end. Top of pier cap is split or concrete column has undergone shear failure. Scour is sufficient that substructure is near state of collapse. Pier has settled. Consultation with cognizant engineer necessary. Bridge should be closed until corrective action is taken.
Reference: Chapter 1, Art. 1.3, 1.6, 1.10, 1.11, 1.12; Chapter 2, Art. 2.1, 2.2, 2.3; Chapter 3, Art. 3.2; Chapter 4, Art. 4.3; Chapter 5, Art. 5.3

1 **"Imminent" Failure Condition:** Bridge closed but corrective action may put back into light service.

0 **Failed Condition:** Out of service. Beyond corrective action.

POSSIBLE INSPECTION ACTION

Problem: Concrete deck with greater than 5 percent spalled and/or 20 percent to 40 percent heavy scaling. Rating 5 or less.
Possible cause: Extensive use of deicing salts.
Conceivable action: In-depth inspection including sounding, electrical potential measurements, and chloride content determination.
Reference: Chapter 4, Art. 4.2.

Problem: Exposed reinforcing steel.
Conceivable action: Determine if primary tension steel. If tension steel, check anchorage; loss of anchorage can be serious. Measure steel section loss. Recalculation of capacity conceivable.
Probable cause: Extensive use of deicing salts. Salt water running onto component.
Reference: Chapter 4, Art. 4.3; Chapter 5, Art. 5.1.

Problem: Large flexure cracks, diagonal tension cracks.
Possible cause: Overload causing yielding of steel. Loss of anchorage.
Conceivable action: Look for yielding. Look at termination points of steel. Recalculate capacity. Consider load restricting bridge.
Reference: Chapter 1, Art. 1.10; Chapter 5, Art. 5.1.

Problem: Extensive leaching.
Possible cause: Poor drainage; extensive use of deicing salts.
Conceivable action: Electrical potential measurements, chloride content determination.
Reference: Chapter 4, Art. 4.4, 4.6.

Problem: Frozen bearings.
Possible cause: Poor housekeeping, water running directly onto bearing.

Conceivable action: Check for duress near bearings, free bearings. Report and reevaluate maintenance procedures.
Reference: Chapter 1, Art. 1.8, 1.10.

Problem: Excessive movement indicated by cracks.
Possible cause: Settlement, stream bed scour, portland cement pavement pushing bridge.
Conceivable action: Determine cause, report, and monitor.
Reference: Chapter 1, Art. 1.6, 1.10, 1.12; Chapter 4, Art. 4.1.

Problem: Rusty crack parallel to reinforcing steel.
Possible cause: Corrosion of reinforcing steel.
Conceivable action: Record with extent of duress. Check loss of steel section and anchorage.
Reference: Chapter 1, Art. 1.6; Chapter 4, Art. 4.3.

Problem: Traffic damage to superstructure.
Possible cause: Impact.
Conceivable action: Check alignment, broken steel, bearings, section loss.
Reference: Chapter 1, Art. 1.8, 1.10; Chapter 5, Art. 5.1.

Problem: Large flexure cracks, reinforced concrete or prestressed concrete.
Possible cause: Overload.
Conceivable action: Reevaluate with regard to capacity.
Reference: Chapter 5, Art. 5.1.

Problem: Moving crack.
Possible cause: Overload.
Conceivable action: Monitor, consider restricting load. Reevaluate capacity.
Reference: Chapter 1, Art. 1.10; Chapter 2, Art. 2.1; Chapter 5, Art. 5.1.

Problem: Exposed steel.
Possible cause: Corrosion, impact, poor concrete.
Conceivable action: Check anchorage. Check steel section loss.
Reference: Chapter 1, Art. 1.6; Chapter 2, Art. 2.1; Chapter 4, Art. 4.3.

Problem: Vertical crack near support of prestressed beam bridge made continuous for live load.
Possible cause: Creep stressing restrained end at continuous support.
Conceivable action: Note and notify cognizant engineer. Potentially serious.
Reference: Chapter 1, Art. 1.7.

Problem: Rusty crack along prestress strand.
Possible cause: Salt-water penetration.
Conceivable action: Note extent and number of strands affected. Check anchorage. Potentially serious. Notify cognizant engineer.
Reference: Chapter 1, Art. 1.7; Chapter 4, Art. 4.3, 4.4; Chapter 5, Art. 5.5.

