MANUAL FOR CONDITION EVALUATION OF BRIDGES 1994

Prepared by the
AASHTO Subcommittee on Bridges and Structures

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1. INTRODUCTION

1.1 PURPOSE

The purpose of this Manual is to serve as a standard and to provide uniformity in the procedures and policies for determining the physical condition, maintenance needs and load capacity of our Nation's highway bridges.

1.2 SCOPE

This Manual has been developed to assist Bridge Owners by establishing inspection procedures and load rating practices that meet the National Bridge Inspection Standards (NBIS). The Manual has been divided into seven sections, with each section representing a distinct phase of an overall bridge inspection and load rating program.

Section 1 contains introductory and background information on the maintenance inspection of bridges as well as definitions of general interest terms. Key components of a comprehensive bridge file are defined in Section 2. The record of each bridge in the file provides the foundation against which changes in physical condition can be measured. Changes in condition are determined by field inspections. The types and frequency of field inspections are discussed in Section 3 as well as specific inspection techniques and requirements. Conditions at a bridge site may require more elaborate material tests, and various testing methods are discussed in Section 4. Field load testing is a means of supplementing analytical procedures in determining the live load capacity of a bridge and for improving the confidence in the assumptions used in modeling the bridge. This is described in Section 5. Section 6 discusses the load rating of bridges and includes optional rating methods. The evaluation of fatigue and other special conditions are discussed in Section 7.

The successful application of this Manual is directly related to the organizational structure established by the Bridge Owner. Such a structure should be both effective and responsive so that the unique characteristics and special problems of individual bridges are considered in developing an appropriate inspection plan and load capacity determination.

1.3 APPLICABILITY

The provisions of this Manual apply to all highway structures which qualify as bridges in accordance with the AASHTO definition for a bridge (see Article 1.6.1). These provisions may be applied to smaller structures which do not qualify as bridges.

1.4 QUALITY MEASURES

In order to maintain the accuracy and consistency of inspections and load ratings, bridge owners should implement appropriate quality control and quality assurance measures. Typical quality control procedures include the use of checklists to ensure uniformity and completeness and the review of reports and computations by a person other than the originating individual. Quality assurance measures may include the periodic field review of inspection teams and their work.

1.5 BRIDGE MANAGEMENT SYSTEMS (BMS)

Bridge Management Systems may be used as a tool in allocating limited resources to the inspection, maintenance, rehabilitation and replacement of bridges. The integrity of BMS is directly related to the quality and accuracy of the bridge inventory and physical condition data obtained through field inspections. A good data base is the foundation of an effective BMS.

1.6 DEFINITIONS AND IMPORTANT REFERENCES

1.6.1 Definitions

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

Bridge: A structure including supports erected over a depression or an obstruction, such as water, highway, or railway, and having a track or passage-way for carrying traffic or other moving loads, and having an opening measured along the center of the roadway of more than 20 feet between undercopings of abutments or spring lines of arches, or extreme ends of openings for multiple boxes; it may also include multiple pipes, where the clear distance between openings is less than half of the smaller contiguous opening (from the AASHTO Transportation glossary).

Bridge Management System (BMS): A system designed to optimize the use of available resources for the inspection, maintenance, rehabilitation and replacement of bridges.

Bridge Owner: An organization or agency responsible for the inspection and load rating of highway bridges.

Condition Rating: The result of the determination of the functional capability and the physical condition of bridge components including the extent of deterioration and other defects.

FHWA: Federal Highway Administration, U.S. Department of Transportation.

Load Rating: The determination of the live load carrying capacity of an existing bridge using existing bridge plans supplemented by information gathered from a field inspection.

MUTCD: The Manual of Uniform Traffic Control Devices.

National Bridge Inspection Standards (NBIS): Federal regulations establishing requirements for inspection procedures, frequency of inspections, qualifications of personnel, inspection reports, and preparation and maintenance of bridge inventory records. The NBIS apply to all structures defined as bridges located on or over all public roads.

NICET: National Institute for Certification in Engineering Technologies.

Quality Control: Procedures that are intended to maintain the quality of a bridge inspection and load rating at or above a specified level.

Quality Assurance: The use of sampling to verify or measure the level of the entire bridge inspection and load rating program.

Structure Inventory and Appraisal Sheet (SI&A): A summary sheet of bridge data required by NBIS. A copy of the SI&A sheet is contained as Appendix A1.

1.6.2 Important References

AASHTO, *Standard Specifications for Highway Bridges*, Washington, D.C. 1989 with annual interim updated specifications.

AASHTO, *Manual for Bridge Maintenance*, Washington, D.C., 1988.

AASHTO, *Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges*, Washington, D.C., 1989.

AASHTO, *Transportation Glossary*, Washington, D.C., 1983.

AASHTO, *Guide Specifications for Fatigue Evaluation of Existing Steel Bridges*, Washington, D.C., 1990.

AASHTO, *Guide Specifications for Strength Design of Truss Bridges (Load Factor Design)*, Washington, D.C., 1985.

AASHTO, *Guide Specifications for Fatigue Design of Steel Bridges*, Washington, D.C., 1989.

AASHTO, *Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members*, Washington, D.C., 1986.

AASHTO, *Standard Specifications for Movable Highway Bridges*, Washington, D.C., 1988.

Federal Highway Administration, U.S. Department of Transportation, *Inspection of Fracture Critical Bridge Members*, Washington, D.C., 1986.

Federal Highway Administration, U.S. Department of Transportation, *Bridge Inspector's Training Manual 90*, Washington, D.C., 1990.

Federal Highway Administration, U.S. Department of Transportation, *Bridge Inspector's Manual for Movable Bridges*, Washington, D.C., 1977.

Federal Highway Administration, U.S. Department of Transportation, *Culvert Inspection Manual*, Washington, D.C., 1986.

Federal Highway Administration, U.S. Department of Transportation, *Non-Destructive Testing Methods for Steel Bridges*, Washington, D.C., 1986.

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., Dec. 1988.

Federal Highway Administration, U.S. Department of Transportation, *Technical Advisory—Revisions to the National Bridge Inspection Standards (NBIS)*, T5140.21, Washington, D.C., Sept. 1988.

Federal Highway Administration, U.S. Department of Transportation, *Technical Advisory—Evaluating Scour at Bridges*, T5140.23, Washington, D.C., Oct. 1991.

Federal Highway Administration, U.S. Department of Transportation, *Manual of Uniform Traffic Control Devices*, Washington, D.C., 1988.

Federal Highway Administration, U.S. Department of Transportation, *Underwater Inspection of Bridges*, Washington, D.C., 1989.

Minor, J. K., et. al., *Condition Surveys of Concrete Bridge Components*, NCHRP Report 312, Transportation Research Board, National Research Council, Washington, D.C., Dec. 1988.

Ritter, Michael A., *Timber Bridges-Design Construction, Inspection, and Maintenance*, EM 7700-8, Forest Service, U.S. Department of Agriculture, Washington, D.C., June 1990.

U.S. Government, *National Bridge Inspection Standards*, Code of Federal Regulations, Title 23, Part 650, Subpart C, Oct. 1988.

2. BRIDGE FILE (RECORDS)

2.1 GENERAL

Bridge Owners should maintain a complete, accurate and current record of each bridge under their jurisdiction. Complete information, in good usable form, is vital to the effective management of bridges. Furthermore, such information provides a record which may be important in legal action.

A bridge record contains the cumulative information about an individual bridge. It should provide a full history of the structure including damages and all strengthening and repairs made to the bridge. The bridge record should provide data on the capacity of the structure, including the computations substantiating reduced load limits, if applicable.

A bridge file describes all of the bridges under the jurisdiction of the Bridge Owner. It contains one bridge record for each bridge and other general information which applies to more than one bridge.

Items which should be assembled as part of the bridge record are discussed in Article 2.2. Information about a bridge may be subdivided into three categories: base data which is normally not subject to change; data which is updated by field inspection; and data which is derived from the base and inspection data. General requirements for these three categories of bridge data are presented in Articles 2.3, 2.4, and 2.5, respectively.

2.2 COMPONENTS OF BRIDGE RECORDS

Some of the components of good bridge records are described below. It is recognized that, in many cases (particularly for older bridges), only a portion of this information may be available. The components of data entered in a bridge record should be dated and include the signature of the individual responsible for the data presented.

2.2.1 Plans

2.2.1.1 Construction Plans

Each bridge record should include one full-size or clear and readable reduced-size set of all drawings used to construct or repair the bridge.

2.2.1.2 Shop and Working Drawings

Each bridge record should include one set of all shop and working drawings approved for the construction or repair of the bridge.

2.2.1.3 As-Built Drawings

Each bridge record should include one set of final drawings showing the "as-built" condition of the bridge complete with signature of the individual responsible for recording the as-built conditions.

2.2.2 Specifications

Each bridge record should contain one complete copy of the technical specifications under which the bridge was built. Where a general technical specification was used, only the special technical provisions need be incorporated in the bridge record. The edition and date of the general technical specification should be noted in the bridge record.

2.2.3 Correspondence

Include all pertinent letters, memorandums, notices of project completion, daily logs during construction, telephone memos and all other related information directly concerning the bridge in chronological order in the bridge record.

2.2.4 Photographs

Each bridge record should contain at least two photographs, one showing a top view of the roadway across and one a side elevation view of the bridge. Other photos necessary to show major defects, or other important features, such as utilities on the bridge, should also be included.

2.2.5 Materials and Tests

2.2.5.1 Material Certification

All pertinent certificates for the type, grade and quality of materials incorporated in the construction of the bridge such as steel mill certificates, concrete delivery slips and other manufacturer's certifications should be included in the bridge record. Material certifications should be retained in accordance with the policies of the Bridge Owner and the applicable statute of limitations.

2.2.5.2 Material Test Data

Reports of non-destructive and laboratory tests of materials incorporated in the bridge during construction or subsequently should be included in the bridge record.

2.2.5.3 Load Test Data

Reports on any field load testing of the bridge should be included in the bridge record.

2.2.6 Maintenance and Repair History

Each bridge record should include a chronological record documenting the maintenance and repairs that have occurred since the initial construction of the bridge. Include details such as date, description of project, contractor, cost, contract number and related data for in-house projects.

2.2.7 Coating History

Each bridge record should document the surface protective coatings used including surface preparation, application methods, dry-film thickness, and types of paint, concrete and timber sealants and other protective membranes.

2.2.8 Accident Records

Details of accident or damage occurrences including date, description of accident, member damage and repairs, and investigative reports should be included in the bridge record.

2.2.9 Posting

Each bridge record should include a summary of all posting actions taken for the bridge including load capacity calculations, date of posting and description of signing used.

2.2.10 Permit Loads

A record of the most significant special single-trip permits issued for use of the bridge along with supporting documentation and computations should be included in the bridge record.

2.2.11 Flood Data

For those structures over waterways, a chronological history of major flooding events including high water marks at the bridge site and scour activity should be included in the bridge record, where available.

2.2.12 Traffic Data

Each bridge record should include the frequency and type of vehicles using the bridge and their historical variations, when available. Average Daily Traffic (ADT) and Average Daily Truck Traffic (ADTT) are two important parameters in fatigue life determination which should be routinely monitored for each bridge and each traffic lane on the bridge. Weights of vehicles using the bridge, if available, should also be included in the bridge record.

2.2.13 Inspection History

Each bridge record should include a chronological record of all inspections performed on the bridge including the date and type of inspection. The original of the report for each inspection should be included in the bridge record. When available, scour evaluation studies, earthquake data, fracture critical information, deck evaluations and corrosion studies should be part of the bridge record.

2.2.14 Inspection Requirements

To assist in planning and conducting the field inspection of the bridge, a list of specialized tools and equipment as well as descriptions of unique bridge details or features requiring non-routine inspection procedures or access should be provided. Special requirements to ensure the safety of the inspection personnel and/or the public should be noted, including a traffic management plan.

2.2.15 Structure Inventory and Appraisal Sheets

The bridge record should include a chronological record of Inventory and Appraisal Sheets used by the Bridge Owner. A sample Structure Inventory and Appraisal Sheet is shown in Appendix A1.

2.2.16 Inventories and Inspections

The bridge record should include reports and results of all inventories and bridge inspections such as construction and repair inspections.

2.2.17 Rating Records

The bridge record should include a complete record of the determinations of the bridge's load-carrying capacity.

2.3 INVENTORY DATA

2.3.1 General

The bridge inventory data provides information about a bridge which is generally not subject to change. As a minimum, the following information should be recorded for each bridge:

(1) *Structure Number.* The official number assigned to the structure by the Bridge Owner.

(2) *Name.* The full name of the bridge. Other common names by which it is known may be placed in parentheses following the official name.

(3a) *Year Built.* Year of original construction.

(3b) *Year Reconstructed.* The year(s) during which major reconstruction or widening occurred.

(4) *Highway System.* State whether or not the bridge is located on the Federal Aid System. Describe the type of Federal aid system and show the Route Number where applicable.

(5) *Location.* Location of the bridge must be sufficiently described so that it can be readily spotted on a map or found in the field. Normally, the bridge should be located by Route number, County, and log mile.

(6) *Description of Structure.* Briefly give all pertinent data concerning the type of structure. Include the type of superstructure for both main and approach spans, the type of piers and type of abutments along with their foundations. If the bridge is on piles, the type of piles should be stated. If it is unknown whether piles exist, this should be so stated. If data is available, indicate type of soil upon which footings are founded, maximum bearing pressures, and pile capacities.

(7) *Skew.* The skew angle is the angle between the centerline of a pier and a line normal to the roadway centerline. Normally the skew angle will be taken from the plans and it is to be recorded to the nearest degree. If no plans are available, the angle should be measured, computed or estimated. If the skew angle is 0°, it should be so stated.

(8) *Spans.* The number of spans and the span lengths are to be listed. These shall be listed in the same direction as the log mile. Spans crossing State highways will be normally listed from left to right looking in the same direction as the log mile for the route under the bridge. Span lengths shall be recorded to the nearest foot and it shall be noted whether the measurement is center to center (*c/c*) or clear open distance (*clr*) between piers, bents, or abutments. Measurements shall be along the centerline of the bridge.

(9) *Structure Length.* This shall be the overall length to the nearest foot and shall be the length of roadway which is supported on the bridge structure. This will normally be the length from paving notch to paving notch or between back faces of backwalls measured along centerline.

(10) *Bridge Roadway Width.* This shall be the most restrictive of the clear width(s) between curbs, railings, or other restrictions for the roadway on the bridge. On divided roadways, the roadway width will be taken as the traveled way between shoulders; but, also, the shoulders and median width will be given.

(11) *Deck Width.* The out-to-out width of the bridge to the nearest tenth of a foot.

(12) *Clearances.* A vertical and horizontal clearance diagram should be made for each structure which restricts the vertical clearance over the highway, such as overcrossings, underpasses, and through truss bridges.

The minimum number of vertical measurements shown on the diagram will be at each edge of the traveled way and the minimum vertical clearance within the traveled way.

The report will state the minimum roadway clearance. This will include each roadway on a divided highway. When a structure is of a deck or pony truss type so that no vertical obstruction is present, the vertical clearance shall be noted on the report as "Unimpaired".

Vertical measurements are to be made in feet and inches and any fractions of an inch will be truncated to the nearest inch, i.e., a field measurement of 15′-7 3/4" will be recorded as 15′-7."

Horizontal measurements are to be recorded to the nearest one-tenth of a foot.

(13) *Wearing Surface and Deck Protective System.* The type and thickness of wearing surface and the type of deck protective system should be noted.

(14) *Curb or Sidewalk Widths.* The widths of the left and right curbs or sidewalks to the nearest tenth of a foot. If only one is present, the sidewalk should be noted thus: "1@5.0′ (east)." Sidewalks

on both sides are noted thus: "2@5.0′." If there are no sidewalks, note "None."

(15) *Railing and Parapets.* List the type and material of the railing and/or parapet. The dimensions of the railing and/or parapet should also be recorded.

(16) *Bridge Approach Alignment.* Note whether the bridge is tangent or on a curve. If the bridge is on a curve, state the radius of the curve if plans are available for this information. On the older roads and bridges, a comparison of the alignment with the general alignment of the road should be made. Note if there are any posted speed restrictions.

(17) *Lanes On and Under the Structure.* State the number of traffic lanes carried by the structure and being crossed by the structure.

(18) *Average Daily Traffic and Average Daily Truck Traffic.* State the ADT and the ADTT, if known, along with the date of record. This information should be updated at intervals of approximately 5 years.

(19) *Design Load.* The live loading for which the bridge was designed should be stated if it is known. A structure widened or otherwise altered so that different portions have different live load designs is to have each live loading specified. If the design live loading is not known, this should be so indicated.

(20) *Features Intersected.* List facilities over which the structure crosses in addition to the main obstacle. For example, a bridge with the name "Wetwater River" obviously carries traffic over the river; it may also cross over a railroad, other roads, etc.

(21) *Plans and Dimensions.* State what plans are available, where they are filed, and if they are as-built. When plans are available, dimensions and size of structural components should be field checked. When plans are not on file, sufficient drawings should be prepared during field investigations to permit an adequate structural analysis of the entire structure, where practical.

(22) *Critical Features.* Special structural details or situations, such as scour critical locations, fracture critical members, fatigue-prone details, pins and hangers, cathodic protection, and weathering steels should be emphasized and highlighted for special attention during field inspections.

2.3.2 Revised Inventory Data

When a bridge is significantly altered by widening, lengthening, or by some other manner which extensively modifies the structure, the bridge inventory data should be updated to reflect the changes made to the bridge. The bridge inventory data should also be updated to reflect changes in wearing surface, railings and other similar items.

2.4 INSPECTION DATA

2.4.1 General

Inspection data may be subject to change with each inspection cycle. In addition to the results of the physical condition inspections conducted in accordance with Section 3, each Bridge record should contain the following inspection information, as a minimum:

(1) *Waterway.* The adequacy of the waterway opening should be classed as "Not a Factor," "Excessive," "Sufficient," "Barely Sufficient," or "Insufficient." The velocity of the stream should be classed with reference to its scouring probabilities, such as "Normally High Velocity," "Normally Medium Velocity." A statement also should be made describing the material making up the stream bed.

An assessment of the scour vulnerability of the substructure should be included. If a bridge has been evaluated as scour critical and is being monitored, or if it has experienced severe scour, or if for other reasons its structural stability is in question for higher discharges, the inspection personnel should coordinate with hydraulics and maintenance personnel in placing a painted line on the piling or abutment which would indicate a water surface at which concern and extra precaution should be exercised. This type of indicator could serve as the trigger for closing a bridge.

When substructures are located within the waterway, indicate the type and location of substructure protection devices. If none are provided, this should be so stated.

If the waterway is navigational, the type and placement of navigation lights should be noted and a clearance diagram of the navigable portion of the waterway should be made.

Bridges may be designed to allow or may experience the overtopping by floods. A statement should be made describing floods that have occurred or that may be possible.

(2) *Channel Profile.* A sheet showing the channel profile at the upstream side of a bridge over a

waterway should be a part of the bridge report. The sketch should show the foundation of the structure and, where available, a description of material upon which footings are founded, the elevation of the pile tips, and/or the footings of piers and abutments. This information is valuable for reference in anticipating possible scour problems through yearly observation and is especially useful to detect serious conditions during periods of heavy flow.

Channel cross sections from the current and past inspections should be plotted on a common plot to observe scouring or stream instability.

Vertical measurements should be made or referenced to a part of the structure such as the top of curb or top of railing which is readily accessible during high water.

Soundings in addition to the single line channel profile are necessary at some river piers to provide adequate information on scour conditions and how the piers may be affected. Such requirements will vary with stream velocity and general channel stability. The necessity of additional soundings must be determined by the Engineer. These soundings will normally be limited to an area within a radius of 100 feet from a pier.

(3) *Restrictions on Structure.* Note any load, speed or traffic restrictions in force on the bridge and if known, record date of establishment and identification of agency who put the restrictions in force.

(4) *Utility Attachments.* An attachment sheet should be submitted when there is one or more utilities on the structure. A utility in the immediate area, though not fastened to the bridge, should also be included, such as a sewer line crossing the ROW and buried in the channel beneath the bridge.

(5) *Environmental Conditions.* Any unusual environmental conditions which may have an effect on the structure such as salt spray, industrial gases, etc., should be noted in the report.

(6) *Miscellaneous.* Include information on high-water marks, unusual loadings or conditions, and such general statements as cannot be readily incorporated into the other headings. Identify the requirements for miscellaneous structural inspections such as those for sign structures, catwalks and other special features.

2.4.2 Revised Inspection Data

The bridge record should reflect the information in the current bridge inspection report. The date upon which the field investigation was made should be noted. All work that has been done to the bridge since the last inspection should be listed. When maintenance or improvement work has altered the dimensions of the structure and/or channel, the new dimensions should be recorded.

2.5 CONDITION AND LOAD RATING DATA

2.5.1 General

This data defines the overall condition and load capacity of the bridge and is based on the Inventory and Inspection data. As a minimum, the following information should be included:

(1) *Bridge Condition Rating.* Document the bridge condition inspection results including observed conditions and recommended maintenance operations or restrictions regarding the deck, superstructure, substructure, and if applicable, channel.

(2) *Inventory and Operating Ratings.* A record should be kept of the calculations to determine the operating and inventory ratings of a bridge and where necessary the load limits for posting. A general statement of the results of the analysis with note of which members were found to be weak, what rating methods were used, and any other modifying factors which were assumed in the analysis, should be given. See Section 6 for the load rating procedures.

2.5.2 Revised Condition and Load Rating Data

When maintenance or improvement work or change in strength of members or dead load has altered the condition or capacity of the structure, the Inventory and Operating ratings should be recalculated.

2.6 LOCAL REQUIREMENTS

Bridge Owners may have unique requirements for collecting and recording bridge data mandated by local conditions and/or legislative actions. These requirements should be considered in establishing the database and updating procedures for the bridge file.

3. INSPECTION

3.1 GENERAL

Bridge inspections are conducted to determine the physical and functional condition of the bridge, to form the basis for the evaluation and load rating of the bridge, as well as analysis of overload permit applications, to initiate maintenance actions, to provide a continuous record of bridge condition and rate of deterioration, and to establish priorities for repair and rehabilitation programs. Cooperation between individuals in those departments responsible for bridge inspection, load rating, permits, and maintenance is essential to the overall effectiveness of such programs.

Successful bridge inspection is dependent on proper planning and techniques, adequate equipment, and the experience and reliability of the personnel performing the inspection. Inspections should not be confined to searching for defects which may exist, but should include anticipating incipient problems. Thus inspections are performed in order to develop both preventive as well as corrective maintenance programs.

The inspection plan and techniques should ensure that:

- Unique structural characteristics and special problems of individual bridges are considered in developing an inspection plan.
- Current technology and practice are applied during the inspection.
- The intensity and frequency of inspection is consistent with the type of structure and details, and the potential for failure.
- Inspection personnel are assigned in accordance with their qualifications, as determined by the Bridge Owner.

Each of the these items is discussed in detail in the following articles.

3.2 TYPES

The type of inspection may vary over the useful life of a bridge in order to reflect the intensity of inspection required at the time of inspection. The five types of inspections listed below will allow a Bridge Owner to establish appropriate inspection levels consistent with the inspection frequency and the type of structure and details.

Each type of inspection requires different levels of intensity. Such items as the extent of access to structural elements, the level of detail required for the physical inspection and the degree of testing will vary considerably for each type of inspection.

3.2.1 Initial Inspections

An Initial Inspection is the first inspection of a bridge as it becomes a part of the bridge file, but the elements of an Initial Inspection may also apply when there has been a change in the configuration of the structure (e.g., widenings, lengthenings, supplemental bents, etc.) or a change in bridge ownership. The Initial Inspection is a fully documented investigation performed by persons meeting the required qualifications for inspection personnel and it must be accompanied by an analytical determination of load capacity. The purpose of this inspection is twofold. First, it should be used to provide all Structure Inventory and Appraisal (SI&A) data required by Federal and State regulations, and all other relevant information normally collected by the Bridge Owner. The second important aspect of the Initial Inspection is the determination of baseline structural conditions and the identification and listing of any existing problems or locations in the structure that may have potential problems. Aided by a prior detailed review of plans, it is during this inspection that any fracture critical members or details are *noted*, and assessments are made of other conditions that may later warrant special attention. If the bridge subjected to an Initial Inspection is anything other than a newly constructed structure, it may be necessary to include some or all of the elements of an In-Depth Inspection.

3.2.2 Routine Inspections

Routine inspections are regularly scheduled inspections consisting of observations and/or measurements needed to determine the physical and functional condition of the bridge, to identify any changes from "Initial" or previously recorded conditions, and to ensure that the structure continues to satisfy present service requirements.

The Routine Inspection must fully satisfy the requirements of the National Bridge Inspection Standards with respect to maximum inspection frequency, the updating of Structure Inventory and Appraisal data and the qualifications of the inspection personnel. These inspections are generally conducted from the deck; ground and/or water levels, and from permanent work platforms and walkways, if present. Inspection of underwater portions of the substructure is limited to observations during low-flow periods and/or probing for signs of undermining. Special equipment, such as under-bridge inspection equipment, rigging or staging, is necessary for Routine Inspection in circumstances where its use provides for the only practical means of access to areas of the structure being monitored.

The areas of the structure to be closely monitored are those determined by previous inspections and/or load rating calculations to be critical to load-carrying capacity. In-depth inspection of the areas being monitored should be performed in accordance with Article 3.2.4. If additional close-up, hands-on inspection of other areas is found necessary during the inspection, then an in-depth inspection of those areas should also be performed in accordance with Article 3.2.4.

The results of a Routine Inspection should be fully documented with appropriate photographs and a written report that includes any recommendations for maintenance or repair and for scheduling of follow-up In-Depth Inspections if necessary. The load capacity should be re-evaluated to the extent that changed structural conditions would affect any previously recorded ratings.

3.2.3 Damage Inspections

A damage inspection is an unscheduled inspection to assess structural damage resulting from environmental factors or human actions. The scope of inspection should be sufficient to determine the need for emergency load restrictions or closure of the bridge to traffic, and to assess the level of effort necessary to effect a repair. The amount of effort expended on this type of inspection may vary significantly depending upon the extent of the damage. If major damage has occurred, inspectors must evaluate fractured members, determine the extent of section loss, make measurements for misalignment of members and check for any loss of foundation support. A capability to make on-site calculations to establish emergency load restrictions may be desirable. This inspection may be supplemented by a timely In-Depth Inspection as described below to document more fully the extent of damage and the urgency and magnitude of repairs. Proper documentation, verification of field measurements and calculations and perhaps a more refined analysis to establish or adjust interim load restrictions are required follow-up procedures. A particular awareness of the potential for litigation must be exercised in the documentation of Damage Inspections.

3.2.4 In-Depth Inspections

An In-Depth Inspection is a close-up, hands-on inspection of one or more members above or below the water level to identify any deficiency(ies) not readily detectable using Routine Inspection procedures. Traffic control and special equipment, such as under-bridge inspection equipment, staging and workboats, should be provided to obtain access, if needed. Personnel with special skills such as divers and riggers may be required. When appropriate or necessary to fully ascertain the existence of or the extent of any deficiency(ies), nondestructive field tests and/or other material tests may need to be performed.

The inspection may include a load rating to assess the residual capacity of the member or members, depending on the extent of the deterioration or damage. Non-destructive load tests may be conducted to assist in determining a safe bridge load-carrying capacity.

This type of inspection can be scheduled independently of a Routine Inspection, though generally at a longer interval, or it may be a follow-up for Damage or Initial Inspections.

On small bridges, the In-Depth Inspection, if warranted, should include all critical elements of the structure. For large and complex structures, these inspections may be scheduled separately for defined segments of the bridge or for designated groups of elements, connections or details that can be efficiently addressed by the same or similar inspection techniques. If the latter option is chosen, each defined bridge segment and/or each designated group of elements, connections or details should be clearly identified as a matter of record and each should be assigned a frequency for re-inspection. To an even greater extent than is necessary for Initial and Routine

Inspections, the activities, procedures and findings of In-Depth Inspections should be completely and carefully documented.

3.2.5 Special Inspections

A Special Inspection is an inspection scheduled at the discretion of the Bridge Owner. It is used to monitor a particular known or suspected deficiency, such as foundation settlement or scour, member condition, and the public's use of a load-posted bridge, and can be performed by any qualified person familiar with the bridge and available to accommodate the assigned frequency of investigation. The individual performing a Special Inspection should be carefully instructed regarding the nature of the known deficiency and its functional relationship to satisfactory bridge performance. In this circumstance, guidelines and procedures on what to observe and/or measure must be provided, and a timely process to interpret the field results should be in place.

The determination of an appropriate Special Inspection frequency should consider the severity of the known deficiency. Special inspections usually are not sufficiently comprehensive to meet NBIS requirements for biennial inspections.

3.3 FREQUENCY

Each bridge should be inspected at regular intervals not to exceed two years or at longer intervals for certain bridges where such action is justified by past reports and performance history and analysis.

If the Bridge Owner proposes to inspect some bridges at greater than the specified two-year interval, a detailed plan which includes supporting rationale must be developed and submitted to Federal and State agencies for approval. Such a plan should include the criteria for classifying structures by inspection intervals and the intended intensity of inspections at each interval. It should consider such factors as age, traffic volume, size, susceptibility to collision, extent of deterioration, performance history of the bridge type, load rating, location, national defense designation, detour length, and social and economic impacts due to the bridge being out of service. The plan should also outline the details of the types and intensity of inspection to be applied. The evaluation of these factors should be the responsibility of the person in charge of the overall inspection program.

Underwater inspection frequencies are described in Articles 3.10.1 and 3.10.2.

3.4 QUALIFICATIONS AND RESPONSIBILITIES OF INSPECTION PERSONNEL

3.4.1 General

Qualified personnel should be used in conducting bridge inspections. Minimum qualifications for the top two levels of responsibility are described below.

3.4.2 Inspection Program Manager

At the highest level, the individual in charge of the organizational unit that has been delegated the responsibilities for bridge inspection, reporting, and inventory shall possess the following minimum qualifications:

(1) Be a registered professional engineer; or
(2) Be qualified for registration as a professional engineer under the laws of the State; or
(3) Have a minimum of 10 years experience in bridge inspection assignments in a responsible capacity and have completed a comprehensive training course based on the *Bridge Inspector's Training Manual.*

The inspection program manager provides overall supervision and is available to team leaders to evaluate problems. Ideally, the position requires a general understanding of all aspects of bridge engineering including design, load rating, new construction, rehabilitation, and maintenance. Good judgment is important to determine the urgency of problems and to implement the necessary short-term remedial actions to protect the safety of the public. When appropriate, the specialized knowledge and skills of associate engineers in such fields as structural design, construction, materials, maintenance, electrical equipment, machinery, hydrodynamics, soils, or emergency repairs should be utilized.

3.4.3 Inspection Team Leader

The second level of responsibility is the Inspection Team Leader. The minimum qualifications of a Team Leader shall be:

(1) Have the qualifications specified for the organizational Unit Leader, or

(2) Have a minimum of 5 years experience in bridge inspection assignments in a responsible capacity and have completed a comprehensive training course based on the *Bridge Inspector's Training Manual*, or

(3) NICET Level III or IV certification in Bridge Safety Inspection.

The Inspection Team Leader is responsible for planning, preparing and performing the field inspection of a bridge. There should be at least one team leader at the bridge at all times during each inspection.

3.5 SAFETY

3.5.1 General

Safety of both the inspection team members and the public is paramount. Bridge Owners should develop a safety program to provide inspection personnel with information concerning their safety and health including the proper operation of inspection tools and equipment. This program should embody applicable State and Federal legislation governing safety and health in the bridge inspection work environment.

3.5.2 Personnel Safety

Personal protective clothing should be worn at all times including hard hats, vests, safety glasses (where needed), and appropriate footwear. Proper hearing, sight, and face protection methods should be practiced whenever using manual and power tools. All equipment, safety devices, and machinery should be kept in the best possible operating condition.

Inspection vehicles should be operated in accordance with the operating manuals provided by the manufacturer. Personnel should be trained in the safe use of the vehicles and emergency procedures in the event of equipment failure.

Belts, lanyards, harnesses, and other personal safety equipment should be used in accordance with applicable standards. All lifelines, belts, lanyards and other equipment should be maintained in good repair. Worn or damaged equipment should be discarded. In addition, inspection personnel should be cautioned to keep safety equipment clean and away from potentially harmful chemicals such as gasoline, dye penetrant and/or oil.

Proper safety precautions should be employed when entering confined spaces such as the interior of a box girder. Air testing, air changes and/or the use of air packs may be required.

Safety programs provide a guide to inspection personnel but do not substitute for good judgment and common sense. It should be recognized that each bridge site is unique. In situations where unusual working conditions may exist, specialized safety precautions may be required. Inspection personnel should have first aid training.

3.5.3 Public Safety

In the interest of public safety, Bridge Owners should employ proper procedures for traffic control and work zone protection during the inspection of a bridge. The *Manual of Uniform Traffic Control Devices* as supplemented by state and local authorities should be used as a guide for such procedures.

3.6 PLANNING, SCHEDULING AND EQUIPMENT

3.6.1 Planning

The key to the effective, safe performance of any bridge inspection is proper advance planning and preparation. The inspection plan should be developed based on a review of the Bridge Record (see Section 2), and may require a pre-inspection site visit. The following items should be considered:

(a) Determine the type of inspection required.

(b) Determine the number of personnel and type of equipment and tools necessary to perform the inspection.

(c) Determine which members or locations are noted in previous inspections or maintenance records to have existing defects or areas of concerns.

(d) Estimate the duration of the inspection and the scheduled work hours.

(e) Establish coordination with or notification of other agencies or the public, as needed.

(f) Assemble field recording forms and prepare appropriate pre-drafted sketches of typical details.

(g) Determine the extent of underwater inspection required and the vulnerability to scour.

Special needs such as diving or scour studies should be identified.

(h) Decide whether non-destructive or other specialized testing is appropriate.

(i) Determine whether the structure contains members or details requiring special attention, such as fracture critical members, fatigue-prone details and non-redundant members.

(j) Determine whether there are structures nearby which are also scheduled for inspection which require a similar crew with similar tools and equipment.

It is advisable for the individual making the inspections to confer with the local highway maintenance superintendent or foreman regarding the bridges to be inspected. The local maintenance person sees the bridges at all times of the year under all types of conditions and may point out peculiarities which may not be apparent at the time of the investigation. Stream action during periods of high water and position of expansion joints at times of very high and low ambient temperatures are examples of conditions observed by local maintenance personnel which may not be seen by the inspector.

3.6.2 Scheduling

So far as is practicable, bridge inspections should be scheduled in those periods of the year which offer the most desirable conditions for thorough inspections. Substructures of bridges over streams or rivers can best be inspected at times of low water, and structures requiring high climbing should be inspected during those seasons when high winds or extremes of temperature are not prevalent. Inspections during temperature extremes should be made at bearings, joints, etc., where trouble from thermal movement is suspected. These examples illustrate the importance of proper scheduling.

3.6.3 Equipment

Bridge inspection equipment consists of those items used for access, and those used to perform actual inspection tasks. Once the equipment requirements are established for a bridge, it should become part of the bridge record (see Article 2.2.14).

3.6.3.1 Access Methods and Equipment

The variation in types of structures to be inspected requires that a broad range of techniques and equipment be used by the bridge inspectors to gain access to the structural elements to perform the inspection. The methods and equipment used to gain access to bridge members include ladders, power lift vehicles, power lift staging, rigging and scaffolds, boats, assisted free climbing, and diving equipment.

In selecting the use of such equipment the following items must be considered:

(a) The ability of the ground, pavement, or bridge structure to safely support the access equipment;

(b) The need for traffic control and/or lane closure, depending on the location of the equipment. The MUTCD and/or State and local requirements should be used as a guide in planning such measures;

(c) The presence of utilities. If utilities are present, special care may be required to prevent accidents;

(d) The need for permits, flagmen and other special considerations for bridges over railroads.

Experienced personnel should be responsible for planning the use of inspection equipment.

3.6.3.2 Inspection Methods and Equipment

The inspection methods and equipment to be employed will depend on the type of inspection as described in Article 3.2. In planning the inspection, a pre-inspection site visit by the Team Leader may be helpful. If plans are available, the pre-inspection should be done plans-in-hand to allow preliminary verification of structure configuration and details.

The pre-inspection should determine means of access, disclose areas of potential concern which will require close attention during subsequent inspections and form the basis for decisions on timing, weather conditions, traffic controls, and utility de-energizations.

3.7 INSPECTION FORMS AND REPORTS

Inspection forms and reports prepared for field use should be organized in a systematic manner and contain sketches and room for notes. The completed report should be clear and detailed to the extent that notes and sketches can be fully interpreted at a later date. Photographs should be taken in the field to illustrate defects and cross referenced in the forms and reports where the various defects are noted. Sketches and photographs should be used to supplement written notes concerning the location and physical characteristics of deficiencies. The use of simple elevation and section sketches of deteriorated members permits the drawing and dimensioning of defects clearly, without resorting to lengthy written notes.

The sources of all information contained in a report should be clearly evident, and the date of the inspection or other sources of data should be noted. A report should be made for each bridge inspection even though it may be only a Special Inspection.

All signs of distress and deterioration should be noted with sufficient accuracy so that future inspectors can readily make a comparison of condition. If conditions warrant, recommendations for repair and maintenance should be included.

Bridge Owners should develop and use standardized abbreviations, legends and methodologies for systematic numbering of bridge components to facilitate note taking and produce uniform results which are easily understood by all inspection teams and office personnel. The use of photographs and sketches to define areas and extent of deterioration should be encouraged. Nomenclature used to describe bridge components should be consistent. Basic highway bridge nomenclature is shown in Appendix A2.

3.8 PROCEDURES

3.8.1 General

The field investigation of a bridge should be conducted in a systematic and organized manner that will be efficient and minimize the possibility of any bridge item being overlooked. To achieve this objective, consideration should be given to standardizing the sequence for inspection of a bridge.

Defects found in various portions of the structure will require thorough investigation to determine and evaluate their cause. The cause of most defects will be readily evident; however, it may take considerable time and effort to determine the cause of some defects and to fully assess their seriousness.

If possible, bridges should be observed during passage of heavy loads to determine if there is any excessive noise, vibration or deflection. If detected, further investigation should be made until the cause is determined. Careful measurement of line, grade, and length may be required for this evaluation. Seriousness of the condition can then be appraised and corrective action taken as required.

Possible fire hazards should be identified including accumulations of debris such as drift, weeds, brush, and garbage. The storage of combustible material under or near a bridge, in control houses on movable bridges, or in storage sheds in the vicinity of the bridge should be reported.

The procedures should include, but not necessarily be limited to, observations described in Articles 3.8.2 through 3.8.10. Unusual or unique bridges or portions of bridges may require special considerations and these should be defined in the inspection plan for the bridge. Items common to these procedures are discussed below.

3.8.1.1 Field Measurements

Field measurements are made to provide baseline data on the existing bridge components and to track changes such as crack width and length, which may occur over time.

Measurements may be required on bridges for which no plans are available and to verify data shown on plans. Measurements are to be made only with sufficient precision to serve the purpose for which they are intended. Unnecessarily precise measurements lead to a waste of time and a false sense of value of the derived results. The following limits of accuracy are generally ample for field measurement:

Timber Members	Nearest 1/4"
Concrete Members	Nearest 1/2"
Asphalt Surfacing	Nearest 1/2"
Steel Rolled Sections	Necessary accuracy to identify section
Span Lengths	Nearest 0.1 foot

When plans are available for a bridge which is to be load rated, dimensions and member types and sizes will normally be taken from the plans. However, many of the plans for older structures are not as-built plans, nor do they reflect all changes made to the bridge. Sufficient checking must be done during field inspections to insure that the plans truly represent the structure before they are used in structural calculations. Special attention should be given to checking for possible changes in dead load, such as a change in the type of decking, additional overlays, and/or new utilities.

Measurements sufficient to track changes in joint opening, crack size or rocker position may need to be made and recorded. Measurements to monitor suspected or observed substructure tilting or movement may be required. In these cases it is necessary that permanent markings be made on the structure and recorded in field notes by the inspector to serve as a datum for future readings. A log of the readings should be kept in the inspection file, and updated with the readings after each inspection cycle.

Direct measurement of the surface area, depth and location of defects and deterioration is preferred to visual estimates of "percentage loss."

3.8.1.2 Cleaning

It is a good inspection practice to clean selected areas to allow close "hands on" inspection for corrosion, deterioration or other hidden defects. Debris, vegetation, fungus, marine growth, vines, litter, and numerous other obscuring coverings can accumulate and hide problem areas.

On metal structures, particularly on fracture critical members, it may be necessary to remove alligatored, cracked and peeling paint for proper inspection. Metal structures with heavy plate corrosion will require chipping with a hammer or other means to remove corrosion down to the base metal in order to measure the remaining section. Provisions should be made to recoat such areas exposed during the inspection which are critical to the structural integrity of the bridge.

On concrete structures, leaching, lime encrustation and debris may cover heavily corroded reinforcing, cracks or other deterioration. Debris on piles can obscure heavy spalling or salt deterioration and vegetation (particularly vines) can obscure large defects such as cracks or spalls.

Timber structures are particularly susceptible to termites and decay in areas where debris causes a wet/dry condition. Inspectors should give particular attention to cleaning and carefully inspecting such areas, especially when they are present near end grain.

3.8.1.3 Guidelines for Condition Rating of Bridge Components

Guidelines for evaluating the condition of bridge components should be developed to promote uniformity in the inspections performed by different teams and at different times. Coding systems, similar to the 0-9 numeric system used by the FHWA, have proven to be effective in establishing such uniformity in condition evaluation.

3.8.1.4 Critical Deficiency Procedures

Critical structural and safety-related deficiencies found during the field inspection and/or evaluation of a bridge should be brought to the attention of the Bridge Owner immediately, if a safety hazard is present. Bridge Owners should implement standard procedures for addressing such deficiencies, including:

- Immediate critical deficiency reporting steps
- Emergency notification to police and the public
- Rapid evaluation of the deficiencies found
- Rapid implementation of corrective or protective actions
- A tracking system to ensure adequate follow-up actions
- Provisions for identifying other bridges with similar structural details with follow-up inspections.

3.8.2 Substructure

An inspection of the substructure of a bridge is generally comprised of an examination and recording of signs of damage, deterioration, movement and, if in water, evidence of scour.

