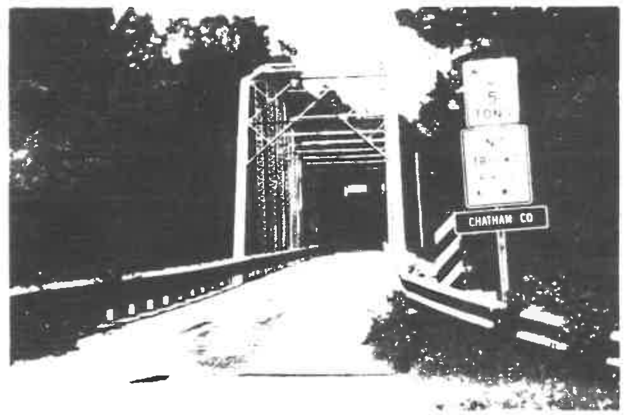
Bridge Evaluation and Posting

An overview of present Federal bridge legislation, policies, and procedures and how they can assure the safety of bridges under your management



Center for Transportation Engineering Studies
Department of Civil Engineering
North Carolina State University

Prepared for the
Federal Highway Administration
Office of Technology Applications
December, 1993

Institute for Transportation
Research and Education
North Carolina State University

NOTICE

BRIDGE EVALUATION AND POSTING

by

David W. Johnston
Professor

Shuaib H. Ahmad
Professor

Edwin C. Weaver
Graduate Research Assistant

and

Martha A. Laverde
Graduate Research Assistant

December 1993

Center for Transportation Engineering Studies
Department of Civil Engineering
North Carolina State University
Raleigh, North Carolina 27695

TABLE OF CONTENTS

The December 1967 collapse of the Silver Bridge over the Ohio River between West Virginia and Ohio killed 46 people and focused national attention on bridge conditions. The resulting Congressional investigation concluded that many state and local governments had inadequate programs for inspecting, maintaining, and improving bridges.

The following year Congress mandated the National Bridge Inspection Program in the Federal-Aid Highway Act of 1968, which established the first Federal-Aid bridge inspection standards. Subsequent legislation defined bridge classifications and inventory standards and authorized Federal funding to help replace unsafe bridges. Although significant progress has been over the years toward improving the safety of the bridge system, periodic failures have been a reminder of the need for continuous improvement in the quality of the inspection and management process.

The five sections of this handbook discuss present Federal bridge legislation, policies, and procedures and how they can help you assure the safety of the bridges under your management.

Preface 'Silver Bridge After Collapse

SECTION 1
FEDERAL BRIDGE LEGISLATION AND RESULTING STANDARDS: UNDERSTANDING BASIC BRIDGE TYPES, COMPONENTS, AND EVALUATIONS

This section discusses legislation affecting national bridge inspection and the resulting standards. Included are the requirements for structural inventory and appraisal, bridge inspectors, inspection frequency, inspection types, and special inspections as for fracture critical bridge members and underwater members.

In addition, basic bridge types and components are described and diagrammed to help you understand information provided for the National Bridge Inventory data base. Finally, evaluation classifications and ratings are explained.

FEDERAL BRIDGE LEGISLATION

The 1968 Federal-Aid Highway Act and subsequent legislation in 1970, 1978, 1987, and 1991 directs the Secretary of Transportation, in consultation with state highway departments and other knowledgeable groups, to establish standards for inspecting all bridges on public roads. These standards were to specify (1) safety inspection methods, (2) minimum time lapse between inspections, and (3) inspector qualifications. In addition, the legislation directs each state to maintain written reports and a current inventory of these bridges and to implement a bridge management system. The legislation also established a training program for Federal and state employees and authorized states to use federal funds for training, inventory, and inspection.

As a result, the National Bridge Inventory (NBI) now contains data for all bridges in the Federal-Aid highway system and for virtually all of the bridges on all other roads as shown in Table 1.1. Inventory, inspection and appraisal data, compiled by state and local governments, is sent to the FHWA at least once a year for inclusion in the NBI.

NATIONAL BRIDGE INSPECTION STANDARDS (NBIS)

In order to assure that bridges are safe for public use, National Bridge Inspection Standards (NBIS) were first issued in April 1971 and have been periodocally updated. The NBI data help state and local officials understand and meet bridge management needs and provide information for Federal fund distribution, defense record preparation, and response to Congressional inquiries.

Structure Inventory and Appraisal

Each state or local government must inventory and inspect its bridges according to National Bridge Inspection Standards, which require current, accurate data for all bridges on public roads. To conduct the inventory, each state must first (a) count all bridges (b) assign each bridge an identifying number (c) record the location of each bridge and (d) establish ownership of each bridge in the state.

Inspection procedures must include rating the condition of various structural elements and appraising overall structural conditions.

Each state or local government must then submit specific inventory data (including inspection results) to the FHWA for its National Bridge Inventory. This data is initially recorded on an NBI Structure Inventory and Appraisal Sheet, shown in Figure 1.1, and must conform with the "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's

Table 1.1 Status of the Nation's Bridges*

	Federal-aid	Non Federal-aid	Total
No. of Bridges Inventoried and classified	276,510	298,903	575,413
No. of structurally deficient bridges**	33,799	84,764	118,563
No. of functionally obsolete bridges**	44,042	36,672	80,714
No. of bridges that are load posted	18,572	99,836	118,408
Additional bridges that should be load posted***	4,154	12,469	16,623
Total No. of bridges that are or should be load posted***	22,726	112,305	135,031
No. of bridges closed to all traffic	462	4,248	4,710

* From NBI June 30, 1992.

** A structurally deficient bridge, as defined by FHWA, is one that (1) has been restricted to light vehicles only, (2) is closed or (3) requires immediate rehabilitation to remain open; a functionally obsolete bridge is one on which the deck geometry, load carrying capacity, clearance, or approach roadway alignment no longer meet the usual criteria for the system of which it is an integral part.

*** Bridges that require load posting fall into two groups. One group includes structurally deficient bridges that have deteriorated to the extent that they cannot carry the load for which they were designed. The second group includes functionally obsolete bridges that are in good condition but the current State legal load exceeds the original design load and therefore, the bridges require posting.

Bridges." This data is entered into the computer record in a similar format.

In order to maintain complete records, the FHWA substitutes approximate values when states provide inadequate data. Therefore, to insure valid national records, it is essential that all reported inventories be as complete and precise as pos-sible. Inadequate inspection programs and reports can lead to higher rehabilitation costs and inconvenience or decreased safety for users.

Requirements for Bridge Inspectors

It is essential that bridge inspectors have a uniform standard of training to enable them to make objective judgements consistent with the evaluations of inspectors from other states or agencies. Therefore, the individual in charge of state inspections must

1. Be a registered professional engineer, or
2. Be qualified for registration as a professional engineer under the laws of that state, or
3. Have at least 5 years of ex-perience and completed a comprehensive training course based upon the "Bridge Inspection Training Manual," or
4. Have NICET (National Certification of Engineering Technologies) certification

To further assure consistency, it is also important that each inspector use the bridge description ratings established by the FHWA instead of any ratings established by the individual states or local governments.

Inspectors must be thoroughly familiar with bridge design and construction features. They must be able to recognize any structural deficiencies, assess their seriousness, determine the safe load carrying capacity of each bridge, and act to maintain its safety (Figure 1.2).

Typically, the bridge inspector will:

- Assist in planning and preparing for bridge inspections
- Inspect bridge components for deterioration
- Sketch bridge components
- Photograph specific bridge problems
- Take technical measurements
- Make basic computations
- Report hazardous conditions to supervisor
- Maintain inspection records

Bridge inspectors will also supervise divers in underwater inspections. Because most commercial divers have no experience inspecting and evaluating bridge conditions, the FHWA has been urged to adopt minimum qualifications for underwater inspectors. Some bridges may require divers with bridge inspection expertise; however, other bridges requiring underwater inspection may be inspected by a fully qualified inspector on the surface directing a diver below (Figure 1.3).

Therefore, those bridges requiring inspector-divers should be determined on a case-by-case basis by the person in overall charge of the agencies inspection program. Such determinations should only be made after considering substructure type, past inspection experience, scour studies and predictions, assessment of the potential for impurities within the water to attack the substructure, and other appropriate evaluations.

Types of Inspections

There are five National Bridge Inspection procedures:

1. Inventory Inspection occurs when a bridge first becomes part

Figure 1.2 Inspectors Evaluating Bridge Deck

Figure 1.3 Underwater Inspection

of the bridge inventory or when the configuration of the structure has been changed. This inspection determines all Structure Inventory and Appraisal data required for the NBI Record. This inspection helps identify any existing structural problems or the exact location of potential probems.

The NBI Standards require inventory inspection data to be reported within 90 days (This data, however, does not appear in the national inventory for 4 to 6 months). Data for bridges under local jurisdiction must be reported within 180 days.

2. Routine Inspection occurs regularly to determine whether the bridge continues to satisfy present service requirements. It identifies physical and functional conditions and any developing problems of the structure. This inspection must meet maximum inspection frequency requirements, bridge inspector qualifications, and include any Structure Inventory and Appraisal updating. Routine inspections must be documented with photographs and written reports recommending maintenance or repair procedures.

3. Damage Inspection is performed to diagnose structural damage from environmental or man-made causes. A timely in-depth inspection should be made to determine the extent of damage, the magnitude and urgency of repair, and the necessity for load restriction or bridge closing. Damage inspection should include complete documentation of the inspection, including field measurements and calculations.

4. In-Depth Inspection, requiring special skills and equipment, is a close-up, hands-on determination of deficiencies not visible during routine inspection. Non-destructive and/or other physical and chemical tests may be needed to ascertain the existence or extent of any deficiency. This inspection can be a follow-up for damage or inventory inspections or can be performed in place of a routine inspection, but at a longer interval.

5. Interim Inspection monitors a suspected or particular known deficiency. It is performed at the direction of the person responsible for bridge inspection activities and is repeated as often as the severity of the known deficiency requires.

Frequency of Inspections

Current regulations generally require bridge inspection every two years. However, NBI standards allow states and local governments to request approval for inspection intervals longer than two years for certain bridges with relatively new structures in good condition and with load capacities at least equal to the state legal load. Other bridges will require more frequent inspection as indicated by condition, age, load capacity, or specific existing or potential deficiencies.

Special inspections are necessary after significant traumatic events such as fires, floods, earthquakes, and collisions.

Special Inspections

A 1988 NBIS revision expanded the inspection for bridges with fracture critical members and the requirements for underwater inspection. In these cases, requirements must be designated on individual inspection and inventory records and compiled in master lists which should include:

1. Bridge type and location
2. Required inspection type and frequency
3. Location of inspected members
4. Inspection procedures
5. Required equipment
6. Dates of previous inspections
7. Most recent inspection findings
8. Follow-up actions taken on last inspection findings

Underwater Inspection is necessary to determine the structural safety of underwater members and the condition of the stream bed. Inspection level and frequency depend upon age, design, construction materials, stream bed material, flood flow depth and velocity, etc.

Several bridge failures, such as the one shown in Figure 1.4, have increased the enforcement of federal regulations for underwater inspection and inspectors.

There are three levels of underwater inspections:

Level I -- A "swim-by" overview of the entire underwater structure with minimal cleaning to remove marine growth.

Level II -- A measure of the damage on at least 10% of the underwater members selected randomly. Marine growth is cleaned from sample underwater members in 10"wide bands at designated depths. More extensive Level II inspection is performed when the condition of underwater members is not determined after the routine inspection or when section losses must be determined.