Problem: Cracks in strange locations.
Possible cause: Foundation movement.
Conceivable action: Determine rate of movement. Determine if continuing.
Reference: Chapter 2, Art. 2.1; Chapter 4, Art. 4.3.

Problem: Uplift or movement of slope pavement, abutment, or wing walls.
Possible cause: Accumulated water pressure.
Conceivable action: Check weep holes, drainage patterns.
Reference: Chapter 1, Art. 1.11.

Problem: Truck burns under bridge causing spalling and cracking.
Possible cause: Heat and rapid cooling.
Conceivable action: Allow to cool slowly, check for spalling, large flexure or shear cracks, condition of core of columns, permanent deformation.
Reference: Chapter 1, Art. 1.10, 1.8.12; Chapter 2, Art. 2.1; Chapter 4, Art. 4.4; Chapter 5, Art. 5.3.3.4.

Problem: Popouts and possibly map cracking.
Possible cause: Reactive aggregates.
Conceivable action: Sample for laboratory study.
Reference: Chapter 1, Art. 1.6; Chapter 4, Art. 4.6.

Problem: Cracking near deck joints.
Possible cause: Debris in expansion joints.
Conceivable action: Review maintenance procedures.
Reference: Chapter 1, Art. 1.8.3.1.

Problem: Water dripping onto pier cap.
Possible cause: Leaking deck joint, inadequate drains.
Conceivable action: Record and recommend corrective action.
Reference: Chapter 1, Art. 1.8.1.

Problem: Bearing fully expanded in cold weather.
Possible cause: Abutment movement.
Conceivable action: Determine rate and amount of movement, cause, and recommend corrective action.
Reference: Chapter 2, Art. 2.3.1.

Problem: Skewed bridge moving laterally.
Possible cause: Portland cement concrete pavement pushing bridge.
Conceivable action: Recommend relief joint.
Reference: Chapter 1, Art. 1.8.3.1; Chapter 2, Art. 2.3.1.

Problem: Scaling, random cracking, exposed aggregate.
Possible cause: Chemical attack.
Conceivable action: Record, seek source.
Reference: Chapter 1, Art. 1.8.9.

Problem: Hollow sound with hammer.
Possible cause: Delamination.
Conceivable action: Determine extent and cause.
Reference: Chapter 1, Art. 1.6.7, 1.7.4; Chapter 4, Art. 4.2.4.

Problem: Crack in deck along supporting member.
Possible cause: Excessive stress.
Conceivable action: Record, look for other signs of duress. Recommend traffic load survey.
Reference: Chapter 1, Art. 1.8.6.

Problem: Deterioration of deck in high traffic situation.
Possible cause: Salts; impact.
Conceivable action: In-depth survey with high-tech methods from vehicle.
Reference: Chapter 4, Art. 4.4.

Problem: Unknown capacity, indeterminate parameters in structure, potential service load increase.
Possible cause: Lost plans, new industry.
Conceivable action: Load tests.
Reference: Chapter 4, Art. 4.7; Chapter 5, Art. 5.5.

Problem: Unusual ride, no apparent duress of superstructure.
Possible cause: Settlement of pier.
Conceivable action: Reevaluate foundation, waterway.
Reference: Chapter 1, Art. 1.6.10, 1.12; Chapter 2, Art. 2.3.1.

Problem: Need for reevaluation of capacity of reinforced concrete bridge.
Possible cause: New loading, traffic damage, loss of steel section.
Conceivable action: Recommend that capacity be recalculated.
Reference: Chapter 5, Art. 5.1, Appendix A.

Problem: Need for capacity evaluation of damaged reinforced concrete column.
Possible cause: Fire, deterioration, or traffic damage.
Conceivable action: Recommend that capacity be recalculated.
Reference: Chapter 5, Art. 5.1, Appendix A.

Problem: Need for reevaluation of capacity of prestressed concrete bridge.
Possible cause: Damage.
Conceivable action: Recommend that capacity be recalculated.
Reference: Chapter 5, Art. 5.1, Appendix B, Appendix C.

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