3.8.2.1 Abutments

The footing of the abutment should be investigated for evidence of significant scour or undercutting. Probing is normally performed if all or part of the abutment is located in water. Those underwater situations which require diving to establish the structural integrity are described in Article 3.10. Typical evidence of abutment scour for spill-through abutments is an observable instability of the slope protection due to removal of material at the toe of slope.

Particular attention should be given to foundations on spread footings where scour or erosion is more critical than for a foundation on piles. However, be aware that scour and undercutting of a foundation on piles can also occur. Any exposed piling should be inspected in accordance with the applicable procedures listed in Article 3.8.2.4. The vertical support capacity of the piles normally should not be greatly affected unless the scour is excessively severe, but the horizontal stability may be jeopardized.

When erosion has occurred on one face of the abutment only, leaving solid material on the opposite face, horizontal instability may result. Horizontal instability may also result from earth or rock fills piled against abutments or on the slopes retained by wingwalls.

All exposed concrete should be examined for the existence of deterioration and cracks. The horizontal surfaces of the tops of abutments are particularly vulnerable to attack from deicing salts. In some areas, corrosion of reinforcing steel near the surface can result in cracking, spalling and discoloration of the concrete.

Devices installed to protect the structure against earthquakes should be examined for evidence of corrosion, broken strands, missing bolts, nuts or cable clamps, and proper adjustment. Check for evidence of horizontal or vertical movement of the superstructure relative to the abutment.

Structural steel partially encased in substructure concrete should be inspected at the face of the concrete for deterioration and for movement relative to the concrete surface.

Stone masonry should be checked for cracking in the mortar joints and to see that the pointing is in good condition. Check the stone masonry for erosion, cavities, cracking, and other signs of deterioration of the stones.

Abutments should be checked for evidence of rotation of walls, lateral or longitudinal shifting, or settlement of foundations as compared to previous records. Such movement is usually evidenced by the opening or closing of cracks or joints, by bearings being off center or at a changed angle, or by changes in measured clearances between ends of girders and the abutment backwall. This type of inspection should be performed after an earthquake has occurred in the vicinity.

Examine the abutment drains and weep holes to see if they are functioning properly. Seepage of water at cracks or joints away from the weep holes may indicate an accumulation of water and improper functioning of the weep holes. Mounds of earth adjacent to drains indicate the probable presence of burrowing animals.

3.8.2.2 Retaining Walls

If the retaining wall is adjacent to water, the footings should be examined for scour as described for abutments in Article 3.8.2.1. The toes of all retaining walls should be examined for soil settlement as well as for erosion and scour. Loss of full bearing at the toe can bring about failure of the wall.

Exposed concrete and stone masonry should be examined for the existence and severity of cracks and any deterioration of the concrete, masonry or mortar. The exposed ends of headers of concrete crib walls should be closely examined for cracks which could indicate possible future loss of the interlocking feature and failure of the wall.

Wall faces, tops and joints should be checked for bulging or settlement since the last inspection. Cracks in the slope behind a wall can indicate settlement of the toe and rotation of the wall. Bulges in the faces of sheet pile walls or mechanically stabilized earth walls can indicate failure of individual anchors.

Any exposed piling, whether exposed as a feature of the wall (sheet pile and soldier pile walls) or by adverse action (scour, erosion or settlement), should be inspected as described in the applicable portions of Article 3.8.2.4.

3.8.2.3 Piers and Bents

Piers and bents located in or adjacent to water should be inspected for evidence of scour as described in Article 3.8.2.1 for abutments. Footings in some locations should also be examined for undercutting caused by soil settlement or wind erosion. Exposed piling should be inspected as described in applicable portions of Article 3.8.2.4.

Riprap that has been placed as a countermeasure against pier scour should be evaluated for stability. It should be verified that the material being observed as riprap is actually riprap. It may be larger material deposited at the pier by the stream and may not be providing adequate protection. The key to making the evaluation is the shape of the material. Angular rock is typically specified for riprap while material deposited by a stream is usually rounded.

Examine all exposed concrete and stone masonry for the existence and severity of cracks and any deterioration of the concrete, masonry or mortar. Areas of special vulnerability are the water line and splash zones, the ground line, and locations where the concrete is exposed to roadway drainage, including the tops of piers or bents. Bearing seats, grout pads and pedestals should be examined for cracks, spalls or other deterioration.

Steel piers and bents should be checked for corrosion, especially at joints and splices. Cable connections, bolts and rivets are especially vulnerable to rust. Article 3.8.3 contains a more complete discussion on examinations of structural steel members.

All bents and piers should be checked for lateral movement, tilt or settlement, particularly after periods of high water, storm or earthquakes. Observe bent members, rockers, pins and bearings during passage of heavy loads to determine whether movements are unusual or as expected.

Any material deposited against a bent or pier which was not provided for in the original design should be noted. Horizontal instability could result from such loads.

3.8.2.4 Pile Bents

This article covers those bridge supports which are composed of concrete, steel or timber piles extending to a cap which may be separate from the bridge superstructure or integral with it.

Timber piles should be checked for decay, especially in areas where they are alternately wet and dry. The most likely place for this condition to be found is at the ground line or tidal zone in coastal areas. Often, the earth has to be removed from around the pile to a depth of a foot or so and the timber probed or bored. Holes made for testing which might promote decay should be filled with treated wooden plugs. The timing of such borings will vary greatly from area to area because of climatic variations, specie of wood used for piling, and the preservative treatment that has been given the timber. Although piles may appear sound on the outer surface, some may contain advanced interior decay. Creosoted piles, for example, may become decayed in the core area where the treatment has not penetrated, even though the outside surface shows no evidence of deterioration. Sounding with a hammer may reveal an unsound pile.

Timber piles in salt water should be checked for damage by marine organisms which will attack timber in the area at and below tide line down to mud line. Footing piles which have been exposed by scour below the mud line are highly vulnerable to attack. Attack may also occur in treated piles where checks in the wood, bolt holes, daps, or other connections provide an entrance to the untreated heartwood area.

In addition to the above, special attention should be given to the following:

(1) contact surfaces of timber when exploring for decay;
(2) areas where earth or debris may have accumulated;
(3) areas such as the top of piles where the cap bears;
(4) areas where the bracing members are fastened; and
(5) checked or split areas.

Caps must be examined for decay, cracks, checking, and any evidence of overstress. Further information on the inspection of timber members is found in Article 3.8.3.4.

Examine steel and concrete piles both in the splash zone and below the water surface for corrosion and deterioration.

Inspect all submerged piles for deterioration and loss of section. Special attention should be given to exposed piles in or near salt water. Corrosion of exposed steel piles may be more active at the terminus of concrete encasements on partially encased structural steel members, at the waterline or tide affected zone, and at the mudline.

When subjected to a corrosive environment, structural steel substructure elements should be inspected below the waterline and in the splash zone by manned or unmanned underwater surveillance. Coastal streams may be brackish due to tidal effects for several miles upstream and should be considered a potentially corrosive environment until confirmed otherwise. Additional information on underwater inspections is given in Article 3.10.

Observe the caps under heavy loads to detect unusual movement or any excessive deflection. Steel and timber caps should be observed for any rotational movement resulting from eccentric connections. Bracing members must be checked to see that they are adequate, sound, and that they are securely fastened. Bearings are designed to move freely about their pins or bearings and, if feasible, should be inspected carefully under passage of heavy loads to confirm that their movement is not being restrained (see Article 3.8.3.12).

3.8.2.5 Bridge Stability and Movements

The baseline condition of the structure should be established during the Initial Inspection and should be the basis for the future determination of movement.

Check for transverse movement by sighting along the top of railing, edge of deck or along a girder. Similarly, one can check for differential vertical movements by sighting along the top of railing or edge of deck. On large structures or structures on complex alignment, it may be necessary to use a level or transit to detect movement. Differential settlement between one side of a bridge and the other may also require checking with a level.

Use of a transit is suggested for checking bents, piers, and faces of abutments and retaining walls for rotational movements or tilt. A plumb bob may be used where heights are not great or where only a preliminary determination is desired.

Vertical movement in the superstructure is usually evidence of foundation settlement or rotation of the abutments or piers. Lateral or longitudinal sliding is caused by high water, ice pressure, earthquake, or other application of horizontal forces. Small, relatively equal movements should be noted, but usually are of little consequence. Large or differential movements should be investigated further to determine the probable cause with a view toward corrective measures being taken.

Examine rockers, rollers and hanger elements for movements or inclinations not consistent with the temperature. Compare with notes from previous inspections to see if movements or inclinations are signs of settlement or shifting of foundations.

Inspect joints at abutments, bents, piers and at hinges. Jamming, unusually large openings and elevation differentials on opposite sides of the joint are evidences of substructure movement (or bearing failure).

Check abutment backwalls and ends of beams for cracking, spalling or improper clearances. Causes could be rotation or sliding of the abutment or pressure from the roadway pavement against the back of the abutment.

Examine abutments, wingwalls, and retaining walls for distortion, unusual cracking, or changes in joint widths or inclination. This damage could have been caused by settlement or a change in pressure against the walls. Look for cracks, slipouts or seepage in the earth slopes in front of or behind the walls as well as for unbalanced, post-construction embankment exerting pressure against these walls.

3.8.2.6 Dolphins and Fenders

Dolphins and fenders are used to protect substructure units from impacts by floating debris or maneuvering vessels. The term "dolphin" refers to the standalone unit placed upstream or downstream from the pier. The term "fender" refers to the protective unit or cover placed around the pier or abutment face and which is frequently attached to the substructure.

Piles used in dolphins or fenders are to be inspected as described in Articles 3.8.2.4, "Pile Bents."

Steel piles, frame members, fasteners, and cables should be inspected for rust damage, particularly in the "splash zone." Since both dolphins and fenders may suffer frequent hits and abrasion, the inspection must include a close examination for the results of these actions.

Timber piles and other timber members should be examined for decay, insect damage, marine organisms, abrasion and structural damage. Check at the waterline for weathering of material. (See Article 3.8.3.4.) Note whether protective treatment needs patching or replacement. Cable ties and bolts should be examined for rust. Catwalks and their fastenings should also be examined for decay and other damage.

Concrete members should be examined for spalling, cracking, rusting of the reinforcing steel and damage from abrasion or collisions. For concrete surfaces which have a protective treatment, indicate the condition of the treatment and the need for patching or replacement.

Rubber elements should be examined for missing parts, deterioration, cracking and other damage to elements or fastening devices. Pneumatic and hydraulic elements should be examined for damage and to see if they are functioning properly under impact.

Lighting devices on dolphins or fenders should be checked for rust, broken or missing lenses, and to see whether the lights are functioning correctly. Wiring, conduits, and fastening devices should be examined for rust, breaks or loose connections.

3.8.3 Superstructure

This article includes discussions covering inspection of all commonly-encountered types of superstructures composed of reinforced concrete, structural steel or timber, including bearings, connection devices and protective coatings. The discussion covering inspection of bridge decks, joints, sidewalks and curbs is included in Article 3.8.4. Inspection of the more unusual types of bridges is covered in Article 3.9.

Girders over a traveled way should be checked for any damage resulting from being struck by overheight loads passing under the bridge. If feasible, note any excessive vibration or deflection as truck loads move across the superstructure.

Where the deck obscures the steel top flange or the steel member is totally encased, the inspector may recommend that portions of the covering material be removed at random locations to determine if significant section loss has occurred.

The inspector should note if flammable material is stored under or near a bridge and check for the accumulations of debris, weeds, bushes and, if over water, driftwood.

3.8.3.1 Steel Beams, Girders and Box Sections

Steel beams, girders and box sections should be evaluated as to whether or not they are Fracture Critical Members (FCM) or contain fatigue-prone details, as defined in the AASHTO Design Specifications. More information on fatigue prone details and FCMs may be found in Articles 3.11 and 3.12, respectively. The bridge record should contain a complete listing of all FCMs and the type and location of various fatigue-prone details found on the structure.

Structural steel members should be inspected for loss of section due to rust. Where a build-up of rust scale is present, a visual observation is usually not sufficient to evaluate section loss. Hand scrape areas of rust scale to base metal and measure remaining section using calipers, ultrasonic thickness meters, or other appropriate method. Sufficient measurements should be taken to allow the evaluation of the effect of the losses on member capacity.

Members should be checked for out-of-plane bending in webs or connection plates. Compression flanges should be checked for buckling.

The tension zone of members should be checked for cracking near erection or "tack" welds and at other fatigue prone details.

Box members should be entered and inspected from within where accessible. Check enclosed members for water intrusion. Access points to enclosed box members should be closed or screened to prevent entry of birds, rodents and other animals. Check for collection of debris, bird/animal excrement and other deleterious materials.

Check for fatigue cracks which typically begin near weld terminations of stiffeners and gusset plates due to secondary stresses or out-of-plane bending. Any evidence of cracking should be carefully documented for evaluation and appropriate follow-up, as necessary.

On FCMs perform periodic inspections at a level of effort sufficient to detect very small cracks.

Inspect uncoated weathering steel structures for:

(a) Details or conditions which promote continuous wetting of the uncoated steel
(b) Bridge geometrics which result in salt spray (marine or traffic generated) reaching the uncoated steel
(c) Pitting of the surface of the steel indicating unacceptable degradation of the steel.

3.8.3.2 Reinforced Concrete Beams and Girders

All reinforced concrete superstructures should be inspected for cracking. The locations of the cracks and their size should be carefully noted for future reference and comparison. An effort should be made to determine the probable cause of the cracking: shrinkage, overstress, settlement of substructure, or possible chemical action.

Stems of members should be checked for abnormal cracking and any disintegration of the concrete, especially over bearings. Diagonal cracks radiating from the bearings toward the center of span indicate overstress caused by shear. Vertical cracks extending upward from the girder soffit near centerline of span indicate overstress in tension. High-edge pressure at the bearings may cause spalling in the girder stems.

Examine the soffit of the lower slab in box girder structures and the outside face of the girders for significant cracking. Note any offset at the hinges which might indicate problems with the hinge bearing. An abnormal offset may require further exploration to determine the cause and severity of the condition. Examine the inside of box girders for cracks and to see that the drains are open and functioning properly. Check the diaphragms for cracks.

If there are earthquake restrainer mechanisms at abutments, bents, or hinges, the inspection should cover close examination of these elements for damage due to corrosion or stress. Vertical, lateral and longitudinal movements relative to the substructure should be noted.

3.8.3.3 Prestressed Concrete, Beams, Girders and Box Sections

Prestressed concrete girders should be examined for alignment, cracking, and deterioration of the concrete. Check for cracking or spalling in the area around the bearings, and at cast-in-place diaphragms where creep and humping of the girders may have had an effect. The location of any cracks and their size should be carefully noted for future reference and comparison. Evidences of rust at cracks can mean possible damage to prestressing steel.

Pretensioned box sections should be checked during the passage of heavy loads to see whether any unit is acting independently of the others. Such independent action would indicate spreading of the girders or failure of the longitudinal key between girders.

On bridges with underpassing traffic the exterior faces and the soffits of all types of prestressed girders should be examined. Spalling, cracking or damage to prestressing steel should be noted.

Inspections of earthquake restrainer mechanisms and for earthquake damage should be conducted as outlined in Article 3.8.3.2.

3.8.3.4 Timber Systems

Examine timber stringers for splitting, cracking, and excessive deflection. Look for crushing and evidence of decay where they bear on the bent caps or abutment seats and at their top edge where the floor is supported. Stringers should be kept clear of dirt accumulations to help prevent decay from starting and to help prevent its acceleration once it has started.

The bridging between the timber stringers should be checked to see that it is tight and functioning properly. Timber connections should be checked for loose or missing fasteners.

In order to evaluate the capacity of existing timber structures, the following information should be recorded:

(a) The beam size, spacing and span length;
(b) The type of beam: rough sawn, dressed, nail-laminated, or glue-laminated;
(c) Horizontal shear capacity is controlled by beam depth. Have beams been cut or notched at the bearing and to what extent?
(d) Age of timber should be estimated;
(e) The moisture content of the timber should be estimated or measured;
(f) The species and grade of the lumber should be identified. Original and repair construction records should be checked for material delivery slips. Where no information is available, the inspector must use judgment based upon local experience, visual appearance, odor, cross grain, etc. Where more exact information is required, obtain a sample for testing by a laboratory.

The age, moisture content, species and grade of timber are used in establishing values for the allowable timber stresses to be used in the load rating computations. Field grading and/or estimates of allowable stresses may be necessary.

3.8.3.5 Floor Systems

Truss and deck girder structures are constructed with a system of stringers, floor beams and, if present, brackets to transmit the live load from the deck to the main load-carrying members (girders or trusses). The transverse floor beams and/or brackets can be Fracture Critical Members depending on the framing used. A U-bolt floor beam connection to the truss may be an example of a fracture critical detail. The bridge record should clearly indicate whether or not the floor system contains FCMs.

Inspect stringers, floor beams and overhang brackets for cracks and losses due to rust. Floor beams and connections located below deck relief joints frequently show severe rust due to leakage through the deck joint. Floor beam overhanging tie plates should be carefully examined for evidence of cracking or section loss.

Stringer systems are usually provided with simple expansion devices such as slotted holes at the floor beam connections. These expansion devices should be checked for freedom of movement, uplift or other evidence that the floor system is not functioning as designed.

The floor beams are frequently subjected to out-of-plane bending due to restraints imposed by stringer, girder and bracing connections. Check for evidence of fatigue cracks adjacent to the various connection points.

On those bridges where the deck does not bear directly on the main longitudinal members, there is a tendency for the deck and main longitudinal members not to respond to dynamic loading in synchronization which can cause twisting and out-of-plane bending in the floor beams. Check for evidence of fatigue cracks adjacent to the floor beam/girder connections.

3.8.3.6 Trusses

The examination of any truss will normally begin with sighting along the roadway rail or curb and along the truss chord members to determine any misalignment either vertical or horizontal. Check alignment of trusses carefully for any sag which may indicate partial failure in joints or improper adjustments of the steel verticals or counters. Any deviation from the normal alignment should be fully investigated to determine its cause. Each of the truss members must be checked.

Steel compression members should be examined to see if they are straight with no kinks or bows. Also, compression members should be checked to see that their connections are intact. Eccentricity in the connecting details has a great influence on the strength of the member and, therefore, warrants a close check.

Steel tension members in trusses should be identified as to whether or not they are fracture critical members. All fracture critical members should be inspected closely in accordance with the provisions of Article 3.12.

When a tension member consists of more than one component, each component should be checked to see that the stresses are being divided equally. Counter members should be checked to see that they are in proper adjustment. Counters are sometimes carelessly tightened in order to prevent vibration or rattling, thus throwing abnormal stresses into the counters or other members. Looped rod tension members found in old trusses should be checked carefully for abnormal cracking where the loop is formed and eyebar members examined for cracks in the eyes.

Examine truss and bracing members for traffic damage. Portal bracing usually is the most restrictive overhead clearance and consequently is most susceptible to damage from overheight vehicles.

Check all upper and lower lateral bracing members for damage and observe if they are properly adjusted and functioning satisfactorily. In old bridges, an appraisal of the lateral and sway bracing should be made to determine its adequacy. This appraisal will normally be a judgment of the Engineer based on observation of transverse vibration or movement of the structure under traffic.

Check the conditions of the pins at the connections and see that the nuts and keys are in place. Also, see that spacers on the pins are holding eye-bars and looped rods in their proper position.

Check rivets and bolts to see that none are loose, worn, or sheared.

All timber members should be examined for checks, splits, and decay. Decay is most often found at the joints where there are contact surfaces, daps in the timbers where moisture can enter, and around holes through which truss rod bolts are fitted. End panel joints are likely areas for decay because of the dirt and debris which tends to accumulate on the bridge seat.

Check for any evidence of crushing at the ends of compression chord and diagonal members.

All splice points should be checked for soundness in the shear connections. All bolts should be checked to see that they are tight and in good condition.

Roof and sides of covered bridges should be investigated for adequacy of protecting the structural members from the elements.

Report any fire hazards which exist and need correction to safeguard the structure.

3.8.3.7 Cables

Inspect wire rope cables for breakage, fraying, and surface pitting. Inspect cable terminations for "fretting fatigue" due to flexure. Inspect saddles, socket assemblies and connections for cracking and evidence of internal rusting. Where severe surface deterioration or wire breakage is present, a more detailed inspection of the cable such as spreading with wedges or nondestructive testing techniques should be required to determine the extent of loss.

Long runs of cable should be observed for excessive vibration due to the passage of trucks or wind. Special attention should be given to cable in the vicinity of saddles and at low points. Cable hangers should be closely examined for cracked wires at the socket attachment.

Cable anchorages should be entered, and the wire terminations examined for loss of section and the presence of moisture.

3.8.3.8 Diaphragms and Cross Frames

Diaphragms and cross frames on steel multi-girder bridges should be checked for condition particularly at the points of attachment to the main structural elements. Welded attachments and gusset plates in the tensile zones of girders are fatigue sensitive and may induce out-of-plane bending in girder webs. The inspector should check for cracking or distortion in the diaphragm/cross frame and the girder web. Riveted or bolted connection points should be checked for evidence of prying and soundness of the fasteners.

3.8.3.9 Lateral Bracing, Portals and Sway Frames

Check lateral bracing and sway frame connection plates for fatigue cracking due to wind or live load induced vibrations. Build-up of debris at gussets should be removed to examine for loss of section. Note any lateral brace or sway frame which vibrates excessively due to wind or live load passage

Truss portal members should be examined for collision damage or misalignment. Measure the vertical clearance to knee braces or other portal connections and record the actual minimum clearance.

3.8.3.10 Rivets, Bolts and Welded Connections

Connections between structural members are either welded or mechanically fastened using rivets or bolts. Bolted connections are either designed to be in bearing (load transferred through the bolts) or in friction where the bolts clamp the joined pieces together relying on friction to transfer the load. The inspector should be familiar with the types of connections present on each bridge. The details of these connections should normally be a part of the bridge record.

Friction type, high-strength bolted connections should be checked to verify that all bolts are fully tightened. Look for signs of rubbing or broken paint or rust around the bolts. For example, the presence of red lead dust and corrosion stains near the connection is an indication of abrasion caused by slipping of the joint. Sound suspect bolt heads with a hammer for audible sounds of distress and observe any movement of the bolt when struck.

Riveted and bearing type high-strength bolted connections in shear should be checked for condition and loose elements. Severe loss to the heads of rivets should be recorded.

Rivets and bolts which act in tension should be hammer sounded for the presence of distress or movement. Missing or unsound rivets or bolts in such a connection should be reported and follow-up repairs should be made to avoid the possibility of a progressive failure of the connection.

Welded connections should be checked for the development of fatigue cracking which occurs most commonly at weld terminations and returns. Examine the weld for fine cracks which frequently exhibit rust staining. Where such areas are visually detected, microscopic or nondestructive tests can be performed to confirm and define the cracks present (see Section 4). Fracture Critical Members must receive immediate attention when weld cracks are detected.

3.8.3.11 Pins and Hangers

Pin and hanger assemblies are generally provided to allow an increased clear span without an increased member depth on multi-span bridges and to allow for a statically determinant structural system. When present on trusses or two-girder systems, a pin and hanger assembly is fracture critical. On multi-girder systems, the hanger may not be fracture critical if sufficient cross-framing is present to redistribute the load to adjacent members without causing progressive failure. The hanger connecting the pins is usually a cut steel plate on girder bridges. On truss bridges, the hanger is usually constructed similarly to the adjacent chord members.

Pin and hanger assemblies can fail in many ways, including fracture of the hanger, fracture or shear in the pin, or by movement of the hanger off the pin. They are usually located next to an open joint and therefore vulnerable to corrosion.

Pin and hanger assemblies are frequently used to provide for thermal movement of adjacent spans. Such movement is provided for by longitudinal translation of the upper pin past the lower pin causing rotation of the hanger. These assemblies often become bound due to rusting of the components which places unanticipated torsional stresses on the pins and bending stresses in the hangers. Inspect these assemblies for evidence of transverse movement at the pins. Fatigue cracking can develop along the entire length of the hanger assembly. Measure the relative position of the pins in both the longitudinal and lateral directions. Record these measurements along with the ambient temperature to establish an on-going record at each inspection. Check the hangers for evidence of misalignment or bowing.

Some pin and hanger assemblies are built with a limited distance between the end of the pin and the hanger plate. The pin retainer plates or nuts should be able to restrain the hangers against the main structural element. Check for rust buildup between the elements and evidence of lateral movement along the pin. Impacted rust build-up between the element can develop enough force to move the hanger laterally to a point where the bearing area is insufficient and the pin shears or the hanger falls off the pin. Cap plates may not be strong enough to restrain this movement. The retainer nuts or cap plates must be checked to see that they are adequately secured. All welds on pin and hanger assemblies should be carefully checked.

The pins are frequently obscured from direct view. Check for evidence of fracture or distress such as displacement of connected elements or leaking abrasion dust. Where the end of the pin is exposed such as with threaded nuts, ultrasound testing may be used to check for cracks in the pins parallel to the tested face of the pins. On those pins which are covered by cap plates, the Bridge Owner should establish a program to routinely remove the cap plates and test the pins by ultrasound, consistent with the testing program established for matted pins.

Pin and hanger assemblies at "fixed" connections usually are provided with a restrainer or thrust plate to prevent longitudinal movement. Check that this restrainer is not subject to flexure or distortion.

3.8.3.12 Bearings

All bearing devices should be examined to determine that they are functioning properly. Small changes in other portions of the structure, such as pier or abutment settlement, may be reflected in the bearings.

Bearings and lateral shear keys are subject to binding and damage from creep in bridges with a relatively high skew. Make a careful examination for any such defects.

Expansion bearings should be checked to see that they can move freely and are clear of all foreign material. Rollers and rockers should bear evenly for their full length and should be in the proper position relative to the temperature at the time of the inspection. Lubricated type bearings should be checked to see that they are being properly lubricated.

Check anchor bolts for any damage and to see that nuts are secure. See that anchor bolt nuts are properly set on the expansion bearings to allow normal movement.

Note the physical condition of the elastomeric bearing pads and any abnormal flattening, bulging or splitting which may indicate overloading or excessive unevenness of loading.

Examine pot, disc, and spherical bearings and note any instances of extruded or deformed elastomer, polyether urethane or TFE (polytetrafluorethylene), damaged seals or rings, and cracked steel.

Examine grout pads and pedestals for cracks, spalls, or deterioration.

Bearings, keys, and earthquake restrainer mechanisms should be examined carefully after unusual occurrences such as heavy traffic damage, earthquake, and batterings from debris in flood periods.

Examine the concrete for cracks and spalls at abutment seats and pier caps. If feasible, check the bearings under passage of heavy and rapidly moving loads to detect rattles. Determine and note the probable cause of such "noise."

3.8.3.13 Paint

The bridge file should provide a record of the paint system(s) present, the date(s) of application and the nature of surface preparation used prior to the last application.

Most Bridge Owners standardize on one or more paint systems. A copy of the when-installed paint specification should be available to the inspector. On older structures without an identifiable record of coating types, the inspector should identify in the field the approximate number of paint layers present and any identifying paint characteristics which might assist in identifying the paint system(s) present.

The inspector should make an overall judgment as to the condition of the paint based on the condition of the majority of surfaces, not on localized areas of rusting. The painted surfaces should be free of rust pitting, chalking, crazing or generalized rust staining. Report individual areas of more severe rust for touchup painting.

Examine the condition of the paint and document the extent of corrosion. Check carefully around bolt and rivet heads. Truss chord and panel point connection details are particularly susceptible to corrosion, especially where contaminants from the roadway surface such as deicing salts may be deposited on the steel. It is difficult to inspect many of the areas around connection details for condition of paint and to determine if any corrosion is beginning. However, these areas should not be overlooked as they frequently are the spots where the corrosion will first start. Look for deformation in riveted or bolted multi-plate sections where moisture may have entered and corroded the contact surfaces of the plates causing them to be pushed apart.

The inspector should investigate cracks on painted surfaces which may indicate a crack in the underlying material. This is especially true if rust staining is present.

3.8.3.14 Utilities

The bridge record should contain a clear description of the utilities present on the bridge, the owner of the utility, the agency responsible for maintaining the utility, the date of installation or modification of the utility encroachment and a party to notify both prior to the inspection and in case any defects are uncovered by the inspection.

The inspector should be familiar with the type of utility present and the nature of hazards which may be present during the inspection.

Utilities are frequently retrofitted on bridges. The nature and type of the retrofitted support system should be inspected for the presence of improper welded connections which may be fatigue sensitive or which may result in overloading secondary bridge elements.

Failures in the utilities can introduce several different types of problems:

(1) Structural deterioration may occur as a result of pipes carrying liquids leaking onto superstructure or substructure elements. They may also cause a build-up of ice during cold weather periods.
(2) Utilities on bridges over waterways may cause restriction in the hydraulic capacity of the structure.
(3) Leaks in gas or sewer lines can cause asphyxiation or light-headedness in the inspector, leading to loss of balance. The risk of fire or explosion in an enclosed area, or adjacent to a major structural element, should be evaluated.
(4) Electrical short circuits can cause any construction material to become electrically charged and a danger to the inspector or the general public.

The inspector should immediately report the presence of a utility deficiency. The bridge inspector will frequently be the first person to detect and report such a failure, and cannot assume that the utility is aware of the problem.

3.8.3.15 Arches

This article covers steel, timber, concrete and masonry arch bridge superstructures and long-span concrete arch culverts. Since arches are compression members, any cracking in the arch ring should be carefully noted as indicative of improper loading or movement of supports.

Elements of steel and timber arches should be inspected as generally covered for steel and timber members in Articles 3.8.3.1 and 3.8.3.4, respectively.

The concrete in the arch ring and in the elements supporting the deck is to be inspected as generally covered in Article 3.8.3.2, and any cracking, spalling or other deterioration noted and compared with previous inspection reports.

Masonry arches or masonry-faced concrete arches should be checked for mortar cracks, vegetation, water seepage through the cracks, loose or missing stones or blocks, weathering, and spalled or split blocks and stones.

Concrete arch culverts should be inspected as described for concrete box culverts in Article 3.8.8. Special attention should be paid to the footing area for evidences of undermining, settlement or outward movement, and to the soffit of the arch ring roughly 1/3 of the distance outward between crown and springing. Longitudinal cracks in this area of the soffit indicate shear or flexure problems.

3.8.4 Decks

This Article covers decks constructed of reinforced concrete, prestressed concrete, steel, and timber regardless of type of superstructure; expansion joints; railings, sidewalks and curbs; bridge drainage; and lighting which is affixed to the bridge.

Many decks were designed to act compositely under live load with the supporting superstructure members. The inspector should check to see that composite decks are acting as intended by the designer. Movement between the bottom of the deck and top flange of supporting members or the loss of camber may be indicative of a breakdown in the composite action.

3.8.4.1 Concrete Decks

Concrete decks should be checked for cracking, leaching, scaling, pot-holing, spalling, and other evidence of deterioration. Each item should be evaluated to determine its effect on the structure and the need to restore the loss of structural integrity and maintain a smooth riding surface. Evidence of deterioration in the reinforcing steel should be examined closely to determine its extent. Decks which are treated with deicing salts or are located in a salt air environment are likely to be affected.

The extent of spalling and/or delamination can be determined by tapping lightly with a hammer or by dragging a chain across the deck in the vicinity of the spall. A hollow sound indicates a separation or fracture plane in the concrete beneath the surface.

The hollow areas should be mapped and recorded. These and other nondestructive field test methods are discussed in Section 4.

The underside of the deck slab should always be examined for indications of deterioration or distress. Any loose concrete which could fall and harm individuals under the bridge is a critical condition and should be reported immediately. Note any evidence of water passing through cracks in the slab. When permanent stay-in-place forms have been used in construction of the deck, the inspector may recommend that some panels at random locations be removed to check the condition of the slab.

Asphaltic or other type wearing surface on a deck may hide defects in the deck until they are well advanced. The surfacing must be examined very carefully for evidence of deterioration in the deck or the wearing surface. Such defects may show as cracking or breaking up of the surfacing. In areas where deck deterioration is suspected the inspector may recommend the removal of small sections of the wearing surface for a more thorough investigation.

Concrete decks should be examined for rutting and wear that may result in reduced skid resistance. Concrete containing certain varieties of limestone aggregate is especially susceptible to wear and the polishing action of tires. Skid resistance tests may be requested and performed to determine the need for remedial action to restore the surface skid resistance.

3.8.4.2 Prestressed Concrete Deck Panels

This Article covers precast prestressed concrete deck slabs with or without composite action. The slab units may or may not be covered with a wearing surface. Not included in this discussion are those precast panels used as stay-in-place forms for cast-in-place concrete deck.

As with conventionally reinforced concrete, the surfaces of prestressed concrete deck panels should be checked for cracking, leaching, scaling, pot-holing, spalling, and other evidences of deterioration. See Article 3.8.4.1. Notations should be made of the location and extent of damage for comparison with previous reports and as a basis for future reports.

The ends of slab units should be examined for evidences of deterioration or failure in the anchorage zone.

The joints between adjacent slab units should be examined for spalling and for intrusion of foreign material.

Where the slab units are covered by a wearing surface of asphalt concrete or other material, defects will tend to be hidden from view. This will require very close inspection for cracking, lifting, or loss of bond of the wearing surface as well as a close inspection of the underside of the slabs.

Evidence of cracking, spalling, water leakage through cracks, or separation at the joints between slabs should be noted during inspection of the underside of slabs. Areas where the slab units bear on the girders must be examined closely for cracking and spalling of concrete in the deck slabs or on the edges of concrete girders.

The neoprene or fabric shims between slabs and girders should be examined for tearing, bulging or loosening. Check to see that nuts or bolt heads on slab anchoring bolts are tight. Check the slab units under passage of heavy loads to see that keys or other connecting devices between adjacent slab units are functioning properly.

3.8.4.3 Steel Decks

The inspector should check the steel deck section since any wearing system which may be present is for riding quality only and is not structural.

Open grid decks should be checked carefully for broken tie down welds. Fatigue cracking of all bars is common in open grid decks. Check for wear in the wheel lines which reduces skid resistance.

Closed grid decks are either filled full depth, or partial depth with concrete. They should be checked for the same defects as open grids. In addition, these decks are susceptible to a build-up of rust on the grid elements embedded in concrete which can cause expansion of the deck and break the tie-down welds or distort the supporting structure. The concrete fill wearing surface should be examined for spalling or scaling which exposes the grid. Where the grid is visible, check for evidence of water ponding which can cause a traffic hazard and promote further concrete deterioration and/or rusting of the grid. The underside of the filled grid should be checked for evidence of water leakage and rusting of grid elements.

Corrugated metal pan decks consist of a corrugated sheet metal structural element with either a Portland cement concrete or, more usually, asphalt concrete fill which forms the wearing surface. Check this type of deck for evidence of rust-through of the bottom corrugations where water collects. This type of deck is usually attached to the stringers with plug welds which are not directly observable. Vertical movement of the deck under the passage of live load may indicate weld failure. The fill material of the wearing surface should be examined for cracks or depressions. Open cracks in the wearing surface will allow rust through of the deck elements to occur at an accelerated rate.

Orthotropic steel plate decks consist of a flat steel plate with a series of stiffening web elements. A wearing surface is bonded to the top of the steel plate. On some structures the steel plate is itself a flange element of a box girder section. The inspector should check for debonding of the overlay, rust through or cracks in the steel plate, and for the development of fatigue cracks in the web elements or connecting welds. The connection between the orthotropic plate deck and supporting members should be checked, where visible, and any evidence of live load movement noted.

3.8.4.4 Timber Decks

Timber decks should be examined for decay especially at their contact surfaces where they bear on the stringers and between layers of planking or laminated pieces. Note any looseness which may have developed from inadequate nailing or bolting, or where the spikes have worked loose. Observation under passing traffic will reveal looseness or excessive deflection in the members.

3.8.4.5 Expansion Joints

Expansion joints provide for thermal expansion of the deck and superstructure. They should be checked for freedom of expansion. The clear opening of the joint should provide for adequate expansion of the adjacent superstructure elements considering the span lengths and temperature at the time of inspection. The inspector should measure expansion joint openings and ambient temperature at easily identifiable locations, so that future inspections can establish a record of joint movement over time. Inspect for solid objects (noncompressibles) which can become wedged in the joint and prevent joint contraction.

On joints without armoring, inspect for proper joint alignment, the presence and condition of any joint sealant material, and for evidence of spalls or "D" cracking in the slab edges which would prevent proper sealing of the joint.

Armored joints, without sealant material, such as sliding plate dams or finger joints should be inspected both above and below deck for the condition of the supports. Any horizontal or vertical misalignment of the joint elements should be recorded and checked at future inspections. Where drainage troughs are provided, check for a build-up of debris that prevents proper drainage and causes spill over onto the superstructure and substructure components, or impedes joint movement.

Sealed armored joints such as strip seals or compression seals should be checked for the presence of defects such as tears, separations, sagging, protrusions or embedment of foreign material. Ultraviolet degradation of the seal material is evidenced by hardening and brittleness of the surface and by the appearance of pattern cracking. The underside of all sealed deck joints should be checked for evidence of active joint leakage shown by water staining of the underlying structural elements. Areas of water staining should be clearly marked on drawings or in the field notes, so that future inspections can more accurately assess the extent of active leakage.

Reinforced elastomeric joints are composed of various proprietary combinations of steel supports and sealant material. Inspect for missing anchor bolt covers, separation of joint elements and audible or visual evidence of loose joint panels under traffic. Loose joint panels should be repaired immediately because the bolt failure is progressive and may result in one of the joint panels breaking loose under traffic.

Modular joints are composed of single or multiple support systems working together to accommodate large bridge movements. Inspect for surface damage to seals and separation beams. Examine underside for evidence of leakage and also for unusual noise which may indicate fractured welds or bolts.

3.8.4.6 Railings, Sidewalks and Curbs

3.8.4.6.1 Railings

Bridge railing and parapets, if present, should be evaluated as to condition and as to adequacy of geometry and structural capacity. The inspector should be familiar with the railing requirements of the Bridge Owner. On through-truss bridges, the structural elements, especially fracture critical members such as eyebars, hangers, etc., should be separated from traffic by an adequate vehicular railing system to prevent vehicle impact from causing major structural damage and to protect the vehicle. Inspect reinforced concrete parapets and curbline barriers for evidence of impact damage or rotation. Record areas of collision damage or movement. On precast parapet elements check for evidence of anchorage failure. Anchor bolts, if exposed, should be hammer sounded. Check for separations of the base of the precast element from the deck, or evidence of active water leakage between the parapet and the deck.

Inspect post and beam railing systems for collision damage and deterioration of the various elements. Post bases should also be checked for loss of anchorage. The exposed side of vehicular railing should be smooth and continuous.

3.8.4.6.2 Sidewalks and Curbs

Sidewalk areas should be inspected for structural defects and pedestrian safety items such as tripping hazards, ponding of water or ice, and a walking surface which will not be slippery in wet weather.

The type, condition and alignment of the curbs should be examined by the inspector. Curbs should also be checked to see that they are properly anchored.

3.8.4.7 Bridge Drainage

Examine bridge drainage for both its adequacy and condition.

Check that the grating over the scupper or drain is intact. Report broken or missing grates that are a traffic hazard immediately. Clogged scuppers and down spouts should be documented and reported.

Drainage through open joints, cracks or spalls in the curbs or parapets, or other routes that are not intended should be noted.

Check that the bridge drainage travels through the down spouting and is adequately terminated in drainage facilities or splash blocks. Record any areas of erosion or undermining caused by down spout outfalls. Water ponding on the bridge deck due to clogged scuppers can accelerate freeze-thaw deterioration of the deck and poses a hazard to the traveling public. The Bridge Owner should establish a clear line of authority for reporting and clearing clogged bridge drainage.

3.8.4.8 Lighting

The inspector should inspect lighting standards and supports for proper anchorage and fatigue damage. Any missing or broken luminaires, exposed wiring or missing junction box covers should be reported.

3.8.4.9 Deck Overlays

The inspector should assess the condition of the deck overlay. The condition of the overlay at the curb lines, joints and scuppers should be reported. The extent of surface deterioration should also be reported as well as the overlay thickness.

3.8.5 Approaches

3.8.5.1 Pavement

Approach pavement condition should be checked for cracking, unevenness, settlement, or roughness. Existence of one or more of these defects may cause vehicles coming onto the bridge to induce undesirable impact stresses in the structure. Cracking or unevenness in a concrete approach slab may indicate a void under the slab from fill settlement or erosion.

Joints between the approach pavement and the abutment backwall should be examined. Some of these joints are designed for thermal movement and when inspecting them, a determination should be made whether or not there is adequate clearance to provide for this movement. If the joint was intended to be sealed, determine if the seal is adequate to prevent leakage.

3.8.5.2 Drainage

The approach roadway drainage should be directed away from the bridge. Check that roadway drainage facilities adjacent to the bridge are functioning, and that runoff flows into the drainage facilities and does not pond in the roadway or shoulder areas and does not erode the approach fill. Settlement of the approach pavement or fill can significantly alter the roadway profiles and cross slope and redirect water away from the drainage facilities.

3.8.5.3 Traffic Safety Features

This article covers the inspection of traffic safety features such as steel rail or wire cable approach guide rail, slope-faced concrete barriers, and impact attenuation devices. Inspectors should be familiar with the current agency standards for approach guide rail types, installation heights, and any minimum clearances. Each approach guide rail assembly should be checked as to its conformance to current standards.

The inspector should check the guide rail condition for collision damage, cracks, rust or breakage. Check that connections between rails and posts are secure and tight. Check the alignment of the rail. All areas of settlement or frost heave should be noted. The posts, either wood, concrete or steel, should be embedded in the ground and cannot be moved by hand. Posts which have been hit by vehicles and displaced horizontally should be reported. Wood posts should be checked for rot or insect damage especially at the ground line. The slope beyond the guide rail posts should be checked for settlement or erosion which may reduce the embedment of the posts. Guide rail approach ends and connections to the bridge parapet or railing should be checked for conformance to current standards.

Check impact attenuation devices adjacent to bridge elements for evidence of damage due to impact, and that the energy absorbing elements, such as water or sand filled tubes, have not ruptured. Check that cables and anchorages are secure and undamaged.

On structures over highways, the inspector should review the adequacy and condition of traffic safety devices for both the upper and lower roadways.