Level III -- A measure of in-ternal soundness of underwater mem-bers. This inspection utilizes nondestructive tests (ultrasound) or minimal destructive tests such as

Figure 1.4 General Bridge Failure Due to Underwater Problem

coring of wood or concrete and insitu hardness tests.

Underwater inspection must be performed at least every five years. However, more frequent inspections are recommended where deterioration is evident but repairs are not yet essential, or where the water environment is very harmful to structural members. An engineering evaluation to determine whether the structure is susceptible to scour damage should be performed after the underwater inspection.

Inspection of Fracture Critical Bridge Members

Fracture critical members are tension members or bridge components whose failure would probably result in the collapse of the bridge, such as the Mianus River Bridge in Connecticut which collapsed in 1983 (Figure 1.5). This collapse resulted in an emergency order by the FHWA to reinspect all bridges with critical members similar to those in the Mianus River Bridge.

Before beginning field operations, the person(s) in charge must decide whether to conduct (1) a close-up, hands-on inspection using standard, readily available tools or (2) an inspection using more sophisticated, nondestructive methods. Sometimes, based on inspection findings, inspection procedures must be modified in the field. After inspection, a report must be filed clearly identifying and giving details of all critical members receiving special attention.

IMPORTANT BRIDGE TYPES

Bridges serve two general functions: to span natural barriers (ravines, waterways) and to provide man-made

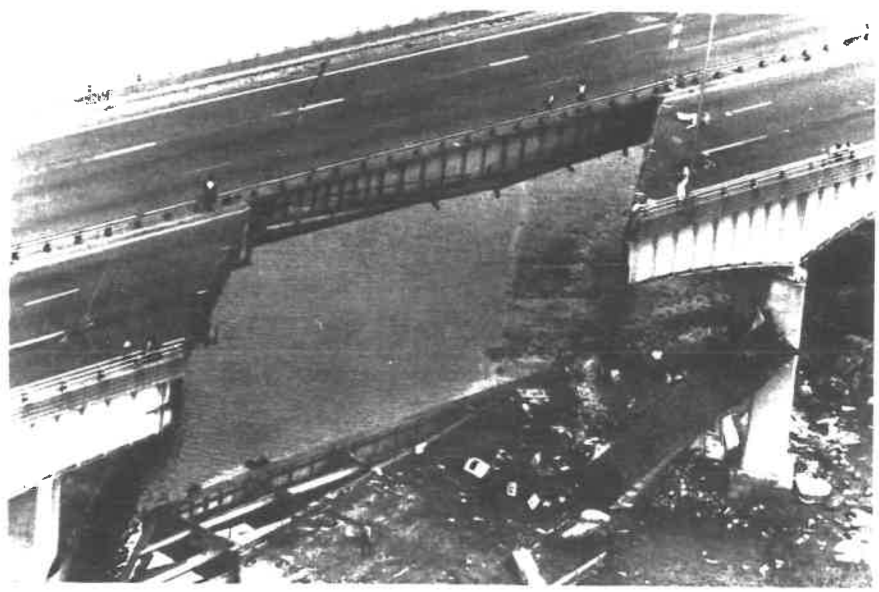


Figure 1.5 Mianus River Bridge Collapse

grade separations over other roadways, railroads, pedestrian rights of way, etc.

There are many possible bridge classifications. For example, bridges are fixed or movable, have different types of spans, and are constructed of different materials such as concrete, steel, timber, masonry, aluminum, wrought iron or cast iron. Following is a list of the major types encountered.

Span Types

The three types of span arrangements are (Figure 1.6):

1. Simple Span is a superstructure whose length is the distance between its two supports. The beams or girders have a pin or hinge at one end and a roller at the other or pads at both ends.

2. Continuous Span has a continuous superstructure over one or more supports. Continuous beams or girders have a pin or hinge bearing one support and usually have rollers at other supports.

3. Cantilever Span has a continuous superstructure with one or more hinges in the beams to simplify design and construction.

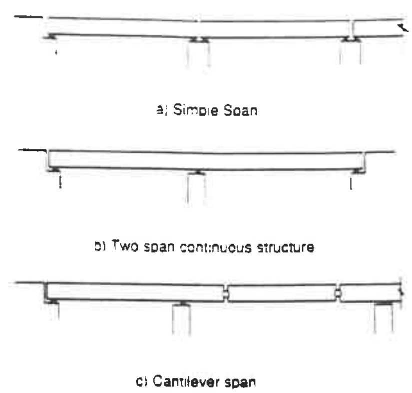


Figure 1.6 Bridge Span Types

Structure Types

1. Thru Bridges have the floor system near the elevation of the bottom flanges of the girders (Figure 1.7).

2. Deck Bridges have the floor system at or above the elevation of the top flanges of the girders (Figure 1.7).

3. Truss Bridges have frame structures divided into a series of

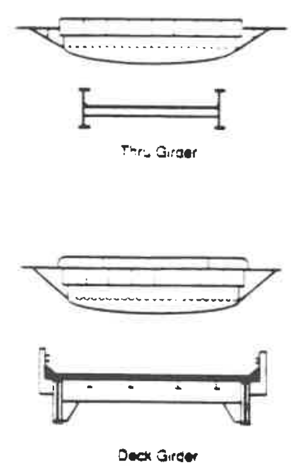


Figure 1.7 Through Bridge and Deck Bridge

triangular figures with perpendicular members giving lateral support. Because trusses are efficient structurally, they can span great lengths. There are three types of truss bridges (Figure 1.8):

a. Thru truss
b. Pony truss
c. Deck truss

4. Girder Bridges have superstructures consisting of two or more girders supporting a separate floor system of slab and floor beams or slab, stringer, and floor beams (Figure 1.9).

5. Stringer Bridges have multiple longitudinal beams or girders supporting the deck slab (Figure 1.10).

6. Tee Beam Bridges have multiple longitudinal concrete girders cast integral with the deck slab.

7. Slab Bridges have a short span superstructure composed of uniform thickness concrete which also serves as the deck.

8. Box Beam Bridges are constructed of large rectangular-shaped concrete beams, either reinforced or prestressed, which also form the bridge deck (Figure 1.11).

9. Movable Bridges span navigable waterways where it was not feasible to construct a higher bridge. There are three types of movable bridges (Figure 1.12):

   a. Bascule bridges have a two-leaf structure in which the leaves open upward with an angular rotation providing a navigation passage.
   b. Swing bridges balance on a center pier and rotates 90 degrees horizontally to allow navigation to pass.

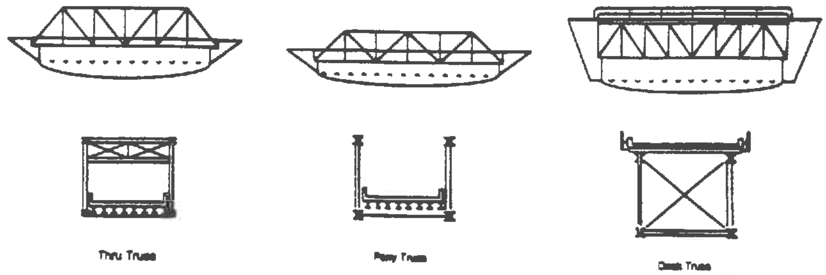


Figure 1.8 Truss Bridges

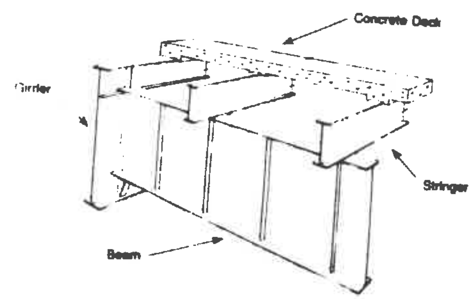


Figure 1.9 Girder Bridge

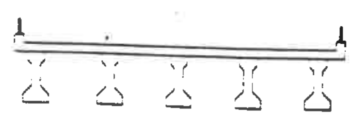


Figure 1.10 Stringer Bridge

Figure 1.11 Box Girder

c. Vertical lift bridges are lifted by cables at each end of the structure which remains horizontal.

10. Suspension and Stayed Girder Bridges (Figure 1.13) have the floor system suspended by cables supported upon towers at one or more locations; the cables are anchored at their extreme ends.

11. Culverts are structures usually less than 20 feet in span which conduct water or sometimes pedestrians under a highway, railroad, or other embankment, and may directly support traffic loads. Culverts are constructed of various materials, such as concrete, corrugated steel, corrugated aluminum, and masonry. Culverts have many different shapes and sizes, depending upon structural and hydraulic requirements, depth of cover, headwater elevation limits, and debris clogging potential (Figure 1.14). They may involve single units or multiple units side-by-side.

IMPORTANT BRIDGE ELEMENTS

Both the superstructure and substructure, composed of the following components, are evaluated in the bridge inspection process.

Superstructure

The superstructure may include the following components (Figure 1.9):

1. Stringers are longitudinal beams which receive and distribute loads from the deck to the supporting elements.

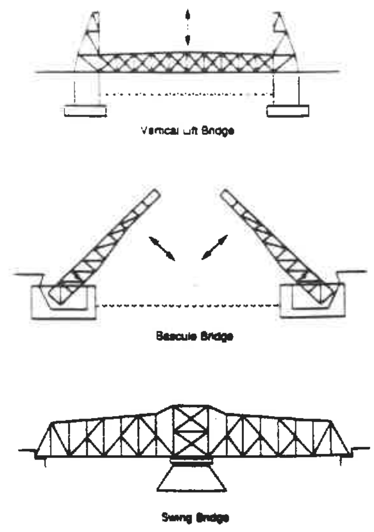


Figure 1.12 Movable Bridges

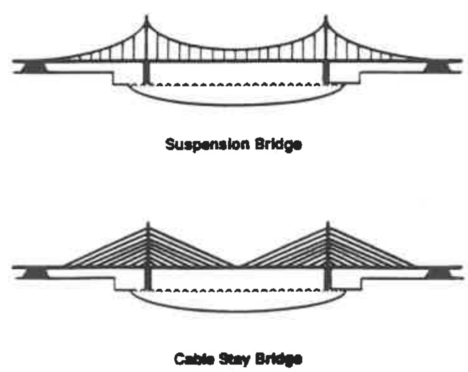


Figure 1.13 Suspension Bridges

2. Floor Beams are simple or compound elements that receive load and transmit it transversely (crosswise).

3. Girders are members which provide primary support of the span by receiving loads from the floor beams and the stringers or directly from the slab.

The major superstructure elements include the following:

1. The Deck (Figure 1.9) is a concrete slab, timber planking, a steel grid, or a steel plate which directly supports and distributes the live load to the floor system.

2. The Floor system is either longitudinal stringers carried by transverse beams or closely spaced crosswise or transverse beams.

3. Main supporting members (beams, girders, rigid frames, trusses, arches, or cables) transmit all loads from the deck or floor system to the substructure.

4. Bearings (Figure 1.15) are hinge and roller or pad elements located between the top of the substructure and the bottom of the superstructure. They transmit the superstructure load to the substructure and enable the superstructure ends to rotate and slide with deflection, expansion, or contraction.