3.8.5.4 Embankment Slopes

Check approach slope embankment for evidence of excessive erosion, settlement, undermining of pavements, curbing or guide railing. Also check for slope failure in vicinity of abutments. Often such slope failures result in lateral loading of the first interior pier from the abutment, and in some cases cause tilting and/or bending of the pier.

3.8.6 Signs

Check to see that all signs required to show restricted weight limit, reduced speed limit, impaired vertical clearance or closure are in their proper place. This inspection should include signs at or on the structure and any necessary advance warning signs. Check the signs to see that the lettering is clear and legible and that they are in generally good physical condition. Inspections which occur in the colder months of the year should account for summer foliage in assessing sign visibility.

Any revision made which will alter the vertical clearances, such as addition of surfacing to the roadway, will necessitate remeasurement of the clearances and correction of the signs and records to reflect the change.

For bridges over navigable channels, check to see that the required navigational signs for water traffic are in place and in good condition. The inspector should be familiar with the regulations of the United States Coast Guard to the extent necessary for making these determinations. The navigational lights should be examined to see that they are properly installed in their intended positions and functioning. The aerial obstruction lights on high bridges should be inspected to see if they are functioning.

Sign framing members including the connections and anchor bolts should be inspected for structural integrity. Connections used in sign framing members may be fatigue prone and should be inspected in accordance with Article 3.11.

The Bridge Owner should designate the parties responsible for replacing missing or damaged signs and for removal of vegetation and otherwise restoring sign visibility. The inspector should know to whom sign deficiencies are to be reported.

3.8.7 Waterways

The adequacy of the waterway opening under the structure should be assessed. When assessing the adequacy of the waterway opening, the inspector should bear in mind the potential for debris build-up during periods of high flow and the hazard posed by ice jamming under the bridge during winter and early spring periods.

A channel profile record for the structure should be developed and revised as significant changes occur. This provides an invaluable record of the tendency toward scour, channel shifting, degradation, or aggradation. Evidence of materials mining should be observed. A study of these characteristics can help predict when protection of pier and abutment footings may be required to avoid or minimize future problems.

Existing bank protection and other protective devices such as groins and guide banks (spur dikes) should be checked to observe if they are sound and functioning properly. Determine if changes in the channel have caused the present protection to be inadequate and if it may be advisable to place more protection or to revise the existing protection.

See that the waterway is not obstructed but that it affords free flow of water. Obstructions such as debris or growth may contribute to scour and may present a fire hazard to the structure. Watch for sand and gravel bars deposited in the channel which may direct stream flow in such a manner as to cause harmful scour at piers and abutments.

Areas upstream and downstream of the bridge should be checked to see if the bridge and its approaches are causing any problems or potential problems. Items to look for will include possible flooding from inadequate openings at the structure, erosion of banks or levees from improper location, or skew of the piers or abutments. Evidence of over-topping of the bridge by floods should also be recorded.

3.8.8 Box Culverts as Bridges

This article covers reinforced concrete single- or multiple-cell box culverts which are classified as bridges in accordance with the AASHTO definition of a bridge (see Article 1.6.1). Much of the material is also applicable to concrete arch culverts and to reinforced concrete facilities constructed in an opened box, either without a bottom slab or with a bottom slab not rigidly connected to the side walls.

Check for outward evidences of settlement or other movement by sighting for a sag in the profile of the roadway overhead, sag of the culvert floor or in the underside of the top slab, differential movement at joints in the box, and for rotation of the wingwalls at the ends of the box.

Inspect the side walls, base slab and any footings for abrasion, cracking or other deterioration of the concrete surfaces. Check for leakage of water through the expansion joints and for any undermining of the structure at the outlet due to scour. Check for accumulations of debris, particularly at the inlet and immediately upstream from the inlet, which could block the entrance. Note whether brush or trees are interfering with proper flow through the culvert. Note excessive accumulations of earth in the culvert. Check for slides in the roadway embankment and in the banks of the waterway which could affect the performance or structural integrity of the culvert. The downstream cut-off wall, if present, should be checked for potential scour behind the wall in the upstream direction.

Inspect the underside of the top slab for cracks and spalls. Note the location and size for comparison with previous and subsequent reports. Longitudinal cracks usually indicate shear or tension stresses due to loadings in excess of those the structure can safely carry. Transverse cracks usually indicate differential settlement along the barrel of the box.

Masonry facing, if present, should be checked for mortar cracks, loose or missing stones or blocks, weathering, and spalled or split blocks and stones.

3.8.9 Corrugated Metal Plate Structures

Corrugated Metal Plate (CMP) Structures depend on the interaction with the backfill soil for their stability and ability to carry loads. The CMP Arch is a compression ring with little bending resistance. The shape of the CMP Arch should be inspected and compared to the as-built shape. Any flattening of the top arch elements or sides should be highlighted, and all changes from the as-built condition or previous inspection should be noted. The base of the CMP arch should be checked for differential settlement or undermining. The backfill material at the outlet should be inspected for evidence of material being removed from underneath and alongside of the structure due to water infiltrating the material from the inlet. Coring or test pits may be required to determine the extent of loss at backfill material. The entire length of the barrel of the CMP arch should be checked for misalignment of plate elements, leakage at seams and dents or other local defects.

All CMP structures should be checked for cracks and distortions, especially at bolt locations.

CMP structures should be checked for partial or full concrete headwalls at the inlet to which the structures should be anchored. In the absence of headwalls, evidence of an upward displacement of the inlet should be checked. For those installations with an inlet end mitered to the embankment slope, evidence of the edges folding inward should be checked.

3.8.10 Encroachments

Encroachments at or adjacent to a bridge site are man-made or natural elements which restrict the clearance under a bridge or, in some cases, over the bridge. Signs and sign structures, utilities, dense vegetation and debris are examples of encroachments which reduce the horizontal and vertical clearances for the passage of vehicles. The encroachment of waterways is discussed in Articles 3.8.7 and the inspection of utilities carried by the bridge is described in Article 3.8.3.14.

The inspector should note if the encroachment is located where there is a possibility that it may be hit and damaged by traffic. The horizontal and vertical clearances should be checked by field measurements particularly after repaving projects.

Note the aesthetic effect encroachments may have on the bridge. This item must be considered in permitting encroachments to remain on a bridge. The general appearance of the vicinity around the structure will be a factor in making this determination.

3.9 SPECIAL STRUCTURES

The Bridge Owner should develop a separate inspection plan for each unusual or special bridge to reflect the unique characteristics of such structures. Some of the special structures and their inspection requirements are briefly described below.

3.9.1 Movable Bridges

The most common types of movable bridges are the swing span, vertical lift spans, and bascule spans (single or double leaf). Movable bridges and their inspections are described in detail in the FHWA *Bridge Inspector's Manual for Movable Bridges.*

Inspection of the bridge portion of the structure should be accomplished in accordance with the normal inspection procedures described in the articles of this Manual. Other portions of the structure do exist, however, and must be examined. Counterweights should be checked to see that all parts are sound and secure. Check closely for corrosion of the steel which extends into the concrete. Water may pocket in these locations and penetrate the joint, thus contributing to corrosion. Stains on the concrete around steel embedments should be thoroughly investigated, as they may indicate corrosion and loss of cross-sectional area in the steel at the surface of the concrete and possibly just beneath the surface. Drains in the counterweight pockets should be checked to ensure that they are open and functioning properly.

Counterweight cables plus the uphaul and downhaul cables on vertical lift structures must be checked for wear, corrosion, and to see that they are adequately lubricated. Check the travel rollers and guides for adequate clearance and to see that there is no excessive wear. A significant change in the clearances may indicate pier movement and will require further investigation to determine the cause.

Check to see that the joints in the roadway surface are not misaligned vertically and that there is adequate clearance at the joints where the movable span meets the fixed span and at the joint between leaves of a double-leafed bascule bridge. Also note differential vertical movement at the joint between the two leaves of the double-leafed bascule span under the passage of heavy loads. Shear locks are subject to heavy wear and pounding under traffic. Excessive movement should be investigated and reported.

Steel grid decks, both open and closed, are commonly used on movable spans. See Article 3.8.4.3 for inspection details.

Examination of the electrical, mechanical and/or hydraulic aspects of the structure should be performed by an inspector qualified in these areas, who is familiar with the functioning and design of these systems. The machinery should be checked generally for proper lubrication, unusual noise, looseness in the shafts and bearings.

Trial openings should be made as necessary to insure that all operations are functioning properly and that the movable span is properly balanced. No trial opening for inspection is to be made concurrently with an opening for the passage of vessels where the attention of the bridge operator might be divided between the two interests.

Auxiliary standby power sources are to be started and checked thoroughly in addition to the normal routine periodic operations of the plant. Such routine operations are normally done by the bridge operator on a weekly basis.

The bridge operator should be consulted as part of the investigation. He is a good source of information on the general handling of the operation and can point out any changes from the normal which may have developed.

The inspection team should include an individual well qualified in the electrical aspects of the operation. This, of course, may be the same person qualified for the mechanical and/or hydraulic inspection. Many of the mechanical, hydraulic and electrical operations complement each other and inspection of these areas should be a well-planned coordinated effort.

Inspection of the electrical system should be thorough and will include such items as the controls, wiring, conduits, motors, and lights. Be watchful for any worn or broken lines which may be hazardous. Check for conditions which may exist that could be hazardous or could be potentially hazardous to the operator or anyone using the structure.

During these inspections, the safety of the operator and other personnel in performing normal operational and maintenance activities must be considered in planning and conducting the inspection program.

Submarine cables carrying power and control circuits should be examined in areas above the water line at each inspection. The underwater portion should be inspected by divers after unusually high water or at any other time when there is reason to suspect damage may have occurred.

Examine traffic gates, barriers, and signal systems for highway and marine traffic, to see that all are functioning properly.

Examine fenders and dolphins for damage from marine traffic. Inspect all timber sheathing, wales, and piles for decay, for damage from marine borers, and to see that bolts and cables are tight. Observe the overall setup of the fender system to see if it is reasonably maintained.

3.9.2 Suspension Spans

Suspension spans include cable suspended and eyebar chain suspension systems.

For cable suspension systems, examine the main suspension cables to see that their protective covering or coating is in good condition and protecting the steel from corrosion. Special attention should be given to the cable areas adjacent to the cable bands, at the saddles over the towers, and at the anchorages.

Emphasis should be placed on checking the condition of caulking, when it exists, at cable band locations on suspension bridge main cables.

Examine the bands holding the suspenders to the main suspension cable to see that no slippage has occurred and that all bolts appear to be tight.

Check anchorages for corrosion and to see that there is adequate protection against moisture entering or collecting where it may cause corrosion. Special attention should be given to steel anchor bars embedded in concrete at the interface of the steel and the concrete.

Nondestructive testing may be helpful in evaluating the condition of cables (see Section 4).

Inspection of the stiffening trusses, floor system, towers, and cable bands are to be made in detail as covered in other sections of this Manual.

Eyebar suspension systems have flat steel bars fabricated into a chain, with each link member consisting of two or more eyebars, connected by pins. In general, a member consisting of two eyebars or less should be considered as fracture critical unless evaluation indicates otherwise (see Article 3.12).

Eyebars used in a chain suspension span are very similar to those in a truss. The same type of inspection should be used on a suspension chain as that used on the truss chord. The inspector should:

(1) Inspect carefully the area around the eye and the shank for cracking.
(2) Examine the spacers on the pins at the end of each eyebar to be sure they are holding the eyebars in their proper position.
(3) Observe the eyebars under live load to assure that the load is distributed evenly to each member of the link.
(4) Examine closely-spaced eyebars at the pin for corrosion buildup (pack rust) between each member.
(5) Look for weld repairs.
(6) Inspect pins, pin nuts, pin caps, through bolts and other similar components very carefully.

3.9.3 Cable-Stayed Bridges

Cable-stayed bridges consist of concrete or steel box girders or trusses supported by cables originating from a tall tower. These cables may be fracture critical elements and inspection is paramount. Prior to inspection, an engineer who is knowledgeable in the design, construction, and operation of cable-stayed bridges should review this type of structure to select and identify areas which are fracture critical.

The inspection of the cable stays should be made using procedures given for cables, Article 3.8.3.7, and cable suspension bridges, Article 3.9.2. The inspection of the other structural elements, box girders or trusses, should be done in accordance with appropriate Articles of this Manual.

3.9.4 Prestressed Concrete Segmental Bridges

Prestressed concrete segmental bridges may be made up of cast-in-place segments or precast segments. The inspection of the superstructure of a segmentally cast-in-place or precast bridge is much the same as that for prestressed concrete bridges, as discussed in Article 3.8.3.3. The inspection of substructure, bearings, deck, and expansion joints should be carried out in accordance with the applicable discussions in Article 3.8. The deck surface should be closely examined for longitudinal cracks at the edge of the exterior girder web. Cracking could have resulted from heavy loads on the overhang or by casting or curing methods which resulted in transverse bowing of individual units and resultant cracking during stressing.

Particular attention should also be paid to the profile of the roadway surface (by sighting the top of railing or edge of deck). Humps or sags of an entire span length are evidences of long-term creep of tendons or concrete not anticipated in the original design. Localized sags or humps are indications of problems deserving closer inspection to see if there has been a failure of prestressing units or their anchorages. Such an inspection will require entry into the box sections and examination of the interior anchorages. The areas around the keys in the girder stems and the slabs should be examined closely for cracks, particularly at interlocking corners. The deck soffit must be inspected for cracks and spalls and for evidences of water leakage through cracks or joints.

While inside the box, check the underside of the deck at joints between segments under passage of heavy loads. Differential movements indicate improper functioning of keys in the girder stems, or possible failure of the bearings under an end unit at its support. Differential movement between segments will also show up as cracks in the wearing surface on the deck.

3.10 UNDERWATER INSPECTIONS

Underwater inspection is the combined effort of sounding to locate the channel bottom, probing to locate deterioration of substructure and undermining, and/or diving to visually inspect and measure bridge components. It should be an integral part of a total bridge inspection plan.

Underwater members must be inspected to the extent necessary to determine structural safety with certainty. In addition to structure elements, underwater inspections must include the stream bed. In wadable water, underwater inspections can usually be accomplished visually or tactually from above the water surface; however, inspections in deep water will generally require diving or other appropriate techniques to determine underwater conditions.

Scour evaluations are to be conducted for all existing bridges that have been screened by Bridge Owners and found to be scour susceptible. Special attention should be given to monitoring scour critical bridges during and after major flood events.

3.10.1 Routine Underwater Inspections

Observations during low-flow periods and/or probing for signs of undermining or substructure deterioration should be done during all routine inspections. Additional observations may be required at high-water levels for those structures located in or adjacent to alluvial stream beds. Observations should also be made to review the structural integrity of the foundations.

3.10.2 In-Depth Underwater Inspections

In-depth underwater inspections of structural members that cannot be inspected visually or by wading are required at least every five years. Typical occurrences which should result in a decision to make an underwater inspection at a shorter interval are structural damage, scour and erosion due to water movement, drift, streambed load, ice loading, navigation traffic collision, and deleterious effects of water movement or deleterious effects of elements in the water. If more frequent underwater inspection is determined to be required, the inspection interval should be established by the Bridge Owner.

3.11 FATIGUE PRONE DETAILS

Fatigue cracks may occur at locations of stress concentrations, where the rigidity of the member changes. Connection details, damaged components, and corrosion-notched sections are examples of such locations.

Various connection details have been identified and assigned a fatigue stress category. (See AASHTO Design Specifications Table 10.3.1B and Figure 10.3.1C.) Generally, Category E′ details have the shortest fatigue life and are the most prone to fatigue cracking. The susceptibility of the detail to cracking decreases from Category E′ to Category A. Many of the problems associated with these details are related to weld terminations and weld defects. Welds made in the field are especially susceptible to fatigue cracking, including tack welds.

Bridge inspectors should be trained to identify fatigue prone details. All locations prone to fatigue cracking should be given a close visual inspection. Bridge Owners may establish the frequency of such inspections based on the category of the detail, the size and number of repetitions of truck loads, and other related factors. The inspection of fatigue prone details may include nondestructive testing (see Section 4).

3.12 FRACTURE CRITICAL MEMBERS

Fracture critical members or member components (FCMs) are steel tension members or steel tension components of members whose failure would be expected to result in collapse of the bridge.

Tension components of a bridge member consist of components of tension members and those portions of a flexural member that are subject to tension stress. Any attachment having a length in the direction of the tension stress greater than 4 inches (10 cm) that is welded to the tension area of a component of a "fracture critical" member shall be considered part of the tension component and, therefore, shall be considered "fracture critical."

FCMs have all or part of their cross section in tension. Most cracks in steel members occur in the tension zones, generally at a flaw or defect in the base material. Frequently the crack is a result of fatigue, occurring near a weld, a material flaw and/or changes in member cross section (see Article 3.11).

After the crack occurs, failure of the member could be sudden and may lead to the collapse of the bridge. For this reason steel bridges with the following structural characteristics or components should receive special attention during the inspection:

- One or two-girder systems, including single boxes with welding
- Suspension systems with two eyebar components
- Steel pier caps and cross girders
- Two-truss systems
- Suspended spans with two girders
- Welded tied arches
- Pin and hanger connections on two or three girder systems.

Inspection of steel bridges should include the identification of fracture critical members and the development of a plan for inspecting such members. The FCM inspection plan should identify the inspection frequency and procedures to be used. A very detailed, close visual "hands-on" inspection in the field is the primary method of detecting cracks. This requires that critical areas be specially cleaned prior to the inspection and additional lighting and magnification be used. Other non-destructive testing procedures (see Section 4) may be used at the discretion of the Bridge Owner. Photographs and sketches should be made of the conditions found and on-site comparisons of photographs and sketches should be made at follow-up inspections.

Where the fracture toughness of the steel is not documented, some tests may be necessary to determine the threat of brittle fracture at low temperatures.

4. MATERIAL TESTING

4.1 GENERAL

This Section describes the more common testing procedures for assessing the strength and condition of materials and structural components of bridges. New testing procedures are evolving rapidly as a result of improved technology. Material testing should be performed by properly trained personnel.

4.2 FIELD TESTS

Numerous field test procedures are available for concrete, steel and timber structures. Many of these procedures are non-destructive, while others result in some removal or damage of the material.

4.2.1 Concrete Field Tests

Typical field test procedures for concrete bridge components are described below. A comparison of the test methods in terms of their capability of detecting defects in concrete components is shown in Table 4.2.1. This table should be used as a guide in selecting an appropriate field test method for concrete components.

Table 4.2.1 Capability of Investigating Techniques for Detecting Defects in Concrete Structures in Field Use

Method Based on	Capability of Defect Detection[a] Cracking	Scaling	Corrosion	Wear and Abrasion	Chemical Attack	Voids in Grout
Strength	N	N	P	N	P	N
Sonic	F	N	G[c]	N	N	N
Ultrasonic	G	N	F	N	P	N
Magnetic	N	N	F	N	N	N
Electrical	N	N	G	N	N	N
Nuclear	N	N	F	N	N	N
Thermography	N	G[b]	G[c]	N	N	N
Radar	N	G[b]	G[c]	N	N	N
Radiography	F	N	F	N	N	F

[a]G = Good; F = Fair; P = Poor; N = Not suitable.
[b]Beneath bituminous surfacings.
[c]Detects delaminations.

4.2.1.1 Strength 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 C 805, "Test Method for Rebound Number for Hardened Concrete." 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 C 803, "Test Method for Penetration Resistance of Hardened Concrete." 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.

4.2.1.2 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 such 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.

The practice for measuring delaminations in concrete bridge decks is discussed in ASTM D 4580.

Portable automated acoustic methods have 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.

4.2.1.3 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.

4.2.1.4 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.

4.2.1.5 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 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. 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.

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

4.2.1.6 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.

4.2.1.7 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.

The test method for detecting delaminations in bridge decks using infrared thermography is discussed in ASTM D 4788.

4.2.1.8 Radar

Ground-penetrating radar has been used to detect deterioration of bridge decks. 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. More than a little experience is necessary for proper interpretation of the data.

4.2.1.9 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.

4.2.1.10 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. Although this is a viewing instrument, some destruction of material is necessary for its proper use with concrete.

4.2.2 Steel Field Tests

Typical field test procedures for detecting defects in steel bridge components are described below.

A general summary of the relative capabilities of the steel test methods is given in Table 4.2.2. This table should be used as a guide in selecting an appropriate field test method for steel components.

Table 4.2.2 Capability of Nondestructive Examination Techniques for Detecting Defects in Steel Structures in Field Use

Method Based on		Capability of Defect Detection[a]									
		Minute Surface Cracks	Deeper Surface Cracks	Internal Cracks	Fatigue Cracks	Internal Voids	Porosity and Slag in Welds	Thickness	Stress Corrosion	Blistering	Corrosion Pits
Radiography		N	F[b]	F[b]	P	G	G	F	F	P	G
Magnetic Particle (A.C.)	Wet	G	G	N	G	N	N	N	G	N	N
	Dry	F	G	N	G	N	N	N	F	N	P
Eddy Current		F	G	N	N	N	P	P	N	N	N
Dye Penetrants		F	G	N	G	N	N	N	G	N	F
Ultrasonics[c]		P	G	G	G	G	F	G	F	F	P

[a]G = Good; F = Fair; P = Poor; N = Not suitable.
[b]If beam is parallel to cracks.
[c]Capability varies with equipment and operating mode.

4.2.2.1 Radiography

Nondestructive examination by use of X-rays depends on the fact that X-radiation, produced either by a commercial X-ray machine or by radioactive decay of a radioisotope, will be absorbed by a material in proportion to the thickness of the part examined and the atomic number. Thus, if a defective piece of material is examined by this method, the X-ray absorption at the region of the defect will be different (usually less) than sound material next to this region. The X-radiation coming through the part is recorded on a film or a fluorescent screen; the image is usually darker in the area where the defect is located. The X-ray image on film provides a permanent record of the defect, and also shows the size and shape of the defect in two dimensions. It does not show its position in depth in the part.

It follows from this description that defects such as slag inclusions or porosity in welds or castings are easily detected by this method. Planar defects such as cracks are also detectable; but only if oriented approximately parallel to the axis of the X-ray beam. Cracks or planar defects perpendicular to the X-ray beam axis will not change the X-ray absorption significantly and thus will be undetected. Intermediate orientations will produce varying degrees of defect detectability.

Advantages of this method of nondestructive examination are the permanent record that normally results, the ability to determine internal defect size and shape (and thus defect nature), and its almost universal acceptance in codes and by the engineering profession in general. The prime disadvantages to this method are its inability to locate the depth of the defect, its inability to locate poorly oriented planar defects, and the need to use, in general, large or hazardous equipment. It may also be difficult to apply in some field locations. One special consideration with this method which makes it particularly attractive is the fact that the resulting film is, in fact, a photograph of the part, and thus is immediately geometrically relatable to the part examined. No secondary analysis of the data is necessary.

4.2.2.2 Magnetic Particle Examination

This method of inspection, like the dye penetrant one, is limited to surface or near-surface defects. An additional limitation placed on the process is the fact that only magnetic materials may be examined. In the shop application of the method, the part to be examined is placed in a magnetic field and fine powdered iron is sprayed (in suspension) or blown on it. If the magnetic field is undisturbed by any surface or subsurface discontinuities, the iron powder aligns itself with the field in a uniform film. If a discontinuity (such as a crack) disturbs the field, a concentration of magnetic lines of force will occur, and thus a concentration of iron powder. This concentration will show the presence of the crack during visual inspection. In order to detect the crack, it must be aligned transverse or nearly transverse to the magnetic field. For this reason, the magnetic field must either be aligned perpendicular to the expected direction of defect formation or must be varied in direction. For shop tests, this is usually accomplished by sequentially magnetizing the part in a large circular coil to produce a longitudinal magnetic field and passing current through the part to produce a circular magnetic field.

In field applications, the part is locally magnetized by use of two current-carrying copper prods that are placed on the surface of the part. These prods produce a circular magnetic field about each contact point when current flows between them and surface defects transverse to the field are detected by use of iron powder. If the prods are moved about the part or structure to be examined, defects at any orientation may be detected. Application of this procedure may produce surface defects which could result in crack initiation sites.

The advantages to this method are its relative portability, the minimum skills required to operate it, and its ability to detect even tight cracks. Of course, it is limited in the materials that it may be applied to, and the type of defects it may detect. Again, in some applications, it has the additional limitation that it leaves the part in the magnetized condition. Although this is not normally a problem, it may interfere with some subsequent operations, such as welding. It is possible to demagnetize the area examined by this method, but this is time consuming and adds to the cost.

4.2.2.3 Eddy Current Examination

This method operates very similarly to magnetic particle inspection but the defect is detected by a perturbation in the electrical, not magnetic, field in the material examined. In this technique, a coil carrying alternating current produces eddy currents in a conductor nearby. The conductor eddy currents, in turn, create impedance in the exciting or, if desired, a separate search coil. The impedance produced depends on the nature of the conductor and the exciting coil, the magnitude and frequency of the current, and the presence or absence of discontinuities in the conductor. The method is therefore instrumented such that a coil is scanned over the surface of the area to be examined and defects produce a characteristic change in impedance as read from a dial or meter (output can be put on a chart if desired).

This method has been given only limited application for several reasons, most important of which has been that generally only simple geometries can be examined. Complex geometries change the impedance readings in themselves, and thus limit the usefulness of the procedure. Again, as with magnetic particle examination, only conductors can be examined.

There is some potential for this method. Defects in depth can be detected, or with suitable frequency control, examination may be limited to the surface. Defect size can also be estimated from the response of the area examined. It is insensitive to many surface conditions (for example, paint) which limit other methods. This method appears to need further development, however, to be generally applicable. Certainly the geometry sensitivity of the method is a real disadvantage.

4.2.2.4 Dye Penetrant Examination

The dye penetrant method of inspection is probably the most commonly employed shop and field method of defect detection. Although it is limited entirely to defects that penetrate the surface of the structure, it is inexpensive, easily applied, and easily interpreted. The method itself is simple. The surface of the part to be examined is cleaned, usually mechanically and/or with a chemical degreasing agent. A fluid is placed on the surface to be examined, often with an aerosol spray, and allowed to penetrate cracks or surface defects by capillary attraction or other surface wetting phenomena. After a period of time, usually minutes, the penetrant is removed and a second solution is sprayed on the surface. The second coating, called a developer, usually dries to a chalky powder and remains unchanged in the regions where no defect exists. In the location of a crack, the penetrant seeps from the crack where it is trapped and stains the developer. For this reason bright-colored (often red) penetrants are used. The red penetrant stains on the white chalky developer indicate the presence of a crack or other defect when visually inspected by the examiner. Modifications of the system include penetrants of different viscosity to detect different size cracks, wet rather than dry developers, and penetrants that fluoresce under ultraviolet light. These penetrants, used in conjunction with ultraviolet light examination, make smaller defects visible.

The principal advantages of the method are the ease with which the tests are conducted, the minimal skills required, and the low cost. Tests are not time consuming and may be made frequently during other operations (for example, to determine if a defect being removed by grinding is completely eliminated). It must be considered the most portable of all methods.

The principal disadvantage is that only surface defects can be detected. This places a limitation on the usefulness of the method for the defect depth determination and "code" approval of most structures. However, from the practical shop viewpoint, many defects that occur during construction (for example, weld cracks) are detectable if dye penetrant is used at intermediate stages in the construction. Thus, defects that are later buried can be detected and repaired before they are hidden from view. Use of dye penetrant during fabrication may prevent later rejection when ultrasonic or X-ray examination is used. The more sophisticated dye penetrant methods, using ultraviolet light, are rarely used in field applications.

4.2.2.5 Ultrasonic Examination

Ultrasonic testing relies on the wave properties of sound in materials to detect internal flaws. High-frequency sound waves in the form of mechanical vibrations are applied to the part to be tested and the waves, passing through the material, strike either a defect or, eventually, an external surface. The sound vibrations are then reflected and the nature of the return signal indicates the location and type of reflecting surface. Normal instrumentation includes a sound wave generator and pick-up device (usually combined in one unit) and a display screen on which the initial and reflected pulse is displayed. Display instrumentation permits an estimation of the position (in depth) of the defect, the nature of the defect and, by moving the detection portion of the unit (called the search unit) along the part to be examined, the size of the defect. The test sensitivity is influenced by a great number of testing variables, such as sound frequency, design of the search unit, instrumentation, electronic processing of the return signal, and the skill of the operator. Normal results of the examination are a form prepared by the operator based on his observations of the display screen.

The major advantages of this system of nondestructive examination are its portability, sensitivity and ability to detect the location of cracks or defects in depth. On the other hand, the major fault of the system is that, until very recent times, no permanent record of the defect was produced. It is now possible to make photographic records of the display, and equipment is now available to permit the storage of field data in a format suitable for subsequent computer processing and reporting. Another characteristic of the system often cited as a difficulty is the sensitivity of the method. It is possible to see too much; i.e., grain size in metals and minor defects not observable by other methods. The system cannot detect surface defects very well. The dependency of the method on operator skill must also be considered an unfavorable factor.

More research has been undertaken to modify this method and make it more widely applicable than most of the others, so advances in technology are more likely in this field.

4.2.3 Timber Field Tests

Typical field test procedures for detecting defects and deterioration in timber bridge components are described below.

A summary of the capabilities of each of the test methods for detecting defects and deterioration in timber components is given in Table 4.2.3. This table should be used as a guide in selecting an appropriate field test procedure for timber components.

4.2.3.1 Penetration Methods

Any probe, such as a knife, ice pick, nail, or brace and bit, can be used to test for internal decay or vermin infestation. The ease with which a member can be penetrated is then a measure of its soundness. Only a qualitative assessment is obtained because the pressure on the instrument is neither controlled nor measured. Although the procedure is rather crude, it is rapid and an overall assessment of the condition

Table 4.2.3 Capability of Investigative Techniques for Detecting Defects in Timber Structures in Field Use

Method Based on	Capability of Defect Detection[a]				
	Surface Decay and Rot	Internal Decay and Voids	Weathering	Chemical Attack	Abrasion and Wear
Penetration	G	G	F	F	N
Electrical	F	F	N	N	N
Ultrasonics	N	G	G	N	N

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

of a structure can be obtained quickly. The use of a probe is much more satisfactory than attempting to identify a hollow member by sounding because the load on the member affects the response and may lead to erroneous conclusions.

An increment borer, which consists of a sharpened hollow tube, usually about 1/4-in. (6-mm) internal diameter, can also be used to penetrate the wood. The borer is superior to a nail or ice pick because it gives a more accurate record of the depth of decay or infestation. It also allows samples to be removed from the interior of the member for detailed examination or testing for such items as moisture content and preservative penetration, or to be cultured for positive evidence of decay fungi.

4.2.3.2 Electrical Methods

The main application of electrical methods is to measure the moisture content of timber. There are several electrical techniques available for measuring moisture content.

Resistance meters are based on a direct current measurement of electrical resistance between point or blade electrodes pushed into the timber. The resistance is related to the moisture content, which is displayed on a calibrated scale. The results are affected by the species of timber and correction factors must be applied. Resistance moisture meters are light, compact, and inexpensive but the major disadvantage is that they measure the moisture content of the surface layers unless special deep probes are used. Readings over 30 percent moisture content are not reliable and contamination by some chemicals, such as salt, affects the readings.

Capacitance meters are based on an alternating current measure of the dielectric constant of wood, which is proportional to its moisture content. The results are a function of the relative density of the wood and correction factors must be applied. The meters measure primarily surface moisture content, and, on lumber thicker than 2 in. (50 mm), do not respond to internal moisture adequately. Capacitance meters have a wider range (0 to at least 35 percent moisture content) than resistance meters and are less affected by the presence of chemicals.

Radio frequency power-loss meters operate in the frequency range 0 to 25 MHz and are based on an alternating current measurement of the impedance (combined effect of resistance) and capacitance of timber. They need to be calibrated for wood species and density. The meters use plate-type electrodes and the field penetrates about 3/4 in. (20 mm) but the surface layers have the predominant effect. The cost of the meters is similar to that of capacity-type meters, being higher than that of simple resistance types.

Electrical resistance measurements are also the basis of an instrument designed to detect internal rot. The device consists of a resistance probe, which is inserted to various depths in a hole 3/32 in. (2.4 mm) in diameter. A marked change in electrical resistance is an indication of decay. Although the device effectively detects rot, it is susceptible to false indications of decay in apparently sound wood.

4.2.3.3 Ultrasonic Techniques

The same ultrasonic pulse-velocity equipment and techniques described in Article 4.2.1.3 for application to concrete members can also be used for the in-situ testing of timber structures, both above and below the water surface.

Pulse-velocity measurements relate to the elastic properties of the wood and are therefore sensitive to the direction of the grain. However, pulse-velocity measurements have been found to follow similar trends to strength changes caused by fluctuations in density and local defects. Consequently, the strength and stiffness properties of the timber can be assessed. The ultrasonic method can also be used to identify internal decay and hollow areas, as well as internal knots, checks, and shakes. Because a discontinuity, such as a crack or a hollow area caused by decay, reflects part of the sound wave and changes the velocity of the transmitted wave, the technique is most sensitive to detecting defects that are oriented perpendicularly to the pulse. For this reason, the direct transmission mode with transducers on opposite faces of the member is generally the most useful configuration. However, in some situations, it may be necessary to investigate other relative positions of the transducers in order to produce a maximum response. To simplify interpretation of the results it is common practice to compare the pulse velocity from a suspected area of deterioration with that from an area known to be sound (measured using the same transducer configuration), thereby eliminating the need to measure the density of the timber. In all cases, a good contact between the transducer and the surface of the timber is essential. A light grease or glycerol are suitable for the coupling medium. Bentonite paste has also been found satisfactory.

4.3 MATERIAL SAMPLING

Tests which require the removal of material from the structure should be used only when a particular piece of information is desired, and only when the results can provide something useful in the overall evaluation of the bridge.

A few common material sampling standards are shown in Table 4.3-1. Samples should be removed from those areas of a bridge subjected to low stress levels as determined by the Engineer. An adequate number of samples should be obtained to provide results representative of the entire structure being evaluated. Normally, a minimum of three samples would be required.

The removal of material from a structure will leave a hole or void in one or more members. Repairs can be readily made to concrete, masonry and timber members. Repairs to steel members may be much more complex, particularly if welding is used, and should be carried out by experienced personnel. Care should be taken to minimize any residual stress resulting from the repair.

Table 4.3-1 Standard ASTM and AASHTO Methods for Material Sampling

Designation	Title
C 42/T 24	Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
T 260	Sampling and Testing for Total Chloride Ion in Concrete Raw Materials
C 823	Standard Practice for Examination and Sampling of Hardened Concrete in Constructions
A 610	Sampling Ferroalloys for Size (Before or After Shipment)
A 673	Sampling Procedures for Impact Testing of Structural Steel (Charpy Test)
A 370	Standard Test Methods and Definitions for Mechanical Testing of Steel Products

TABLE 4.4-1 Standard ASTM and AASHTO Test Methods for Concrete for Use in the Laboratory

Designation[a]	Title
C 39/T 22	Test Method for Compressive Strength of Cylindrical Concrete Specimens
C 85/T 178	Test Method for Cement Content of Hardened Portland Cement Concrete
C 174/T 148	Method of Measuring Length of Drilled Concrete Cores
C 457	Practice for Microscopical Determination of Air-Void Content and Parameters of the Air-Void System in Hardened Concrete
C 469	Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression
C 496	Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens
C 617/T 231	Method of Capping Cylindrical Concrete Specimens
C 642	Test Method for Specific Gravity, Absorption, and Voids in Hardened Concrete
C 666/T 161	Test Method for Resistance of Concrete to Rapid Freezing and Thawing
C 856	Recommended Practice for Petrographic Examination of Hardened Concrete
T 259	Method of Test for Resistance of Concrete to Chloride Ion Penetration[b]
T 260	Method of Sampling and Testing for Total Chloride Ion in Concrete and Concrete Raw Materials
T 277	Interim Method of Test for Rapid Determination of the Chloride Permeability of Concrete

[a]ASTM test methods are designated C. AASHTO test methods are designated T.
[b]Corrosion threshold is about 1.3 to 2.0 pounds of chloride per cubic yard.

TABLE 4.4-2 Standard ASTM and AASHTO Test Methods for Steel for Use in the Laboratory

Designation[a]	Title
A 370/T 244	Methods and Definitions for Mechanical Testing of Steel Products
E 3	Methods of Preparation of Metallographic Specimens
E 8/T 68	Methods of Tension Testing of Metallic Materials
E 10/T 70	Test Method for Brinnel Hardness of Metallic Materials
E 92	Test Method for Vickers Hardness of Metallic Materials
E 103	Method of Rapid Indentation Testing of Metallic Materials
E 110	Test Method for Indentation Hardness of Metallic Materials by Portable Hardness Testers
E 112	Methods for Determining Average Grain Size
E 340	Method for Macroetching Metals and Alloys
E 384	Test Method for Microhardness of Materials
E 407	Methods for Microetching Metals and Alloys
E 807	Practice for Metallographic Laboratory Evaluation
E 883	Practice for Metallographic Photomicrography

[a] ASTM test methods are designated A or E. AASHTO test methods are designated T.

4.4 LABORATORY TESTS

To supplement field tests and observations, there are many laboratory tests which have been standardized and used routinely in the evaluation of materials used in bridges. Tables 4.4.-1, 4.4-2 and 4.4-3 list the ASTM and AASHTO Standards governing the laboratory testing of concrete, steel and timber components respectively.

Laboratory tests should be conducted by testing laboratories familiar with the AASHTO, ASTM and Bridge Owner standards to be employed.

4.5 INTERPRETATION AND EVALUATION OF TEST RESULTS

Field and laboratory test results must be interpreted and evaluated by a person experienced in such activity. If the same test has been previously run on material from this structure, the test results should be compared, differences noted, and those differences evaluated. When more than one type of test is used to measure the same material property, the individual test results should be compared and differences explained.

TABLE 4.4-3 Standard Test Methods for Timber for Use in the Laboratory

Designation	Title
D 143	Method of Testing Small Clear Specimens of Timber
D 198	Method for Static Tests of Timbers in Structural Sizes
D 1860	Test Method for Moisture and Creosote-Type Preservation in Wood [a]
D 2016	Test Methods for Moisture Content of Wood
D 2017	Method for Accelerated Laboratory Test of Natural Decay Resistance of Woods
D 2085	Test Method for Chloride for Calculating Pentachlorophenol in Solutions for Wood (Lime Ignition Method)
D 2395	Test Methods for Specific Gravity of Wood and Wood-Base Materials
D 2915	Method for Evaluating Allowable Properties for Grades of Structural Lumber
D 3345	Method for Laboratory Evaluation of Wood and Other Cellulosic Materials for Resistance to Termites

[a] Substantially the same as AWPA-A6.

4.6 TESTING REPORTS

It is important that all field and laboratory tests be documented in writing and become part of the bridge file. Where instrumentation is used in the conduct of the test, the report should include the type of equipment, including the manufacturer and serial number, a copy of the most recent calibration certificate, and the name of the trained operator.

For laboratory tests, the results should be submitted in a formal report using the laboratory letterhead, signed by a responsible official of the laboratory.

6. LOAD RATING

6.1 GENERAL

Bridge load rating calculations provide a basis for determining the safe load capacity of a bridge. Load rating requires engineering judgment in determining a rating value that is applicable to maintaining the safe use of the bridge and arriving at posting and permit decisions. Bridge load rating calculations are based on information in the bridge file including the results of a recent inspection. As part of every inspection cycle, bridge load ratings should be reviewed and updated to reflect any relevant changes in condition or dead load noted during the inspection.

Bridge Owners should implement standardized procedures for determining the load rating of bridges based on this Manual.

This Manual provides a choice of load rating methods. Load ratings at Operating and Inventory levels using the allowable stress method can be calculated and may be especially useful for comparison with past practices. Similarly, load ratings at Operating and Inventory levels based on the load factor method can also be calculated. Each of these rating methods is presented below.

In addition, some Bridge Owners may elect to determine the bridge rating by the load and resistance factor rating method (LRFR). This method is described in the AASHTO *Guide Specifications for the Strength Evaluation of Existing Steel and Concrete Bridges*.

6.1.1 Assumptions

The safe load capacity of a bridge is based on existing structural conditions. To maintain this capacity, it is assumed that the bridges are subject to competent inspections as often as the existing conditions of the structures require, and that sound judgment will be exercised in determining an appropriate safety margin.

6.1.2 Substructure Consideration

Careful attention should be given to all elements of the substructure for evidence of instability which affects the load-carrying capacity of a bridge. Evaluation of the conditions of a bridge's substructure will in many cases be a matter of good engineering judgment.

The adequacy of the substructure should be based on information from as-built plans, construction plans, design calculations, inspection results and other appropriate data. When such information is available, the substructure elements, including piers and abutments, should be checked to ensure that they have at least the capacity of the lowest rated superstructure member. If such information is not available, the substructure should be assumed to be adequate if it is judged by the engineer to be stable after examining the alignment, condition and performance of the substructure elements over time.

6.1.3 Safety Criteria

In general, the safety factors to be used should be taken from this Manual. However, there are some cases where judgment must be exercised in making an evaluation of a structure and the safety factor may be adjusted based on site conditions and/or structure conditions as recorded in the most recent inspection report. This determination most commonly applies to timber which may be of substandard grade or where the material is weathered or otherwise deteriorated. In determining the safety factor for a bridge, consideration should be given to the types of vehicles using the bridge routinely. Every effort should be made to minimize hardships related to economic hauling without jeopardizing the safety of the public.

All data used in the determination of the safety factor should be fully documented.

6.1.4 Application of Standard Design Specifications

For all matters not covered by this Manual, the current applicable AASHTO *Standard Specifications for Highway Bridges* (AASHTO Design Specifications) should be used as a guide. However, there may be instances in which the behavior of a member under traffic is not consistent with that predicted by the controlling specification. In this situation, deviations from the controlling specifications based on the known behavior of the member under traffic may be used and should be fully documented. Diagnostic load tests may be helpful in establishing the safe load capacity for such members (see Section 5).

For ease of use and where appropriate, reference is made to specific articles in the AASHTO *Standard Specifications for Highway Bridges*, 14th Edition, 1989 with Interims through 1990.

6.1.5 Nonredundant Structures

There may exist in a structure critical components whose failure would be expected to result in the collapse of the bridge. Special considerations of these nonredundant components may be required in load rating the structure.

6.1.6 Load Rating for Complex Structures

This Manual is intended for use in rating the types of bridges commonly in use in the United States. The computation of the load-carrying capacity of more complex structures, such as suspension bridges, cable-stayed bridges, curved steel girder bridges, arches, continuous trusses, and those bridges with variable girder depth and spacing, requires special analysis methods and procedures. General guidance and direction is available in this Manual, but more complex procedures must be used for the actual determination of the load rating.

6.2 QUALIFICATIONS AND RESPONSIBILITIES

The individual charged with the overall responsibility for load rating bridges should be a licensed professional engineer and preferably have a minimum of 5 years of bridge design and inspection experience. The engineering knowledge and skills necessary to properly evaluate bridges may vary widely depending on the complexity of the bridge involved. The specialized knowledge and skills of other engineers may be needed to ensure proper evaluation.

6.3 RATING LEVELS

Each highway bridge should be load rated at two levels, Inventory and Operating levels.

6.3.1 Inventory Rating Level

The Inventory rating level generally corresponds to the customary design level of stresses but reflects the existing bridge and material conditions with regard to deterioration and loss of section. Load ratings based on the Inventory level allow comparisons with the capacity for new structures and, therefore, results in a live load which can safely utilize an existing structure for an indefinite period of time.

6.3.2 Operating Rating Level

Load ratings based on the Operating rating level generally describe the maximum permissible live load to which the structure may be subjected. Allowing unlimited numbers of vehicles to use the bridge at Operating level may shorten the life of the bridge.

6.4 RATING METHODS

In the load rating of bridge members, two methods for checking the capacity of the members are provided in this Manual, the Allowable Stress method and Load Factor method.

6.4.1 Allowable Stress (AS)

The allowable or working stress method constitutes a traditional specification to provide structural safety. The actual loadings are combined to produce a maximum stress in a member which is not to exceed the allowable or working stress. The latter is found by taking the limiting stress of the material and applying an appropriate factor of safety.

6.4.2 Load Factor (LF)

The Load Factor method is based on analyzing a structure subject to multiples of the actual loads (factored loads). Different factors are applied to each type of load which reflect the uncertainty inherent in the load calculations. The rating is determined such that the effect of the factored loads does not exceed the strength of the member.

6.5 RATING EQUATION

6.5.1 General

The following general expression should be used in determining the load rating of the structure:

$$RF = \frac{C - A_1D}{A_2L(1 + I)} \tag{6-1a}$$

where:

RF = the rating factor for the live-load carrying capacity. The rating factor multiplied by the rating vehicle in tons gives the rating of the structure (see equation 6-1b)
C = the capacity of the member (see Article 6.6)
D = the dead load effect on the member (see Article 6.7.1). For composite members, the dead load effect on the noncomposite section and the dead load effect on the composite section need to be evaluated when the Allowable Stress method is used
L = the live load effect on the member (see Article 6.7.2)
I = the impact factor to be used with the live load effect (see Article 6.7.4)
A_1 = factor for dead loads (see Articles 6.5.2 and 6.5.3)
A_2 = factor for live load (see Articles 6.5.2 and 6.5.3)

In the equation above "load effect" is the effect of the applied loads on the member. Typical "load effects" used by engineers are axial force, vertical shear force, bending moment, axial stress, shear stress and bending stresses. Once the "load effect" to be evaluated is selected by the engineer, the "capacity" of a member to resist such a load effect may be determined (see Article 6.6).

The Rating Factor (RF) may be used to determine the rating of the bridge member in tons as follows:

$$RT = (RF)W \tag{6-1b}$$

where:

RT = bridge member rating in tons
W = weight (tons) of nominal truck used in determining the live load effect (L)

The rating of a bridge is controlled by the member with the lowest rating in tons.

6.5.2 Allowable Stress

For the allowable stress method, $A_1 = 1.0$ and $A_2 = 1.0$ in the general rating equation.

The capacity (C) depends on the rating level desired, with the higher value for "C" used for the Operating level. The determination of the nominal capacity of a member is discussed in Article 6.6.2.

6.5.3 Load Factor

For the load factor method, $A_1 = 1.3$ and A_2 varies depending on the rating level desired. For Inventory level, $A_2 = 2.17$ and for Operating level, $A_2 = 1.3$.

The nominal capacity (C) is the same regardless of the rating level desired (see Article 6.6.3).

6.5.4 Condition of Bridge Members

The condition and extent of deterioration of structural components of the bridge should be considered in the computation of the dead load and live load effects when stress is chosen as the evaluation approach, and for the capacity when force or moment is chosen for use in the basic rating equation.

The rating of an older bridge for its load-carrying capacity should be based on a recent thorough field investigation. All physical features of a bridge which have an effect on its structural integrity should be examined as discussed in Section 3. Note any damaged or deteriorated sections and obtain adequate data on these areas so that their effect can be properly evaluated in the analysis. Where steel is severely corroded, concrete deteriorated, or timber decayed, make a determination of the loss in a cross-sectional area as closely as reasonably possible. Determine if deep pits, nicks or other defects exist that may cause stress concentration areas in any structural member. Lowering load capacities below those otherwise permitted or other remedial action may be necessary if such conditions exist.

Size, number, and relative location of bolts and rivets through tension members should be determined and recorded so that the net area of the section can be calculated. Also, in addition to the physical condition, threaded members such as truss rods at turn-buckles should be checked to see if the rod has been upset so that the net area will be properly calculated. This information will normally be taken from plans when they are available, but should be determined in the field otherwise. Any misalignment, bends, or kinks in compression members should be measured carefully. Such defects will have a great effect on the load-carrying capability of a member and may be the controlling factor in the load-carrying capacity of the entire structure. Also, examine the connections of compression members carefully to see if they are detailed such that eccentricities are introduced which must be considered in the structural analysis.

The effective area of members to be used in the calculations shall be the gross area less that portion which has deteriorated due to decay or corrosion. The effective area should be adjusted for rivet or bolt holes in accordance with the AASHTO Design Specifications.

6.5.5 Bridges with Unknown Structural Components

For redundant bridges where necessary details, such as reinforcement in a concrete bridge, are not available from plans or field measurements, a physical inspection of the bridge by a qualified inspector and evaluation by a qualified engineer may be sufficient to approximate Inventory and Operating ratings. Load tests may be helpful in establishing the safe load capacity for such structures (see Section 5).

6.6 NOMINAL CAPACITY (C)

6.6.1 General

The nominal capacity to be used in the rating equation depends on the structural materials, the rating method and rating level used. Nominal capacities based on the Allowable Stress method are discussed in Article 6.6.2 and those based on the Load Factor method are discussed in Article 6.6.3.

The Bridge Owner is responsible for selecting the rating method. The method used should be identified for future reference.

6.6.2 Allowable Stress Method

In the Allowable Stress method, the capacity of a member is based on the rating level evaluated: Inventory level-Allowable Stress, or Operating level-Allowable Stress.

The properties to be used for determining the allowable stress capacity for different materials follow. For convenience, the tables provide, where appropriate, the Inventory, Operating and yield stress values. Allowable stress and strength formulas should be those provided herein or those contained in the AASHTO Design Specifications. When situations arise that are not covered by these specifications, then rational strength of material formulae should be used consistent with data and plans verified in the field investigation. Deviations from the AASHTO Design Specifications should be fully documented.

When the bridge materials or construction are unknown, the allowable stresses should be fixed by the engineer, based on field investigations and/or material testing conducted in accordance with Section 4, and should be substituted for the basic stresses given herein.

6.6.2.1 Structural Steel

The allowable unit stresses used for determining safe load capacity depend on the type of steel used in the structural members. When non-specification metals are encountered, coupon testing may be used to determine a nominal yield point. When information on specifications of the steel is not available, allowable stresses should be taken from the applicable "Date Built" column of Tables 6.6.2.1-1 and 6.6.2.1-2.

Table 6.6.2.1-1 gives allowable Inventory stresses and Table 6.6.2.1-2 gives the allowable Operating stresses for structural steel. The nominal yield stress, F_y, is also shown in Tables 6.6.2.1-1 and 6.6.2.1-2. Tables 6.6.2.1-3 and 6.6.2.1-4 give the allowable Inventory and Operating Stresses for bolts and rivets. For compression members, the effective length (KL) may be determined in accordance with the AASHTO Design Specifications or taken as follows:

KL = 75% of the total length of a column having riveted end connections
= 87.5% of the total length of a column having pinned end connections

The modulus of Elasticity (E) for steel should be 29,000,000 lbs. per sq. in.

If the investigation of shear and stiffener spacing is desirable, such investigation may be based on the AASHTO Design Specifications.

6.6.2.1.1 Combined Stresses

The allowable combined stresses for steel compression members may be calculated by the provisions of AASHTO Design Specifications as modified below or by the procedure contained in Appendix A11.

In using the AASHTO Design Specifications (Article 10.36), the allowable compressive axial stress (F_a) and the allowable compressive bending stresses (F_{bx} and F_{by}) should be based on Tables

TABLE 6.6.2.1-1 INVENTORY RATING ALLOWABLE STRESSES (psi)

			DATE BUILT-STEEL UNKNOWN					Silicon Steel			
			Prior to 1905	1905 to 1936	1936 to 1963	After 1963	Carbon Steel	Over 2" to 4" incl	Nickel Steel	1-1/8" and Under	Over 1-1/8" to 2" incl
AASHTO Designation [1]							M 94(1961)	M 95(1961)	M 96(1961)		
ASTM Designation [1]							A 7(1967)	A 94(1966)	A 8(1961)	A 94	A 94
Minimum Tensile Strength		F_u	52,000	60,000			60,000	70,000	90,000	75,000	72,000
Minimum Yield Point		F_y	26,000	30,000	33,000	36,000	33,000	45,000	55,000	50,000	47,000
Axial tension in members with no holes for high strength bolts or rivets. Use net section when member has any open holes larger than 1-1/4" diam. such as perforations		$0.55F_y$	14,000	16,000	18,000	20,000	18,000	24,000	30,000	27,000	25,000
		$0.46F_y$				NOT APPLICABLE					
Axial tension in members with holes for high strength bolts or rivets and tension in extreme fiber of rolled shapes, girders, and built-up sections subject to bending	use whichever is smaller	Gross* Section $0.55F_y$	14,000	16,000	18,000	20,000	18,000	24,000	30,000	27,000	25,000
• When the area of holes deducted for high strength bolts or rivets is more than 15 percent of the gross area, that area in excess of 15 percent shall be deducted from the gross area in determining stress on the gross section. In determining gross section, any open holes larger than 1-1/4" diam. such as perforations shall be deducted.		Net Section $0.50F_u$	26,000	30,000	30,000	30,000	30,000	35,000	45,000	37,500	36,000
		Net Section $0.46F_u$				NOT APPLICABLE					
Axial tension in members without holes. Axial compression, gross section: stiffeners of plate girders. Compression in splice material, gross section		$0.55F_y$	14,000	16,000	18,000	20,000	18,000	24,000	30,000	27,000	25,000
Compression in extreme fibers of rolled shapes, girders and built-up sections, subject to bending, gross section, when compression flange is. (A) Supported laterally its full length by embedment in concrete		$0.55F_y$	14,000	16,000	18,000	20,000	18,000	24,000	30,000	27,000	25,000
(B) Partially supported or unsupported[2]											

$$F_b = \frac{91 \times 10^6 C_b}{(F.S.)\, S_{xc}} \left(\frac{I_{yc}}{l}\right) \sqrt{0.772 \frac{J}{I_{yc}} + 9.87 \left(\frac{d}{l}\right)^2} \le 0.55F_y$$

C_b = 1.75 + 1.05 (M_1 / M_2) + 0.3 $(M_1 / M_2)^2 \le 2.3$ where M_1 is the smaller and M_2 is the larger end moment in the unbraced segment of the beams; M_1 / M_2 is positive when the moments cause reverse curvature and negative when bent in single curvature.

C_b = 1.0 for unbraced cantilevers and for members where the moment within a significant portion of the unbraced segment is greater than or equal to the larger of the segment end moments.

F.S. = Factor of Safety at Inventory Level = 1.82

TABLE 6.6.2.1-1 INVENTORY RATING ALLOWABLE STRESSES (psi) (continued)

		DATE BUILT-STEEL UNKNOWN								
		Prior to 1905	1905 to 1936	1936 to 1963	After 1963	Carbon Steel	Silicon Steel Over 2" to 4" incl	Nickel Steel	1-1/8" and Under	Over 1-1/8" to 2" incl
Compression in concentrically loaded columns[3]										
with $C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$		148.4	138.1	131.7	126.1	131.7	112.8	102.0	107.0	110.4
$F_a = \frac{F_y}{F.S.}\left[1 - \frac{\left(\frac{KL}{r}\right)^2 F_y}{4\pi^2 E}\right]$ when $\frac{KL}{r} \le C_c$		$12{,}260 - 0.28\left(\frac{KL}{r}\right)^2$	$14{,}150 - 0.37\left(\frac{KL}{r}\right)^2$	$15{,}570 - 0.45\left(\frac{KL}{r}\right)^2$	$16{,}980 - 0.53\left(\frac{KL}{r}\right)^2$	$15{,}570 - 0.45\left(\frac{KL}{r}\right)^2$	$21{,}230 - 0.83\left(\frac{KL}{r}\right)^2$	$25{,}940 - 1.25\left(\frac{KL}{r}\right)^2$	$23{,}580 - 1.03\left(\frac{KL}{r}\right)^2$	$22{,}170 - 0.91\left(\frac{KL}{r}\right)^2$
$F_a = \frac{\pi^2 E}{F.S.\left(\frac{KL}{r}\right)^2} = \frac{135{,}008{,}740}{\left(\frac{KL}{r}\right)^2}$ when $KL \ge C_c$ with F.S. = 2.12										
Shear in girder webs, gross section		8,500	9,500	11,000	12,000	11,000	14,000	17,500	16,500	15,500
Bearing on milled stiffeners and other steel parts in contact. Stress in extreme fiber of pins	$0.80F_y$	20,000	24,000	26,000	29,000	26,000	36,000	44,000	40,000	37,000
Bearing on pins not subject to rotation		20,000	24,000	26,000	29,000	26,000	32,000	40,000	40,000	37,000
Bearing on pins subject to rotation (such as rockers and hinges)		10,000	12,000	13,000	14,000	13,000	16,000	18,000	20,000	18,000
Shear in pins	$0.40F_y$	10,000	12,000	13,000	14,000	13,000	18,000	22,000	20,000	18,000
Bearing on Power-Driven Rivets and high strength bolts (or as limited by allowable bearing on the Fasteners)	$1.35F_u$	70,000	81,000			81,000	94,500	121,000	100,000	97,500

(1) Number in parenthesis represents the last year these specifications were printed

(2) For the use of larger C_b values, see *Structural Stability Research Council Guide to Stability Design Criteria for Metal Structures*, 3rd Ed., pg. 135. If cover plates are used, the allowable static stress at the point of theoretical cutoff shall be as determined by the formula.

l = length in inches, of unsupported flange between lateral connections, knee braces, or other points of support

I_{yc} = moment of inertia of compression flange about the vertical axis in the plane of the web in.^{4}

d = depth of girder, in.

$J = \frac{[(bt^3)_c + (bt^3)_t + Dt_w^3]}{3}$, in.^{4}, where b and t represent the flange width and thickness of the compression and tension flange, D is the web depth, and t_w is the web thickness.

S_{xc} = Section modulus with respect to the compression flange, in^3.

(3) E = modulus of elasticity of steel

r = governing radius of gyration

L = actual unbraced length

K = effective length factor

Note: The formulae do not apply to members with variable moment of inertia

TABLE 6.6.2.1-1 INVENTORY RATING ALLOWABLE STRESSES (psi) (continued)

			1-1/2" max	1/2" max	Over 2-1/2" to 4" incl	3/4" and under	To 2-1/2" incl (A 514) All thick (A 517)	Ov. 4" to 5" incl (A 588) Ov. 3/4" to 1-1/2" incl
AASHTO Designation [1]								
ASTM Designation [1]			A 572	A 572	A 514	A 242, A 440, A 441	A 514/A 517	A 242, A 440, A 441, A 588
Minimum Tensile Strength		F_u	60,000	80,000		70,000	115,000	67,000
Minimum Yield Point		F_y	45,000	65,000	90,000	50,000	100,000	46,000
Axial tension in members with no holes for high strength bolts or rivets. Use net section when member has any open holes larger than 1-1/4" diam. such as perforations		$0.55F_y$ $0.46F_u$	25,000 NOT APPLICABLE	36,000	N.A. 48,300	27,000 N.A.	55,000 53,000	25,000 N.A.
Axial tension in members with holes for high strength bolts or rivets and tension in extreme fiber of rolled shapes, girders, and built-up sections subject to bending	use whichever is smaller	Gross* Section $0.55F_y$	25,000	36,000	49,000	27,000	55,000	25,000
• When the area of holes deducted for high strength bolts or rivets is more than 15 percent of the gross area, that area in excess of 15 percent shall be deducted from the gross area in determining stress on the gross section. In determining gross section, any open holes larger than 1-1/4" diam. such as perforations shall be deducted.		Net Section $0.50F_u$	30,000	40,000	N.A.	35,000	N.A.	33,500
		Net Section $0.46F_u$	NOT APPLICABLE		48,300	N.A.	53,000	N.A.
Axial tension in members without holes. Axial compression, gross section: stiffeners of plate girders. Compression in splice material, gross section		$0.55F_y$	25,000	36,000	49,000	27,000	55,000	25,000
Compression in extreme fibers of rolled shapes, girders and built-up sections, subject to bending, gross section, when compression flange is. (A) Supported laterally its full length by embedment in concrete		$0.55F_y$	25,000	36,000	49,000	27,000	55,000	25,000

(B) Partially supported or unsupported[2]

$$F_b = \frac{91 \times 10^6 C_b}{(F.S.)\, S_{xc}} \left(\frac{I_{yc}}{l}\right) \sqrt{0.772\frac{J}{I_{yc}} + 9.87\left(\frac{d}{l}\right)^2} \le 0.55F_y$$

$C_b = 1.75 + 1.05\,(M_1 / M_2) + 0.3\,(M_1 / M_2)^2 \le 2.3$ where M_1 is the smaller and M_2 is the larger end moment in the unbraced segment of the beams; M_1 / M_2 is positive when the moments cause reverse curvature and negative when bent in single curvature.

C_b = 1.0 for unbraced cantilevers and for members where the moment within a significant portion of the unbraced segment is greater than or equal to the larger of the segment end moments.

F.S. = Factor of Safety at Inventory Level = 1.82

TABLE 6.6.2.1-1 INVENTORY RATING ALLOWABLE STRESSES (psi) (continued)

		1-1/2" max	1/2" max	Over 2-1/2" to 4" incl	3/4" and under	To 2-1/2" incl (A 514) All thick (A 517)	Ov. 4" to 5" incl (A 588) Ov. 3/4" to 1-1/2" incl
Compression in concentrically loaded columns[1]							
with $C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$		112.8	93.8	79.8	107.0	75.7	111.6
$F_a = \frac{F_y}{F.S.}\left[1 - \frac{\left(\frac{KL}{r}\right)^2 F_y}{4\pi^2 E}\right]$ when $\frac{KL}{r} \le C_c$		$21{,}230 - 0.83\left(\frac{KL}{r}\right)^2$	$30{,}660 - 1.74\left(\frac{KL}{r}\right)^2$	$42{,}450 - 3.34\left(\frac{KL}{r}\right)^2$	$23{,}580 - 1.03\left(\frac{KL}{r}\right)^2$	$47{,}170 - 4.12\left(\frac{KL}{r}\right)^2$	$21{,}700 - 0.87\left(\frac{KL}{r}\right)^2$
$F_a = \frac{\pi^2 E}{F.S.\left(\frac{KL}{r}\right)^2} = \frac{135{,}008{,}740}{\left(\frac{KL}{r}\right)^2}$ when $KL \ge C_c$ with F.S. = 2.12							
Shear in girder webs, gross section		15,000	22,000	30,000	17,000	30,000	15,000
Bearing on milled stiffeners and other steel parts in contact. Stress in extreme fiber of pins	$0.80F_y$	37,000	52,000	72,000	40,000	80,000	37,000
Bearing on pins not subject to rotation		37,000	52,000	72,000	40,000	80,000	37,000
Bearing on pins subject to rotation (such as rockers and hinges)		18,000	26,000	36,000	20,000	40,000	18,000
Shear in pins	$0.40F_y$	18,000	26,000	36,000	20,000	40,000	18,000
Bearing on Power-Driven Rivets and high strength bolts (or as limited by allowable bearing on the Fasteners)	$1.35F_u$	81,000	108,000	142,000	94,500	155,000	90,500

TABLE 6.6.2.1-1 INVENTORY RATING ALLOWABLE STRESSES (psi) (continued)

		1-1/2" max	1" max	Over 5" to 8" incl" (A 588) ov. 1-1/2" to 4" incl	Over 4" to 8" incl
AASHTO Designation [1]					M 188
ASTM Designation [1]		A 572	A 572	A 242, A 440, A 441, A 588, A 572	A 441
Minimum Tensile Strength	F_u	70,000	75,000	63,000	60,000
Minimum Yield Point	F_y	55,000	60,000	42,000	40,000
Axial tension in members with no holes for high strength bolts or rivets. Use net section when member has any open holes larger than 1-1/4" diam. such as perforations	$0.55F_y$ $0.46F_y$	30,000 NOT APPLICABLE	33,000	23,000	22,000
Axial tension in members with holes for high strength bolts or rivets and tension in extreme fiber of rolled shapes, girders, and built-up sections subject to bending (use whichever is smaller)	Gross* Section $0.55F_y$	30,000	33,000	23,000	22,000
	Net Section $0.50F_u$	35,000	37,500	31,500	30,000
• When the area of holes deducted for high strength bolts or rivets is more than 15 percent of the gross area, that area in excess of 15 percent shall be deducted from the gross area in determining stress on the gross section. In determining gross section, any open holes larger than 1-1/4" diam. such as perforations shall be deducted.	Net Section $0.46F_u$	NOT APPLICABLE			
Axial tension in members without holes. Axial compression, gross section: stiffeners of plate girders. Compression in splice material, gross section	$0.55F_y$	30,000	33,000	23,000	22,000
Compression in extreme fibers of rolled shapes, girders and built-up sections, subject to bending, gross section, when compression flange is. (A) Supported laterally its full length by embedment in concrete	$0.55F_y$	30,000	33,000	23,000	
(B) Partially supported or unsupported[2]					

$$F_b = \frac{91 \times 10^6 C_b}{(F.S.)\, S_{xc}} \left(\frac{I_{yc}}{l}\right) \sqrt{0.772 \frac{J}{I_{yc}} + 9.87 \left(\frac{d}{l}\right)^2} \leq 0.55F_y$$

TABLE 6.6.2.1-1 INVENTORY RATING ALLOWABLE STRESSES (psi) (continued)

		1-1/2" max	1" max	ov. 5" to 8" incl" (A 588) ov. 1-1/2" to 4" incl	Over 4" to 8" incl

$C_b = 1.75 + 1.05\,(M_1/M_2) + 0.3\,(M_1/M_2)^2 \leq 2.3$ where M_1 is the smaller and M_2 is the larger end moment in the unbraced segment of the beams; M_1/M_2 is positive when the moments cause reverse curvature and negative when bent in single curvature.

$C_b = 1.0$ for unbraced cantilevers and for members where the moment within a significant portion of the unbraced segment is greater than or equal to the larger of the segment end moments.

F.S. = Factor of Safety at Inventory Level = 1.82

		1-1/2" max	1" max	ov. 5" to 8" incl" (A 588) ov. 1-1/2" to 4" incl	Over 4" to 8" incl
Compression in concentrically loaded columns[3]					
with $C_c = \sqrt{\dfrac{2\pi^2 E}{F_y}}$		102.0	97.7	116.7	
$F_a = \dfrac{F_y}{F.S.}\left[1 - \dfrac{\left(\frac{KL}{r}\right)^2 F_y}{4\pi^2 E}\right]$ when $\dfrac{KL}{r} \leq C_c$		$25{,}940 - 1.25\left(\frac{KL}{r}\right)^2$	$28{,}300 - 1.48\left(\frac{KL}{r}\right)^2$	$19{,}810 - 0.73\left(\frac{KL}{r}\right)^2$	
$F_a = \dfrac{\pi^2 E}{F.S.\left(\frac{KL}{r}\right)^2} = \dfrac{135{,}008{,}740}{\left(\frac{KL}{r}\right)^2}$ when $KL \geq C_c$ with F.S. = 2.12					
Shear in girder webs, gross section		18,000	20,000	14,000	
Bearing on milled stiffeners and other steel parts in contact. Stress in extreme fiber of pins	$0.80F_y$	44,000	48,000	34,000	32,000
Bearing on pins not subject to rotation		44,000	48,000	34,000	32,000
Bearing on pins subject to rotation (such as rockers and hinges)		22,000	24,000	17,000	16,000
Shear in pins	$0.40F_y$	22,000	24,000	17,000	
Bearing on Power-Driven Rivets and high strength bolts (or as limited by allowable bearing on the Fasteners)	$1.35F_u$	94,500	101,000	85,000	81,000

TABLE 6.6.2.1-2 OPERATING RATING ALLOWABLE STRESS (psi)

		DATE BUILT-STEEL UNKNOWN						
		Prior to 1905	1905 to 1936	1936 to 1963	After 1963	Carbon Steel	Silicon Steel Over 2" to 4" incl	Nickel Steel
AASHTO Designation [1]						M 94(1961)	M 95(1961)	M 96(1961)
ASTM Designation [1]						A 7(1967)	A 4(1966)	A 8(1961)
Minimum Tensile Strength	F_u	52,000	60,000			60,000	70,000	90,000
Minimum Yield Point	F_y	26,000	30,000	33,000	36,000	33,000	45,000	55,000
Axial tension in members with no holes for high strength bolts or rivets. Use net section when member has any open holes larger than 1-1/4" diam. such as perforations	$0.75F_y$	19,500	22,500	24,500	27,000	24,500		
	$0.60F_u$				NOT APPLICABLE		33,500	41,000
Axial tension in members with holes for high strength bolts or rivets and tension in extreme fiber of rolled shapes, girders, and built-up sections subject to bending • When the area of holes deducted for high strength bolts or rivets is more than 15 percent of the gross area, that area in excess of 15 percent shall be deducted from the gross area in determining stress on the gross section. In determining gross section, any open holes larger than 1-1/4" diam. such as perforations shall be deducted. use whichever is smaller	Gross* Section $0.75F_y$	19,500	22,500	24,500	27,000	24,500	33,500	41,000
	Net Section $0.67F_u$	35,000	40,000	40,000	40,000	40,000	46,500	60,000
	Net Section $0.60F_u$				NOT APPLICABLE			
Axial tension in members without holes. Axial compression, gross section: stiffeners of plate girders. Compression in splice material, gross section	$0.75F_y$	19,500	22,500	24,500	27,000	24,500	33,500	41,000
Compression in extreme fibers of rolled shapes, girders and built-up sections, subject to bending, gross section, when compression flange is. (A) Supported laterally its full length by embedment in concrete	$0.75F_y$	19,500	22,500	24,500	27,000	24,500	33,500	41,000

(B) Partially supported or unsupported[2]

$$F_b = \frac{91 \times 10^6 C_b}{(F.S.)\, S_{xc}} \left(\frac{I_{yc}}{l}\right) \sqrt{0.772 \frac{J}{I_{yc}} + 9.87 \left(\frac{d}{l}\right)^2} \leq 0.75F_y$$

C_b = 1.75 + 1.05 (M_1 / M_2) + 0.3 $(M_1 / M_2)^2 \leq 2.3$ where M_1 is the smaller and M_2 is the larger end moment in the unbraced segment of the beams; M_1 / M_2 is positive when the moments cause reverse curvature and negative when bent in single curvature.

C_b = 1.0 for unbraced cantilevers and for members where the moment within a significant portion of the unbraced segment is greater than or equal to the larger of the segment end moments.

F.S. = Factor of Safety at Operating Level = 1.34

TABLE 6.6.2.1-2 OPERATING RATING ALLOWABLE STRESS (psi) (continued)

		DATE BUILT-STEEL UNKNOWN						
		Prior to 1905	1905 to 1936	1936 to 1963	After 1963	Carbon Steel	Silicon Steel Over 2" to 4" incl	Nickel Steel
Compression in concentrically loaded columns[3]								
with $C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$		148.4	138.1	131.7	126.1	131.7	112.8	102.0
when $\frac{KL}{r} \geq C_c$								
$F_a = \frac{F_y}{F.S.}\left[1 - \frac{\left(\frac{KL}{r}\right)^2 F_y}{4\pi^2 E}\right]$ when $\frac{KL}{r} \geq C_c$		$15{,}290 - 0.35\left(\frac{KL}{r}\right)^2$	$17{,}650 - 0.46\left(\frac{KL}{r}\right)^2$	$19{,}410 - 0.56\left(\frac{KL}{r}\right)^2$	$21{,}180 - 0.67\left(\frac{KL}{r}\right)^2$	$19{,}410 - 0.56\left(\frac{KL}{r}\right)^2$	$26{,}470 - 1.04\left(\frac{KL}{r}\right)^2$	$32{,}350 - 1.55\left(\frac{KL}{r}\right)^2$
$F_a = \frac{\pi^2 E}{F.S.\left(\frac{KL}{r}\right)^2} = \frac{168{,}363{,}840}{\left(\frac{KL}{r}\right)^2}$ with F.S. = 1.70								
Shear in girder webs, gross section	$0.45F_y$	11,500	13,500	15,000	16,000	15,000	20,000	24,500
Bearing on milled stiffeners and other steel parts in contact. Stress in extreme fiber of pins	$0.90F_y$	23,000	27,000	29,500	32,000	29,500	40,500	49,500
Bearing on pins not subject to rotation	$0.90F_y$	23,000	27,000	29,500	32,000	29,500	40,500	49,500
Bearing on pins subject to rotation (such as rockers and hinges)	$0.55F_y$	14,000	16,500	18,000	19,500	18,000	24,500	30,000
Shear in pins	$0.55F_y$	14,000	16,500	18,000	19,500	18,000	24,500	30,000
Bearing on Power-Driven Rivets and high strength bolts (or as limited by allowable bearing on the Fasteners)	$1.85F_u$	96,000	111,000	111,000	111,000	111,000	129,500	166,500

(1) Number in parenthesis represents the last year these specifications were printed

(2) For the use of larger C_b values, see *Structural Stability Research Council Guide to Stability Design Criteria for Metal Structures*, 3rd Ed., pg. 135. If cover plates are used, the allowable static stress at the point of theoretical cutoff shall be as determined by the formula.

l = length in inches, of unsupported flange between lateral connections, knee braces, or other points of support

I_{yc} = moment of inertia of compression flange about the vertical axis in the plane of the web, in.^{4}

d = depth of girder, in.

$J = \frac{[(bt^3)_c + (bt^3)_t + Dt_w^3]}{3}$, in.^{4}, where b and t represent the flange width and thickness of the compression and tension flange, D is the web depth, and t_w is the web thickness.

S_{xc} = section modulus with respect to the compression flange, in^3.

(3) E = modulus of elasticity of steel

r = governing radius of gyration

L = actual unbraced length

K = effective length factor

Note: The formulae do not apply to members with variable moment of inertia.

TABLE 6.6.2.1-2 OPERATING RATING ALLOWABLE STRESS (psi) (continued)

		8" and Under	1-1/8" and Under	Over 1-1/8" to 2" incl	1-1/2" max	1/2" max	Over 2-1/2" to 4" incl	3/4" and under 4" and under (A 588)
AASHTO Designation [1]								
ASTM Designation [1]		A 36	A 94	A 94	A 572	A 572	A 514	A 242, A 440, A 441, A 588, A 572
Minimum Tensile Strength	F_u	58,000	75,000	72,000	60,000	80,000	105,000	70,000
Minimum Yield Point	F_y	36,000	50,000	47,000	45,000	65,000	90,000	50,000
Axial tension in members with no holes for high strength bolts or rivets. Use net section when member has any open holes larger than 1-1/4" diam. such as perforations	$0.75F_y$	27,000	37,500	35,000	33,500	48,500	N.A.	37,500
	$0.60F_u$	NOT APPLICABLE					63,000	N.A.
Axial tension in members with holes for high strength bolts or rivets and tension in extreme fiber of rolled shapes, girders, and built-up sections subject to bending (use whichever is smaller)	Gross* Section $0.75F_y$	27,000	37,500	35,000	33,500	48,500	67,500	37,500
	Net Section $0.67F_u$	38,000	50,000	48,000	40,000	53,000	N.A.	46,500
• When the area of holes deducted for high strength bolts or rivets is more than 15 percent of the gross area, that area in excess of 15 percent shall be deducted from the gross area in determining stress on the gross section. In determining gross section, any open holes larger than 1-1/4" diam. such as perforations shall be deducted.	Net Section $0.60F_u$	NOT APPLICABLE					63,000	N.A.
Axial tension in members without holes. Axial compression, gross section: stiffeners of plate girders. Compression in splice material, gross section	$0.75F_y$	27,000	37,500	35,000	33,500	48,500	67,500	37,500
Compression in extreme fibers of rolled shapes, girders and built-up sections, subject to bending, gross section, when compression flange is. (A) Supported laterally its full length by embedment in concrete	$0.75F_y$	27,000	37,500	35,000	33,500	48,500	67,500	37,500

(B) Partially supported or unsupported[2]

$$F_b = \frac{91 \times 10^6 C_b}{(F.S.)\,S_{xc}}\left(\frac{I_{yc}}{l}\right)\sqrt{0.772\,\frac{J}{I_{yc}} + 9.87\left(\frac{d}{l}\right)} \le 0.75F_y$$

TABLE 6.6.2.1-2 OPERATING RATING ALLOWABLE STRESS (psi) (continued)

		8" and Under	1-1/8" and Under	Over 1-1/8" to 2" incl	1-1/2" max	1/2" max	Over 2-1/2" to 4" incl	3/4" and under 4" and under (A 588)
$C_b = 1.75 + 1.05\,(M_1/M_2) + 0.3\,(M_1/M_2)^2 \leq 2.3$ where M_1 is the smaller and M_2 is the larger end moment in the unbraced segment of the beams; M_1/M_2 is positive when the moments cause reverse curvature and negative when bent in single curvature. $C_b = 1.0$ for unbraced cantilevers and for members where the moment within a significant portion of the unbraced segment is greater than or equal to the larger of the segment end moments. F.S. = Factor of Safety at Operating Level = 1.34								
Compression in concentrically loaded columns[3]								
with $C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$		126.1	107.0	110.4	112.8	93.8	79.8	107.0
when $\frac{KL}{r} \leq C_c$ $F_a = \frac{F_y}{F.S.}\left[1 - \frac{\left(\frac{Kl}{r}\right)^2 F_y}{4\pi^2 E}\right]$ when $\frac{KL}{r} \geq C_c$		$21{,}180 - 0.67\left(\frac{KL}{r}\right)^2$	$29{,}410 - 1.28\left(\frac{KL}{r}\right)^2$	$27{,}650 - 1.13\left(\frac{KL}{r}\right)^2$	$26{,}470 - 1.04\left(\frac{KL}{r}\right)^2$	$38{,}240 - 2.17\left(\frac{KL}{r}\right)^2$	$52{,}940 - 4.16\left(\frac{KL}{r}\right)^2$	$29{,}410 - 1.28\left(\frac{KL}{r}\right)^2$
$F_a = \frac{\pi^2 E}{F.S.\left(\frac{KL}{r}\right)^2} = \frac{168{,}363{,}840}{\left(\frac{KL}{r}\right)^2}$ with F.S. = 1.70								
Shear in girder webs, gross section	$0.45F_y$	16,000	22,500	21,000	20,000	29,000	40,500	22,500
Bearing on milled stiffeners and other steel parts in contact. Stress in extreme fiber of pins	$0.90F_y$	32,000	45,000	42,000	40,500	58,500	81,000	45,000
Bearing on pins not subject to rotation	$0.90F_y$	32,000	45,000	42,000	40,500	58,500	81,000	45,000
Bearing on pins subject to rotation (such as rockers and hinges)	$0.55F_y$	19,500	27,500	25,500	24,500	35,500	49,500	27,500
Shear in pins	$0.55F_y$	19,500	27,500	25,500	24,500	35,500	49,500	27,500
Bearing on Power-Driven Rivets and high strength bolts (or as limited by allowable bearing on the Fasteners)	$1.85F_u$	107,000	138,500	133,000	171,000	148,000	194,000	129,500

TABLE 6.6.2.1-2 OPERATING RATING ALLOWABLE STRESS (psi) (continued)

		To 2-1/2" incl (A 514) All thick (A 517)	Ov. 4" to 5" incl (A 588) Ov. 3/4" to 1-1/2" incl	1-1/2" max	1" max	Ov. 5" to 8" incl (A 588) Ov. 1-1/2" to 4" incl	Over 4" to 8" incl
AASHTO Designation [1]							
ASTM Designation [1]		A 514-A 517	A 242, A 440, A 441, A 588	A 572	A 572	A 242, A 440, A 441, A 588, A 572	A 441
Minimum Tensile Strength	F_u	115,000	67,000	70,000	75,000	63,000	60,000
Minimum Yield Point	F_y	100,000	46,000	55,000	60,000	42,000	40,000
Axial tension in members with no holes for high strength bolts or rivets. Use net section when member has any open holes larger than 1-1/4" diam. such as perforations	$0.75F_y$	75,000	34,500	41,000	45,000		30,000
	$0.60F_u$	69,000		NOT APPLICABLE			
Axial tension in members with holes for high strength bolts or rivets and tension in extreme fiber of rolled shapes, girders, and built-up sections subject to bending • When the area of holes deducted for high strength bolts or rivets is more than 15 percent of the gross area, that area in excess of 15 percent shall be deducted from the gross area in determining stress on the gross section. In determining gross section, any open holes larger than 1-1/4" diam. such as perforations shall be deducted. (use whichever is smaller)	Gross* Section $0.75F_y$	75,000	34,500	41,000	45,000	31,500	30,000
	Net Section $0.67F_u$	N.A.	44,500	46,500	50,000	42,000	40,000
	Net Section $0.60F_u$	69,000		NOT APPLICABLE			
Axial tension in members without holes. Axial compression, gross section: stiffeners of plate girders. Compression in splice material, gross section	$0.75F_y$	75,000	34,500	41,000	45,000	31,500	30,000
Compression in extreme fibers of rolled shapes, girders and built-up sections, subject to bending, gross section, when compression flange is. (A) Supported laterally its full length by embedment in conc,ete	$0.75F_y$	75,000	34,500	41,000	45,000	31,500	

(B). Partially supported or unsupported[2]

$$F_b = \frac{91 \times 10^6 C_b}{(F.S.)\, S_{xc}} \left(\frac{I_{yc}}{l}\right) \sqrt{0.772 \frac{J}{I_{yc}} + 9.87 \left(\frac{d}{l}\right)^2} \leq 0.75F_y$$

TABLE 6.6.2.1-2 OPERATING RATING ALLOWABLE STRESS (psi) (continued)

		To 2-1/2" incl (A 511) All thick (A 517)	Ov. 4" to 5" incl (A 588) Ov. 3/4" to 1-1/2" incl	1-1/2" max	1" max	Ov. 5" to 8" incl (A 588) Ov. 1-1/2" to 4" incl	Over 4" to 8" incl
$C_b = 1.75 + 1.05\,(M_1/M_2) + 0.3\,(M_1/M_2)^2 \le 2.3$ where M_1 is the smaller and M_2 is the larger end moment in the unbraced segment of the beams; M_1/M_2 is positive when the moments cause reverse curvature and negative when bent in single curvature. $C_b = 1.0$ for unbraced cantilevers and for members where the moment within a significant portion of the unbraced segment is greater than or equal to the larger of the segment end moments. F.S. = Factor of Safety at Operating Level = 1.34							
Compression in concentrically loaded columns[3] with $C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$ with $\frac{KL}{r} \le C_c$		75.7	111.6	102.0	97.7	116.7	
$F_a = \frac{F_y}{F.S.}\left[1 - \frac{\left(\frac{KL}{r}\right)^2 F_y}{4\pi^2 E}\right]$ when $\frac{KL}{r} \ge C_c$		$58{,}820 - 5.14\left(\frac{KL}{r}\right)^2$	$27{,}060 - 1.09\left(\frac{KL}{r}\right)^2$	$32{,}350 - 1.55\left(\frac{KL}{r}\right)^2$	$35{,}290 - 1.85\left(\frac{KL}{r}\right)^2$	$24{,}710 - 0.91\left(\frac{KL}{r}\right)^2$	
$F_a = \frac{\pi^2 E}{F.S.\left(\frac{KL}{r}\right)^2} = \frac{168{,}363{,}840}{\left(\frac{KL}{r}\right)^2}$ with F.S. = 1.70							
Shear in girder webs, gross section	$0.45F_y$	45,000	20,500	24,500	27,000	18,500	18,000
Bearing on milled stiffeners and other steel parts in contact. Stress in extreme fiber of pins	$0.90F_y$	90,000	41,000	49,500	54,000	37,500	36,000
Bearing on pins not subject to rotation	$0.90F_y$	90,000	41,000	49,500	54,000	37,500	36,000
Bearing on pins subject to rotation (such as rockers and hinges)	$0.55F_y$	55,000	25,000	30,000	33,000	23,000	22,000
Shear in pins	$0.55F_y$	55,000	25,000	30,000	33,000	23,000	22,000
Bearing on Power-Driven Rivets and high strength bolts (or as limited by allowable bearing on the Fasteners)	$1.85F_u$	213,000	124,000	129,500	138,500	116,500	111,000

Table 6.6.2.1-3 Allowable Inventory and Operating Stresses For Low Carbon Steel Bolts and Power Driven Rivets (PSI)

Type of Fastener	Rating Level	Tension	Bearing	Shear: Bearing Type Connection
(A) Low Carbon Steel Bolts: Turned Bolts (ASTM A 307) and Ribbed Bolts	INV OPR	(1) 18,800(1)(2)	20,000 27,000	11,000 15,000
(B) Power Driven Rivets (rivets driven by pneumatically or electrically operated hammers are considered power driven)				
Structural Steel Rivet (ASTM A 502 Grade 1 or ASTM A 141)	INV OPR	— —	40,000 54,500	13,500 18,000
Structural Steel Rivet (High Strength) (ASTM A 502 Grade 2)	INV OPR	— —	40,000 54,500	20,000 27,000

(1) The AASHTO Design Specifications indicate that ASTM A 307 bolts shall not be used in tension in connections subject to fatigue.
(2) Based on area at the root of thread.