Substructure

Seven important substructure components are:

1. Abutments (Figure 1.16) support the ends of the extreme span of the superstructure and usually retain or support the

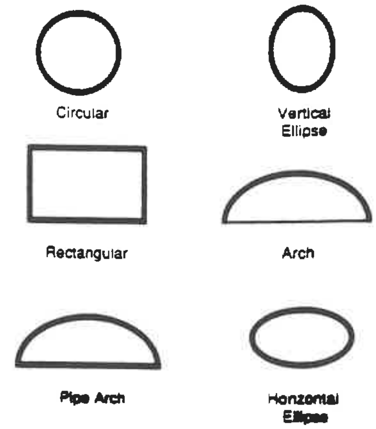


Figure 1.14 Types of Culverts

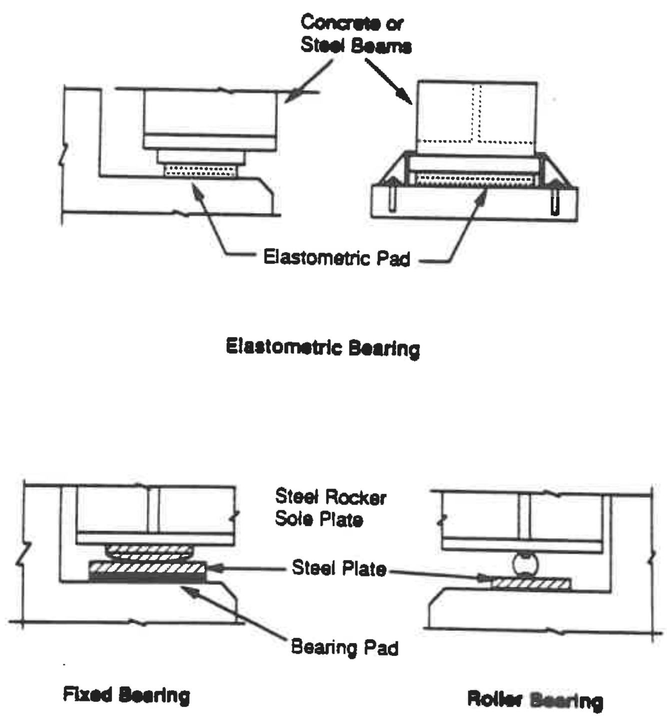


Figure 1.15 Example Bearings

approach fill. They consist of a footing, a stem or breast wall, a bridge seat, a back-wall, and wing walls.

2. Pier caps (Figure 1.16) are cross-beams located on top of piers or piles to receive and distribute superstructure loads to the substructure.

3. Piers are column-like members supporting the ends of the spans in multispan superstructures at intermediate locations between abutments.

4. Footings (Figure 1.16) are slabs that transmit the load from piers or abutments to the soil or rock.

5. Pile caps are slab-like elements which distribute loads from piers or abutments to the tops of piles or foundation piers.

6. Piles are linear elements of concrete, steel, timber, or composite material driven into the earth to carry loads through weak soil layers to those layers capable of providing support.

7. Foundation piers (in some places called caissons) are larger diameter, predominately concrete, piles installed by drilling or excavating.

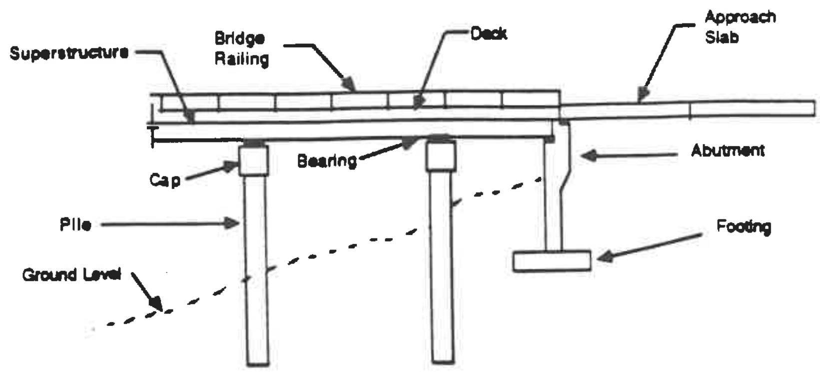


Figure 1.16 Bridge Elements

BRIDGE IDENTIFICATION AND CLASSIFICATION DATA

1. Identification: All bridges in the National Bridge Inventory must first be identified according to:

   - State
   - Structure number
   - Inventory route on/under
   - State highway department district

- Facility (roadway #) carried
- Location
- Latitude and longitude

2. Classification: Bridges must then be classified and coded by highway function and possible historical significance.

The functional classification identifies the service provided by the route the bridge carries. Functional codes are listed in Table 1.2.

Historical significance codes indicate whether a bridge is historical because

A) It is a unique example of engineering history
B) The crossing is historically significant
C) The bridge is associated with historical property or events.

Structure, service, geometric, and navigation data must be included in the evaluation. These data provide current bridge characteristics such as:

- Type of structure and material
- Type of service
- Structure length and number of spans
- Year built
- ADT (average daily traffic)
- Design load
- Lanes on/under structure
- Width of bridge and approach roadways
- Skew
- Under/over clearance
- Navigation control and pier protection

Figure 1.2 provides a complete list of the required data items.

Table 1.2 Functional Classification

Code	Description
	Rural
01	Principal Arterial - Interstate
02	Principal Arterial - Other
06	Minor Arterial
07	Major Collector
08	Minor Collector
09	Local
	Urban
11	Principal Arterial - Interstate
12	Principal Arterial - Other Freeways or Expressways
14	Other Principal Arterial
16	Minor Arterial
17	Collector
19	Local

BRIDGE EVALUATION RATINGS

Four different rating systems are used to record bridge strength and adequacy data. These systems and coding requirements are described in detail in the "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nations Bridges." A summary description is provided here.

Inventory and operating ratings indicate the magnitude and frequency with which bridges can safely support loads: The inventory rating indicates the maximum load a bridge can support through indefinite repetitions; the operating rating indicates the maximum load which can be supported with very few repetitions.

Condition ratings describe current bridge maintenance condition compared with conditions when the bridge was new. Appraisal ratings, on the other hand, compare the adequacy of the existing structure with a new bridge built to current standards for that particular highway system. Condition codes record maintenance condition, degree of deterioration of major structural elements, and in short, what the bridge looks like. Appraisal codes indicate how adequate the existing bridge is for current load capacities and roadway service needs. This adequacy is not necessarily a measure of the bridge's physical quality.

Condition Rating

The condition rating codes shown in Table 1.3 are used to evaluate the deck, superstructure, substructure, channel, and culverts.

Bridge condition ratings should not be influenced by

- The load carrying capacity of the bridge

Table 1.3 Condition Rating Codes

Code	Description
N	NOT APPLICABLE
9	EXCELLENT CONDITION
8	VERY GOOD CONDITION - no problems noted
7	GOOD CONDITION - some minor problems
6	SATISFACTORY CONDITION - structural elements show some minor deterioration
5	FAIR CONDITION - all primary structural elements are sound but may have minor section loss, cracking, spalling or scour.
4	POOR CONDITION - advanced section loss, deterioration, spalling or scour.
3	SERIOUS CONDITION - loss of section, deterioration, spalling or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.
2	CRITICAL CONDITION - advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.
1	"IMMINENT" FAILURE CONDITION - major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic but corrective action may put back in light service.
0	FAILED CONDITION - out of service - beyond corrective action.

- Temporary members used to support or strengthen part of the structure
- The fact that a bridge is complete, but not yet open to traffic

1. Deck Evaluation The complete deck should be inspected for:

   - Cracking
   - Corrosion
   - Scaling
   - Splitting
   - Spalling
   - Crushing
   - Leaching
   - Fastener failure
   - Broken welds
   - Deterioration from rot
   - Broken grids

   Although not considered in the evaluation, the condition of the following should be recorded during the inspection: joints, curbs, sidewalks, parapets, rails, scuppers, expansion devices, and wearing surface.

2. Superstructure Evaluation Structural members should be inspected for cracking, deterioration, section loss, and bearing malfunction and misalignment. The superstructure rating may be affected by the deck condition if the deck is an integral part of the superstructure. The condition of fracture critical members must be included in this evaluation.

3. Substructure Evaluation The substructure evaluation includes the condition of caps, piers, abutments, piles, fenders, and footings, which must be inspected for cracking, section loss, settlement, misalignment, scour, collision damage, and corrosion.

4. Evaluation of Channels and Channel Protection Evaluation codes reflect the effects of water flow on the channel, riprap, slope protection, and stream stability. The major concern is how much the water velocity is eroding the banks, realigning the stream, and undermining the footings and slope protection. Channel quality is recorded using the general condition rating codes plus the special codes listed in Table 1.4.

5. Culvert Evaluation Culverts are evaluated and rated using both the general and special codes mentioned above. Culvert conditions are indicated by evaluating alignment, settlement, joints, structural conditions, and the impact of scour.

Operating Rating

An operating rating indicates the absolute maximum permissible load to which a bridge may be subjected, provided the load is not often repeated. This rating is given as both the vehicle type (number and spacing of axles) and the gross load in tons.

Special consideration should be given to temporary bridges and bridges shored up or temporarily repaired. Operating ratings can be calculated by either the working stress or load factor methods.

Inventory Rating

An inventory rating indicates the load level which can safely be used on an existing bridge for an indefinite period. This rating uses the same codes as the operating rating.

Table 1.4 Channel and Channel Protection Codes

Code	Description
N	Not applicable. Use when bridge is not over a waterway.
9	There are no noticeable or noteworthy deficiencies which affect the condition of the channel.
8	Banks are protected or well vegetated. River control devices such as spur dikes and embankment protection are not required or are in a stable condition.
7	Bank protection is in need of minor repairs. River control devices and embankment protection have a little minor damage. Banks and/or channel have minor amounts of drift.
6	Bank is beginning to slump. River control devices and embankment protection have widespread minor damage. There is minor stream bed movement evident. Debris is restricting the waterway slightly .
5	Bank protection is being eroded. River control devices and/or embankment have major damage. Trees and brush restrict the channel.
4	Bank and embankment protection is severely undermined. River control devices have severe damage. Large deposits of debris are in the waterway.
3	Bank protection has failed. River control devices have been destroyed. Stream bed aggradation, degradation or lateral movement has changed the waterway to now threaten the bridge and/or approach roadway.
2	The waterway has changed to the extent the bridge is near a state of collapse.
1	Bridge closed because of channel failure. Corrective action may put back in light service.
0	Bridge closed because of channel failure. Replacement necessary.

Appraisal Rating

An appraisal rating compares current roadway service needs with the adequacy of a bridge in its current condition to handle those needs. Included in this rating is a comparison of needed and current load capacities.

It is very important that the nationally established level-of-service criteria be used in appraisal ratings instead of any corresponding evaluation criteria developed by individual states or local governments. Uniform national ratings can only be achieved if all agencies use the same national criteria shown in Table 1.5 to make the following evaluations:

Structural Appraisal evaluates the overall strength of a bridge, including all major structural deficiencies. The structural appraisal rating code considers the inventory rating in relation to the volume of traffic served. This rating should not be higher than the lowest of the condition ratings of the superstructure and the substructure.

Deck Geometry Appraisal determines how adequate a bridge roadway is for current traffic demands by evaluating (1) the width curb-to-curb, or face-to-face in rail bridges and (2) the minimum vertical clearance over the bridge roadway. The tables used to rate deck geometry include consideration of bridge roadway width, ADT, vertical overclearance, and number of lanes.

Vertical and Horizontal Underclearance Appraisal evaluates the vertical and horizontal underclearances from the through roadway to the superstructure or substructure. The lowest vertical and horizontal codes obtained after evaluation are used. This code is also part of the functional classification of the underpassing route.

Waterway Appraisal evaluates how well the waterway opening allows water to flow through it. This appraisal

Table 1.5 Appraisal Rating Codes

Code	Description
N	Not applicable
9	Superior to present desirable criteria
8	Equal to present desirable criteria
7	Better than present minimum criteria
6	Equal to present minimum criteria
5	Somewhat better than minimum adequacy to tolerate being left in place as is
4	Meets minimum tolerable limits to be left in place as is
3	Basically intolerable requiring high priority of corrective action
2	Basically intolerable requiring high priority of replacement
1	This value of rating code not used
0	Bridge closed

rating is part of the bridge's functional classification; rating codes range from zero to nine. A zero rating indicates a bridge closed to traffic because the waterway is so inadequate that floodwater very frequently overtops the roadway. A nine rating means that the chance of floodwater overtopping the bridge roadway is so remote that it will not occur more than once in over 100 years.

<u>Approach Roadway Alignment Appraisal</u> evaluates how closely the roadway approaching the bridge is aligned with the general highway the bridge is on. The alignment appraisal rating is determined by comparing the posted speed on that highway section with the reduced speed a vehicle must assume to approach the bridge. Evaluation codes range from 3 or less for intolerable alignment to 8 for very close alignment.

Note that this evaluation differs from other appraisal evaluations which compare existing conditions with current standards.

<u>Scour Critical Bridge Appraisal</u> determines whether bridge pier foundations or abutments are or may become unstable because of scour (soil erosion around footings or piles). A bridge is considered to have problems if scour has been observed at the bridge site or if a scour evaluation study has determined a scour potential.

This evaluation must be conducted by a foundation or a hydraulic engineer. The rating codes for this appraisal are listed in Table 1.6. If the rating code is less than four, it will also be necessary to revise the substructure condition rating in order to include the impact of scour on the bridge condition.

Table 1.6 Scour Appraisal Ratings

Code	Description
N	Bridge not over waterway
9	Bridge foundations (including piles) well above flood water elevations.
8	Bridge foundations determined to be stable for calculated scour conditions; calculated scour is above top of footing. (Example A)
7	Countermeasures have been installed to correct a previously existing problem with scour. Bridge is no longer scour critical.
6	Scour calculation/evaluation has not been made. (Use only to describe case where bridge has not yet been evaluated for scour potential.)
5	Bridge foundations determined to be stable for calculated scour conditions; scour within limits of footing or piles. (Example B)
4	Bridge foundations determined to be stable for calculated scour conditions; field review indicates action is required to protect exposed foundations from effects of additional erosion and corrosion.
3	Bridge is scour critical; bridge foundations determined to be unstable for calculated scour conditions: - Scour within limits of footing or piles. (Example B) - Scour below spread-footing base or pile tips. (Example C)
2	Bridge is scour critical; field review indicates that extensive scour has occurred at bridge foundations. Immediate action is required to provide scour countermeasures.
1	Bridge is scour critical; field review indicates that failure of piers/abutments is imminent. Bridge is closed to traffic.
0	Bridge is scour critical. Bridge has failed and is closed to traffic.

SECTION 2
BRIDGE DEFICIENCIES AND DETERIORATION

Bridge deterioration increases maintenance costs and ultimately causes the traveler increased travel time and expense. In addition, bridge deficiencies increase the potential for vehicular accidents.

The leading cause of bridge deficiency in the United States is age. Many structures have passed or are approaching the limits of their service life.

This section discusses bridge deficiencies and deterioration: how they are classified, how they affect users, how they are caused by various environmental conditions, and how they are detected during inspection.

BRIDGE DEFICIENCY CLASSIFICATIONS

Bridge deficiencies are caused by age, environmental conditions, or functional demands which exceed the originally intended use of the design (such as traffic volume, speed limit, vehicle size and vehicle weight).

Bridge deficiencies are measured by determining the extent to which bridges fail to meet level-of-service goals or standards. The FHWA evaluates deficient bridges by considering

1. Condition ratings, which compare current bridge maintenance conditions with conditions when the bridge was new, and
2. Appraisal ratings, which compare the adequacy of the existing structure with a new bridge built to current standards for the particular highway being served (See Section 1).

Based on these ratings, the FHWA classifies deficient bridges as either <u>structurally deficient</u> or <u>functionally obsolete</u>.

The Structurally Deficient Bridge

Structural deficiencies develop because of wear, improper maintenance, poor initial design, inferior materials, and exposure to the elements. The FHWA classifies a bridge as structurally deficient if it has:

a) a condition rating of 4 or less for its deck, superstructure, culvert, or retaining walls, <u>or</u>
b) an appraisal rating of 2 or less for its structural condition or waterway adequacy.

The structurally deficient bridge is usually either restricted to light vehicles only, or requires immediate rehabilitation to remain open if it is not already closed to traffic.

The Functionally Obsolete Bridge

Functional obsolescence results from geometric deficiencies caused by increased traffic, changing traffic patterns, and increased safety standards. The functionally obsolete bridge no longer has adequate deck geometry, load carrying capacity, clearance, or approach roadway alignment. The FHWA classifies a bridge as functionally obsolete if it has:

a) an appraisal rating of 3 or less for its deck geometry, underclearance, or approach roadway alignment, <u>or</u>
b) an appraisal rating of 3 for its structural condition or waterway adequacy.

The effects of geometric deficiencies for a functionally obsolete bridge can be somewhat reduced by using warning signs, roadway striping, and crash cushions.

TYPES OF BRIDGE DEFICIENCIES

Deficiencies may be due to inadequancies of the original design versus today's standards, changed needs, or deterioration. Deficiencies in the following four bridge conditions are the most common causes of user costs due to delays, detours, and, in some cases, serious accidents.

1. Load capacity: Low bridge load capacity can be a result of low capacity members, from the loss of a load-carrying member or from material degradation causing reduced or lost load-carrying capacity of individual members (Figure 2.1). This deterioration is usually caused by corrosion, spalling, cracking, scouring, rotting, or infestation.

   A bridge over water is most likely to develop such deficiencies in its substructure. A bridge carrying high volume traffic will more likely suffer from deck and superstructure fatigue (See Section 3) and overloading. Load capacity deficiencies can also result from inferior materials and construction which accelerate deterioration.

2. Roadway Geometrics: Roadway geometric deficiencies include poor vertical and horizontal alignment, roadway width, vertical over and under clearance, and vehicle sight distance. These deficiencies are often due to limitations of the original design. As an example, vehicles taller than the vertical clearance of a bridge must detour. Most states limit vehicle height to 13.5 feet consistent with current needs of the trucking industry and try to provide higher clearance.

   Sometimes, however, changes in bridge use, such as increased traffic volume, cause geometric deficiencies to become critical. Bridges with narrow and/or poor approach alignment have higher accident potentials. Narrow bridges reduce traffic flow and sometimes require wide vehicles to detour. Curved bridges and approach roadways also increase safety hazards.

3. Channel Protection: Deficient channel protection may allow flowing water to undermine the

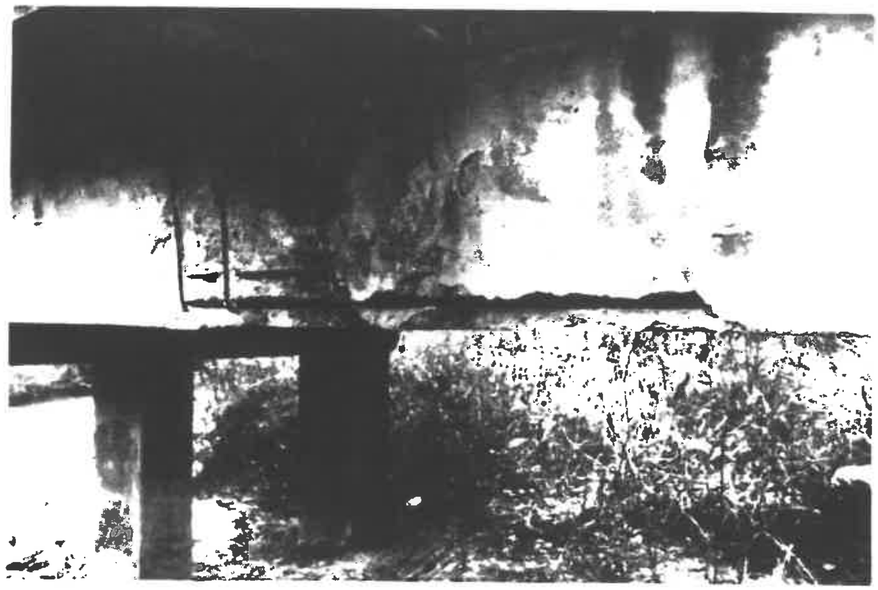


Figure 2.1 Material Degradation of Concrete Bridge Girder

slope protection for the bridge or erode of soil from around and under footings. Increased water velocity can even realign the stream course, presenting additional potential problems. Other channel protection deficiencies result from the accumulation of debris on the superstructure and substructure (Figure 2.2)

4. Drainage: Drainage deficiencies can result in damage to bridge structures, especially in colder climates. Poor deck drainage can contribute to deck disintegration, and superstructure bearing and pier cap corrosion. Therefore, it is essential to maintain an effective drainage system that carries trapped or ponded water away from the structure as quickly as possible.

It should be noted that even properly drained bridges can develop deck corrosion because drainage water frequently carries corrosive elements, especially salt, which attacks concrete and drainage pipes.

Figure 2.2 Debris Collected Against Bridge Substructure

HOW BRIDGE DEFICIENCIES AFFECT USERS

Bridge deficiencies most commonly cause detours, which cost users additional travel time and vehicle operating expense. In addition, bridge deficiencies decrease safety by increasing the possibility of bridge-related accidents or collapse.

Vehicular accident rates increase on narrow bridges (Figure 2.3) and those with poor approach alignment. The majority of accidents at bridge sites are caused by narrow bridge roadways and curved approach roadways. Curved bridges, bridge lane width reduction, shoulder reduction, roadside distraction, higher ADT and more larger trucks are other documented causes of bridge accidents. Accident rates are also increased by high speeds and undivided roadways.

Figure 2.3 Poor Approach Alignment

Bridge collapse, although fortunately rare, while vehicles are in transit commonly causes loss of life and property. In addition, users not directly involved in the collapse are severely affected by the resulting detours which frequently overload the alternate routes and bridges, causing additional delays and decreased safety.

CAUSES OF BRIDGE DETERIORATION

Adverse environmental and loading conditions cause deterioration of steel, concrete, timber, and stone masonry bridge members. Steel members can deteriorate because of air, moisture, industrial fumes, deicing agents, seawater, thermal strains, fatigue, and galvanic action. Concrete members can deteriorate because of freezing and thawing, salt action, differential thermal strains, unsound aggregates, sulfate compounds, leaching, chemical attack, wear, and abrasion. Timber members may deteriorate because of fungi, vermin, weathering, chemicals, fire, and abrasion or mechanical wear.

Underwater members are particularly subject to material and structural damage. In addition, flow of water can cause scour at bridge foundations.

Environmental Deterioration of Bridge Elements

Bridge elements start to deteriorate as soon as the bridge is open to traffic. Therefore, it is necessary to provide timely maintenance to prevent environmental deterioration that can develop gradually possibly leading to the collapse of the bridge.