6.6.2.1-1 and 6.6.2.1-2. The safety factor (F.S.) to be used in computing the Euler buckling stress (F'_e) should be as follows:

F.S. = 2.12 at Inventory Level
= 1.70 at Operating Level

6.6.2.1.2 Batten Plate Compression Members

To allow for the reduced strength of batten plate compression members, the actual length of the member shall be multiplied by the following factor to obtain the adjusted value of L/r to be substituted in the compression member formulae discussed in Articles 6.6.2.1 and 6.6.2.1.1.

Actual L/r	FACTOR Spacing center-to-center of batten plates: Up to 2d	4d	6d	10d
40	1.3	2.0	2.8	4.5
80	1.1	1.3	1.7	2.3
120	1.0	1.2	1.3	1.8
160	1.0	1.1	1.2	1.5
200	1.0	1.0	1.1	1.3

d = depth of member perpendicular to battens

For compression members having a solid plate on one side and batten plates on the other, the foregoing factors shall be reduced 50 percent.

Adjusted L/r (batten plate both sides) =
Actual L/r x factor.

Adjusted L/r (batten plate one side) =
Actual L/r x [1 + 1/2 (factor − 1)].

6.6.2.2 Wrought Iron

Allowable maximum unit stress in wrought iron for tension and bending:

Operating 20,000 psi
Inventory 14,600 psi

Where possible, coupon tests should be performed to confirm material properties used in the rating.

6.6.2.3 Reinforcing Steel

The following are the allowable unit stresses in tension for reinforcing steel. These will ordinarily be used without reduction when the condition of the steel is unknown:

	Stresses (psi): Inventory Rating	Operating Rating	Yield
Structural or unknown grade prior to 1954	18,000	25,000	33,000
Grade 40 billet, intermediate, or unknown grade (after 1954)	20,000	28,000	40,000
Grade 50 rail or hard	20,000	32,500	50,000
Grade 60	24,000	36,000	60,000

6.6.2.4 Concrete

Unit stresses in concrete may be determined in accordance with the Service Load Design Method of the AASHTO Design Specifications (Article 8.15) or be based on the articles below. When the ultimate strength (f'_c) of the concrete is unknown and the concrete is in satisfactory condition, f'_c may be determined from the following table:

Year Built	f'_c (psi)
Prior to 1959	2,500
After 1959	3,000

6.6.2.4.1 Bending

The following maximum allowable bending unit stresses in concrete in lbs/sq. in. may be used:

	Compression Due to Bending f'_c (psi)		
f'_c (psi)	Inventory Level	Operating Level	n
2000–2400	800	1200	15
2500–2900	1000	1500	12
3000–3900	1200	1900	10
4000–4900	1600	2400	8
5000 or more	2000	3000	6

The value of "n" may be varied according to the above table.

6.6.2.4.2 Columns

The determination of the capacity of a compression member based on the AASHTO Design Specifi-

Table 6.6.2.1-4 Allowable Inventory and Operating Stresses for High Strength Bolts in ksi[a]

Load Condition	Hole Type	Rating Level	AASHTO M 164[e] (ASTM A 325) Bolts	AASHTO M 253 (ASTM A 490) Bolts
Applied Tension (T)	Standard, oversize or slotted	INV	39.5	48.5
		OPR	54	66
Shear (F_v): Friction-Type Connection[b]	Standard	INV	16	20
		OPR	22	27
	Oversize	INV	13.5	17
		OPR	18	23
	Short slotted	INV	13.5	17
		OPR	18	23
	Long slotted	INV	11.5	14.5
		OPR	16	20
Shear (F_v) : Bearing-Type Connection[c]				
Threads in any shear plane	Standard or slotted	INV	19.5	25
		OPR	26	34
No threads in shear plane	Standard or slotted	INV	27	36
		OPR	36	49
Bearing[d](f_p)	Standard or slotted	INV	$\frac{LF_u}{2.2d}$ or $1.35\ F_u$	
		OPR	$\frac{LF_u}{1.75d}$ or $1.85\ F_u$	

[a] The tabulated stresses, except for bearing stress, apply to the nominal area of bolts used in any grade of steel.
[b] Applicable for contact surfaces with clean mill scale.
[c] In bearing-type connections whose length between extreme fasteners in each of the spliced parts measured parallel to the line of an axial force exceeds 50 inches (1.27 m), tabulated value shall be reduced by 20 percent.
[d] L is the distance in inches (m) measured in the line of force from the center line of a bolt to the nearest edge of an adjacent bolt or to the end of the connected part toward the force is directed; d is the diameter of the bolts; and F_u is the lowest specified minimum tensile strength of the connected parts.
[e] AASHTO M 164 (ASTM A 325) high-strength bolts are available in three types, designated as types 1, 2 or 3.

cations (Article 8.15.4) results in an Inventory level capacity. The following simplified approach establishes the maximum Operating level capacity:

Maximum safe axial load in columns at Operating rating:

$$P = f_c A_g + f_s A_s \quad (6\text{-}2)$$

where

P = Allowable axial load on column
f_c = Allowable unit stress of concrete taken from equation 6-3 or 6-4
A_g = Gross area of column
f_s = Allowable stress of steel = $0.55 f_y$
f_y = Yield strength of reinforcing steel
A_s = Area of longitudinal reinforcing steel

Compression, short columns, in which L/D is 12 or less:

$$f_c = 0.3 f'_c \quad (6\text{-}3)$$

Compression, long columns, in which L/D is greater than 12:

$$f_c = 0.3 f'_c (1.3\text{-}0.03 L/D) \quad (6\text{-}4)$$

L = Unsupported length of column
D = Least dimension of column

6.6.2.4.3 Shear (Diagonal Tension)

The Inventory level shear strength should be determined in accordance with the Service Load Design method of the AASHTO Design Specifications (Article 8.15.5).

The Operating level shear strength in beams showing no diagonal tension cracking may be found as follows:

(Total Unit Shear) = (Shear Taken by Steel) + (Shear Taken by Concrete)

$$\text{or } v = v_s + v_c = v_s + 0.05 f'_c \quad (6\text{-}5)$$

Maximum value of $0.05 f'_c$ to be used = 160 psi

Where severe diagonal tension cracking has occurred, v_c should be considered as zero and all shear stress should be taken by the reinforcing steel.

6.6.2.5 Prestressed Concrete

The Inventory level rating should be based on the in-service allowable stresses of the AASHTO Design Specifications (Article 9.15.2.2) or those established by the Bridge Owner, if more stringent.

For prestressed concrete members which meet the ductility limitations of Article 9.18 of the AASHTO Design Specifications, the Operating rating should result in moments not to exceed 75 percent of the ultimate moment capacity of the member (Article 9.17, AASHTO Design Specifications). In situations of unusual design with wide dispersion of the tendons, the Operating rating might further be controlled by stresses not to exceed 0.90 of the yield point stress in the prestressing steel nearest the extreme tension fibre of the member.

6.6.2.6 Masonry

Stone, concrete, and clay brick masonry structures should be evaluated using the allowable stress rating method. Mortar used to bind the individual masonry units should be classified in accordance with ASTM C 270.

The allowable Inventory level compressive stresses for masonry assemblies are shown in Table 6.6.2.6. These are minimum values and may be used in the absence of more reliable data such as the results of a prism test conducted in accordance with ASTM E 447. The condition of the masonry unit and mortar should be considered when assigning an allowable stress.

Allowable Operating level stresses for masonry are not included in this Manual. Masonry components should be evaluated at the Inventory level.

Reinforced masonry construction may be evaluated using the allowable unit stresses for reinforcing steel, Article 6.6.2.3 and an appropriate allowable stress in the masonry.

6.6.2.7 Timber

Determining allowable stresses for timber in existing bridges will require sound judgment on the part of the engineer making the field investigation.

(1) Inventory Stress
The Inventory unit stresses should be equal to the allowable stresses for stress-grade lumber given in the AASHTO Design Specifications.

Allowable Inventory unit stresses for timber columns should be in accordance with the applicable provisions of the AASHTO Design Specifications.

(2) Operating Stress

Table 6.6.2.6 Allowable Inventory Compressive Stresses for Evaluation of Masonry

Construction; Compressive Strength of Unit, gross area, psi	Allowable Inventory Compressive Stresses gross cross-sectional area, psi	
	Type M or S Mortar [a]	Type N Mortar [a]
Solid masonry of brick and other solid units of clay or shale; sand-lime or concrete brick:		
8000 or greater	350	300
4500	225	200
2400	160	140
1500	115	100
Grouted masonry, of clay or shale; sand-lime or concrete:		
4500 or greater	225	200
2500	160	140
1500	115	100
Solid masonry of solid concrete masonry units:		
3000 or greater	225	200
2000	160	140
1200	115	100
Masonry of hollow load bearing units:		
2000 or greater	140	120
1500	115	100
1000	75	70
700	60	55
Stone ashlar masonry:		
Granite	720	640
Limestone or marble	450	400
Sandstone or cast stone	360	320
Rubble stone masonry Coarse, rough, or random	120	100

[a] Mortar is classified in accordance with ASTM C-270.

The maximum allowable Operating unit stresses should not exceed 1.33 times the allowable stresses for stress-grade lumber given in the current AASHTO Design Specifications. Reduction from the maximum allowable stress will depend upon the grade and condition of the timber and should be determined at the time of the inspection.

Allowable Operating stress in pounds per square inch of cross-sectional area of simple solid columns should be determined by the following formulae but the allowable Operating stress should not exceed 1.33 times the values for compression parallel to grain given in the design stress table of the AASHTO Design Specifications.

$$\frac{P}{A} = \frac{4.8E}{(l/r)^2} \tag{6-6}$$

in which

P = total load in pounds
A = cross-sectional area in square inches
E = modulus of elasticity
l = unsupported overall length, in inches, between points of lateral support of simple columns
r = least radius of gyration of the section in inches

For columns of square or rectangular cross section, this formula becomes:

$$\frac{P}{A} = \frac{0.40E}{(l/d)^2} \tag{6-7}$$

in which d = dimension in inches of the narrowest face.

The above formula applies to long columns with (l/d) over 11, but not greater than 50.

For short columns, (l/d) not over 11, use the allowable design unit stress in compression parallel to grain times 1.33 for the grade of timber used.

6.6.3 Load Factor Method

Nominal capacity of structural steel, reinforced concrete and prestressed concrete should be the same as specified in the load factor sections of the AASHTO Design Specifications. Nominal strength calculations should take into consideration the observable effects of deterioration, such as loss of concrete or steel-sectional area, loss of composite action or corrosion.

Allowable fatigue strength should be checked based on the AASHTO Design Specifications. Special structural or operational conditions and policies of the Bridge Owner may also influence the determination of fatigue strength.

6.6.3.1 Structural Steel

The yield stresses used for determining ratings should depend on the type of steel used in the structural members. When nonspecification metals are encountered, coupon testing may be used to determine yield characteristics. The nominal yield value should be substituted in strength formulas and is typically taken as the mean test value minus 1.65 standard deviations. When specifications of the steel are not available, yield strengths should be taken from the applicable "Date Built" column of the tables set forth in Article 6.6.2.1.

The capacity of structural steel members should be based on the load factor requirements stated in the AASHTO Design Specifications. The capacity (C) for typical steel bridge members is summarized in Appendix C. For beams, the overload limitations of Article 10.57 should also be considered.

The Operating rating for welds, bolts, and rivets should be determined using the maximum strengths from Table 10.56A in the AASHTO Design Specifications.

The Operating rating for friction joint fasteners (A 325 bolts) should be determined using a stress of 21 ksi. A_1 and A_2 should be taken as 1.0 in the basic rating equation.

6.6.3.2 Reinforced Concrete

The following are the yield stresses for reinforcing steel.

Reinforcing Steel	Yield Point F_y (psi)
Unknown steel (prior to 1954)	33,000
Structural Grade	36,000
Billet or Intermediate Grade and unknown after 1954 (Grade 40)	40,000
Rail or Hard Grade (Grade 50)	50,000
Grade 60	60,000

The capacity of concrete members should be based on the strength requirements stated in AASHTO Design Specifications (Article 8.16). Appendix C contains formulas for the capacity (C) of typical reinforced concrete members. The area of tension steel at yield to be used in computing the ultimate moment capacity of flexural members should not exceed that available in the section or 75 percent of the reinforcement required for balanced conditions.

6.6.3.3 Prestressed Concrete

The capacity of prestressed concrete members should be evaluated for strength requirements stated in the AASHTO Design Specifications (Article 9.17). Formulas for the capacity (C) of typical prestressed concrete members are included in Appendix C. At the Inventory level, serviceability requirements should also be considered, including the ability of the section to resist cracking. The basic rating equation (6-1a) may be used to check the cracking serviceability limit state with $A_1 = 1.0$, $A_2 = 1.0$, and $C = M^*_{CR}$. M^*_{CR} is calculated in accordance with Article 9.18.2.1 of the AASHTO Design Specifications.

6.7 LOADINGS

This section discusses the loads to be used in determining the load effects in the basic rating equation (6-1a).

6.7.1 Dead Load (D)

The dead load effects of the structure should be computed in accordance with the conditions existing at the time of analysis. Minimum unit weight of materials to be used in computing the dead load stresses should be in accordance with current AASHTO Design Specifications.

For composite members, the portion of the dead load acting on the noncomposite section and the portion acting on the composite section should be determined.

Care should be exercised in estimating the weight of concrete decks since significant variations of deck thickness have been found, particularly on bridges built prior to 1965.

Nominal values of dead weight should be based on dimensions shown on the plans with allowances for normal construction tolerances.

The approximate overlay thickness should be measured at the time of the inspection.

6.7.2 Rating Live Load

The live load to be used in the basic rating equation (6-1a) should be the HS20 truck or lane loading as defined in the AASHTO Design Specifications and shown in Figures 6.7.2.1 and 6.7.2.2. Other loadings used by Bridge Owners for posting and permit decisions are discussed in Section 7.

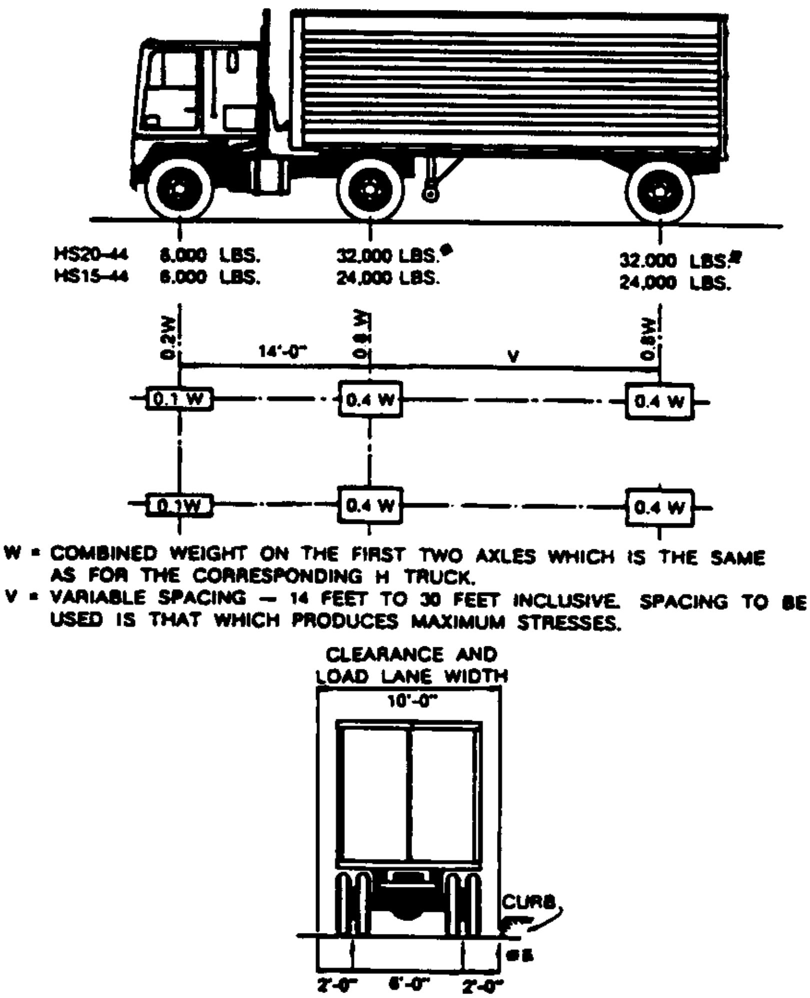


Figure 6.7.2.1 Standard HS Truck

*In the design of timber floors and orthotropic steel decks (excluding transverse beams) for HS20 loading, one-axle load of 24,000 pounds or two-axle loads of 16,000 pounds each, spaced 4 feet apart may be used, whichever produces the greater stress, instead of the 32,000-pound axle shown.

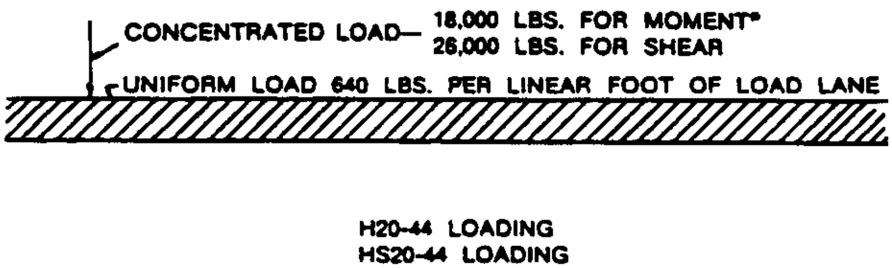


Figure 6.7.2.2 Standard HS Lane Load

*For the determination of maximum negative moment in continuous spans, the lane load shown shall be modified by the addition of a second, equal weight concentrated load placed in one other span in the series in such position to produce the maximum effect.

6.7.2.1 Wheel Loads (Deck)

In general, stresses in the deck do not control the load rating except in special cases. The calculation of bending moments in the deck should be in accordance with AASHTO Design Specifications. Wheel loads should be in accordance with the current AASHTO Design Specifications.

6.7.2.2 Truck Loads

The live or moving loads to be applied on the deck for determining the rating should be the Standard AASHTO "HS" loading.

The number of traffic lanes to be loaded, and the transverse placement of wheel lines should be in conformance with the current AASHTO Design Specifications and the following:

(1) Roadway widths from 18 to 20 feet should have two design lanes, each equal to one-half the roadway width. Live loadings should be centered in these lanes.
(2) Roadway widths less than 18 feet should carry one traffic lane only.

When conditions of traffic movements and volume would warrant it, fewer traffic lanes than specified by AASHTO may be considered.

6.7.2.3 Lane Loads

The Bridge Owner may use the Standard AASHTO HS lane load for all span lengths where it may result in load effects which are greater than those produced by the AASHTO standard HS truck.

6.7.2.4 Sidewalk Loadings

Sidewalk loadings used in calculations for safe load capacity ratings should be the probable maximum loads anticipated. Because of site variations, the determination of loading to be used will require engineering judgment, but in no case should it exceed the value given in AASHTO Design Specifications.

The Operating level should be considered when full truck and sidewalk live loads act simultaneously on the bridge.

6.7.2.5 Live Load Effects (L)

Live load moments in longitudinal stringers and girders may be calculated using the moment table, Appendix A3, for live load moments produced by the HS20 load.

Live load moments in the intermediate and end floor beams of trusses and through girders may be calculated by using the tables of live load reactions, Appendices A4 and A5. The tables, along with the moment formulas on the same sheets, provide a convenient means of computing the live load moments based on the HS20 load.

Live loads in truss members can be calculated by using the formulas for maximum shear and moments given in Appendices A6 through A10. Using these formulas will give the maximum live load stresses for the HS20 truck. Note that the formulas are valid only when used within the given limits. Modifications of the formulas may be required under loadings not meeting these limits. Such modifications may be found necessary when the structure or panels are too short to permit the entire load to be on the structure with the load positioned to produce the maximum shear or moment.

6.7.3 Distribution of Loads

The fraction of live load transferred to a single member should be selected in accordance with the current AASHTO Design Specifications. These values represent a possible combination of diverse circumstances. The option exists to substitute field measured values, analytically calculated values or those determined from advanced structural analysis methods based on the properties of the existing structure. Loadings should be placed in positions causing the maximum response in the components being evaluated.

6.7.4 Impact (I)

Impact should be added to the live load used for rating in accordance with the current AASHTO Design Specifications. However, specification impact may be reduced when conditions of alignment, enforced speed posting, and similar situations require a vehicle to substantially reduce speed in crossing the structure.

6.7.5 Deflection

Live load deflection limitations should not be considered in load rating except in special cases.

6.7.6 Longitudinal Loads

The rating of the bridge members to include the effects of longitudinal loads in combination with dead and live load effects should be done at the Operating level. Where longitudinal stability is considered inadequate, the structure may be posted for restricted speed. In addition, longitudinal loads should be used in the evaluation of the adequacy of the substructure elements.

6.7.7 Environmental Loads

The rating of the bridge members to include the effects of environmental loads in combination with dead and live load effects should be done at the Operating level.

6.7.7.1 Wind

Lateral loads due to wind normally need not be considered in load rating.

However, the effects of wind on special structures such as movable bridges, suspension bridges and other high-level structures should be evaluated.

6.7.7.2 Earthquake

Earthquake loads should not be considered in calculating load ratings or in determining live load restrictions.

To evaluate the resistance of the structure to seismic forces, the methods described in Division I-A, Seismic Design of the AASHTO Design Specifications may be used.

6.7.7.3 Thermal Effects

Stresses caused by thermal changes should not be considered in calculating load ratings except for long-span bridges and concrete arches.

6.7.7.4 Stream Flow

Forces caused by water movements should not be considered in calculating the load rating. However, remedial action should be considered if these forces are especially critical to the structure's stability.

6.7.7.5 Ice Pressure

Forces caused by ice pressure should be considered in the evaluation of substructure elements in those regions where such effect can be significant. If these forces are especially important, then corrective action should be recommended.

6.8 DOCUMENTATION OF RATING

The load rating of a bridge should be completely documented in writing including all background information such as field inspection reports, material and load test data, all supporting computations, and a clear statement of all assumptions used in calculating the load rating. If a computer model was used, the input data file should be retained for future use.

7. ADDITIONAL CONSIDERATIONS

7.1 GENERAL

Additional considerations in the assessment of the short- and long-term load-carrying capacity of bridges are presented in this Section.

7.2 CORRELATION OF INSPECTION, TESTING AND LOAD RATING DATA

The determination of the load-carrying capacity of a bridge should contain a detailed analysis of the structure, including the likelihood of failure due to overloads, lack of redundancy, and other vulnerable bridge conditions and details. It is important that all relevant data from field inspection, testing (if available) and load rating be correlated to provide a safe and reliable evaluation of the bridge. At each inspection, any deterioration or distress which has occurred which will materially affect the load-carrying capacity of the structure should be evaluated.

Normally, the determination of the load-carrying capacity of structures requires a comprehensive analysis, not limited to simply a check at maximum moment and shear points. To insure serviceability and durability, consideration should also be given to the control of permanent deformations under overloads and to the fatigue characteristics under service loads.

7.3 FATIGUE EVALUATION OF STEEL BRIDGES

The evaluation of the safe remaining life of steel bridges may be needed when planning retrofit, rehabilitation or replacement schedules or establishing limitations on permit vehicles. Remaining life is also a consideration in an assessment of critical redundant or nonredundant details for inspection and fracture control.

Procedures for carrying out the assessment of remaining life are provided in the AASHTO *Guide Specifications for Fatigue Evaluation of Existing Steel Bridges (1990).*

7.4 POSTING OF BRIDGES

7.4.1 General

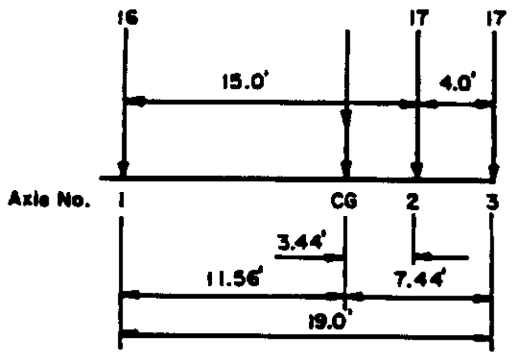
Weight limitations for the posted structure should conform to local regulations or policy within the limits established by this Manual. A bridge should be capable of carrying a minimum gross live load weight of three tons at Inventory or Operating level. When deciding whether to close or post a bridge, the owner may particularly want to consider the volume of traffic, the character of traffic, the likelihood of overweight vehicles and the enforceability of weight posting. A bridge owner may close a structure at any posting threshold, but bridges not capable of carrying a minimum gross live load weight of three tons must be closed.

A concrete bridge need not be posted for restricted loading when it has been carrying normal traffic for an appreciable length of time and shows no distress. This general rule may apply to bridges for which details of the reinforcement are not known. However, until such time as the bridge is either strengthened or replaced, it should be inspected at frequent intervals for signs of distress. In lieu of frequent inspections, a bridge may be load tested to determine its capacity.

The total load on any member caused by dead load, live load, and such other loads deemed applicable to the structure, should not exceed the member capacity as set forth in this Manual or in the rating report. When it becomes necessary to reduce the allowable live loads in order to conform to the capacity of a structure, such a reduction should be based on the assumption that each axle load maintains a proportional relation to the total load of the vehicle or vehicle combination.

7.4.2 Posting Loads

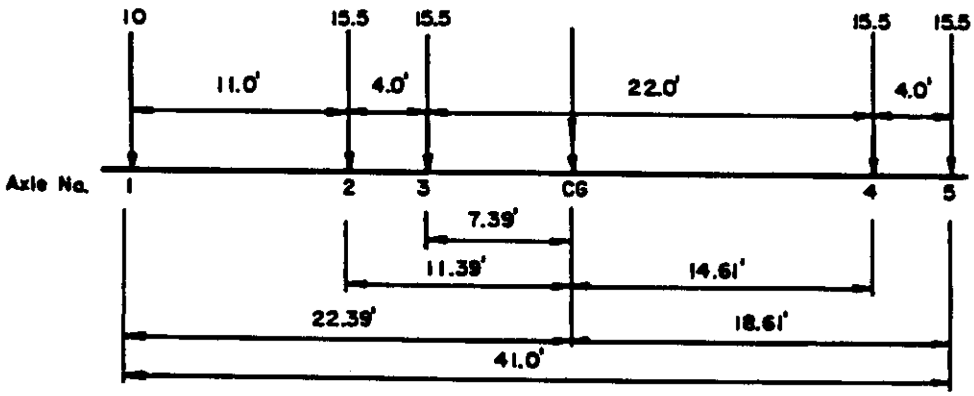
The live load to be used in the rating equation (6-1a) for posting considerations should be any of the three typical legal loads shown in Figure 7.4.3.1 or



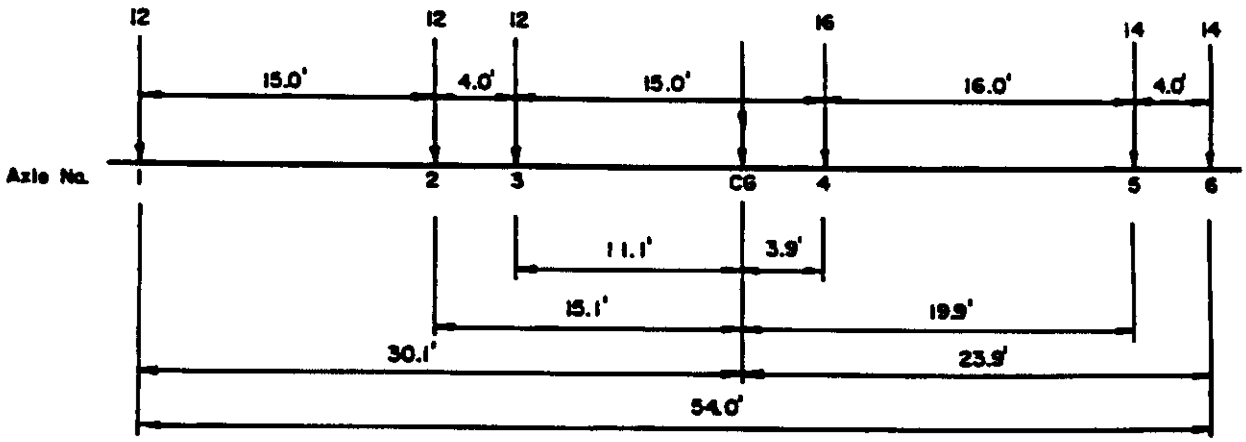
INDICATED CONCENTRATIONS ARE AXLE LOADS IN KIPS

CG = CENTER OF GRAVITY

TYPE 3 UNIT WEIGHT = 50 KIPS (25 TONS)



TYPE 3S2 UNIT WEIGHT = 72 KIPS (36 TONS)



TYPE 3-3 UNIT WEIGHT = 80KIPS (40 TONS)

Figure 7.4.3.1 Typical Legal Loads Used for Posting

state legal loads. For spans over 200 feet in length the selected legal load should be spaced with 30 feet clear distance between vehicles to simulate a train of vehicles in one lane and a single vehicle load should be applied in the adjacent lane(s).

7.4.3 Posting Analysis

The determination of the need to load post a bridge should be made by the Bridge Owner based on the general procedures in Section 6 and established practices of the Bridge Owner.

7.4.4 Regulatory Signs

Regulatory signing should conform to the requirements of the *Manual on Uniform Traffic Control Devices* (MUTCD), and should be established in accordance with the requirements of the agency having authority over the highway.

When a decision is made to close a bridge, signs and structurally sound barriers should be erected to provide adequate warning and protection to the traveling public. If pedestrian travel across the bridge is also restricted, adequate measures to prevent pedestrian use of the bridge should be installed. Signs and barriers should meet or exceed the requirements of the applicable sections of the MUTCD. Bridge closure signs and barriers should be inspected periodically to ensure their continued effectiveness.

7.4.5 Speed Limits

In some cases, lower speed limits will reduce impact loads to the extent that lowering the weight limit may not be required. Consideration of a speed posting will depend upon alignment, general location, volume, and type of traffic. A speed posting should not be considered as a basis for increasing the weight limit in areas where enforcement will be difficult and frequent violations can be anticipated.

7.5 PERMITS

7.5.1 General

Bridge Owners usually have established procedures which allow over-sized/weight vehicles to travel on the highway system. These procedures involve the issuance of a permit which describes the features of the vehicle and/or its load and, in most jurisdictions, will specify the allowable route or routes of travel. Generally speaking, permits should not be approved in situations where the load or the hauling vehicle can be reduced to conform to the size and weight limitations of local regulations.

Most Bridge Owners have methods for checking bridges to determine the effects which would be caused by the passage of vehicles above the legally established weight limitations. One approach is to check permit vehicles by the general methods of Section 6.

The live load to be used in the rating equation (6-1a) for permit decisions should be the actual vehicle size, weight, and type using the highway, together with an impact factor dependent on local conditions. The actual loading used may vary from time to time and from state to state in accordance with local laws and regulations.

The Operating level may be used for evaluating special permits for heavier than normal vehicles. Bridges which have members theoretically stressed to near the Operating level stress should be inspected more frequently than other structures.

7.5.2 Routine Permits

Routine permit vehicles are expected to mix in the random traffic stream and move at normal times and speeds. The maximum load effects of all permit vehicles allowed to move on a routine basis should be evaluated. The structural component with the lowest permit load rating on the route system should determine whether a vehicle should be issued a permit.

For routine permits, it is usually necessary to calculate load effects by assuming that a permit vehicle may occur on the bridge alongside another heavy vehicle.

7.5.3 Controlled Permits

Special or controlled permits are usually valid for a single trip only. These permit vehicles are usually heavier than those vehicles issued routine permits for unlimited trips. Depending on the authorization, these special vehicles may be allowed to mix with random traffic or may be required to be escorted in a manner which controls speed and/or lane position.

7.5.4 Escorted Permits

If a special permit vehicle is escorted, then the loading for that permit vehicle may be applied in a designated lane position. Impact values may be reduced if speed control is ensured. If the escort control is able to ensure that no other trucks will be on the bridge simultaneously with the permit vehicle, then other live loads need not be applied.

7.6 HISTORIC BRIDGES

Procedures should be in place or initiated to recognize the importance of historic bridges and to preserve them. Unfortunately, the older a bridge the greater is the uncertainty concerning its ability to carry modern truck loads. The behavior of older bridges under live loads is further complicated by questions concerning the properties of the materials used in the structure, the maintenance and repair history, the extent of deterioration and the actual response of the structural system.

It is recognized that many bridges are eligible to be placed or are already placed on the Federal and/or State Registers of Historic Places. The inspector should clearly indicate whether such a determination has been made. In the event a bridge is an "Historic Property," all work done to strengthen, repair and rehabilitate the bridge should be conducted in accordance with the applicable State and Federal regulations. The bridge may need to be "Historically Recorded" prior to commencing any repairs or removals.

The evaluation of older bridges should be comprehensive, encompassing the relevant parts of Sections 2 through 7. Consideration should be given to the use of nondestructive load tests to verify both component and system performance under a known live load.

7.7 SPECIAL CONDITIONS

Inspection, testing, load rating and evaluation of the most common bridge types are discussed in this Manual. The evaluation of highly unusual structures and special conditions requires good engineering judgment. In the load rating of such structures, the original design method may be used, with adjustments to reflect the actual condition of the structure.

APPENDIX A
MISCELLANEOUS FIGURES AND TABLES

APPENDIX A1
STRUCTURE INVENTORY AND APPRAISAL SHEET

NATIONAL BRIDGE INVENTORY-----STRUCTURE INVENTORY AND APPRAISAL MM/DD/YY

*************IDENTIFICATION*************
(1) STATE NAME - ______________CODE______
(8) STRUCTURE NUMBER # ______________
(5) INVENTORY ROUTE (ON/UNDER) - ____ = ____
(2) STATE HIGHWAY DEPARTMENT DISTRICT ____
(3) COUNTY CODE _____ (4) PLACE CODE ____
(6) FEATURES INTERSECTED ________________ -
(7) FACILITY CARRIED - ________________
(9) LOCATION - ________________
(11) MILEPOINT ________
(16) LATITUDE __ D __.__' (17) LONGITUDE __D__.__'
(98) BORDER BRIDGE STATE CODE __% SHARE __%
(99) BORDER BRIDGE STRUCTURE NO.__________

*******STRUCTURE TYPE AND MATERIAL*******
(43) STRUCTURE TYPE MAIN: MATERIAL - ________
TYPE - ________________ CODE ______
(44) STRUCTURE TYPE APPR: MATERIAL - ________
TYPE- ________________ CODE ______
(45) NUMBER OF SPANS IN MAIN UNIT ______
(46) NUMBER OF APPROACH SPANS ______
(107) DECK STRUCTURE TYPE- ____ CODE ______
(108) WEARING SURFACE/PROTECTIVE SYSTEM:
A) TYPE OF WEARING SURFACE- _______ CODE__
B) TYPE OF MEMBRANE-___________ CODE__
C) TYPE OF DECK PROTECTION-________ CODE__

*************AGE AND SERVICE*************
(27) YEAR BUILT ________
(106) YEAR RECONSTRUCTED ________
(42) TYPE OF SERVICE: ON- ______________
UNDER - ______________ CODE______
(28) LANES: ON STRUCTURE ______ UNDER STRUCTURE ______________
(29) AVERAGE DAILY TRAFFIC ______
(30) YEAR OF ADT 19__ (109) TRUCK ADT____%
(19) BYPASS, DETOUR LENGTH ______ MI

*************GEOMETRIC DATA*************
(48) LENGTH OF MAXIMUM SPAN ____ FT
(49) STRUCTURE LENGTH ____ FT
(50) CURB OR SIDEWALK: LEFT __.__FT RIGHT __.__FT
(51) BRIDGE ROADWAY WIDTH CURB-TO-CURB __.__FT
(52) DECK WIDTH OUT-TO-OUT __.__FT
(32) APPROACH ROADWAY WIDTH W/SHOULDERS ____FT
(33) BRIDGE MEDIAN - ______________ CODE ____
(34) SKEW __DEG (35) STRUCTURE FLARED ____
(10) INVENTORY ROUTE MIN VERT CLEAR __FT __IN
(47) INVENTORY ROUTE TOTAL HORIZ CLEAR __.__FT
(53) MIN VERT CLEAR OVER BRIDGE RDWY __FT__IN
(54) MIN VERT UNDERCLEAR REF- ______ __FT__IN
(55) MIN LAT UNDERCLEAR RT REF- ______ __.__FT
(56) MIN LAT UNDERCLEAR LT __.__FT

*************NAVIGATION DATA*************
(38) NAVIGATION CONTROL- __________ CODE __
(111) PIER PROTECTION- ______________ CODE__
(39) NAVIGATION VERTICAL CLEARANCE ____ FT
(116) VERT-LIFT BRIDGE NAV MIN VERT CLEAR__ FT
(40) NAVIGATION HORIZONTAL CLEARANCE ____ FT

****
SUFFICIENCY RATING = ___.__
STATUS = ______________

**********CLASSIFICATION********** CODE
(112) NBIS BRIDGE LENGTH - __________ __
(104) HIGHWAY SYSTEM - __________ __
(26) FUNCTIONAL CLASS - __________ __
(100) DEFENSE HIGHWAY - __________ __
(101) PARALLEL STRUCTURE- __________ __
(102) DIRECTION OF TRAFFIC- __________ __
(103) TEMPORARY STRUCTURE- __________ __
(110) DESIGNATED NATIONAL NETWORK- __ __
(20) TOLL - __________ __
(21) MAINTAIN- __________ __
(22) OWNER __________ __
(37) HISTORICAL SIGNIFICANCE- ______ __

*************CONDITION************* CODE
(58) DECK __
(59) SUPERSTRUCTURE __
(60) SUBSTRUCTURE __
(61) CHANNEL & CHANNEL PROTECTION __
(62) CULVERTS __

******LOAD RATING AND POSTING****** CODE
(31) DESIGN LOAD - ________ __
(64) OPERATING RATING - ________ __
(66) INVENTORY RATING - ________ __
(70) BRIDGE POSTING - ________ __
(41) STRUCTURE OPEN, POSTED OR CLOSED - __
DESCRIPTION - ______________

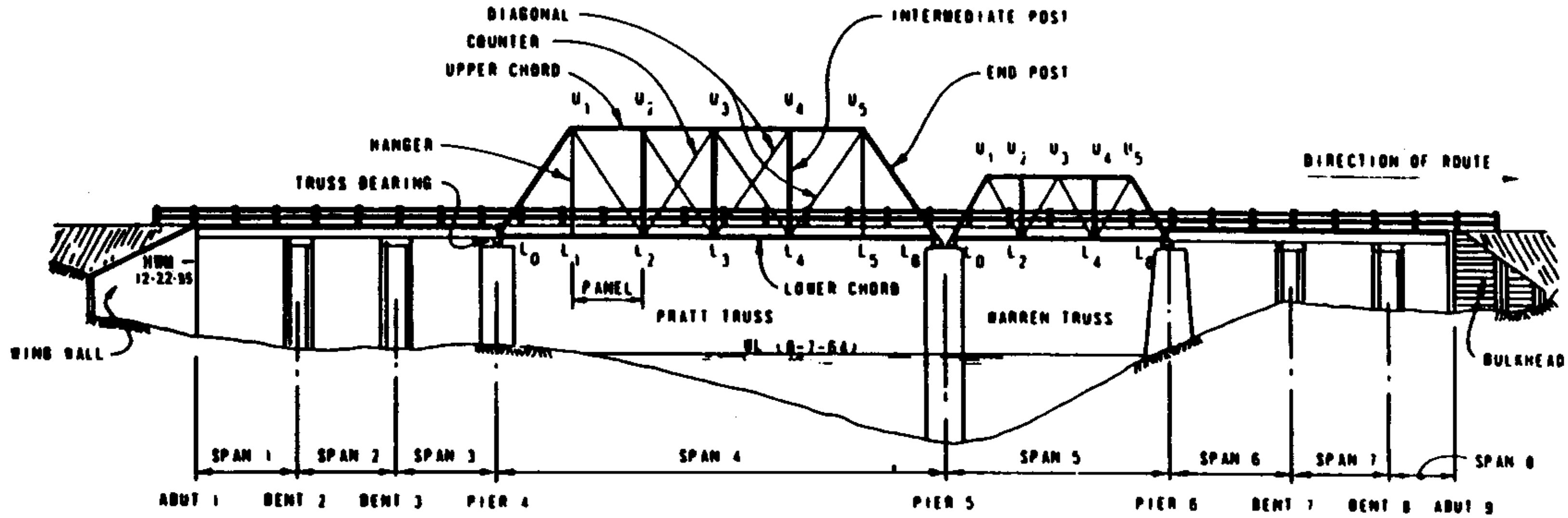
*************APPRAISAL************* CODE
(67) STRUCTURAL EVALUATION __
(68) DECK GEOMETRY __
(69) UNDERCLEARANCES, VERTICAL & HORIZONTAL __
(71) WATERWAY ADEQUACY __
(72) APPROACH ROADWAY ALIGNMENT __
(36) TRAFFIC SAFETY FEATURES __
(113) SCOUR CRITICAL BRIDGES __

*******PROPOSED IMPROVEMENTS******* CODE
(75) TYPE OF WORK- ____________ CODE____
(76) LENGTH OF STRUCTURE IMPROVEMENT___ FT
(94) BRIDGE IMPROVEMENT COST $__,__,000
(95) ROADWAY IMPROVEMENT COST $__,__,000
(96) TOTAL PROJECT COST $__,__,000
(97) YEAR OF IMPROVEMENT COST EST 19/20__
(114) FUTURE ADT ______
(115) YEAR OF FUTURE ADT 20__