A. Steel Elements:

1. Paint deterioration can quickly lead to severe damage to major steel elements. Regular paint maintenance is essential to combat the continual weathering and chemical exposure that steel bridge members are subject to. Spot failures can develop rapidly into large areas of corrosion (Figure 2.4) which, if left untreated over time, cause extensive, irreparable bridge damage.

   Paint deterioration occurs more rapidly in a damp environment. Because paint is the primary protection for structural steel against rust and corrosion, bridges in damp areas require more frequent paint maintenance than bridges in dry areas.

   Paint condition is also affected by paint type, quality of application, the configuration of structural details, and the amount of exposed area to be maintained.

2. Corrosion resulting from paint deterioration can cause a substantial loss of metal section areas. Such a loss can severely reduce the load-carrying capacity of the bridge member which can ultimately result in collapse of the member.

   Corrosion is accelerated by snow-laden chemical deicing agents coming in contact, either as splash, seepage or flow, with primary structural steel elements. Truss members are usually most severely affected.

3. Brittle fracture of a critical member is the sudden separation of the member into two parts (Figure 2.5). It can occur at relatively low stresses and collapse can occur without warning.

   The rapid loading of a steel member, such as the loading caused by a truck collision or an explosion, can impose enough energy to cause a steel member to fail in brittle fracture.

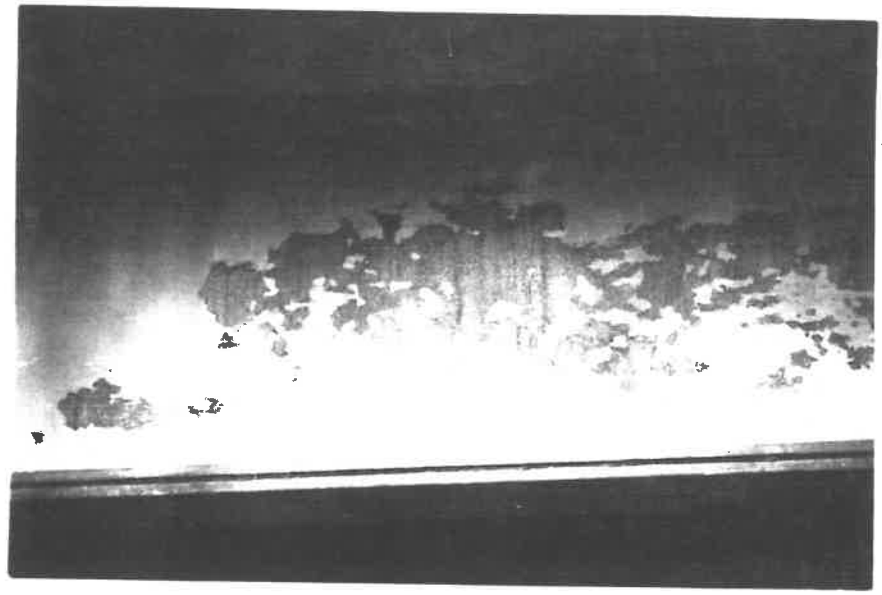


Figure 2.4 Paint Deterioration

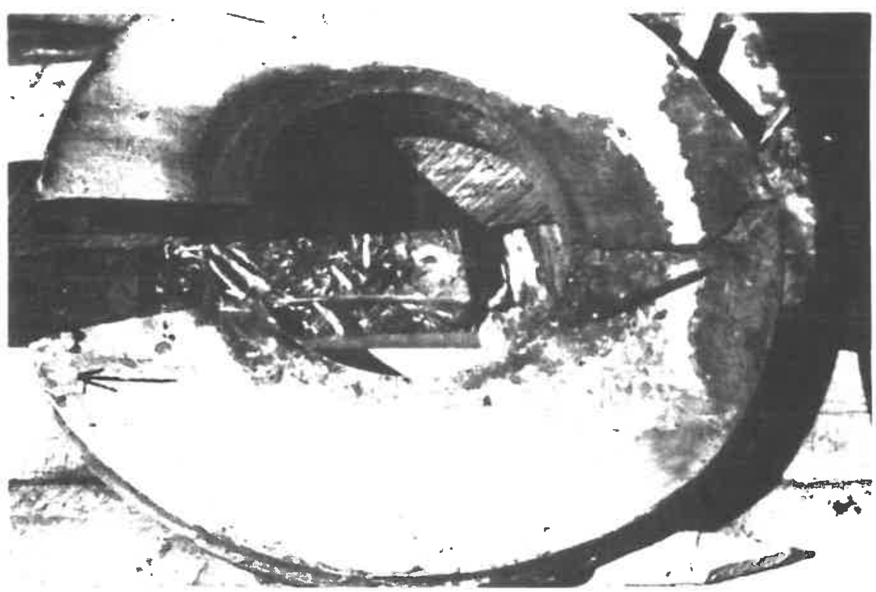


Figure 2.5 Cracked Eyebar from Silver Bridge Collapse

The adverse combination of cold temperatures, rapid loading, and stress concentrations increases the possibilities of brittle fracture. This fracture of the steel member is sudden and catastrophic.

Fracture toughness is the measure of the ability of a material to resist cracking; it is further defined as the ability to carry load and to absorb energy in the presence of cracks or to deform greatly without rupture. In summary, a brittle fracture can occur under one or more of the following conditions:

a. The temperature is low
b. The structure is rapidly loaded
c. High constraint

4. Slippery surfaces and rutting occur on steel grid flooring. Slippery surfaces occur on wet or frost covered open grids. Rutting is the "dishing out" of concrete between the grids of a filled grid floor.

B. Concrete Elements:

1. Scaling is the gradual and continuing loss of surface mortar and aggregate over an area (Figure 2.6). Scaling is classified as light, medium, heavy, or severe: Light scale is the loss of mortar less than 1/4 inch deep. Medium scale is the loss of mortar between 1/4 to 1/2 inch deep. Heavy scale is between 1/2 inch to one inch, and severe scale is greater than one inch. Scaling is caused mainly by improper mixing, placing, finishing, or curing of the concrete. Inadequate air content in the concrete also reduces the durability of the exposed concrete.

2. Spalling is the roughly circular or oval depressions in concrete caused mainly by the removal of a portion of the concrete element by a fracture parallel to or inclined to the surface (Figure 2.7).

Spalling is classified as (a) small spall, usually one inch deep or

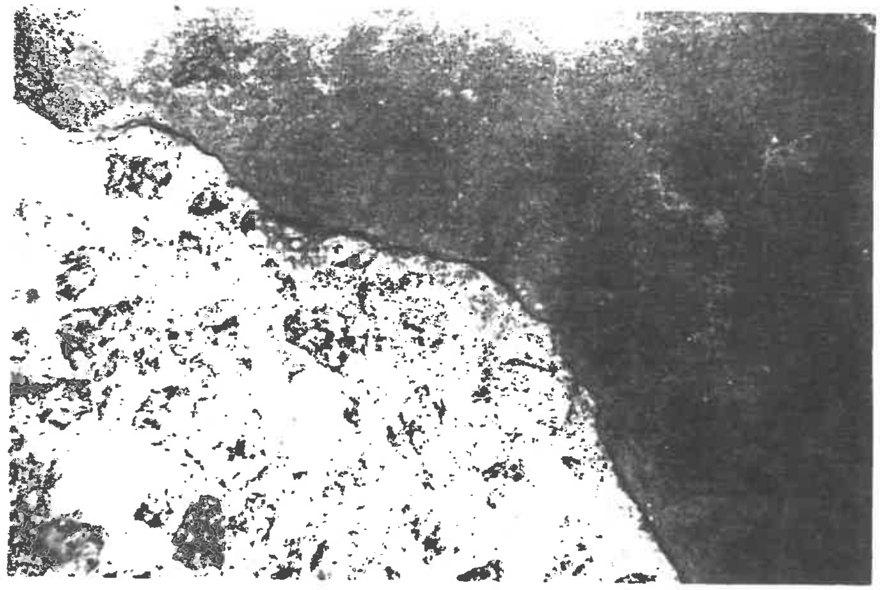


Figure 2.6 Scaling of Concrete Surface

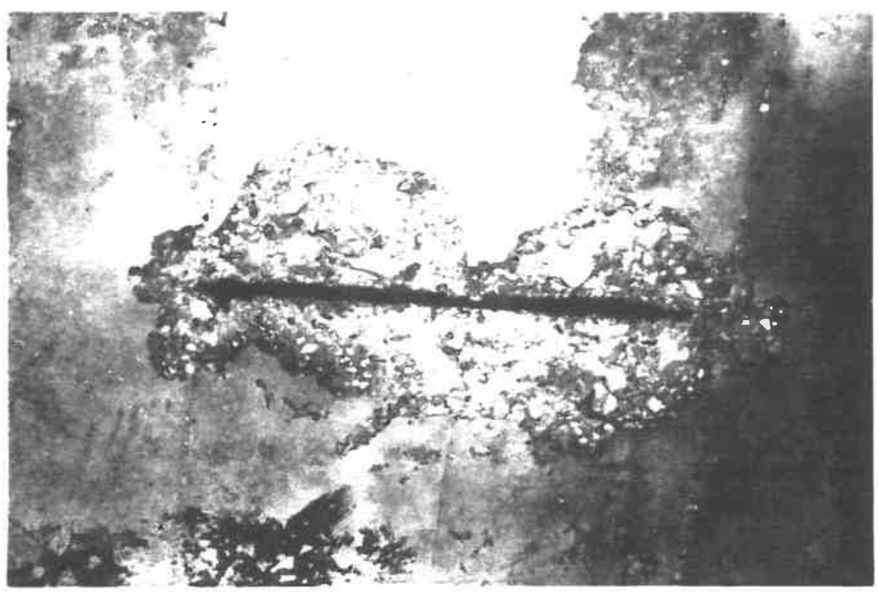


Figure 2.7 Spalling of Concrete Surface

approximately six inches in diameter, (b) large spall, deeper than one inch or more than six inches in diameter, or (c) hollow spall, a fracture in the concrete below the surface. Hollow spalls can be detected by striking a hammer on the surface of the concrete.

3. Cracking is a linear fracture in the concrete (Figure 2.8). Cracks may occur in different sizes, lengths, directions, locations, and types. Longitudinal cracks are straight cracks in the slab running parallel to the center line of the road. Longitudinal cracks are usually caused by shrinkage, settlement, deflection of the superstructure, and corrosion of the reinforcement.

   Transverse cracks are almost straight cracks that are roughly perpendicular to the center line of the roadway. Pattern or map cracks form networks of connected cracks of varying size and are caused by improper curing and rapid shrinkage of concrete. Random cracks have no form or direction.

   Plastic shrinkage cracks are shallow, fairly wide and usually short. They are randomly distributed over the member area. Their length ranges from few inches to many feet and are spaced a few inches to as much as several feet apart. These cracks occur during construction when there is a rapid loss of moisture in fresh concrete caused by low humidity, wind, and/or high temperature differential between the concrete and air. A relative volume change between the surface and the interior portion of the concrete is created leading to the formation of plastic cracks.

4. Corrosion of underlying reinforcement steel occurs when concrete members deteriorate in adverse environmental conditions. Brine solutions penetrate unprotected concrete surfaces, causing the reinforcement steel to expand and oxidize, which

Need General Photo of Longitudinal Concrete Cracking

Figure 2.8 Longitudinal Cracking

eventually leads to spalling and cracking.

Corrosion is common in bridge decks and in primary structural members. It results from high saline levels, salt water splash, or runoff from roadway joints and drainage details which is permitted to come in contact with concrete beams and girders for lengthy periods.

5. Delamination is the separation of concrete along a plane parallel to the surface of the concrete. As the reinforcing steel corrodes and expands, it exerts increasing pressure within the concrete which eventually becomes detached, resulting in a spall. Surface cracks may or may not appear, depending upon the amount of corrosion and the amount of cover given to the reinforcement.

   Delamination usually occurs on bridge decks and on the corners of concrete beams and is a good indication that corrosion is taking place in the reinforcement.

6. Wearing surface breakdown occurs on that portion of the roadway surface that is in direct contact with the vehicles passing over it. This surface is in particular subjected to abrading, crushing, or other disintegrating effects produced by the hammering, rolling, sliding, etc. of the vehicles.

C. Timber Elements:

1. Decay occurs when moisture penetrates treated or untreated wood members around nails, bolt holes, notches, cracks, and end grain (Figure 2.9). Moisture may also cause fungi decay, which can be detected by the stains or discoloration it causes. As fungal decay advances, the wood darkens and the surface becomes punky, soft and spongy, or even crumbly, all of which are obvious signs of disintegration.

Figure 2.9 Decayed Wood Bridge Member

2. Insect infestation is caused by termites, powder-post beetles, carpenter ants, and marine borers. Because termites damage the inside of a surface, termite-inflicted decay is impossible to observe externally. Therefore, timber members should be checked for excessive sagging or crushing, indications of possible termite damage. An ice pick or incremental borer (Figure 2.10) can also detect termite damage.

   Powder-post beetle damage is detected when the outer surface of the wood is poked with small holes, each containing powdery dust on its edge. Similarly, carpenter ants leave sawdust on the ground at the base of the timber.

   Marine borers usually cause the most severe damage on timber members located between high and low water. Where these insects are likely to appear, concrete and steel shielding is necessary. This shielding should be checked periodically for holes or cracks which the borers can easily penetrate.

3. Weathering is normally caused by dimensional changes in the wood due to repeated wetting. Weathering is classified as:

   a. Slight weathering, where the wood surface is rough and corrugated and possibly even warped.
   b. Advance weathering, where large cracks extend deeply or completely through the wood which is crumbly and deteriorated.

4. Mechanical wear is abrasion which can be detected by the gradual loss of section at the points of wear. Mechanical wear is increased by surface dirt or debris and by studded tires.

Figure 2.10 Wood Coring Tool

D. Stone Masonry:

There are three types of stone masonry deficiencies:

1. Weathering occurs when hard surfaces degenerate into small granules, giving stones a smooth, rounded look.
2. Spalling occurs when small pieces of rock break and fall away.
3. Splitting occurs when cracks open up in rocks, eventually breaking them into smaller pieces.

E. Bearings:

1. Metal bearings show deterioration (Figure 2.11) by having corroded and sometimes frozen rockers, pins, and rollers; loose bolts and missing nuts; or improper alignment.

2. Elastometric bearing pads are made of a rubber-like material that may deteriorate by cracking, splitting, tearing or bulging.

Deterioration of Underwater Bridge Elements

It is estimated that as many as half of all bridge failures are caused by the failure of underwater members. The following paragraphs explain how different underwater bridge materials deteriorate and which environmental forces are likely to cause this deterioration.

A. Material Damage:

1. Underwater steel members corrode by converting the iron and adjacent oxygen, through chemical or electrochemical reactions, into a compound form, rust. Rust easily flakes off the parent metal, reducing an area of the section.

   Moisture speeds the corrosion of steel members above the splash zone because of the excess oxygen available in this location. Corrosion of steel members within the splash

Need General Photo of Deteriorated Bearing

Figure 2.11 Deteriorated Metal Bearings

zone is accelerated by salt water and the repeated effects of wetting and drying. Submerged steel members corrode from the concentration of dissolved oxygen in the water.

Chemicals in contact with the steel surface also cause corrosion. That part of the steel member below the mud line or buried in the soil corrodes from electrical conductivity, high acidity, or the presence of dissimilar metals.

Corrosion of steel members can be identified by the characteristic reddish-brown rust color and by loose flakes of the oxides on the pitted surface. The rate of corrosion increases with increased water velocity which brings more oxygen into contact with the metal.

2. Submerged concrete members can deteriorate from a chemical process occurring in marine environments; however, it is difficult to determine the exact cause of this deterioration which is identified by the presence of cracks, spalls, and cavities. In addition, reinforcing bars corrode from the penetration of air and moisture through the concrete cover. Marine borers can also penetrate poor quality concrete.

3. Wood deterioration in submerged timber is mainly caused by marine borers (Figure 2.12). Marine borers are classified either as limnoria or teredo. The limnoria is a crustacean whose tunneling is limited to the outer shell of the wood. Limnoria activity can be identified by the lack of organisms on the surface of the pile or wood structure. Increased infestation eventually leads to mechanical erosion which reduces a section of the pile.

The teredo, also known as the shipworm, is a mollusk whose larvae begin tunneling on the surface, but eventually bore their way into the interior of the wood, leaving their tails behind at the surface. Teredo activity is difficult to identify

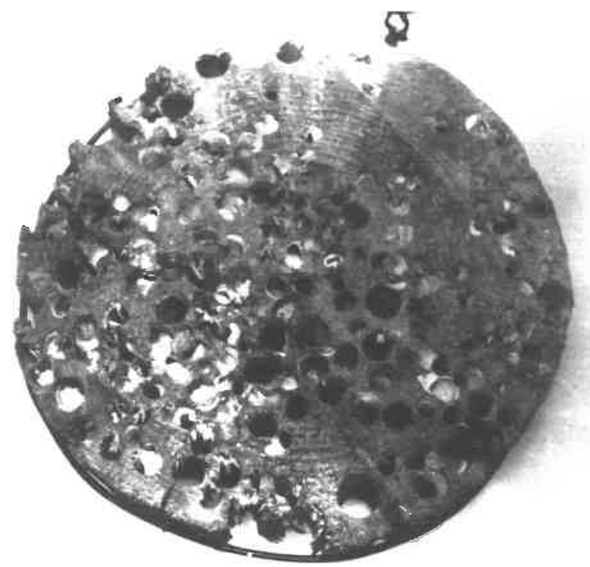


Figure 2.12 Marine Borer Damage

because most of it occurs in the internal wood structure.

B. Causes Of Structural Damage:

1. Storms increase environmental damage to underwater bridge structures. The power of a current to pick up and transport granular material from a channel or river bottom increases as water velocity increases. Thus, the erosive powers of the current increase during storms. In addition, the surface debris generated by storms exerts horizontal forces on the bridge substructure and increases the possibility of scour.

2. Erosion (also known as abrasion) is caused by the wave action of suspended particles of sand and silt. Deterioration from erosion can be identified by the worn, smooth surface of the bridge element. Over time, current velocity combined with suspended particles reduces the area of a bridge section. The deterioration from active erosion is detected by the depressions in the eroding area.

3. Collision damage ranges from substructure surface damage to total failure and destruction of the entire structure. Typical damage occurs as:

   a. Severe spalling on mass concrete footings
   b. Full depth cracks on concrete columns
   c. Crippled flanges on short steel batter piles or similar damage on long vertical piles
   d. Timber piles with severely abraded, splintered surfaces or piles that appear only slightly displaced, but are completely severed below the mud line.

4. Scour is the erosion of stream bed material from around and beneath bridge foundations caused by the action of running water (Figure 2.13). The settlement of piers, the misalignment of joints and bearings, and the binding of movable spans all

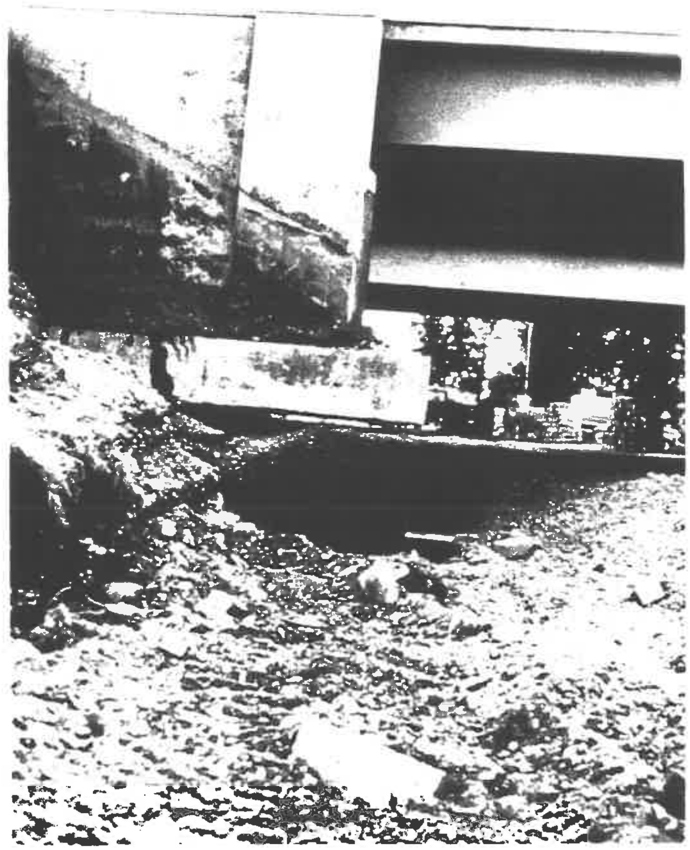


Figure 2.13 Scour at Abutment

indicate serious scour problems. Periodic inspection for scour problems is necessary for all bridges located over waterways. Three classifications of scour are:

a. Aggradation and degradation are long-term stream bed changes. Aggradation is the deposition of material eroded from other sections of a stream. Degradation is the lowering of a stream bed.

b. General scour and contraction scour is the removal of material from the bed and banks of a stream across all or most of the width of a channel. General scour results from an increase in water velocity, or changes in water surface elevation. When a contraction of the flow takes place, the general scour is known as contraction scour.

c. Local scour is the removal of material from the channel bed. Local and general scour differ in the extensiveness of material removal: local scour occurs only in a minor part of the width of the channel, while general scour occurs across all or most of the channel width. Local scour is caused by the acceleration of flow and the resulting vortices formed at the perimeter of the foundation elements.

Scour Detection

Scour problems are more critical during flood periods. Because of the dangerous conditions during these periods scour measurements should be done after the flood.

When the stream velocity decreases, sediment is redeposited in the scour hole. This is known as infilling. Since this material usually has a different density than the material in place, the extent of scour can be measured by determining the interface where the change in density occurs. Scour can be measured with different equipment as listed below:

1. Direct Measurement is accomplished by use of a sounding pole with channel elevations relative to the bridge superstructure or survey monuments.

2. The tuned transducer and video sounder are seismic systems which operate through the transmission and reception of acoustic waves. The black and white video sounder is easy to use and provides indications of depth and bottom profile. The color video sounder distinguishes between different interfaces by assigning them different colors. The data is displayed in a color monitor.

3. Ground penetrating radar (GPR) is used to obtain high resolution, and continuous subsurface profiles in water less than 25 feet deep. The system includes a transmitter, a receiver, a high density tape recorder and player for storage and an antenna.