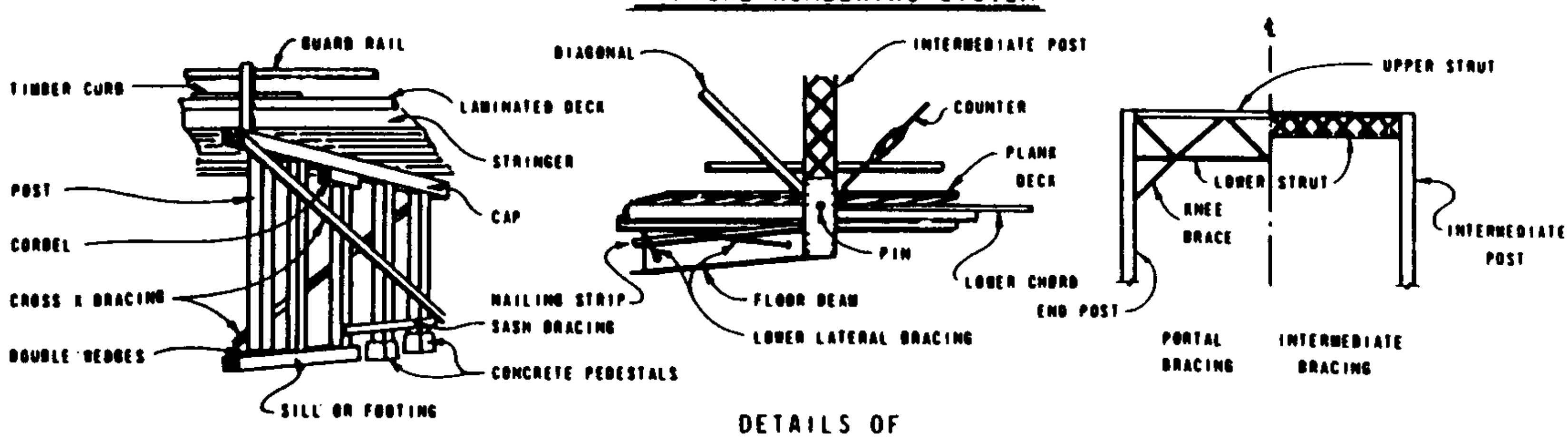
***************INSPECTIONS***************
(90) INSPECTION DATE __/__(91) FREQUENCY __ MO
(92) CRITICAL FEATURE INSPECTION: (93) CFI DATE
A) FRACTURE CRIT DETAIL-__-__MO A) __/____
B) UNDERWATER INSP - ___-__MO B) __/____
C) OTHER SPECIAL INSP - ___-__MO C)____/____

APPENDIX A2

BRIDGE NOMENCLATURE



TYPICAL NUMBERING SYSTEM



TYPICAL TIMBER BENT

DETAILS OF TRUSS AND FLOOR SYSTEM

TYPICAL SWAY FRAMING

ABBREVIATIONS

RC REINFORCED CONCRETE
RW REDWOOD
UF UNTREATED FIR
BTF BRUSH TREATED FIR (WOOD PRESERVATIVE)
CDF CREOSOTED DOUGLAS FIR (PRESSURE TREATED)
BS BEARING SURFACE
WL WATER LEVEL
HWM HIGH WATER MARK
VC VERTICAL CLEARANCE
b b DISTANCE BACK TO BACK
c c DISTANCE CENTER TO CENTER

APPENDIX A3

LIVE LOAD MOMENTS ON LONGITUDINAL STRINGERS OR GIRDERS

Live Load Moments in Foot-Kips Per Wheel Line										
Type of Loading (Without Impact)					Span	Type of Loading (With Impact)				
H-15	**HS-20**	3	3-S2	3-3	Feet c/c	H-15	**HS-20**	3	3-S2	3-3
15.0	20.0	10.6	9.7	10.0	5	19.5	26.0	13.8	12.6	13.0
18.0	24.0	12.8	11.6	12.0	6	23.4	31.2	16.6	15.1	15.6
21.0	28.0	15.2	13.8	14.0	7	27.3	36.4	19.7	18.0	18.2
24.0	32.0	19.1	17.4	16.0	8	31.2	41.6	24.9	22.7	20.8
27.0	36.0	23.1	21.1	19.1	9	35.1	46.8	30.1	27.4	24.8
30.0	40.0	27.2	24.8	22.4	10	39.0	52.0	35.4	32.2	29.1
33.0	44.0	31.3	28.5	25.8	11	42.9	57.2	40.7	37.1	33.5
36.0	48.0	35.4	32.2	29.2	12	46.8	62.4	46.0	42.0	37.9
39.0	52.0	39.6	36.1	32.6	13	50.7	67.6	51.4	46.9	42.3
42.0	56.0	43.7	39.9	36.0	14	54.6	72.8	56.8	51.8	46.8
45.0	60.0	47.9	43.7	39.4	15	58.5	78.0	62.2	56.8	51.3
48.0	64.0	52.1	47.5	42.9	16	62.4	83.2	67.7	61.7	55.7
51.0	68.0	56.3	51.3	46.3	17	66.3	88.4	73.1	66.7	60.2
54.0	72.0	60.4	55.1	49.8	18	70.2	93.6	78.6	71.6	64.7
57.0	76.0	64.6	58.9	53.2	19	74.1	98.8	84.0	76.6	69.2
60.0	80.0	68.9	62.8	56.7	20	78.0	104.0	89.5	81.6	73.7
63.0	84.0	73.1	66.6	60.2	21	81.9	109.2	95.0	86.6	78.2
66.0	88.0	77.3	70.5	63.6	22	85.8	114.4	100.5	91.6	82.7
69.0	92.0	81.5	75.2	67.1	23	89.7	119.6	105.9	97.7	87.2
72.0	96.3	85.7	80.3	70.6	24	93.6	125.2	111.4	104.4	91.8
75.0	103.7	89.9	85.4	74.1	25	97.5	134.8	116.9	111.0	96.3
78.0	111.1	94.2	90.5	77.5	26	101.4	144.4	122.4	117.7	100.8
81.3	118.5	98.4	95.6	81.0	27	105.7	154.1	127.9	124.3	105.3
85.1	126.0	102.6	100.7	84.5	28	110.6	163.8	133.4	131.0	109.8
88.8	133.5	106.8	105.9	88.0	29	115.4	173.6	138.9	137.6	114.4
92.5	141.0	112.9	111.0	91.5	30	120.2	183.3	146.8	144.3	118.9
99.8	156.2	125.3	121.2	101.5	32	130.0	203.1	162.9	157.6	132.0
107.4	171.8	137.6	131.5	112.3	34	139.6	223.3	178.9	170.9	146.0
114.8	189.4	150.0	141.7	123.1	36	149.2	246.2	195.0	184.2	160.1
122.3	207.1	162.4	151.9	134.0	38	159.0	269.2	211.1	197.5	174.1
129.7	224.9	174.8	162.2	144.8	40	168.6	292.4	227.3	210.8	188.3
137.2	242.7	187.2	172.4	155.7	42	178.3	315.3	243.3	224.0	202.3
144.7	260.4	199.7	182.7	166.6	44	187.5	337.5	258.7	236.7	215.8
152.1	278.3	212.1	192.9	177.4	46	196.6	359.6	274.1	249.3	229.3
159.6	296.1	224.5	203.2	188.3	48	205.7	381.7	289.4	261.9	242.8
167.1	314.0	237.0	220.8	199.3	50	214.8	403.8	304.7	283.9	256.2
174.6	331.8	249.4	238.4	214.3	52	223.9	425.5	319.9	305.8	274.8
182.0	349.7	261.8	256.1	231.3	54	232.8	447.3	335.0	327.6	295.9
189.5	367.6	274.3	273.8	248.3	56	241.8	469.1	350.1	349.4	316.9
198.8	385.4	286.8	291.4	265.3	58	253.1	490.6	365.1	371.1	337.7
209.2*	403.3	299.2	309.2	282.3	60	265.8*	512.2	380.1	392.7	358.5
265.1*	492.8	361.5	398.0	372.2	70	333.1*	619.0	454.2	500.1	467.6
327.0*	582.4	423.9	487.1	471.9	80	406.8*	724.5	527.3	605.9	587.0
394.9*	672.2	486.3	576.4	571.7	90	486.7*	828.8	599.4	710.5	704.6
468.8*	762.0	548.7	665.9	671.5	100	572.9*	931.2	670.7	813.9	820.7
634.5*	941.6	673.6	845.1	871.3	120	764.0*	1133.7	811.1	1017.5	1049.1
824.2*	1121.4	798.5	1024.5	1071.1	140	979.8*	1333.3	949.2	1217.8	1273.2
1038.0*	1384.0*	923.5	1204.1	1270.9	160	1220.1*	1626.2*	1085.5	1415.3	1493.9
1275.8*	1701.0*	1048.4	1383.7	1470.8	180	1484.9*	1980.0*	1222.3	1610.6	1712.0
1537.5*	2050.0*	1173.4	1563.5	1670.8	200	1774.0*	2365.7*	1353.9	1804.0	1927.8
2296.9*	3062.5*	1485.8	2013.0	2170.6	250	2603.1*	3469.8*	1683.9	2281.4	2460.0
3206.2*	4275.0*	1798.2	2462.6	2670.5	300	3583.5*	4779.4*	2009.8	2752.4	2984.7

***Based on standard lane loading. All other values based on standard truck loading.**

APPENDIX A4

STRINGER LIVE LOAD REACTIONS ON
TRANSVERSE FLOOR BEAMS & CAPS
(INTERMEDIATE TRANSVERSE BEAMS)
(Simple Span Only)

STRINGER SPAN FEET	LIVE LOAD REACTIONS (R) IN KIPS PER WHEEL LINE NO IMPACT				
	TYPE OF LOADING				
	TYPE 3	TYPE 3-S2	TYPE 3-3	H-15	HS-20
10	13.6	12.4	11.2	12.0	16.0
11	13.9	12.7	11.5	12.0	16.0
12	14.2	13.1	11.7	12.0	16.0
13	14.4	13.7	11.9	12.0	16.0
14	14.6	14.2	12.0	12.0	16.0
15	14.8	14.6	12.2	12.2	17.3
16	15.3	15.0	12.3	12.4	18.5
17	15.8	15.4	12.7	12.5	19.5
18	16.4	15.6	13.3	12.7	20.4
19	16.8	15.9	13.7	12.8	21.3
20	17.2	16.1	14.2	12.9	22.0
21	17.6	16.3	14.5	13.0	22.7
22	18.0	16.5	14.9	13.1	23.3
23	18.3	16.7	15.2	13.2	23.8
24	18.5	16.9	15.5	13.3	24.3
25	18.8	17.0	15.7	13.4	24.8
26	19.0	17.5	16.2	13.4	25.2
27	19.3	18.2	16.8	13.5	25.6
28	19.5	18.8	17.5	13.5	26.0
29	19.7	19.4	18.0	13.6	26.3
30	19.9	20.1	18.8	13.6	26.7

ONE LANE LOADING__________ $M = \frac{(L-3)^2 R}{2L}$

*TWO LANE ROADWAY OVER 18 FEET__________ $M = \left(L - 9 + \frac{2.25}{L}\right) R$

$$\text{*WHEEL LINES/TRUSS: } \begin{cases} \text{ONE LANE LOADING} = \left(1 + \frac{W-9}{C}\right) \\ \text{TWO LANE LOADING} = \left(1 + \frac{W-18}{C}\right)2 \end{cases}$$

Where:

M = Moment in Transverse Beam
R = Reaction (Tabular Value)
L = Span of Transverse Beam
W = Width of Roadway
C = Spacing, Ctr to Ctr of Trusses

All values based on standard truck loadings.
*Based on 9 ft. lane width.

APPENDIX A5

STRINGER LIVE LOAD REACTIONS ON TRANSVERSE FLOOR BEAMS & CAPS (END TRANSVERSE BEAMS) (Simple Span Only)

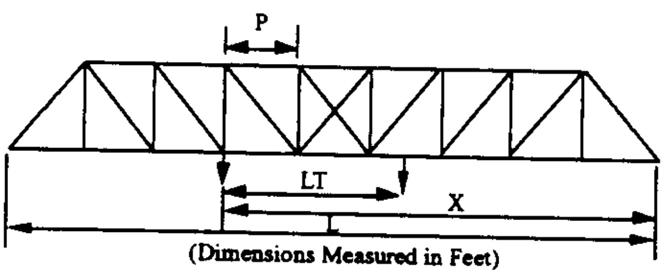
STRINGER SPAN FEET	LIVE LOAD REACTIONS (R) IN KIPS PER WHEEL LINE NO IMPACT				
	TYPE OF LOADING				
	TYPE 3	TYPE 3-S2	TYPE 3-3	H-15	HS-20
10	13.6	12.4	11.2	12.0	16.0
11	13.9	12.7	11.5	12.0	16.0
12	14.2	12.9	11.7	12.0	16.0
13	14.4	13.1	11.9	12.0	16.0
14	14.6	13.3	12.0	12.0	16.0
15	14.7	13.4	12.1	12.2	17.1
16	14.9	13.9	12.3	12.4	18.0
17	15.0	14.3	12.4	12.5	18.9
18	15.1	14.6	12.4	12.7	19.6
19	15.2	14.9	12.5	12.8	20.2
20	15.7	15.2	12.6	12.9	20.8
21	16.1	15.5	13.1	13.0	21.3
22	16.6	15.7	13.5	13.1	21.8
23	16.9	15.9	13.8	13.2	22.2
24	17.3	16.1	14.2	13.3	22.6
25	17.6	16.3	14.5	13.4	23.0
26	17.9	16.4	14.8	13.4	23.4
27	18.1	16.6	15.0	13.5	23.7
28	18.4	16.7	15.3	13.5	24.0
29	18.6	16.8	15.5	13.6	24.4
30	18.8	17.0	15.7	13.6	24.8

All values based on standard truck loadings.

APPENDIX A6

FORMULAE FOR MAXIMUM SHEAR*
AT ANY PANEL POINT
(No Impact Included)
(Simple Span Only)

Type** Load	LT	Min. "X"	Formula	Use for Truss with No. Panels	(1)	(2)
3	19'	19'	$V = \frac{25(X-7.44)}{L}$	All	3	Rt
3-S2	41'	41'	$V = \frac{36(X-18.61)}{L}$	5 or more	5	Rt
		30'	$V = \frac{36(X-11.39)}{L} - \frac{55}{P}$	3,4	2	Lt
		26'	$V = \frac{36(X-7.39)}{L} - \frac{106}{P}$	2	3	Lt
3-3	54'	54'	$V = \frac{40(X-23.9)}{L}$	6 or more	6	Rt
		50'	$V = \frac{40(X-19.9)}{L} - \frac{28}{P}$	4,5	5	Rt
		35'	$V = \frac{40(X-11.1)}{L} - \frac{138}{P}$	3	3	Lt
		34'	$V = \frac{40(X-3.9)}{L} - \frac{252}{P}$	2	4	Rt



(Dimensions Measured in Feet)

L = Length of Truss
LT = Length of Truck
P = Length of Panel
X = Distance from panel point to end of truss
V = Shear at panel point in kips per wheel line.

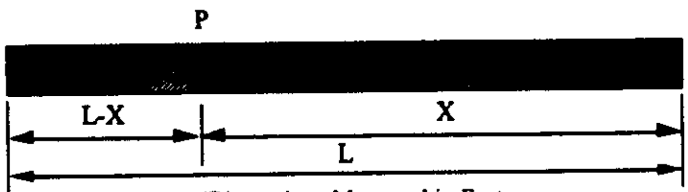
(1) Axle No. @ Panel point
(2) Truck facing

* Applicable when entire truck is on span.
** See Appendix A8 for shear resulting from H and HS load types.

APPENDIX A7

FORMULAE FOR MAXIMUM SHEAR AT ANY POINT ON SPAN
(No Impact Included)
(Simple Spans Only)

Type Load	$\frac{L-X}{L}$	Formula for Maximum Shear (1)	Length of Truck	Minimum	
				L-X	X(2)
3	0-0.500	$V = \frac{25(X-7.44)}{L}$	19'	0	19'
3-S2	0-0.500	$V = \frac{36(X-18.61)}{L}$	41'	0	41'
3-3	0-0.500	$V = \frac{40(X-23.90)}{L}$	54'	0	54'



(Dimensions Measured in Feet

V = Shear at a point "P" which is (L-X) distance from end of span in kips per wheel line.

(1) These formulae are applicable only when dimension "X" exceeds total length of truck.

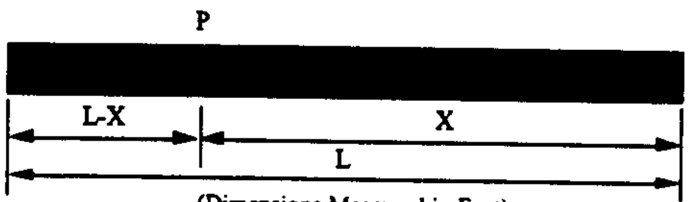
(2) For spans where dimension "X" is less than the minimum, the maximum shears are to be determined from statics.

APPENDIX A8

FORMULAE FOR MAXIMUM SHEAR AT ANY POINT ON SPAN
(No Impact Included)
(Simple Spans Only)

Type Load	$\frac{L-X}{L}$	Use for Girder Lengths	Formula for Maximum Shear (1)	Minimum L-X	Minimum X
HS-20	0-0.500	Under 42'	$V = \frac{36(X-4.67)}{L} - 4$	14	14
		42' to 120'*	$V = \frac{36(X-9.33)}{L}$	0	28
HS-15	0-0.500	Under 42'	$V = \frac{27(X-4.67)}{L} - 3$	14	14
		42' to 120'*	$V = \frac{27(X-9.33)}{L}$	0	28
H-20	0-0.500	to 35'*	$V = \frac{20(X-2.8)}{L}$	0	14
H-15	0-0.500	to 35'*	$V = \frac{15(X-2.8)}{L}$	0	14

(1) All values based on standard truck loadings.
* Truck loading does *not* govern shear beyond the lengths specified. Use lane loading.



(Dimensions Measured in Feet)

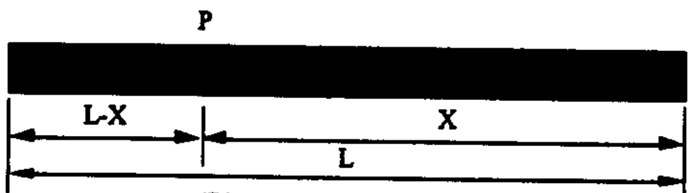
V = Shear to Left of point "P" in kips per wheel line.

APPENDIX A9

FORMULAE FOR MAXIMUM MOMENT AT ANY POINT ON SPAN
(No Impact Included)
(Simple Spans Only)

Type Load	$\frac{L-X}{L}$	Formula for Maximum Moment at "P"	Minimum L-X	Minimum X	(1)	(2)
3	0-0.340	$25\,(X-7.44)\frac{(L-X)}{L}$	0	19.0	3	Rt
	0.340 - 0.500	$25(X-3.44)\frac{(L-X)}{L} - 34$	4.0	15.0	2	Rt
3-S2	0-0.211	$36\,(X-18.61)\frac{(L-X)}{L}$	0	41.0	5	Rt
	0.211-0.354	$36\,(X-11.39)\frac{(L-X)}{L} - 55$	11.0	30.0	2	Lt
	0.354-0.500	$36\,(X-7.39)\frac{(L-X)}{L} - 106$	15.0	26.0	3	Lt
3-3	0-0.175	$40\,(X-23.9)\frac{(L-X)}{L}$	0	54.0	6	Rt
	0.175-0.3125	$40\,(X-19.9)\frac{(L-X)}{L} - 28$	4.0	50.0	5	Rt
	0.3125-0.396	$40\,(X-11.10)\frac{(L-X)}{L} - 138$	19.0	35.0	3	Lt
	0.396-0.500	$40\,(X-3.9)\frac{(L-X)}{L} - 252$	20.0	34.0	4	Rt

(1) Axle No. @ P
(2) Truck facing



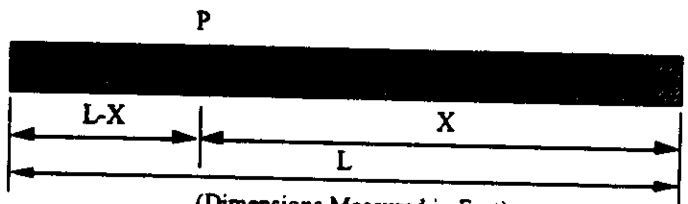
(Dimensions Measured in Feet)

Moments in ft-kips per wheel line at a distance (L-X) from end of span.
Formulae are applicable when entire truck is on span.

APPENDIX A10

FORMULAE FOR MAXIMUM MOMENT AT ANY POINT ON SPAN
(No Impact Included)
(Simple Spans Only)

Type Load	$\frac{L-X}{L}$	Formula for Maximum Moment at "P"	Minimum L-X	Minimum X	Max L*
HS-20	0-0.333	$\frac{36(L-X)(X-9.33)}{L}$	0	28	
	0.333 - 0.500	$\frac{36(L-X)(X-4.67)}{L} - 56$	14	14	144.5
HS-15	0-0.333	$\frac{27(L-X)(X-9.33)}{L}$	0	28	
	0.333-0.500	$\frac{27(L-X)(X-4.67)}{L} - 42$	14	14	144.5
H-20	0-0.500	$\frac{20(L-X)(X-2.8)}{L}$	0	14	56
H-15	0-0.500	$\frac{15(L-X)(X-2.8)}{L}$	0	14	56



(Dimensions Measured in Feet)

Moments in ft-kips per wheel line.
These formulae are applicable when all loads are on the span.

* Span lengths greater than this value are controlled by lane loading.

APPENDIX A11

FORMULAS FOR STEEL COLUMNS*

The allowable combined stresses for steel compression members may be calculated by the provisions of AASHTO Design Specifications or from the following relationship. The permissible average unit stress for steel columns shall be:

** $$f_s = \frac{\frac{f_y}{\eta}}{1+\left(.25 + \frac{e_g c}{r^2}\right) B \text{ Cosec } \Phi} = \frac{P}{A} \quad \text{(A)}$$

P = load parallel to the axis of the member in lbs.
A = gross cross-sectional area of column in sq. in.
f_y = yield point or yield strength (See Tables 6.6.2.1-1 and 6.6.2.1-2)
η = factor of safety based on yield point or yield strength
= 1.82 at Inventory Level
= 1.48 at Operating Level
c = distance from neutral axis to the extreme fiber in compression
r = radius of gyration in the plane of bending
$\Phi = \frac{L}{r}\sqrt{\frac{\eta f_s}{E}}$ radians
L = effective length of the column
= 75% of the total length of a column having riveted end connections
= 87.5% of the total length of a column having pinned end connections
E = modulus of elasticity of steel
= 29,000,000 lbs. per sq. in.

$$B = \sqrt{\alpha^2 - 2\alpha \cos \Phi + 1}$$

$$\alpha = \frac{\frac{e_s c}{r^2} + 0.25}{\frac{e_g c}{r^2} + 0.25}$$

When e_g and e_s lie on the same side of the column axis, α is positive; when on opposite sides, α is negative.

e_g = eccentricity of applied load at the end of column having the greater computed moment, in inches.

e_s = eccentricity at opposite end.

For values of $\frac{L}{r}$ equal to or less than:

$$(\cos^{-1} \alpha) \left[\frac{E\left(1+.25 + \frac{e_g c}{r^2}\right)}{f_y}\right]^{1/2} \quad \text{(B)}$$

the permissible f_s shall be determined from the formula:

$$f_s = \frac{\frac{f_y}{\eta}}{1 + .25 + \frac{e_g c}{r^2}} \quad \text{(C)}$$

For $\alpha = -1$ with values of $\frac{L}{r}$ greater than determined by formula B, the permissible f_s shall be determined by the Euler formula:

$$f_s = \frac{\pi^2 E}{\eta\left(\frac{L}{r}\right)^2} \quad \text{(D)}$$

When the values of end moments are not computed but considered negligible in amount, α shall be assumed equal to +1.

α shall be assumed equal to +1 for a member subject to bending stresses induced by the components of externally applied loads acting perpendicular to its axis. For this case the general formula becomes:

$$f_s = \frac{\frac{f_y}{\eta} - \frac{Mc}{I}}{1 + \left[.25 + (e_g + d)\frac{c}{r^2}\right] \text{Sec } 1/2\ \Phi} \quad \text{(E)}$$

d = deflection due to the transverse components of externally applied loads, in inches
I = moment of inertia of section about an axis perpendicular to the plane of bending, in inches^4
M = moment due to the transverse components of externally applied load, in inch pounds

Note: The value of 0.25 in the above formulae provides for inherent crookedness and unknown eccentricity.

* Refer also to the column formulas given in Tables 6.6.2.1-1 and 6.6.2.1-2.

** When the radius of gyration perpendicular to the plane of bending is less than "r", the column shall be investigated for the case of a long column concentrically loaded, having a greater value of L/r.

APPENDIX B
ILLUSTRATIVE EXAMPLES

Several load rating examples are illustrated in this Appendix. Included are the following:

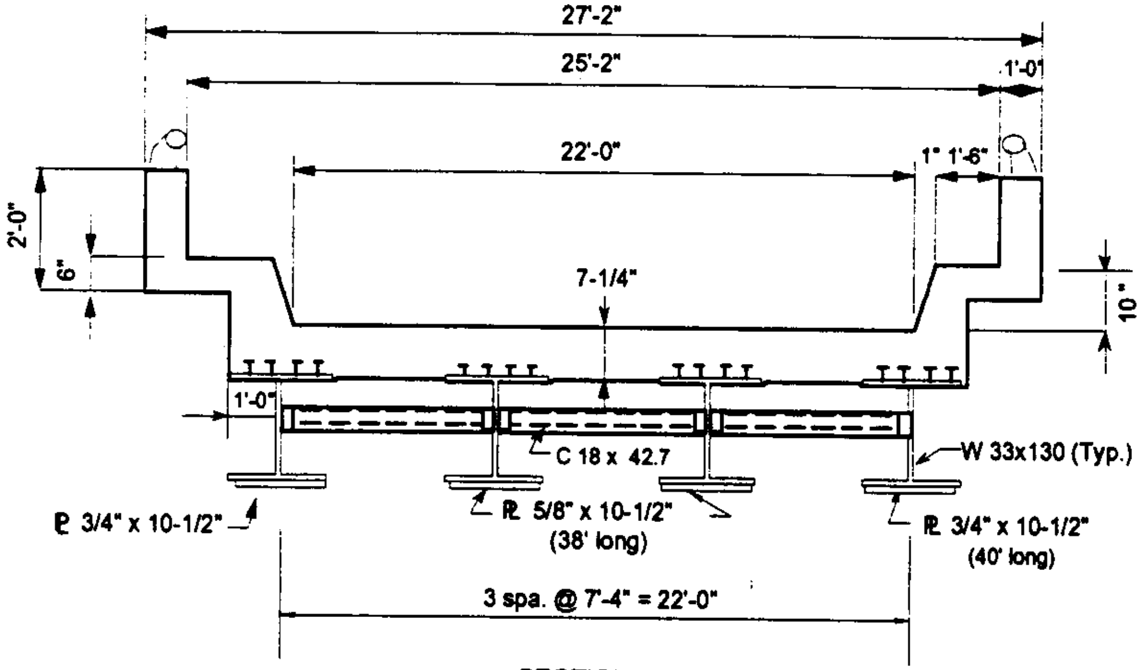
B1 - Simple Span, Interior Steel Stringer with a Composite Deck.
B2 - Simple Span, Reinforced Concrete T-Beam.
B3 - Timber Stringer

The examples represent typical bridge members. Each of the rating methods, including the Load and Resistance Factor rating, is illustrated. The examples are not complete since the rating of connections and investigation of shear and bearing are generally not considered.

In the examples which follow, "AASHTO" refers to the AASHTO "Standard Specifications for Highway Bridges," "MANUAL" refers to the proposed "Manual for Condition Evaluation of Bridges," and "Guide" refers to the AASHTO "Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges."

EXAMPLE B1: Composite Steel Stringer (Adapted from West Virginia Department of Highways)

Given: A 65' long, simple span highway bridge as shown below.



SECTION
No Scale

Materials: A36 Steel - $F_y = 36$ ksi
$f'_c = 3{,}000$ psi

Year Built : 1964
Redundant (multi-Stringer)

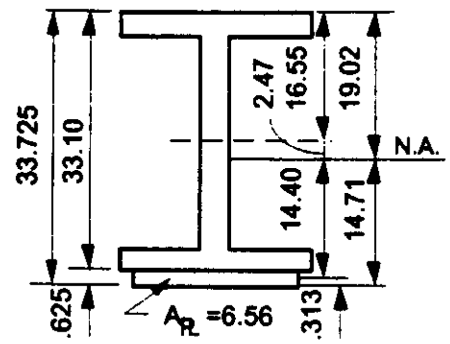
Conditions at Site of Bridge:

ADTT > 1000 with good enforcement.
Maintenance is good and no deterioration was noted.
The approaches and wearing surface are smooth and in good condition.
Inspections are routinely performed.

Rate a typical <u>Interior</u> Stringer

Section Properties: In unshored construction, the steel stringer must support its own weight plus the weight of the concrete slab. For the composite section, the concrete is transformed into an equivalent area of steel by dividing the area of the slab by the modular ratio. Live load plus impact stresses are carried by the composite section using a modular ratio of n. To account for the effect of creep, superimposed dead load stresses are carried by the composite section using a modular ratio of 3n. (AASHTO 10.38.1). The as-built section properties are used in this analysis.

Noncomposite: W33 x 130 & PL 5/8" x 10-1/2"
$t_f = 0.855"$; $b_f = 11.51"$; $t_w = 0.58"$
$A = 38.26 \text{ in}^2$



$$\bar{y} = \frac{(17.175)^{W}(38.26) + (.313)^{PL}(6.56)}{38.26 + 6.56}$$

$$\bar{y} = 14.71"$$

$$I_x = \overset{W}{6699} + \overset{W}{38.26(2.47)^2} + \overset{PL}{6.56(14.40)^2} = 8293 \text{ in}^4$$

$$S_t = \frac{8293}{19.02} = 436.0 \text{ in}^3 = S_t^{DL}$$

$$S_b = \frac{8293}{14.71} = 563.7 \text{ in}^3 = S_b^{DL}$$

Composite Section Properties:

Effective Flange Width: (AASHTO 10.38.3.1)

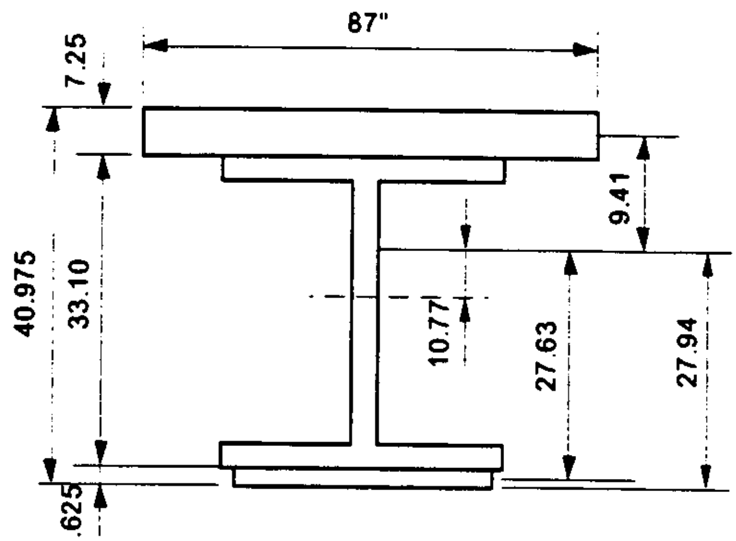
1/4(65)(12)	= 195"
(7.33)(12)	= 88"
(7.25)(12)	= 87" ⇐ Controls

Modular Ratio (n): (MANUAL 6.6.2.4)
for f'_c = 3,000 psi - n = 10

Composite Section Properties cont.:

Typical Interior Stringer:

Composite n = n: W33 x 130, PL 5/8" x 10-1/2" & Conc. 7-1/4" x 87"



$$\bar{y} = \frac{\overset{W}{(17.175)(38.26)} + \overset{PL}{(.313)(6.56)} + \overset{Conc.}{(87 \times 7.25 \div 10)(37.35)}}{38.26 + 6.56 + (87 \times 7.25) \div 10}$$

$$\bar{y} = 27.94"$$

$$I_x = \overset{W}{6699} + \overset{W}{(38.26)(10.77)^2} + \overset{PL}{(6.56)(27.63)^2} + \overset{Conc.}{\frac{(87 \div 10)(7.25)^3}{12}} + \overset{Conc.}{(87 \times 7.25) \div 10\ (9.41)^2}$$

$$= 22007 \text{ in}^4$$

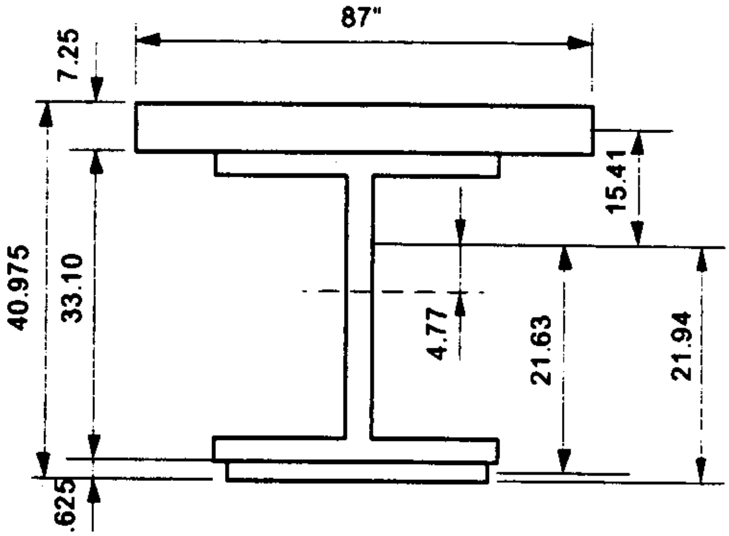
$$S_t = \frac{22007}{5.79} = 3801 \text{ in}^3$$ Section modulus at top of steel

$$S_b = \frac{22007}{27.94} = 787.7 \text{ in}^3 = S_b^L$$

use with Live Load

Composite Section Properties cont.:

Composite n = 3n: W33 x 130, ℄ 5/8" x 10-1/2" & Conc. 7-1/4" x 87"



$$\bar{y} = \frac{\overset{W}{(17.175)(38.26)} + \overset{℄}{(.313)(6.56)} + \overset{Conc.}{(87 \times 7.25 \div 30)(37.35)}}{38.26 + 6.56 + (87 \times 7.25) \div 30}$$

$$\bar{y} = 21.94"$$

$$I_x = \overset{W}{6699} + \overset{W}{(38.26)(4.77)^2} + \overset{℄}{(6.56)(21.63)^2} + \overset{Conc.}{\frac{(87 \div 30)(7.25)^3}{12}} + \left(\frac{87 \times 7.25}{30}\right)(15.41)^2$$

$$I_x = 15{,}725 \text{ in}^4$$

$$S_t = \frac{15725}{11.79} = 1333.8 \text{ in}^3 \text{ (Section modulus at top of steel)}$$

$$S_b = \frac{15725}{21.94} = 716.7 \text{ in}^3 = S_b^{SDL}$$

use with Superimposed Dead Load (SDL)

Loads:

Dead Loads (includes an allowance of 6% of steel weight for connections):

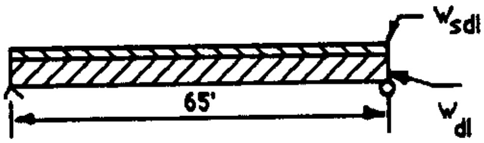
Deck $(7.33)\left(\frac{7.25}{12}\right)(150 \text{ pcf})$	=	664.3 lbs/ft
Stringer (130)(1.06)	=	137.8 lbs/ft
Cover ℄ (.625)(10.5)(490/144)(1.06)(38) ÷ 65	=	13.8 lbs/ft
Diaphragms (3)(42.7)(7.33)(1.06) ÷ 65	=	15.4 lbs/ft
Total per stringer		831.3 lbs/ft

Superimposed Dead Loads: (see AASHTO 3.23.2.3.1.1)

Curb $(1)\left(\frac{10}{12}\right)(150 \text{ pcf}) \div 2$	=	62.5 lbs/ft
Parapet $\left[\left(\frac{6 \times 19}{144}\right) + \left(\frac{18 \times 12}{144}\right)\right](150 \text{ pcf}) \div 2$	=	171.9 lbs/ft
Railing (assume 20 plf) ÷ 2	=	10.0 lbs/ft
Wearing Surface	=	0.0
Total per stringer		244.4 lbs/ft

Live Load: Rate for HS20

Moments:



$w_{sdl} = 0.244$ k/ft $\quad M_{DL} = \frac{w_{dl} L^2}{8} = \frac{.831(65)^2}{8} = 439$ ft-k

$w_{dl} = 0.831$ k/ft $\quad M_{SDL} = \frac{w_{sdl} L^2}{8} = \frac{.244(65)^2}{8} = 129$ ft-k

M_L - From MANUAL, Appendix A3, page 74(1)

Span	M_L
60	403.3
70	492.8

⇐ 65'

$$M_L = \frac{403.3 + 492.8}{2}$$

$M_L = 448$ ft-k

(without Impact, without Dist.)

(1) Note the moments given in MANUAL are for one line of wheels. The values given in AASHTO are for the entire axle and are therefore twice the MANUALvalue.

Allowable Stress Rating (MANUAL 6.4.1, 6.5.2 & 6.6.2)

(Consider Maximum Moment Section only for this example - See general notes.)

Impact - MANUAL 6.7.4. Use standard AASHTO

AASHTO 3.8.3.1

$$I = \frac{50}{L+125} \leq 0.3$$

$$I = \frac{50}{65+125} = 0.26$$

Distribution - MANUAL 6.7.3 indicates that standard AASHTO provisions may be used.

AASHTO 3.23.2.2 and Table 3.23.1

$$DF = \frac{S_s}{5.5} = \frac{7.33^{FT}}{5.5} = 1.33$$

Thus:

$$M_{L+I} = M_L(1+I) * DF = 448(1+0.26)(1.33)$$

$$M_{L+I} = 751 \text{ ft-k}$$

Inventory Level: MANUAL 6.6.2.1, Table 6.6.2.1-1 (bottom steel in tension controls)

For steel with $F_y = 36$ ksi $\rightarrow f_I = 0.55\ f_y$

Thus:

$$f_I = 0.55(36) = 20 \text{ ksi}$$

The Resisting Capacity $(M_{RI}) = f_I\ S_x^L$

$$M_{RI} = 20 \text{ ksi } (787.7 \text{ in.})^3 = 15754 \text{ in-k} = 1313 \text{ ft-k}$$

Then:

$$RF_I = \frac{M_{RI} - M_{DL}\dfrac{S_b^L}{S_b^{DL}} - M_{SDL}\dfrac{S_b^L}{S_b^{SDL}}}{M_{L+I}}$$

$$= \frac{1313 - 439\dfrac{787.7}{563.7} - 129\dfrac{787.7}{716.7}}{751} = \frac{557.8}{751}$$

$$= \underline{\underline{0.74}} \text{ or } 0.74 \times 36 \text{ tons} = \underline{\underline{26.7 \text{ tons}}}$$

Alternatively, in terms of stress:

$$RF_I = \frac{f_s - \dfrac{M_{DL}}{S_b^{DL}} - \dfrac{M_{SDL}}{S_b^{SDL}}}{\dfrac{M_{L+I}}{S_b^{L+I}}}$$

$$= \frac{20\text{ ksi} - \dfrac{439^{\text{ft-k}} \times 12^{\text{in/ft}}}{563.7\text{ in}^3} - \dfrac{129^{\text{ft-k}} \times 12^{\text{in/ft}}}{716.7\text{ in}^3}}{\dfrac{751^{\text{ft-k}} \times 12^{\text{in/ft}}}{787.7\text{ in}^3}}$$

$$= \frac{20 - 9.345 - 2.160}{11.441}$$

$$= \frac{8.495}{11.441} = 0.74 \text{ as above}$$

Operating Level: MANUAL 6.6.2.1, Table 6.6.2.1-2

For steel with $F_y = 36$ ksi $\rightarrow f_o = 0.75\ f_y$

Thus:

$$f_o = 0.75(36) = 27 \text{ ksi}$$

and

$$M_{RO} = 27(787.7) = 21268 \text{ in-k} = 1772 \text{ ft-k}$$

and

$$RF_o = \frac{1772 - 439\dfrac{787.7}{563.7} - 129\dfrac{787.7}{716.7}}{751} = \frac{1016.8}{751}$$

$$RF_o = \underline{\underline{1.35}} \text{ or } 1.35 \times 36 \text{ tons} = \underline{\underline{48.7 \text{ tons}}}$$

Load Factor Rating: (MANUAL 6.4.2, 6.5.3 & 6.6.3)

(Consider maximum moment section only for this example - see General Notes.)

Impact - MANUAL 6.7.4 - use standard AASHTO

From AS Rating I = 0.26

Distribution - MANUAL 6.7.3 - use standard AASHTO

From AS Rating DF = 1.33

$$M_{L+I} = M_L(1+I)\ DF = 448(1+0.26)(1.33)$$

$$= 751 \text{ ft-k (as for AS Rating)}$$

Capacity of Section (M_R) - MANUAL 6.6.3.1

For braced, compact, composite sections:

$$M_R = M_u \text{ (AASHTO 10.50.1.1)}$$

where M_u is found in accordance with applicable load factor provisions of AASHTO.

Check assumptions:

(1) Section is fully braced along top flange by composite deck (for Live Load & SDL)

(2) To check if section is compact, need to apply provisions of AASHTO 10.50. These checks follow.

Checks for Compactness (AASHTO 10.50)

Eqn. 10-93: $\frac{D}{t_w} \leq \frac{19230}{\sqrt{f_y}}$ $D = d - 2t_f = 33.10 - 2(0.855)$

$D = 31.39"$

$$\frac{31.39}{0.58} = 54 \leq \frac{19230}{\sqrt{36000}} = 101$$

54 < 101 OK

Eqn. 10-121: $\frac{b'}{t_f} \leq \frac{2200}{\sqrt{1.3\ f_{dL}}}$ $b' = \frac{b_f - t_w}{2} = \frac{11.51 - 0.58}{2}$

$b' = 5.465"$

$$\frac{5.465}{0.855} \leq \frac{2200}{\sqrt{1.3 \times 12100}}$$

$f_{dL} = \frac{M_{DL}}{S_t^{DL}} = \frac{439 \text{ ft-k} \times 12 \text{ in-ft}}{436 \text{ in}^3}$

$f_{dL} = 12.1$ ksi $= 12{,}100$ psi

6.39 ≤ 17.54 OK

NOTE: The provisions of AASHTO 10.50.1.1.2 will be checked later.