Scour Mitigation

When an existing bridge is susceptible to scour problems the following actions can be followed to prevent or to mitigate the problem:

a) provide rip rap at piers and abutments,
b) construct channel improvements,
c) strengthen bridge foundations, and
d) construct relief bridges or lengthen existing bridges.

For new bridges it is important to:

a) locate the bridge to avoid adverse flood patterns
b) minimize obstructions to the flow,
c) locate foundations at an adequate depth,
d) not constrict the channel to avoid increase in water velocity, and
e) drive piles upstream.

INSPECTION EQUIPMENT, TOOLS AND TECHNIQUES

If the routine inspection either identifies problems or details that are prone to problems, follow-up special inspection with one or more of the following special tools is necessary.

In order to detect the different types of deficiencies and deterioration discussed previously a proper inspection of the bridge is required. The equipment for access and tools for inspection may vary depending on whether it is a routine inspection, a special inspection, or an underwater inspection. Access often requires ladders, boats, and snooper equipment. Underwater inspection requires a variety of diving equipment and often underwater video recorders and cleaning.

Inspection tools can be roughly divided into needs for routine and special inspections.

Routine Inspection Tools

Some of the equipment used in the routine two-year inspection is used as a check to recognize the need for a more in-depth investigation using more sophisticated equipment.

Visual inspection is the first test that is used. At arm's length, visual inspection is the single most important tool in spotting problems. Although quick it only gives information about the location of surface problems or deterioration. It is necessary to use other inspection techniques to find any internal problems in the members being inspected in order to take remedial measures to prevent their growth and possibly the failure of the member.

Equipment and tools used for visual inspection includes measuring tapes, magnifying glasses, wire brusites, flashlights, and inspection mirrors, scrapers, grinding equipment, and cleaning equipment.

Sonic methods are used to sound concrete and timber members. Chain drags, hammers, and sounding rods are used to tap the member. A hollow sound is an indication of subsurface problems.

Coring is also used to detect and measure voids in timber member and piles.

Pointed probe resistance is used to estimate the quality of timber members.

Rebound and penetration tests measure the hardness of concrete and predict concrete strength.

Pachometers are used to determine the location of reinforcing steel in concrete members.

Special Inspection Tools

Ultrasonic techniques are used to detect cracks, slags, laminations, pores or gas pockets, and incomplete weld penetrations that are not visible to the naked eye. It is also used to measure the member thickness.

Nuclear methods are used to measure the moisture content in concrete and help determine if corrosion of the reinforcing steel is likely to occur.

Magnetic methods are used to determine the position of reinforcing steel in concrete members and to locate surface cracks in steel members.

Dye penetrants are used to find surface cracks in steel members.

Thermography is used to detect delamination in concrete members by measuring the differences in surface temperature.

Radiograpy uses X-rays to locate surface and subsurface defects in concrete and steel members. Such defects include cracks, incomplete weld fusion, slags and other inclusions, pores, and gas pockets.

Chemical tests are used to determine if the concrete has an excessive amount of chlorides that can corrode the reinforcing steel.

Voltage potential readings indicate if the reinforcing steel in concrete members is corroded.

Computer assisted tomography (CAT) scanning uses a nuclear source to develop a cross sectional view of the member. Therefore, deterioration and cracks can be found.

Detection Of Fatigue Failure

Proper inspection of bridge members can prevent failure that could lead to the collapse of part or all of the structure. If during inspection a crack is found in any fracture critical bridge member, immediate action should be taken to correct the problem and prevent a failure. When a fatigue crack is discovered by visual inspection the service life of the member has been already reduced by at least 80%. Therefore, it is necessary to use special methods and equipment in order to identify the problem at an early stage. This makes inspection of fracture critical members expensive and expertise of the inspector is required.

Some of the methods used to detect fatigue problems are outlined next. More complete information can be found in "Inspection of Fracture Critical Bridge Members, Supplement to the Bridge Inspector's Training Manual" (FHWA, 1986).

The dye penetrant test is useful to find cracks that are open at the surface (Figure 2.14). The following steps need to be followed:

a) Clean the surface of the member being inspected.
b) Apply an oil-based liquid to the surface of the member.

Figure 2.14 Dye Penetrant Test

c) After a period of time the excess of liquid on the surface is cleaned.
d) A developing agent is applied on the surface to detect existing cracks easier.

Ultrasonic testing is used to detect surface and internal defects in metals (Figure 2.15). It can also be used to measure the thickness of a piece of metal in order to determine the amount of loss due to corrosion. This method of testing uses high-frequency waves introduced by means of a transducer. Knowing the properties of the wave and the material tested helps determine the distance to the flaw and the length of the crack.

Magnetic particle testing induces a magnetic field in the steel member by means of a power source (Figure 2.16). Magnetic particles sprayed on the surface arrange themselves into recognizable patterns at cracks which cause discontinuation in the magnetic field. It is used to detect potential flaws in steel members.

Acoustic emissions (AE) is the measure of the energy released when the material deforms or fractures under load. Basically it consists of listening to the crack growth under load. AE can only locate the general area of the defect. Another method is needed to pinpoint and quantify the cracks.

Radiographic inspection involves the application of x-rays to the member (Figure 2.17). It is primarily used to detect internal cracks, incomplete weld penetration and gas pockets.

Acoustic Crack Detector (ACD) consists of a backpack and a handheld probe. The ACD will detect cracks as small as 1/4-inch long.

The Magnetic crack definers (MCD) indicate the limits of a crack that has been detected by other methods such as the ACD.

Figure 2.15 Ultrasonic Testing

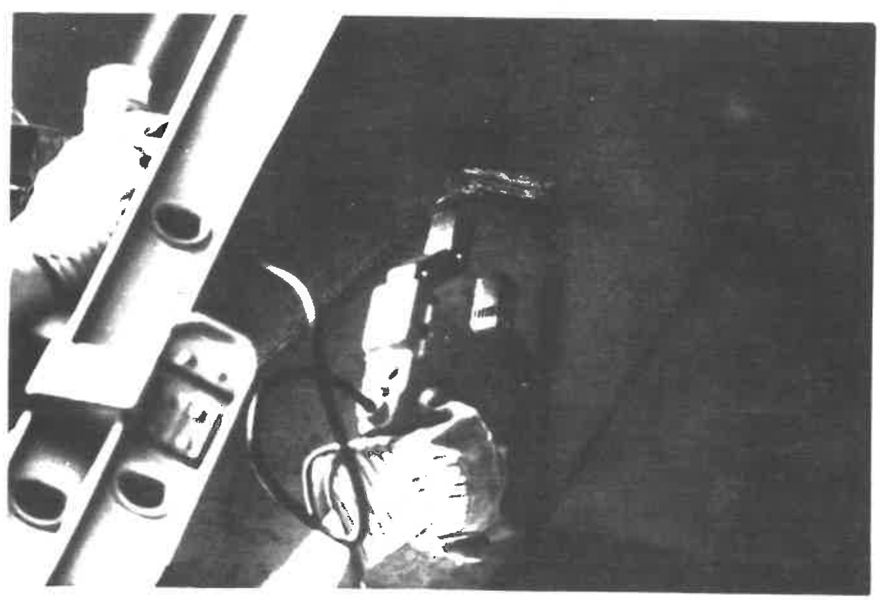


Figure 2.16 Magnetic Particle Test

Defects and Corrective Measures

In order to protect bridge structures against fatigue failure, it is important to design bridges that

a) avoid details which produce high stress concentrations or poor stress distribution,
b) avoid making attachments on parts subject to severe fatigue loadings,
c) avoid details which introduce high localized constraints,
d) avoid abrupt changes in cross section, and
e) keep eccentricities to a minimum.

Fatigue cracks are repaired according to the size and location of the crack. Very large cracks may require replacing the entire bridge member. If repairs are possible, they may be accomplished by adding bracing to redistribute the loads or by adding bolted splice plates. Additional fatigue crack repairs are made by

a) grinding,
b) peening,
c) drilling holes at the crack ends to reduce high stress concentrations,
d) replacing rivets with high strength bolts, or
e) rewelding.

It is necessary to use continuous, rather than intermittent, welds and to use welding procedures that will eliminate

a) gas pockets,
b) gas inclusions,
c) incomplete penetration,
d) undercutting, and
e) weld spatter.

Finally, a satisfactory paint system should be used to protect against corrosion fatigue.

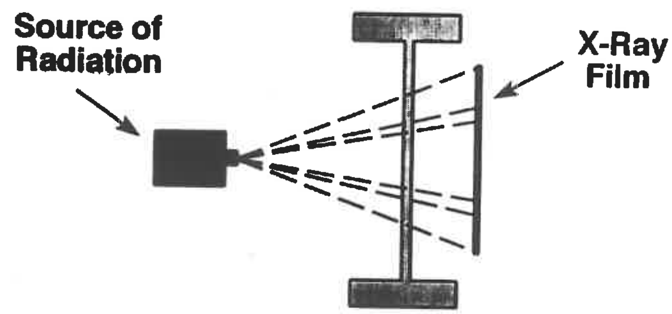


Figure 2.17 Radiographic Inspection

SECTION 3
FORCES AND MATERIAL PROPERTIES AFFECTING BRIDGE STRENGTH

The manager of a sound bridge maintenance program must understand basic bridge design and the forces seeking to undermine bridge stability. Accordingly, this section discusses types of vehicle loadings, basic affects of loadings on stresses in numbers and connections, fundamentals of material strength, the effect of fatigue on strength and how fatigue failure is avoided.

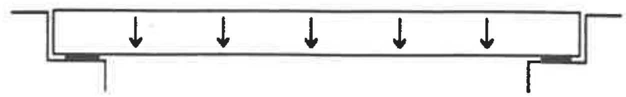


Figure 3.1 Dead Load on Simple Beam

HOW BRIDGE STRENGTH IS AFFECTED BY TRUCK CONFIGURATIONS

Bridges support dead, live, and other loads. The dead load (Figure 3.1) is the weight of the complete structure including roadway, sidewalks, pipes, conduits, cables, and other utility services. The live load (Figure 3.2) is the weight of the applied moving load of vehicles and pedestrians and is the primary subject of this discussion. Other loads include effects of earthquakes, water, wind, ice, etc.

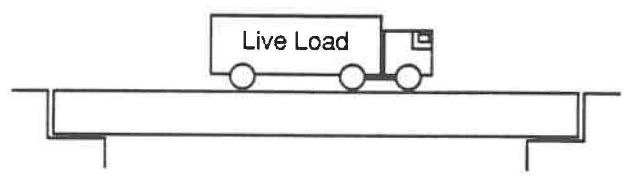


Figure 3.2 Live Load on Simple Beam

Bridge Strength and Load Classifications

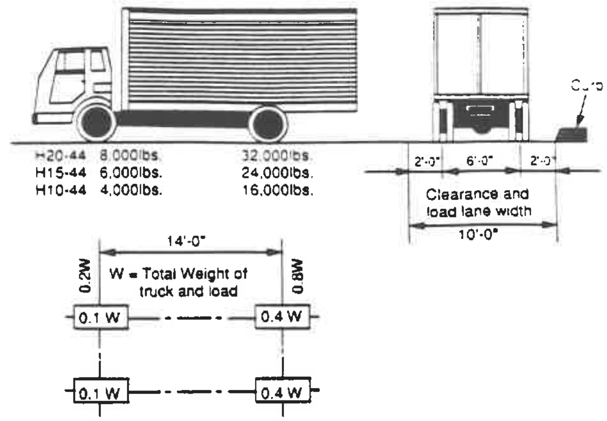
Because of the vast differences in bridge types and vehicle types, it is impossible to develop exact design criteria for every situation. Therefore, engineers have defined a set of hypothetical vehicles to determine bridge load carrying capacities. These vehicles represent all traffic that is expected to use the bridge during its 40-60 year service life. Assuming that the live loads of these hypothetical vehicles represent the most extreme stresses bridges will have to support, bridge members are designed and built strongly enough to withstand these loads with a factor of safety. Thus, basic bridge design incorporates protection from overstress. For an existing bridge, the load capacity is determined as the total weight of the hypothetical vehicle that can safely use the bridge.