At this point section is braced and compact. Find capacity.

Eqn: 10-122: $C_{CONC} = 0.85\ f'_c\ b_{eff}\ t_s = 0.85(3 \text{ ksi})(87 \text{ in})(7.25 \text{ in}) = 1608 \text{ k}^*$

Eqn: 10-123: $C_{STL} = A_s\ f_y = (38.26 \text{ in}^2 + 6.56 \text{ in}^2)(36 \text{ ksi}) = 1613.5 \text{ k}$

$C_{CONC} < C_{STL}\ \therefore C_{CONC} = 1608$ controls (10.50.1.1.1(a))

Capacity - per AASHTO 10.50.1.1.1(c)

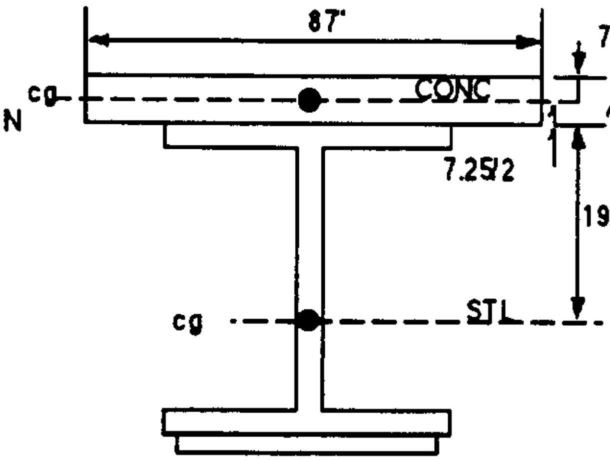
Eqn: 10-126: $C' = \frac{\Sigma (AF_y) - C}{2} = \frac{1613.5 - 1608}{2} = 2.75 \text{ k}$

and applying AASHTO 10.50.1.1.1(d) -

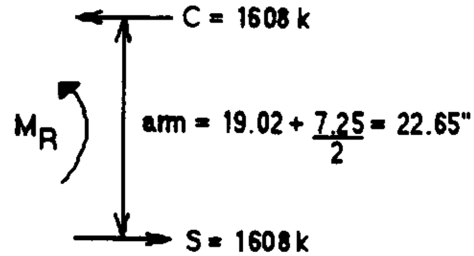
$(AF_y)_{TF} = (11.51 \times .855)(36) = 354 \text{ k} >>> 2.75 \text{ k}\ \therefore$ NA in top flange

Eqn: 10-127: $\bar{y} = \frac{C'}{(AF_y)_{TF}} t_{TF} = \frac{2.75}{354}(.855) = 0.007$ in. neglect. Say NA at top of steel.

* Neglects reinforcement in slab.



X-SECTION



FORCES

M_R = C*arm = S*arm = 1608(22.65) = 36421 in-k = 3035 ft-k

Now check provisions of 10.50.1.1.2

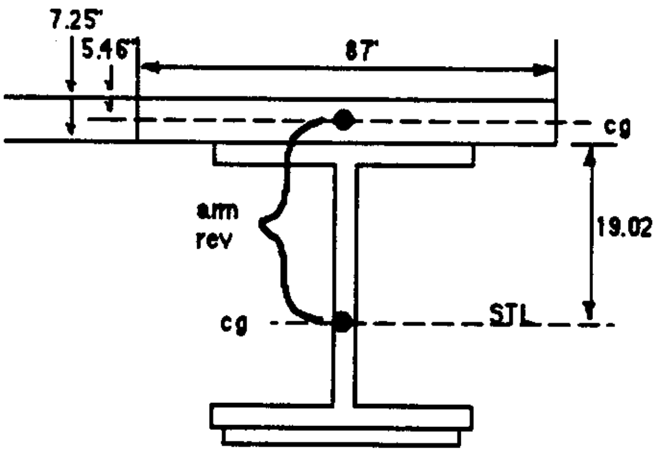
Eqn: 10-128a: $a' \leq \frac{d + t_s}{7.5} = \frac{33.725 + 7.25}{7.5} = 5.46" < 7.25$

Thus this provision governs capacity of section

∴ Non Compact: $M_u = F_y S$

$$a = a' = 5.46$$

$$C = 0.85\, f'_c\, a\, b_{eff} = 0.85(3 \text{ ksi})(5.46 \text{ in})(87 \text{ in}) = 1211 \text{ k}$$



$C_{REVISED}$ = 1211 k

$arm_{rev} = 19.02 + \left(7.25 - \frac{5.46}{2}\right) = 23.54"$

$M_R = C_{rev} * arm_{rev}$ = 1211 k (23.54 in)
= 28507 in-k = 2375 ft-k

$M_R = F_y\, S = 36 * \frac{787.7}{12}$
= 2363 ft-k ⇐ Controls

Inventory Level: MANUAL 6.5.1 and 6.6.3

$$RF_I^{LF} = \frac{M_R - A_1 M_D}{A_2 M_{L+I}} \quad \text{(MANUAL Eqn: 6-1a)}$$

where: (MANUAL 6.5.3)

$A_1 = 1.3$
$A_2 = 2.17$

Thus:

$$RF_I^{LF} = \frac{(2363) - 1.3(439 + 129)}{2.17(751)}$$

$$RF_I^{LF} = \underline{\underline{1.00}} \text{ or } 1.00 \times 36 \text{ tons} = \underline{\underline{36 \text{ tons}}}$$

Operating Level: MANUAL 6.5.3

Only change is $A_2 = 1.3$

Thus:

$$RF_O^{LF} = \frac{2.17}{1.3} RF_I^{LF} = \frac{2.17}{1.3}(1.00)$$

$$RF_O^{LF} = \underline{\underline{1.67}} \text{ or } 1.67 \times 36 \text{ tons} = \underline{\underline{60 \text{ tons}}}$$

Check Serviceability Criteria - AASHTO 10.57.2

At Inventory Level (bottom steel in tension controls)

$$M_D + 1.67\left(RF_I^{LF}\right)(M_{L+I}) \leq \text{Serv. Strength} = S_b^L (.95\ F_y)$$

Thus:

$$RF_I^{LF} = \frac{.95\,(F_y)\,S_b^L - M_D \dfrac{S_b^L}{S_b^{DL}} - M_{SDL} \dfrac{S_b^L}{S_b^{SDL}}}{1.67\,(M_{L+I})}$$

$$= \frac{.95(36 \text{ ksi}) \dfrac{787.7 \text{ in}^3}{12 \text{ in/ft}} - 439 \dfrac{787.7}{563.7} - 129 \dfrac{787.7}{716.7}}{1.67(751)}$$

$$RF_I^{LF} = \underline{\underline{1.19}} \text{ or } 1.19 \times 36 \text{ tons} = \underline{\underline{42.8 \text{ tons}}}$$

At Operating Level

$$M_D + RF_O^{LF}(M_{L+I}) \leq \text{Serv. Strength}$$

Thus:

$$RF_O^{LF} = RF_I^{LF} \times 1.67 = 1.19 \times 1.67$$

$$RF_O^{LF} = \underline{\underline{1.98}} \text{ or } 1.98 \times 36 \text{ tons} = \underline{\underline{71.3 \text{ tons}}}$$

Load Factor Summary

	RF	TONS	CONTROLLED
Inventory	1.00	36	Eqn. 10-128a
Operating	1.67	60	Eqn. 10-128a

Load and Resistance Factor Rating (See AASHTO *Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges*.)

(Again consider maximum moment section only for this example - see General Notes.)

Impact - Guide 3.3.2.3; may vary based on condition of wearing surface. But for comparison purposes use same I as for AS method:

$$I = 0.26$$

Distribution - Guide 3.3.3 use standard AASHTO

$$DF = 1.33$$

Live Load - Guide 3.3.2.2 use HS20 to be consistent with other rating methods. Normally would use rating vehicles (Guide Fig. 2) or lane loading (Guide Fig. 3).

Thus:

$$M_{L+I} = M_R(1+I) \times DF = 448 \times (1+0.26)(1.33)$$

$$= 751 \text{ ft-k}$$

Capacity of Section: Guide 3.3.2.4

M_R is based on AASHTO 10.50 as for Load Factor

Thus: $M_R = 2363$ ft-k (page B.9)

$$RF^{LRF} = \frac{\phi M_R - \gamma_D M_D}{\gamma_L M_{L+I}} \quad \text{(Guide Eqn. 2)}$$

where:

ϕ	=	0.95	(Guide 3.3.4.2, Table 3(b))
γ_D	=	1.2	(Guide, Table 2)
γ_L	=	1.45	(Guide, Table 2)

Load and Resistance Factor Rating cont.

then:

$$RF^{LRF} = \frac{0.95(2363) - 1.2(439 + 129)}{1.45(751)}$$

$$RF^{LRF} = \underline{\underline{1.45}} \text{ or } 1.45 \times 36 \text{ tons} = \underline{\underline{52 \text{ tons}}}$$

SUMMARY OF RESULTS

	RF	HS20 Truck	H20(1) Truck
Allowable Stress:			
Inventory	0.74	26.7	21
Operating	1.35	48.7	38.3
Load Factor:			
Inventory	1.00	36	28.4
Operating	1.67	60	47.4
Load and Resistance Factor	1.45	52	41.2

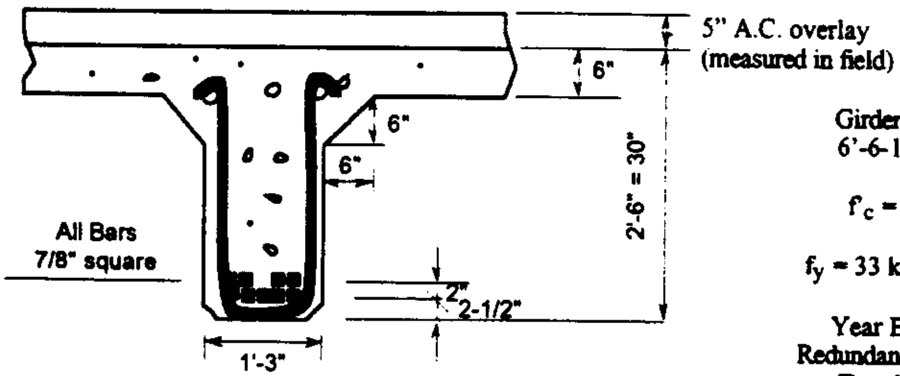
(1) $$H_{TR} = RF \times \frac{M_L^{HS20}}{M_L^{H20}} \times 20^T \qquad M_L^{HS20} = 448 \text{ (page 87)}$$

$$= RF \frac{448}{316} \times 20^T \qquad M_L^{H20} = \frac{20}{15} M_L^{H15} = \frac{20}{15} \frac{(265.1 + 209.2)}{2} = 316 \text{ ft-k}$$

$$H_{TR} = RF \times (1.42) \times 20^T$$

EXAMPLE B2: - Reinforced Concrete Girder

Given: A simple span highway bridge, span 26 ft. Typical interior girder. Cross section:



Girders spaced on
6'-6-1/4" centers

$f'_c = 3{,}000$ psi

$f_y = 33$ ksi (unknown)

Year Built - 1925
Redundant (multi-girder)
Two-lane bridge

Loads:

Dead Loads on interior girder:

Structural Concrete: $0.15\ \text{k/ft}^3 \left[\left(\frac{6"}{12"/\text{ft}} \times 6.52'\right) + (1.25\ \text{ft} \times 2.0\ \text{ft}) + 2\left(\frac{1}{2}\ \frac{6}{12}\ \frac{6}{12}\right)\right]$

$= 0.87$ k/ft

AC Overlay: $0.144\ \text{k/ft}^3 \left(\frac{5"}{12"/\text{ft}} \times 6.52'\right) = 0.39$ k/ft

$W_{dL} = 0.87 + 0.39 = 1.26$ k/ft <u>say 1.3 k/ft</u>

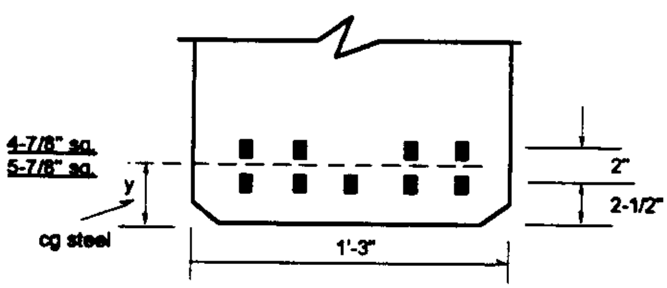
Live Load - Rate for HS20 vehicle. Could use other rating vehicles (Fig. 6.7.2.4).

Conditions at Site of Bridge:

ADTT > 1000 with good enforcement. Maintenance is good and no deterioration noted. Approaches and wearing surfaces are smooth and in good condition. Inspections performed regularly.

Section Properties

Find cg steel



$$y = \frac{4(.766)(2 + 2\text{-}1/2) + 5(.766)(2\text{-}1/2)}{4(.766) + 5(.766)}$$

$y = 3.39"$

$d = 30" - 3.39 = 26.61"$

$A_{1BAR} = 7/8" \times 7/8" = 0.766\ \text{in}^2$

$A_s = 9 \times A_{1BAR} = 6.89\ \text{in}^2$

Effective Slab Width (for T-Girder)

AASHTO 8.10.1.1

$$1/4\ L = \frac{26 \text{ ft x } 12 \text{ in-ft}}{4} = 78"$$

or

$$\text{CC SPCG} = 6' - 6\text{-}1/4" = 78.25"$$

or

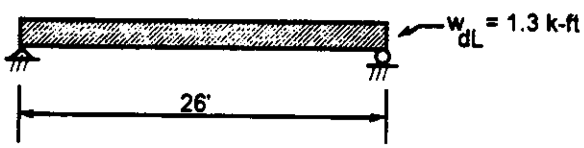
$$12\ t_s = 12 \times 6" = 72" \Leftarrow \text{Controls}$$

$$\rho_{act} = \frac{A_s}{b_{eff}\ d} = \frac{6.89}{72" \times 26.61} = 0.0036$$

(if compression within flange)

Midspan Moments:

Live Load - HS20



$$M_d = \frac{w_{dL}\ L^2}{8} = \frac{1.3 \text{ k/ft x } 26^2 \text{ft}^2}{8} = 109.9 \text{ k-ft}$$

For HS20 - From MANUAL, Appendix A3, page 74[1]. Using Table, select from column "Without Impact"

M_L = 111.1 k-ft (without impact and without distribution)

(1) Note the moments given in the MANUAL are for one line of wheels. The values given in AASHTO are for the entire axle and are therefore twice the MANUAL values.

Allowable Stress Rating (MANUAL 6.4.1, 6.5.2 & 6.6.2)

(For this example we consider only the maximum moment section - see General Notes)

Impact - MANUAL 6.7.4 use standard AASHTO

AASHTO 3.8.2.1

$$I = \frac{50}{L + 125} \leq 0.30$$

$$I = \frac{50}{26 + 125} = 0.33 \text{ use } \underline{\underline{0.30}}$$

Distribution - MANUAL 6.7.3 indicate that standard AASHTO provisions may be used.

AASHTO 3.23.2.2 and Table 3.23.1

$$DF = \frac{S_G}{6.0} \quad \text{Concrete T-Beam}$$

$$DF = \frac{6'\text{— }6\text{-}1/4"}{6.0} = \frac{6.52'}{6} = 1.087$$

Thus:

$$M_{L+I} = M_L\,(1+I)\,(DF) = 111.1(1 + .30)(1.087)$$

$$= 157 \text{ ft-k}$$

Inventory Level: MANUAL 6.5.2 & 6.6.2.4 - The inventory unit stresses are determined in accordance with AASHTO "Service Load Design Method" Article 8.15 or taken from MANUAL 6.6.2.4.

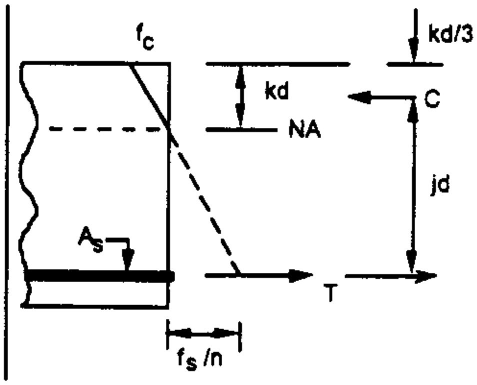
Thus Inventory allowable stresses, AASHTO 8.15.2.1.1

$$f_c^I = 0.4\,f'_c = 0.4\,(3000 \text{ psi}) = 1200 \text{ psi} = 1.2 \text{ ksi}$$

For Reinforcing Steel, MANUAL 6.6.2.3 controls

$$f_s^I = 18000 \text{ psi} = 18 \text{ ksi (unknown steel prior to 1954)}$$

Capacity (Traditional Approach):



Stress & Force Diagram
(nts)

The actual steel and concrete stresses are not known and must be found. Since this is a T-beam, assume neutral axis (na) is within slab. Thus, rectangular beam formulas apply. Check this assumption later.

Position of Neutral Axis:

$$k = \sqrt{2\ \rho n + (\rho n)^2} - \rho n$$

where: $\rho = \frac{As}{bd} = \frac{6.89 \text{ in}^2}{(72 \text{ in})(26.61 \text{ in})}$

$\rho = 0.0036$

$$k = \sqrt{2(.0036)(10) + ((.0036)(10))^2} - (.0036)(10)$$

$n = \frac{E_s}{E_c}$

$n = 10$ (from Article 6.6.2.4)

$k = 0.235$

$$j = 1 - \frac{k}{3} = 1 - \frac{.235}{3} = 0.922$$

Then

Capacity if concrete allowable stress controls—

$$M_c = 1/2\ f_c\, jk\, bd^2$$

$$= 1/2\ (1.2 \text{ ksi})(0.922)(0.235)(72 \text{ in})(26.61 \text{ in})^2$$

$$= 6622.8 \text{ in-k} = 552 \text{ ft-k}$$

Capacity if steel reinforcement allowable stress controls—

$$M_s = A_s\, f_s\, j\, d$$

$$M_s = (6.89 \text{ in}^2)(18 \text{ ksi})(0.922)(26.61 \text{ in})$$

$$M_s = 3042.8 \text{ in-k} = 253 \text{ ft-k} \Leftarrow \text{Controls since } M_s < M_c.$$

Check neutral axis assumption:

k_d = (0.235)(26.61 in) = 6.25" > 6" the slab thickness ∴ NA is below bottom of slab and slightly into web. This could be ignored in this case. However for the sake of completeness, capacity will be figured below based on the NA below the slab and ignoring the compression in the stem concrete.

$$kd = \frac{2nd\,A_s + bt^2}{2n\,A_s + 2bt}$$

$$kd = \frac{2(10)(26.61\text{ in})(6.89\text{ in}) + (72\text{ in})(6\text{ in}^2)}{2(10)(6.89\text{ in}) + 2(72\text{ in})(6\text{ in})} = \frac{6258.9}{1001.8}$$

$$kd = 6.25\text{ in} \rightarrow k = \frac{kd}{d} = \frac{6.25\text{ in}}{26.61\text{ in}} = 0.235$$

$$Z = \left(\frac{3kd - 2t}{2kd - t}\right)\frac{t}{3}$$

$$Z = \left(\frac{3(6.25\text{ in}) - 2(6\text{ in})}{2(6.25\text{ in}) - (6\text{ in})}\right)\frac{6\text{ in}}{3} = \frac{6.75\text{ in}}{6.5\text{ in}}(2\text{ in})$$

$$Z = 2.077\text{ in.}$$

$$j\,d = d - Z$$

$$j\,d = 26.61\text{ in} - 2.077\text{ in} = 24.53\text{ in}$$

$$M_s = A_s\,f_s\,j\,d$$

$$M_s = (6.89\text{ in}^2)(18\text{ ksi})(24.53\text{ in}) = 3042.2\text{ in-k}$$

$$M_s = 253\text{ ft-k} \quad \text{as before}$$

(Note concrete was not checked since capacity of section is limited by steel allowable stress.)

$$RF_I^A = \frac{M_{RI} - M_D}{M_{L+I}} \qquad \text{(MANUAL Eqn. 6-1a)}$$

$$RF_I^A = \frac{253\text{ ft-k} - 109.9\text{ ft-k}}{157\text{ ft-k}} = 0.91$$

<u>Operating Level:</u> MANUAL 6.5.2 & 6.6.2.4

The operating allowable stresses, MANUAL 6.6.2.4 for f'_c = 3,000 psi:

$$f_c^O = 1900\text{ psi} = 1.9\text{ ksi}$$

For Reinforcing Steel, MANUAL 6.6.2.3 controls:

$$f_s^O = 25{,}000 \text{ psi} = 25 \text{ ksi (unknown steel, prior to 1954)}$$

The basic relationships defined previously apply:

Since ρ and n do not change, the neutral axis, k, j and Z terms do not change.

Thus:

$$M_s = A_s f_s j d$$

$$= (6.89 \text{ in}^2)(25 \text{ ksi})(24.53 \text{ in})$$

$$= 4225.3 \text{ in-k} = 352 \text{ ft-k}$$

and checking concrete stress to ensure that concrete does not control

$$f_c = \frac{f_s}{n}\left(\frac{k}{1 - k}\right)$$

$$f_c = \left(\frac{25 \text{ ksi}}{10}\right)\left(\frac{0.235}{1 - .235}\right) = 0.77 \text{ ksi} << 1.9 \text{ ksi allowable}$$

Therefore, capacity of section is controlled by allowable steel stress.

$$M_{R_O} = 352 \text{ ft-k}$$

$$RF_O^A = \frac{M_{R_O} - M_D}{M_{L+I}} = \frac{352 \text{ ft-k} - 109.9 \text{ ft-k}}{157 \text{ ft-k}}$$

$$RF_O^A = 1.54$$

Load Capacity Based on Allowable Stress

Inventory: $0.91 \times 36^T = 32.8^T$ HS

Operating: $1.54 \times 36^T = 55.4^T$ HS

To transform "HS" rating to "H" rating multiply HS rating factor by ratio of "HS" moment to "H" moment:

For 26' span: $M_L^{HS20} = 111.1$ ft-k (see Sheet 97)

and using MANUAL Appendix A3, pg. 74 → $M_L^{H15} = 78$ ft-k

Then

$$M_L^{H20} = \frac{20^T}{15^T} \times 78 \text{ ft-k} = 104 \text{ ft-k}$$

and

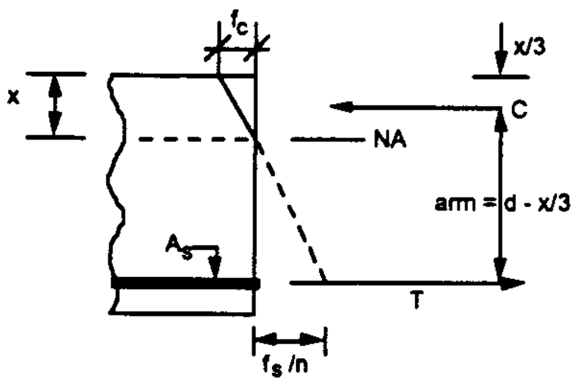
$$\text{Ratio} = \frac{M_L^{HS20}}{M_L^{H20}} = \frac{111.1}{104} = 1.068$$

Thus for H20 Truck:

Inventory: $0.91 \times 1.068 \times 20^T = 19.4^T H$

Operating: $1.54 \times 1.068 \times 20^T = 32.9^T H$

Capacity (Alternate Approach):



Since the location of the neutral axis (NA) and the corresponding stresses in the steel and concrete are not known, these must be determined consistent with the principles of equilibrium of the cross section.

Stress & Force Diagram
(nts)

(1) From the stresses on the cross section:

$$\frac{f_c}{x} = \frac{f_s/n}{d - x} \rightarrow f_c = \frac{f_s}{n}\left(\frac{x}{d - x}\right) \qquad \text{Eqn. 1}$$

(2) Assume the steel allowable stress controls the capacity of the section. This will be checked later. Then

$$T = A_s f_s = (6.89 \text{ in}^2)(18 \text{ ksi}) = 124 \text{ k}$$

and

$$C = 1/2 f_c b x$$

but

$$C = T$$

thus,

$$1/2 f_c b x = A_s f_s$$

$$x = \frac{A_s f_s}{1/2 f_c b} \qquad \text{Eqn. 2}$$

Solve equations 1 and 2 to find location of neutral axis. This may be done by trail and error as follows.

Assume $f_s = 18$ ksi, i.e. steel allowable stress controls.

Try x = 6.0 in. Then by Eqn. 1:

$$f_c = \frac{f_s}{n}\left(\frac{x}{d - x}\right) = \frac{18\text{ ksi}}{10}\left(\frac{6.0\text{ in}}{26.61\text{ in} - 6.0\text{ in}}\right) = 0.524\text{ ksi} < 1.2\text{ ksi} \qquad \text{allowable OK}$$

and by Eqn. 2:

$$x = \frac{A_s\, f_s}{1/2\, f_c\, b} = \frac{(6.89\text{ in}^2)(18\text{ ksi})}{1/2\,(.524\text{ ksi})(72\text{ in})} = 6.57\text{"} > 6.0 \qquad \text{assumed. Try again}$$

Try x = 6.25 in.

$$f_c = \frac{18}{10}\left(\frac{6.25}{26.61 - 6.25}\right) = 0.552 < 1.2\text{ ksi} \qquad \text{allowable OK}$$

and

$$x = \frac{(6.89)(18)}{1/2\,(.552)(72)} = 6.24 \doteq 6.25 \qquad \text{assumed OK}$$

(3) Since x = 6.24 > t = 6.0, NA is below bottom of slab and slightly into web. If web concrete in compression is neglected,

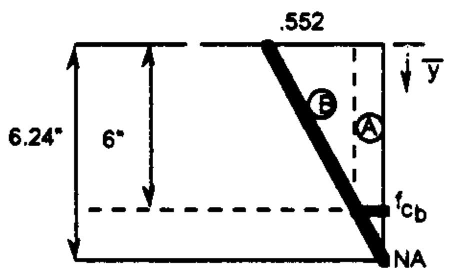
$$\text{arm} \doteq d - \frac{x}{3} \text{ for this example.}$$

$$\text{arm} \doteq 26.61 - \frac{6.24}{3} = 24.53\text{ in}$$

and capacity is

$$M = A_s\, f_s\,(\text{arm}) = (6.89)(18)(24.53) = 3042.2\text{ in-k} = 253\text{ ft-k} \qquad \text{as before.}$$

The exact "arm" may be determined from the concrete stress diagram as follows:



@ bottom of slab

$$f_{c_b} = .552\left(\frac{.24}{6.24}\right) = 0.021$$

Next find centroid of stress diagram from top of slab.

$$\bar{y} = \frac{\Sigma\, A_y}{\Sigma\, A} = \frac{(0.021)(6)(6/2) + (0.552 - 0.021)(6)(1/2)(6/3)}{(0.021)(6) + (0.552 - 0.021)(6)(1/2)}$$

$$\bar{y} = \frac{3.576}{1.722} = 2.08 \text{ in}$$

$\therefore$ arm = 26.61 - 2.08 = 24.53 in as found previously.

(4) The Operating capacity may be found as above and will be the same as for the "traditional method." The rating calculations are not shown here since they too will be the same as for the traditional method.

Load Factor Rating (MANUAL 6.4.2, 6.5.3 & 6.6.3)

(For this example we consider only the maximum moment section - see General Notes.)

Impact - MANUAL 6.7.4 use standard AASHTO

AASHTO 3.8.2.1

$$I = \frac{50}{L + 125} \leq 0.30$$

$$I = \frac{50}{26 + 125} = 0.33 \text{ use } \underline{\underline{0.30}}$$

Distribution - MANUAL 6.7.3 indicate that standard AASHTO provisions may be used.

AASHTO 3.23.2.2 and Table 3.23.1, Concrete T-Beam

$$DF = \frac{S_G}{6.0} = \frac{6.52'}{6} = 1.087$$

Thus:

$$M_{LL+I} = M_L\,(1+I) \times DF = 111.1\,(1+0.30)(1.087)$$

$$= 157 \text{ ft-k}$$

Capacity of Section - MANUAL 6.6.3.2

For unknown steel, prior to 1954 f_y = 33,000 psi = 33 ksi

M_u is found in accordance with applicable strength requirements of AASHTO Article 8.16.

Consider a rectangular section with compression limited to top slab. Then check MANUAL 6.6.3.2 requirement for 75% of balanced condition.

$$\rho_{max} = 0.75\,\rho_{bal} = 0.75\,\frac{0.85\,\beta_1\,f'_c}{f_y}\,\frac{87000}{87000 + f_y} \quad \text{(AASHTO Eqn. 8-18)}$$

$$\rho_{max} = 0.75\,\frac{0.85(.85)(3000)}{33000}\left(\frac{87000}{87000 + 33000}\right)$$

$$\rho_{max} = 0.0357$$

$$\rho_{act} = 0.0036 << \rho_{max} \text{ OK (see Sheet 97)}$$

Then:

$$a = \frac{A_s\, f_y}{0.85\, f'_c\, b_{eff}} \qquad \text{(AASHTO Eqn. 8-17)}$$

$$a = \frac{6.89\ \text{in}^2\ (33\ \text{ksi})}{0.85(3\ \text{ksi})\ 72\ \text{in}} = 1.24" < 6" \qquad \text{OK within slab}$$

$$M_R = A_s\, f_y\, (d - a/2) \qquad \text{(AASHTO Eqn. 8-16)}$$

$$M_R = (6.89\ \text{in}^2)(33\ \text{ksi})\left(26.61\ \text{in} - \frac{1.24}{2}\right)$$

$$M_R = 5909\ \text{in-k} = \underline{\underline{492\ \text{ft-k}}}$$

$$M_u = \phi\, M_R \qquad \phi\text{: AASHTO 8.16.1.2.2} \rightarrow \phi = 0.90$$

$$M_u = 0.90 \times 492 = 443\ \text{ft-k}$$

<u>Inventory Level</u>: MANUAL 6.5.1 & 6.6.3

$$R_I^{LF} = \frac{M_u - A_1\, M_D}{A_2 M_{L+I}} \qquad \text{(MANUAL Eqn. 6-1a)}$$

where in accordance with MANUAL 6.5.3

$$A_1 = 1.3$$
$$A_2 = 2.17$$

Thus:

$$RF_I^{LF} = \frac{443 - 1.3\ (109.9)}{2.17(157)} = 0.88$$

<u>Operating Level:</u> MANUAL 6.5.1 & 6.6.3

$$R_O^{LF} = \frac{M_u - A_1\, M_D}{A_2 M_{L+I}} \qquad \text{(MANUAL Eqn. 6-1a)}$$

where in accordance with MANUAL 6.5.3

$$\gamma_D = 1.3$$
$$\gamma_L = 1.3$$

Thus:

$$RF_O^{LF} = \frac{443 - 1.3(109.9)}{1.3(157)} = 1.47$$

Load capacity based on Load Factor Method, HS20 truck

Inventory: $0.88 \times 36^T = 31.7^T$ HS
Operating: $1.47 \times 36^T = 52.9^T$ HS

and

Inventory: $0.88 \times 1.068^x \times 20 = 18.8^T$ H (see Sheet 102)
Operating: $1.47 \times 1.068^x \times 20 = 31.4^T$ H

Load and Resistance Factor Rating (See AASHTO *Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges*.)

(For this example we consider only the maximum moment section - see General Notes.)

Impact - Guide 3.3.2.3

Based on conditions at site (see sheet 96) select:

$$I = 0.1$$

Distribution - Guide 3.3.3 use standard AASHTO with correction factor of 1.0 (Guide Table 1).

AASHTO 3.23.2.2 and Table 3.23.1, Concrete T-Beam

$$DF = \frac{S_G}{6.0} = \frac{6.52}{6} = 1.087$$

Live Load - Guide 3.3.2.2 use HS20 to be consistent with other rating methods. Normally would use rating vehicles (Guide Fig. 2) or lane loading (Guide Fig. 3).

Thus:

$$M_{LL+I} = M_L\,(1+I)\,DF = 111.1(1+0.1)(1.087)$$
$$= 133 \text{ ft-k}$$

Capacity of Section - MANUAL 6.6.3.2

$$M_R = M_N = \frac{M_u}{\phi} \text{ found in accordance with AASHTO Article 8.16}$$

$$M_R = 492 \text{ ft-k (from sheet 105)}$$

Rating Level

$$RF^{LRF} = \frac{\phi\, M_R - A_1\, M_D}{A_2\, M_{LL+I}} \qquad \text{(MANUAL Eqn. 6-1a)}$$

where:

ϕ	=	0.95	(Guide 3.3.4.2, Table 3(b)) From Sheet 96, concrete girder, redundant, good inspection and maintenance and good condition.
A_1	=	1.2	(Guide, Table 2) AC overlay measured in field (from sheet 96)
A_2	=	1.45	(Guide, Table 2) ADTT > 1000 and good enforcement (from sheet 96)

Then:

$$R^{LRF} = \frac{0.95(492) - 1.2(109.9)}{1.45(133)} = 1.74$$

Load Capacity Based on LRF:

$$1.74 \times 36^T = 62.6 \text{ tons HS}$$

and

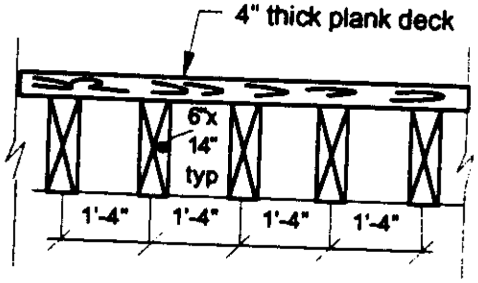
$$1.74 \times 1.068 \times 20^T = 37.2^T\,H \text{ (see sheet 102)}$$

SUMMARY OF RESULTS

Method	RF	HS Truck Max. Load (tons)	H Truck Max. Load (tons)
Allowable Stress:			
Inventory	0.91	32.8	19.4
Operating	1.54	55.4	32.9
Load Factor:			
Inventory	0.88	31.7	18.8
Operating	1.47	52.9	31.4
Load and Resistance Factor	1.74	62.6	37.2

EXAMPLE B3: - Timber Stringer

Given: A simple span, timber stringer bridge with a timber plank deck (two lanes). Span length - 17'-10" c-c bearings (field measured).



Partial X-Section
(nts)

Timber dimensions field measured (actual)

Good maintenance and inspection.

Smooth approaches, fair deck smoothness.

Year Built: 1930
Year Reconstructed: 1967

ADTT << 1000

Timber Species: Southern Pine No. 2

AASHTO Table 13.2.1A for No. 2:

$$\underset{(\text{new})}{F_b = 1200\text{ psi}^*}\ ;\ \underset{(\text{new})}{F_v = 90\text{ psi}}$$

Load rate the 6"x14" stringers:

Dead Loads - Deck $\dfrac{(1'-4")4"}{144\text{ in}^2/\text{ft}^2} \times 50\text{ lbs/ft}^3 = 22.2\text{ lbs/ft}$

Stringer $\dfrac{6"\text{x}14"}{144} \times 50 = \underline{29.2\text{ lb/ft}}$

51.4 lb/ft say 0.055 k/ft

Live Load - Rate for H15 truck

Section Properties: (Again for stringers)

$$I_x = \frac{bh^3}{12} = \frac{6\text{x}14^3}{12} = 1372\text{ in}^4$$

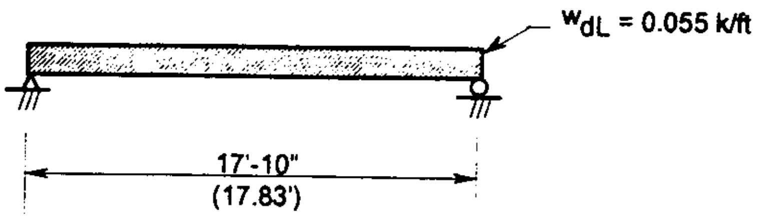
$$S_x = \frac{I_x}{h/2} = \frac{1372}{14/2} = 196\text{ in}^3$$

$$A = bh = 6 \times 14 = 84\text{ in}^2$$

* The provisions of AASHTO 13.3.7.1 should also be applied. For this sample $C_F = 1.0$ was assumed.

Midspan Moments:

For Dead Load -



$$M_D = \frac{w_{dL}\ L^2}{8} = \frac{0.055\ (17.83)^2}{8}$$

$$M_D = 2.19'^k$$

For H15 - From MANUAL Appendix A3, page 74[1]

Span	M_L
17'	$51'^k$
18'	$54'^k$

← For 17.83' span, interpolate

$$M_L = 51 + \frac{17.83 - 17}{18-17}(54-51) = 53.5'^k$$

<u>Allowable Stress Rating</u> (MANUAL 6.4.1, 6.5.2 & 6.6.2)

(Consider stringer only; consider maximum moment and shear sections only for this example - see General Notes.)

Impact - MANUAL 6.7.4 use standard AASHTO

AASHTO 3.8.1.2 - No impact for timber members

$$I = 0$$

Distribution - MANUAL 6.7.3 indicate that standard AASHTO provisions may be used.

AASHTO 3.23.2.2 and Table 3.23.1
For two lanes and plank deck:

$$DF = \frac{S}{3.75} = \frac{16''/12''/ft}{3.75} = 0.36$$

Thus:

$$M_{LL+I} = M_L\,(1+I) \times DF = 53.5'^k \times (1+0) \times 0.36$$

$$M_{LL+I} = 19.26'^k$$

<u>Stresses to be used:</u>

Inventory: MANUAL 6.6.2.7(1) → use AASHTO

(1) Note the moment given in the MANUAL are for one line of wheels. The values given in AASHTO are for the entire axle and are therefore twice the MANUAL values.

For this specie $F_b = 1200$ psi

However since this bridge is more than 10 years old, AASHTO 13.2.4 applies.

$$F_b^{inv} = 0.9(1200)\ C_F = 1080 \text{ psi} = 1.08 \text{ ksi}$$
$$(C_F = 1.0 \text{ (see note sh. 108)})$$

and

$$F_v^{inv} = 90 \times 0.9 = 81 \text{ psi}$$

Operating: MANUAL 6.6.2.7(2)

$$F_b^{op} = F_b^{inv} \times 1.33 \times C_F = 1080 \times 1.33 \times 1.0$$
$$(C_F = 1.0 \text{ (see note sh. 108)})$$

$$F_b^{op} = 1436 \text{ psi say } 1440 \text{ psi} = 1.44 \text{ ksi}$$

and

$$F_v^{op} = 1.33\ F_v^{inv} = 1.33 \times 81 \text{ psi} = 108 \text{ psi}$$

<u>Inventory Level Rating</u>

Capacity: $M_{R_I} = F_b^{inv}\ S_x = 1.08 \text{ ksi} \times 196 \text{ in}^3 = 211.7 \text{ in-k}$

$M_{R_I} = 17.64$ ft-k

then

(MANUAL Eqn. 6-1a) $$RF_I^M = \frac{M_{R_I} - M_D}{M_{L+I}} = \frac{17.64'^k - 2.19'^k}{19.26'^k}$$

$$RF_I^M = \underline{\underline{0.80}} \text{ or } 0.80 \times 15 \text{ tons} = \underline{\underline{12 \text{ tons}}} \text{ H truck}$$

<u>Operating Level Rating</u>

Capacity: $M_{R_O} = F_b^{op}\ S_x = 1.44 \text{ ksi} \times 196 \text{ in}^3 = 282.2 \text{ in-k}$

$M_{R_O} = 23.52'^k$

then

(MANUAL Eqn. 6-1a) $$RF_O^M = \frac{M_{R_O} - M_D}{M_{L+I}} = \frac{23.52'^k - 2.19'^k}{19.26'^k}$$

$$RF_O^M = \underline{\underline{1.11}} \text{ or } 1.11 \times 15 \text{ tons} = \underline{\underline{16.6 \text{ tons}}} \text{ H truck}$$

Check Horizontal Shear

AASHTO 13.3.1 suggests that shear be computed at:

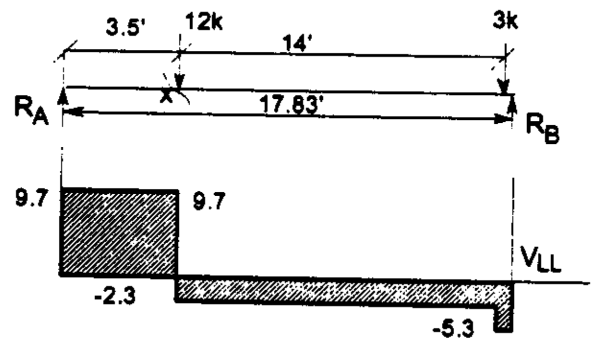
(1) A distance from the support equal to three times the depth of the stringer; or

(2) At the quarter point, whichever is less.

Thus by: (1) $3(14") = 42" \leftarrow$ Controls $= 3.5$ ft

(2) $$\frac{17.83' \times 12"/\text{ft}}{4} = 53.5"$$

For H15 Truck: MANUAL, Appendix A8, pg. 79



$$V_x = \frac{15(x - 2.8)}{L}$$

where $L = 17.83'$ $x = 17.83 - 3.5 = 14.33$

$$V_x = \frac{15(14.33 - 2.8)}{17.83} = 9.7 \text{ k},$$

per wheel line without distribution

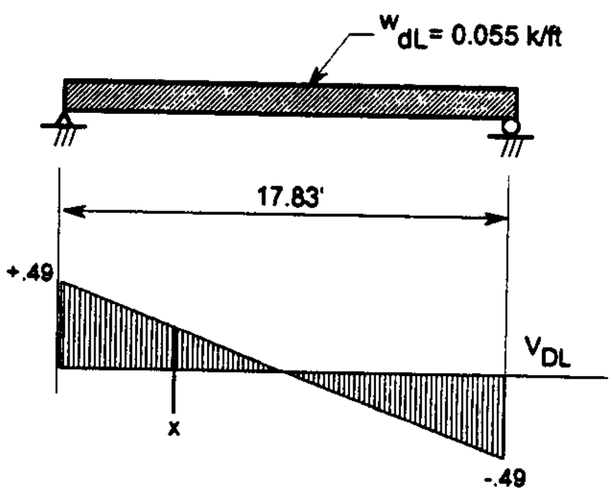
then per AASHTO 13.3.1

$$V_{Lx} = 1/2\left[0.6\, V_x^{L\text{ no dist.}} + DF\, V_x^{L\text{no dist.}}\right]$$

$$V_{Lx} = 1/2\,[0.6\,(9.7) + 0.36\,(9.7)]$$

$$V_{Lx} = 4.7 \text{ k}$$

For $w_{dL} = 0.055$ k/ft



$R_A = R_B = 1/2\, w_{dL}\, L$

$= 1/2\,(0.055) \times 17.83$

$= 0.49$ k

$V_{Dx} = 0.49 - .055 \times 3.5$

$V_{Dx} = 0.3$ k

Rating Based on Shear

Inventory:

Capacity: AASHTO Eqn. 13-1 solve for V_R

$$V_R = \frac{2}{3} bd\, f_v$$

then

$$V_{R_I} = \frac{2}{3}(6)(14)\ 81 \text{ psi} = 4536 \text{ lbs.} = 4.54 \text{ k}$$

(MANUAL Eqn. 6-1a): $RF_I^V = \frac{V_{R_I} - V_{D_x}}{V_{L_x}} = \frac{4.54 \text{ k} - 0.3 \text{ k}}{4.7 \text{ k}}$

$$RF_I^V = \underline{\underline{0.90}} \text{ or } 0.90 \times 15 \text{ tons} = \underline{\underline{13.5 \text{ tons}}} \text{ H truck}$$

Operating:

Capacity:

$$V_{R_O} = \frac{2}{3}(6)(14)(108 \text{ psi}) = 6048 \text{ lbs.} = 6.05 \text{ k}$$

(MANUAL Eqn. 6-1a): $RF_O^V = \frac{V_{R_O} - V_{D_x}}{V_{L_x}} = \frac{6.05 \text{ k} - 0.3 \text{ k}}{4.7 \text{ k}}$

$$RF_O^V = \underline{\underline{1.22}} \text{ or } 1.22 \times 15 \text{ tons} = \underline{\underline{18.5 \text{ tons}}} \text{ H truck}$$

Load Factor Rating

Not currently available for timber.

Load and Resistance Factor Rating

Not currently available for timber.

SUMMARY OF RESULTS

Method/Force	RF	H Truck Max. Load (tons)
Allowable Stress		
Moment:		
Inventory	0.80	12.0
Operating	1.11	16.6
Allowable Stress		
Shear:		
Inventory	0.90	13.5
Operating	1.22	18.3

∴ Rating governed by moment rather than shear

APPENDIX C
FORMULAS FOR THE CAPACITY (C) OF TYPICAL BRIDGE COMPONENTS BASED ON THE LOAD FACTOR METHOD

C.1 GENERAL

When using the Load Factor Method, the capacity (C) in the basic load rating equation (6-1a) is based on procedures in the AASHTO Standard Specifications for Highway Bridges (AASHTO Design Specifications), 1989 with Interims through 1990. This Appendix summarizes the capacity determination for typical bridge members of steel, reinforced concrete or prestressed concrete. For those conditions not covered in this Appendix, the AASHTO Design Specifications should be used.

The formulas shown below have been taken from the AASHTO Design Specifications. All equation and article numbers cited below refer to this Specification. The notation used in the formulas is as defined in the AASHTO Design Specifications. Formulas are limited to those which would apply to simply-supported spans.

C.2 CAPACITY OF STEEL MEMBERS (PART D, STRENGTH DESIGN METHOD)

C.2.1 SECTIONS IN BENDING

C.2.1.1 Compact, Braced, Non-Composite

$$C = F_y Z \qquad (10\text{-}91)$$

C.2.1.2 Compact, Composite

$$C = M_u$$

where M_u is found in accordance with Article 10.50.1.1

C.2.1.3 Non-Compact, Braced, Non-Composite

$$C = F_y S \qquad (10\text{-}97)$$

C.2.1.4 Non-Compact, Composite

$$C = F_y S \text{(Article 10.50.1.2.1)}$$

Also note that Article 10.50(f) requires that the dead (D) and live (L) load effects be evaluated based on the appropriate section properties.

C.2.1.5 Unbraced, Non-Composite

$$C = M_u$$

where M_u is found in accordance with Article 10.48.4.1

C.2.2 SECTIONS IN SHEAR

$$C = V_u \qquad (10\text{-}112)$$

where V_u is found in accordance with Article 10.48.8.1

C.2.3 SECTIONS IN SHEAR AND BENDING (ARTICLE 10.48.8.2)

If $A_1 D + A_2 L (1+I) > 0.75\ M_u$

Then $C = V = \left[2.2 - \left(1.6 \frac{M}{M_u}\right)\right] V_u$(10-117)

where M_u is found as described above for sections in bending and V_u is found as for sections in shear.

C.2.4 COMPRESSION MEMBERS

C.2.4.1 Concentrically Loaded Members

$$C = 0.85\ A_s\ F_{CR} \qquad (10\text{-}150)$$

where F_{CR} is found in accordance with Article 10.54.1.1.

C.2.4.2 Combined Axial Load and Bending

Interaction equations (10-155 and 10-156) must be satisfied by factored axial force (P) and factored axial moment (M). See Article 10.54.2.

C.2.5 CAPACITY BASED ON OVERLOAD PROVISIONS OF ARTICLE 10.57

Note $A_1 = 1.0$ and $A_2 = 1.67$ in the basic rating equation (6-1a) when making this check.

C.2.5.1 Non-Composite Beams

$C = 0.8\ F_y\ S$(Article 10.57.1)

C.2.5.2 Composite Beams

$C = 0.95\ F_y\ S$(Article 10.57.2)

Note the composite section properties should be considered in computing the dead (D) and live (L) load effects.

C.3 REINFORCED CONCRETE MEMBERS (ARTICLE 8.16)

C.3.1 SECTIONS IN BENDING

C.3.1.1 Rectangular Sections with Tension Reinforcement Only

$$C = \phi M_n = \phi \left[A_s\ f_y \left(d - \frac{a}{2}\right)\right] (8\text{-}16)$$

where:

$$a = \frac{A_s\ f_y}{0.85\ f'_c\ b} \qquad (8\text{-}17)$$

C.3.1.2 Tee Section (Flanged) With Tension Reinforcement Only

C.3.1.2.1 Compression Zone Within Flange Area

$C = \phi M_n$ as for C.3.1.1 above

C.3.1.2.2 Compression Zone Includes Both Flange Area and a Portion of the Web

$$C = \phi M_n \qquad (8\text{-}19)$$

where M_n is found in accordance with Article 8.16.3.3.2.

C.3.2 SECTIONS IN COMPRESSION

See Article 8.16.4.

C.3.3 SECTIONS IN SHEAR

$$C = V_u \qquad (8\text{-}46)$$

See Article 8.16.6 for the procedure for computing V_u.

C.4 PRESTRESSED CONCRETE MEMBERS (SECTION 9)

C.4.1 SECTIONS IN BENDING

C.4.1.1 Rectangular Sections Without Non-Prestressed Reinforcement

$$C = \phi M_n = \phi \left[A_s^* f_{su}^* d \left(1 - 0.6 \frac{\rho^* f_{su}^*}{f'_c} \right) \right] \qquad (9\text{-}13)$$

C.4.1.2 Tee (Flanged) Sections Without Non-Prestressed Reinforcement

C.4.1.2.1 Compression Zone Within Flange Area

$C = \phi M_n$ as for Recta..gular Sections, C.4.1.1 above

C.4.1.2.2 Compression Zone Includes Flange Area and Part of Web

$$C = \phi M_n \qquad (9\text{-}14)$$

See Article 9.17.3 for the evaluation of this equation.

C.4.2 SECTIONS IN SHEAR

$$C = V_u \qquad (9\text{-}26)$$

V_u should be found in accordance with Article 9.20.

COMMENTARY

1. INTRODUCTION

1.3 APPLICABILITY

At the discretion of the Bridge Owner, the provisions of this Manual may be applied to highway bridge structures regardless of span or total length of bridge.

Federal regulations entitled the "National Bridge Inspection Standards" (NBIS) have been promulgated which establish minimum requirements for inspection programs and minimum qualifications for bridge inspection personnel. NBIS applies to all bridges on public roads which are more than 20 feet in length.

1.4 QUALITY MEASURES

The Quality Control Plan for bridge maintenance inspection and evaluation should contain at least these basic elements:

(1) Level and frequency of reviews for each major activity performed. Procedures should be established for preparing and checking calculations, preparing and checking drawings.

(2) Elements of structures or specific types of structures which require special quality control or emphasis. Approved practices should be described, including the situations in which outside experts (design, construction, materials) should be consulted.

(3) Responsibilities and authorities within the project team and for the entire unit. The routes for approvals and for dispute resolution should be identified. Organization charts and decision trees are helpful.

(4) Documentation requirements: number of copies, routing and filing procedures.

(5) Timetables: types of activities matched to appropriate response and completion time periods.

1.5 BRIDGE MANAGEMENT SYSTEMS (BMS)

NCHRP Report 300, on Bridge Management Systems, defines the basic concepts of such systems as follows.

Bridge management is not "business as usual." It requires a practical, objective, and systematic consideration of the problem with a set of economic and technical tools not previously combined to solve the problem. Specifically, a bridge management system (BMS) is a rational and systematic approach to organizing and carrying out the activities related to planning, designing, constructing, maintaining, rehabilitating, and replacing bridges vital to the transportation infrastructure. A BMS should assist decision-makers to select optimum cost-effective alternatives needed to achieve desired levels of service within the allocated funds and to identify future funding requirements. Bridge management is a relatively new concept that was adapted from successful application of systems concepts to pavement management functions.

A bridge management system provides benefits to administrators, engineers, and managers at all levels within a transportation agency. The concepts can be implemented in many ways, but should include, in some format, the following: a data base; network level maintenance, rehabilitation and replacement selection procedures; maintenance scheduling; capability to analyze historical data; and a system for generating standard and customized reports. In addition, the system should have a capability for future expansion to provide project level recommendations. The data base should contain information essential to the management of an individual bridge or network of bridges.

2. BRIDGE FILE (RECORDS)

2.1 GENERAL

This section covers the records and reports which make up a complete bridge file, including the SI&A Report. The file should be reviewed prior to the conduct of a bridge inspection, rating or evaluation.

The components of a bridge record indicated in Article 2.2 encompass a wide range of information which may not be practical to assemble in one location. Some items could be filed elsewhere and incorporated in the bridge file by appropriate references.

2.3 INVENTORY DATA

The FHWA *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges* includes detailed descriptions of various bridge items to be inventoried. Where possible, the subheadings used in this Manual follow those used in the Coding Guide.

3. INSPECTION

3.1 GENERAL

This section covers methods and equipment used to make bridge inspections, safety of both the inspecting personnel and the traveling public, guidelines for making field measurements, condition rating of bridge components, cleaning procedures, and "critical condition" procedures. The actual inspection procedures themselves have been listed by bridge element, such as substructure, superstructure, and decks, for ease of use by the inspector.

3.2 TYPES

The description of inspection types is based on the AASHTO *Manual for Maintenance Inspection of Bridges* (Appendix B), 1989 Interim, with some clarifications and editorial changes.

For new and existing bridges, the inspection team should determine if there is a need for greater than a routine inspection regimen for any given bridge. Particular attention should be given to details that are outmoded in the original design or have potential fatigue problems. Special inspections are required for any bridge in questionable condition. All bridges which have weight limits less than established by statute may require special inspections. Special and more intense inspections than for ordinary bridges should also be considered for:

- New structure types
- Structures incorporating details which have no performance history
- Structures with potential foundation and scour problems
- Nonredundant structures

3.3 FREQUENCY

Inspection intervals are not limited to a maximum of two years but may be adjusted where past performance justifies such strategies. However, prior approval by FHWA is required if an inspection interval longer than two years is proposed. Guidelines for obtaining FHWA approval are contained in FHWA *Technical Advisory—Revisions to the National Bridge Inspection Standards (NBIS)*, T5140.21.

The inspection frequency for those bridges which require an underwater inspection for structural integrity is discussed in Article 3.10.

3.4 QUALIFICATIONS AND RESPONSIBILITIES OF INSPECTION PERSONNEL

3.4.1 General

Minimum qualifications have been established in the National Bridge Inspection Standards. The intent of the term "Be qualified for registration. . . ," is that the individual should meet all of the education and experience requirements for licensing but has not obtained the license.

The quality and efficiency of the inspection is influenced by the inspector's knowledge of how the bridge works and what controls its strength and stability. An understanding of material characteristics and construction procedures combined with skills in organizing data, plan reading, sketching, photography and technical report writing are valuable. Team members should have some formal classroom training to supplement on the job training. Short courses have proven to be effective in establishing standards and consistency within the inspection organization.

3.4.3 Inspection Team Leader

It is generally not desirable for bridge inspections to be conducted by a single individual working alone.

3.6.3.2 Inspection Methods and Equipment

Typical inspection equipment and tools are listed in the Bridge Inspection Training Manual (BITM) and other related publications.

3.7 INSPECTION FORMS AND REPORTS

In making a report, keep in mind that money may be allocated or repairs designed based on this information. Furthermore, it is a legal record which may form an important element in some future litigation. The language used in reports should be factual, clear and concise and, in the interest of uniformity, the same phraseology should be used insofar as possible to avoid ambiguity of meaning. The information contained in reports is obtained from field investigations, supplemented by reference to "as-built" or "Field Checked" plans.

Special inspections are made many times for the purpose of checking some specific item where a problem or change may be anticipated. Even though no changes are evident in this inspection and the condition seems relatively unimportant, documenting this information would be valuable in the future.

3.8 PROCEDURES

3.8.1.2 Cleaning

It is inadvisable to estimate corrosion depth from the thickness of corrosion bloom for many reasons. The corrosion thickness varies with environmental conditions and the existing corrosion at the time of inspection could be new deterioration on top of a previously deteriorated and cleaned area.

3.8.2.3 Piers and Bents

This article contains general instructions covering both piers and bents, without attempting to distinguish between the two terms. A separate discussion on open pile bents is contained in Article 3.8.2.4.

3.8.2.5 Bridge Stability and Movements

Articles 3.8.2.1-4 contain references to the need for checking bridge substructure elements for movement. Large movements will cause joints and hinges to jam or function improperly; slabs and deck units to crack; abutments, bents and piers to crack, rotate or slide; superstructure beams and girders to crack, buckle or lose their support; and retaining walls to fail. This article is intended to assist the inspector in locating places where movement has occurred and in tracing damage to determine if movement was its cause.

3.8.3.11 Pins and Hangers

Figure C3.8.3.11 illustrates the many parts that make up one type of pin and hanger assembly.

Ultrasonic testing of pins should be conducted by properly trained personnel. Calibration pins, when available, may be helpful in obtaining more meaningful ultrasonic test results.

3.8.3.12 Bearings

Sharp skewed and curved girder bridges may not have bearings which permit multi-rotation and movements. In such instances uneven wear of the bearing components should be expected. The substructure in

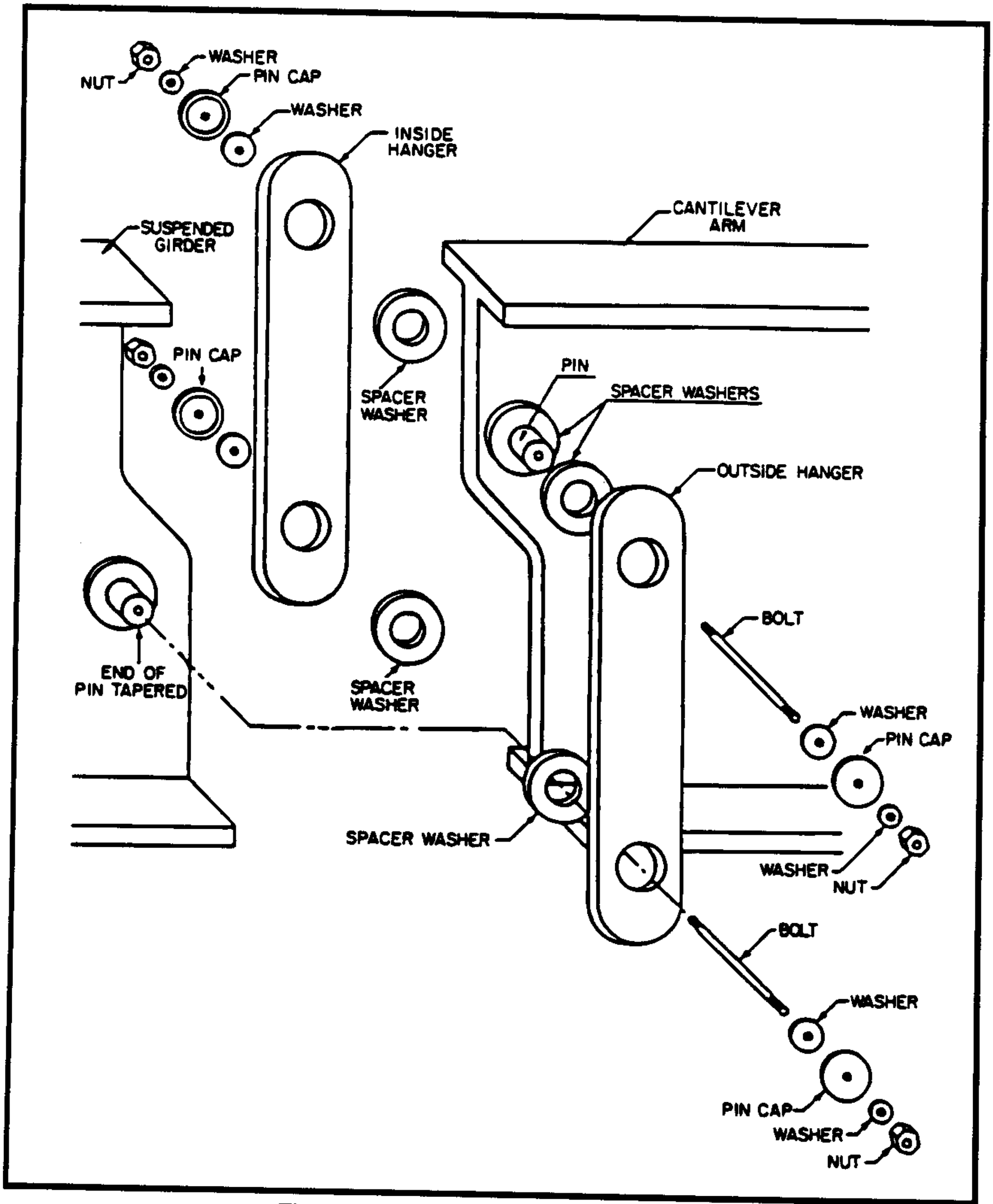


Figure C3.8.3.11 Pin and Hanger Assembly the vicinity of such bearings should be checked for possible distress.

3.8.3.14 Utilities

Bridges frequently are used to support utilities such as water supply, sanitary sewer, gas, electric and telephone. Most commonly these are suspended between beams or girders, below the deck. In most jurisdictions, the utility and the supports are owned, installed, and maintained by the utility company. In certain cases such as lighting circuits, the owner agency may be the same as the Bridge Owner.

3.8.9 Corrugated Metal Plate Structures

For more information on the inspection of CMP Arch culverts see the FHWA *Culvert Inspection Manual.*

3.9.4 Prestressed Concrete Segmental Bridges

Because of the many differences between design details used for segmental bridges, it is advisable to develop a separate inspection plan for each bridge.

Maintenance engineers have noticed a few instances of cracking which are peculiar only to segmental prestressed concrete bridges. A few bridges exhibited longitudinal cracks in the deck surface immediately outboard of the exterior girders. Most of these cracks were felt to have been caused by casting or curing methods which caused differential shrinkage between the overhanging slab and the box section. Cracks showed up when the section was stressed.

Other distress occurred at interior anchorages in the bottom slab. The high stress concentrations in these areas resulted in pop-outs, tension and shear cracking.

3.10 UNDERWATER INSPECTIONS

This article covers underwater inspection procedures and scour evaluation. The article highlights the need to thoroughly inspect substructure elements in a water environment. For additional information see the FHWA *Technical Advisory-Evaluating Scour at Bridges,* 5140.23.

The underwater inspection requirements of Title 23 *Code of Federal Regulations* Section 650.303 pertain to inspections that require diving or other special methods or equipment.

3.11 FATIGUE PRONE DETAILS

Fatigue refers to the process of material damage caused by repeated loads. Bridges that carry a large volume of heavy loads are more likely to experience fatigue problems. For further information see BITM-90.

3.12 FRACTURE CRITICAL MEMBERS

This article contains material on the inspection of fracture critical bridge members. For further information see *Inspection of Fracture Critical Bridge Members,* FHWA Report No. IP-86-26 and BITM-90.

4. MATERIAL TESTING

4.1 GENERAL

This new section defines the types of nondestructive field tests and provides guidance on when to use them. In addition, guidelines are provided for sampling bridge materials and using related laboratory tests. Source material included FHWA Manual on the *Inspection of Fracture Critical Bridge Members,* NCHRP Report 312 on the *Condition Surveys of Concrete Bridge Components,* NCHRP Report 206 on the *Detection and Repair of Fatigue Damage in Welded Highway Bridges,* NCHRP Report 242 on the *Ultrasonic Measurement of Weld Flaw Size,* FHWA Training Course on Nondestructive Testing, NCHRP Project 10-30 on the "Nondestructive Methods for Field Inspection of Embedded or Encased High Strength Steel Rods and Cables," various ASTM specifications, and state manuals.

Properly trained personnel should perform the testing described in this Section. The American Society of Non-Destructive Testing has programs for certifying technicians at various skill levels which may be used as a guide in establishing minimum levels of competency for test personnel.

4.3 MATERIAL SAMPLING

Additional guidance on repairing areas of bridge members from which material was removed for testing may be found in the AASHTO *Manual for Bridge Maintenance*, NCHRP Report 271, *Guidelines for Evaluation and Repair of Damaged Steel Bridge Members* and NCHRP Report 280, *Guidelines for Evaluation and Repair of Prestressed Concrete Bridge Members.*

4.5 INTERPRETATION AND EVALUATION OF TEST RESULTS

Care must be exercised in the interpretation and evaluation of field and laboratory test results. Several issues may play a part in the evaluation, for instance:

- Was sampling done properly? (location, size, number to adequately represent the member being tested)
- Do the results confirm expectations? Any surprises?
- Is there a pattern or consistency to the results of the group of tests or to previous test results?
- Was the test performed by an experienced individual or firm? (the reliability factor)
- Do the results indicate incipient failure, the need for immediate repairs or for weight-limit posting? (If so, must verify data.)
- Are other tests or inspections needed to verify results, to investigate other members in the same structure for like defects, or to look into the possibility of there being companion-type defects in the same member?
- Is there a likelihood that other structures on the system have experienced like problems—or that there may be similar structures where the problem is as yet undiscovered?

5. NONDESTRUCTIVE LOAD TESTING

The intent of this section is to indicate that load testing is an acceptable alternative for determining the response of a bridge to known loads. NCHRP Project 12-28(13)A, Bridge Rating Through Nondestructive Load Testing, is currently underway. When this project is completed, additional guidance on the utilization of the results of the load testing of bridges will be included here.

6. LOAD RATING

6.1 GENERAL

Bridge engineers have recognized that for the same bridge conditions a wide range of ratings may arise, depending on the rating method selected. Historically, several approaches have been used in rating bridges including Inventory and Operating rating levels and the use of allowable stress and load factor methods of analysis.

In recent years, methods have been developed to provide more uniform safety margins for structures in terms of a reliability index. AASHTO has a study underway to produce a new standard bridge design specification utilizing LRFD methods. For bridge evaluation, there is the load and resistance factor rating (LRFR) contained in the AASHTO *Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges* (1989) which provides a flexible selection of dead and live load factors and material resistance factors. However, use of this

Guide Specification will result in only one rating, not the traditional "Inventory" and "Operating" based ratings.

6.1.2 Substructure Consideration

The structural stability at abutments and both the structural stability and strength of pier elements should be checked in accordance with the provisions of this Article. Rarely, except in cases of severe material deterioration, will structural strength considerations govern the load rating at an abutment.

6.1.5 Nonredundant Structures

This section introduces the importance of redundancy in the evaluation and rating of bridges. Further guidelines in this area are being developed in NCHRP Project 12-38, Redundancy in Highway Bridge Superstructures.

6.4 RATING METHODS

In addition to the two methods described in this Manual, the LRFR method may be used. See Article 6.1 for more information on LRFR.

6.5 RATING EQUATION

6.5.1 General

The rating equation may be used regardless of the method used to evaluate a member capacity. The application of the basic rating equation to steel, concrete and timber bridges is illustrated in Appendix B.

For example, at the maximum moment section of a girder, the bending stress may be selected as the "load effect" to be evaluated. The capacity of the girder would be determined based on the maximum stress which the girder cross section could safely carry at the rating level desired. The dead load effect would be the theoretical bending stress due to dead loads at the section being evaluated. The live load bending stress would be computed based on the truck configuration or lane load selected for the rating and AASHTO impact and distribution factors. Appropriate factors (A_1 and A_2) would be selected and RF determined.

The RF would then be multiplied by the total weight (tons) of the nominal truck used in establishing the live load effect (L). Thus, the final rating for a bridge member will be expressed in tons.

6.5.4 Condition of Bridge Members

The effective cross-section properties used in determining the resistance or strength of the section to applied forces should be based on the gross cross section less that portion which has deteriorated. For instance, in a steel tension member, the member should be evaluated based on the least cross section area available to resist the applied tension force.

6.6 NOMINAL CAPACITY

6.6.2.1 Structural Steel

Standard coupon testing procedures (see Article 4.3) may be used to establish the nominal yield point. To provide a 95 percent confidence limit, the nominal yield point would typically be the mean coupon test value minus 1.65 standard deviations.

Mechanical properties of eyebars, high-strength eyebars, forged eyebars, and cables vary depending on manufacturer and year of construction. In the absence of material tests, the Engineer should carefully investigate the material properties using manufacturer's data and compilations of older steel properties before establishing the yield and allowable stresses to be used in load rating the bridge.

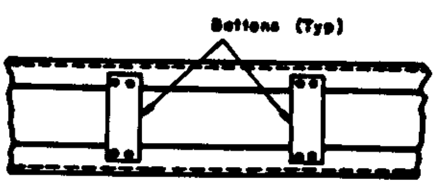
The formulas for the allowable bending stress in partially supported or unsupported compression flanges of beams and girders, given in Tables 6.6.2.1-1 and 6.6.2.1-2 are the corresponding formula based on given in Table 10.32.1A of the allowable Stress Design portion of the AASHTO Standard Specifications. The equation in Table 6.6.2.1-1 is to be used for an Inventory Rating and the equation in Table 6.6.2.1-2 is to be used for an Operating Rating.

The previously used formulas are inelastic parabolic formulas which treat the lateral torsional buckling of a beam as flexural buckling of the compression flange. This is a very conservative approach for beams with short unbraced lengths. The flexural capacity is reduced for any unbraced length greater than zero. This does not reflect the true behavior of a beam. A beam may reach Mp with unbraced lengths much greater than zero. In addition, the formula neglects the St. Venant torsional stiffness of the cross sections. This is a significant contribution to the lateral torsional buckling resistance of rolled shapes, particularly older "I" shapes. The previous formulas must also be limited to the values of l/b listed. This limit is the slenderness ratio when the estimated buckling stress is equal to half the yield strength or 0.275 Fy in terms of an allowable stress. Many floor stringers will have unbraced lengths beyond this limit. If the formulas are used beyond these limits, negative values of the allowable stress can result. The new formulas have no upper limit which allows the determination of allowable stresses for all unbraced lengths. In addition, the influence of the moment gradient upon buckling capacity is considered using the modifier C_b in the new formulas.

The specification formulas are based on the exact formulations of the lateral torsional buckling of beams. They are currently used in the AISC LRFD Specifications and other specifications throughout the world. They are also being used to design and rate steel bridges by the Load Factor method. The new figures on page 131 show a comparison between the specification formulas and the previous specification formulas for two sections. The top figure compares results for a W18 x 46 rolled section. The new specification gives a much higher capacity than the previous specification. The difference is due to the inclusion of the St. Venant torsional stiffness, J, in the proposed specification. The lower figure shows a similar comparison for a plate-girder section. The section, labeled section 3, has 1.5 x 16-inch flanges and a 5/16 x 94-inch web. The previous specification equation gives higher values than the new specification for large unbraced lengths. The previous specification is unconservative in this range. Both graphs show that, for small unsupported lengths, the new specification gives higher allowable stress values. The higher values result from the fact that there is an immediate reduction in capacity versus unsupported length in the previous specification.

6.6.2.1.2 Batten Plate Compression Members

Built-up compression members are generally connected across their open sides. Typical connections include stay plates in combination with single or double lacing, perforated cover plates, and battens. This article covers the use of batten plates only, when used as shown below:



6.6.2.2 Wrought Iron

Allowable maximum unit stresses in wrought iron for tension and bending at the Inventory level should be between 10,000 psi and 14,000 psi depending on material test results.

6.6.2.4 Concrete

Some guidance on the ultimate strength (f'_c) of concrete may be obtained from compression testing of cores removed from the structure. (See Article 4.3)

6.6.2.5 Prestressed Concrete

The limitation on the maximum stress in the prestressing steel ensures sufficient reserve ductility in the prestressing steel.

In the design of prestressed concrete members both the strength at ultimate load (Load Factor) and the allowable stress criteria at the transfer and in-service conditions must be satisfied. The strength design is based on factored loads and the flexural capacity of the section computed in accordance with Article 9.17 of the AASHTO Design Specifications.

In the Allowable Stress Method, Operating Level, the effects of the actual (unfactored loads) should not exceed 75 percent of the ultimate capacity of the member. This was selected by AASHTO to be consistent with the 75 percent of the steel yield stress used in the Allowable Stress Method, Operating Level for steel members.

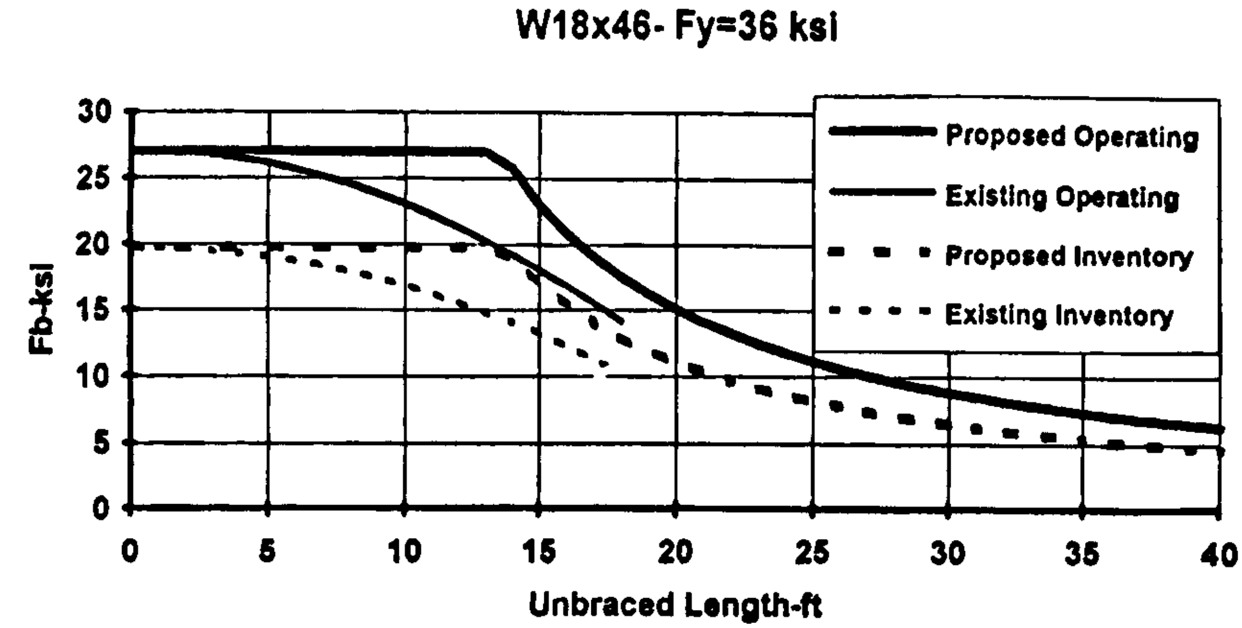
6.6.2.6 Masonry

The allowable stresses for evaluating masonry structures are based on the ACI empirical method (see ACI 530-88). These values are conservative and constitute a lower bound for allowable masonry stresses. The Engineer may use the more rigorous approach in ACI 530-88 as an alternative.

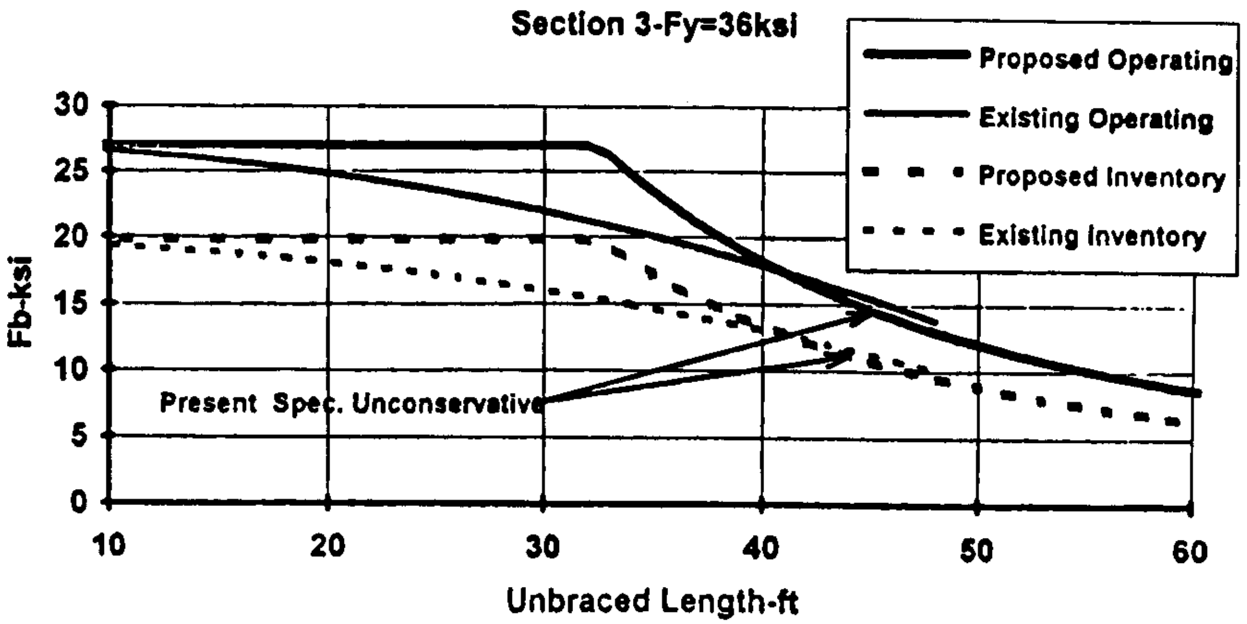
6.6.2.7 Timber

(2) Operating Stress

Equation 6-6 is based on the Euler long-column formula with two adjustments as follows. First E is reduced by dividing by 2.74. This corresponds to a safety factor of 1.66 for solid timber members according to the National Design Specifications for Wood Construction published by the National Forest Products Association. Then the Euler allowable stress is multiplied by 1.33 to provide an Operating level allowable stress as shown in Equation 6-6.



W18x46- Fy=36 ksi
Proposed Operating
Existing Operating
Proposed Inventory
Existing Inventory
Fb-ksi
30
25
20
15
10
5
0
0
5
10
15
20
25
30
35
40
Unbraced Length-ft



Section 3-Fy=36ksi
Proposed Operating
Existing Operating
Proposed Inventory
Existing Inventory
Present Spec. Unconservative
Fb-ksi
30
25
20
15
10
5
0
10
20
30
40
50
60
Unbraced Length-ft

For square and rectangular columns, substituting $d/\sqrt{12}$ for the radius of gyration (r) in equation 6-6 results in equation 6-7. This equation may also be found by multiplying equation 13-15 from the AASHTO Design Specifications by 1.33.

6.6.3 Load Factor Method

Nominal capacities for members in the proposed guidelines are based on AASHTO's Design Specifications contained in the load factor section. This resistance depends on both the current dimensions of the section and the nominal material strength.

Different methods for considering the observable effects of deterioration were studied. The most reliable method available still appears to be a reduction in the nominal resistance based on measured or estimated losses in cross-sectional area and/or material strengths.

At the present time, load factor methods for determining the capacity of timber and masonry structural elements are not available.

6.7.2 Rating Live Load

Federal regulations require the reporting of Inventory and Operating ratings uniformly, based on the AASHTO Standard HS loading.

6.7.2.2 Truck Loads

The probability of having a series of closely-spaced heavy vehicles of the maximum allowable weight becomes greater as the maximum allowed weight for each unit becomes less. That is, it is more likely to have a train of light-weight vehicles than to have a train of heavy-weight vehicles. This makes it necessary to consider more than one vehicle in the same lane under some conditions. For example, vehicles should be spaced at distances of 30 feet clear or more in the same lane to produce maximum load effect when the safe loading per vehicle or vehicle combinations is less than 12 tons.

6.7.2.4 Sidewalk Loadings

The probability that both the full truck and full sidewalk live loads would act simultaneously on the bridge is quite low. This loading case should be evaluated based on the Operating level.

6.7.4 Impact (I)

The condition of the approach roadway and deck joints may also influence the selection of an appropriate impact factor. Some guidelines are provided in the AASHTO *Guide Specifications for the Strength Evaluation of Existing Steel and Concrete Bridges.*

6.7.7.2 Earthquake

Bridge Maintenance Engineers may be called upon to evaluate existing structures for their capacity to resist earthquake forces. This specification permits the investigator to use either the relatively simple methods of the AASHTO Bridge Design Specifications or the more complex analysis procedures described in the AASHTO Specifications for Seismic Design. If facilities and trained personnel are available, the multimodal spectral method of analysis is recommended to provide more thorough and credible results.

For seismic retrofitting of bridges, seismic loads must be considered.

7. ADDITIONAL CONSIDERATIONS

7.3 FATIGUE EVALUATION OF STEEL BRIDGES

Fatigue evaluation procedures for existing steel bridges, and comparable fatigue design procedures for new steel bridges, were developed in NCHRP Project 12-28(3) and are described in detail in NCHRP Report 299. The evaluation procedures are intended to be used as an alternative to Articles 10.3.1 and 10.3.2 in the 1989 AASHTO Design Specifications.

Historically, AASHTO fatigue design procedures did not reflect actual fatigue conditions in bridges; instead they combined an artificially high stress range with an artificially low number of stress cycles. Furthermore, design procedures were too rigid, did not provide consistent levels of reliability for different cases, did not permit calculation of the remaining fatigue life, and, consequently, were not really suitable for checking existing bridges.

Most of the fatigue damage in a bridge is caused by passages of single trucks across the bridge. The total number of truck passages in the 75- to 100-year life of a bridge can exceed 100 million, but is often much less. The effective stress range rarely exceeds 5 ksi and is usually 1 to 3 ksi. Traffic volumes usually grow at an annual rate of about 3 to 5% until they reach a very high limiting value. Traffic volumes on some urban highways are presently at such high levels.

The AASHTO Guide Specifications referenced in the text contain procedures for determining fatigue loading, impact factor, stress ranges in typical sections, stress cycles, appropriate reliability factors and methods to estimate remaining safe life and mean life. Options to be considered if the computed remaining life is inadequate are also described.

7.4 POSTING OF BRIDGES

7.4.1 General

Most structures which require weight limits below statutory limits are old and designed for light loads, and/or are weak as a result of damage. With some exceptions, the weaker elements of older bridges are usually in the superstructure, not in the piers or abutments.

There may be circumstances where the Bridge Owner may utilize load levels higher than those used for Inventory rating, in order to minimize the need for posting of bridges. In no case shall the load levels used be greater than those permitted by the Operating Rating.

For those bridges supporting large dead loads, the use of the Load Factor or Load and Resistance Factor rating methods may result in a live load capacity greater than that determined based on the allowable stress rating method.

Bridges which use a load level above the Inventory Level should be subject to more frequent, competent inspections. Several factors may influence the selection of the load level. For instance:

(1) The factor of safety commonly used in the design or Inventory level rating may have provided for an increase in traffic volume, a variable amount of deterioration and extreme conditions of live loading.

(2) The factor of safety used in rating existing structures must provide for unbalanced distribution of vehicle loads, and possible overloads. For both design and rating, factors of safety must provide for lack of knowledge as to the distribution of stresses, possible minimum strength of the materials used as compared to quoted average values, possible differences between the strength of laboratory test samples and the material under actual conditions in the structure, and normal defects occurring in manufacture or fabrication.

(3) A higher safety factor for a bridge carrying a large volume of traffic may be desirable as compared with the safety factor for a structure carrying few vehicles, especially if the former includes a high percentage of heavy loads.

(4) The probability of having a series of closely-spaced vehicles of the maximum allowed weight should be considered. This effect becomes greater as the maximum allowed weight for each unit becomes less.

(5) Lower load levels may be warranted for nonredundant metal bridge elements due to the consequences of failure. Exceptions may be elements of riveted construction and all floor beams, provided they are in good condition. Examples of nonredundant elements are welded or rolled two-girder bridges, truss members, or pinned eye-bar trusses and truss members on welded trusses.

(6) Bridges with extensive material losses may warrant a lower load level because of the greater uncertainty in evaluating present strength capacity. This is especially true if the loss in material is in a highly stressed area.

(7) Sites for which it is suspected that there are frequent truck overloads should be considered for lower load levels unless enforcement methods are put in place.

(8) The ratio of dead load to live load may have an influence on the selection of appropriate

load level. Structures with high ratios of dead to live load and for which there are no visible signs of distress may be considered for the higher load levels.

For the LRFR Method, additional guidance on selection of Intermediate rating levels based on bridge inspection, condition, maintenance, traffic loading and redundancy, may be obtained from the AASHTO *Guide Specifications for the Strength Evaluation of Existing Steel and Concrete Bridges.*

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