Standard Truck Loads

These hypothetical design vehicles are divided into standard truck classifications according to weight and the number of axles (Figs. 3.3 and 3.4). Vehicle classifications H-10, 15, and 20 (Figure 3.3) designate two-axle truck loads with gross weights of 10, 15 and 20 tons, respectively. Classifications beginning with HS (Figure 3.4) represent the combined gross weight of the first two axle groups of a tractor trailer load. Classifications HS-15 and HS-20 indicate tractor-trailer trucks having combined first two axle group weights of 15 and 20 tons respectively. Additional load is applied at the following trailer axle(s). Thus, an HS-20 truck, representing a 36 ton tractor-trailer or equivalent series load, actually weighs more than twice as much as an H-15 truck. The final digits in the designations, i.e. 44, indicates the year that the loading was adopted by the American Associates of State Highway and Transportation Officials (AASHTO) in the Standard Specifications for Highway Bridges.

Lane Loads

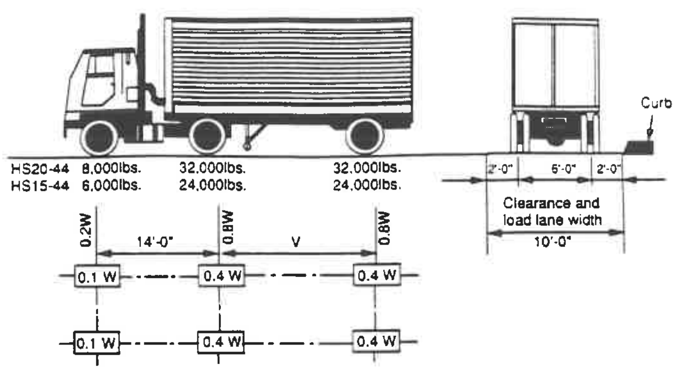
Designated H- or HS- loads (Figure 3.5), these include a number indicating the gross weight in tons of the first two axles of a standard truck or of a truck train. The uniform lane load represents a series of vehicles in line simultaneously using a long bridge. Each lane load consists of a uniform load per linear foot of traffic lane combined with a single concentrated load (or, in the case of continuous spans, two concentrated loads) placed on the span to produce maximum stress.



Type of Truck Configuration*			Weight at Truck Tractor		Weight at Trailer Rear Axles		Total Gross Weight	
A	B	C	Tons	Pounds	Tons	Pounds	Tons	Pounds
H	10	-44	10	20,000	N/A		10	20,000
H	15	-44	15	30,000	N/A		15	30,000
H	20	-44	20	40,000	N/A		20	40,000

* A: letter assigned to truck configuration
B: Number of tons for the truck axles
C: Year truck configurations were adopted

Figure 3.3 Standard H Trucks



W = Combined weight on the first two axles which is the same as for the corresponding H (M) truck.
V = Variable spacing - 14 feet to 30 feet inclusive. Spacing to be used is that which produces maximum stresses.

Type of Truck Configuration*			Weight at Truck Tractor		Weight at Trailer Rear Axles		Total Gross Weight	
A	B	C	Tons	Pounds	Tons	Pounds	Tons	Pounds
HS	15	-44	15	30,000	12	24,000	27	54,000
HS	20	-44	20	40,000	16	32,000	36	72,000

* A: letter assigned to truck configuration
B: Number of tons for the truck axles
C: Year truck configurations were adopted

Figure 3.4 Standard HS Trucks

Both uniform and concentrated loads are considered evenly distributed over a 10-foot lane width.

Design Loads

Secondary Roadway Bridges have often been designed for HS-15-44, the minimum live load for heavy truck traffic, although many bridges have less strength. However, many secondary bridges are now designed for HS-20-44 and increasingly agencies are utilizing this policy.

Primary Highway Bridges are usually designed for HS-20-44 loading. Interstate bridges are designed either for HS-20-44 or for an alternate military loading of two axles four feet apart with each axle weighing 24,000 lbs, whichever loading produces the greater stress. The states sometime also design for particular truck configurations which may be significant for the industries of particular states. These loadings are available from the individual state transportation agencies.

It is very important that bridge strength not be exceeded by applied loading. Bridge posting safeguards against such overloading. The bridge design strength classifications also help determine bridge posting policies (See Section 4). Bridges are posted with signs which indicate the total gross weight in tons of individual vehicles that may safely use the bridge.

Permit Loads

Legal weight limits exist to protect the country's highway and bridge network from the excessive weight sometimes imposed by trucks, construction equipment, etc. The allowable weight, height, width, and length for specific vehicles vary among the states which have defined individual legal limit regulations. Some states grant extensions of legal limits

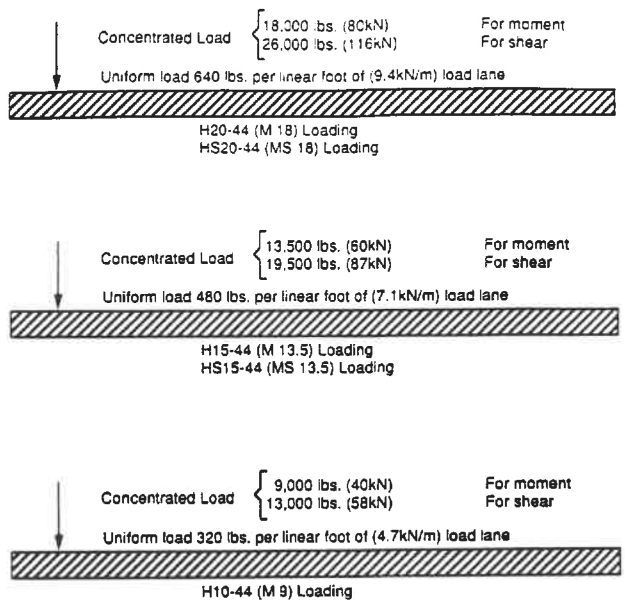


Figure 3.5 Lane Loads

(overlimit permits) to military, agricultural, or local industrial users. Additional weight limit variations are based on seasonal conditions (See Section 4).

THE VEHICLE GROSS WEIGHT FORMULA

Since initial weight limits were adopted in 1944, larger and heavier trucks have gone into service with a variety of axle arrangements to distribute the load. As a result, axle spacing has become just as important to bridge safety as axle weight. Bridges can be overstressed even when the gross weight and individual axle weights of a given vehicle are within legal limits. Consequently, in 1974 a Vehicle Gross Weight Formula was adopted assuring that the axle spacing of heavy trucks now in use correlates with the allowable weight limits and axle patterns adopted in 1944.

The system of hypothetical design vehicles adopted in 1944 includes specific axle weights and spacings applied as an umbrella loading (Figure 3.6). For example, a bridge can support a given weight much easier when the load (Figure 3.7) is spread over a large area (such as the load of a long truck) than when it is concentrated in a small area (the load of a smaller, shorter truck). The Vehicle Gross Weight Formula helps to assure bridge safety by determining whether a given vehicle conforms to this design umbrella.

The Vehicle Gross Weight Formula is:

$$W(\text{max.}) = 500\frac{LxN}{N-1} + 12\,N + 36,$$

where W is the maximum weight in pounds that can be carried on a group of two or more axles to the nearest 500 pounds; L is the spacing in feet between the outer axles of any two or more

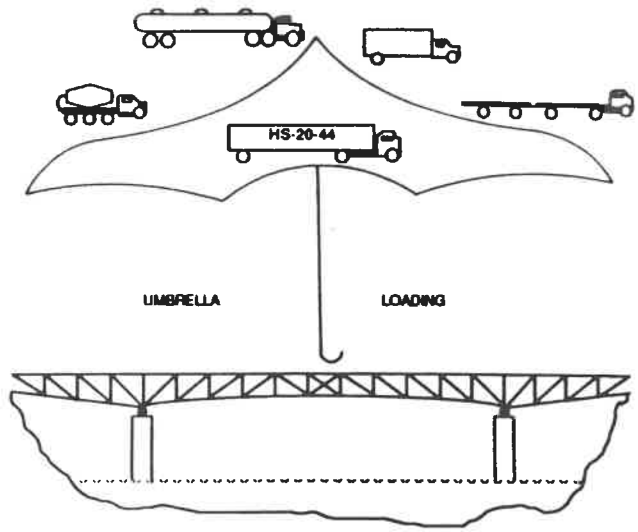
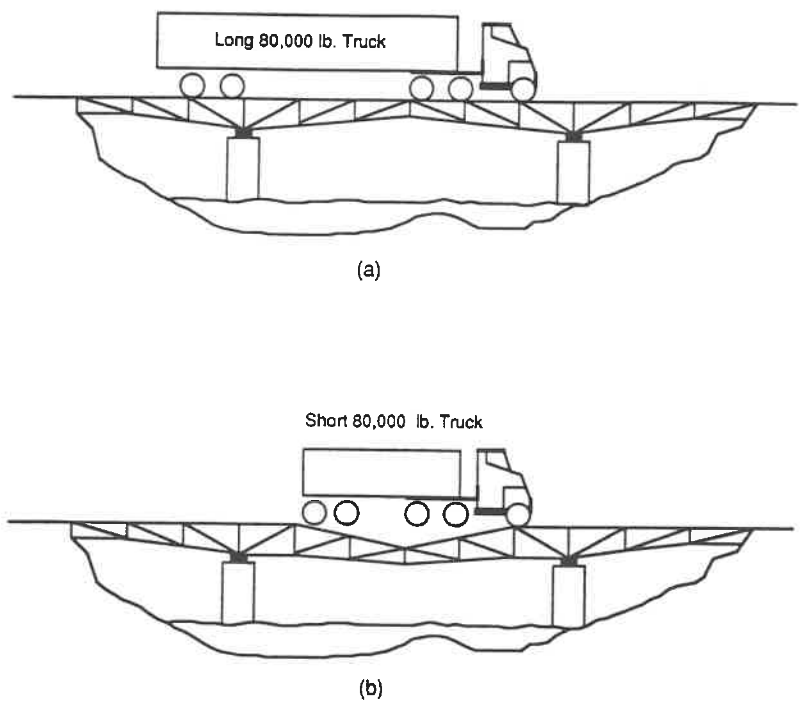


Figure 3.6 Umbrella Loading



(a)

(b)

Figure 3.7 Effect of Distribution

consecutive axles; and N is the number of axles being considered.

The following terms and limits must be defined in order to understand the bridge formula:

Gross weight is the weight of the vehicle and/or vehicle combination without load plus the weight of any load thereon. The Federal Interstate Highway gross weight limit is 80,000 pounds.

Single axle weight is the total weight transmitted by all wheels whose centers may be included between two parallel transverse vertical planes 40 inches apart, extending across the full width of the vehicle. The Federal Interstate Highway single axle weight limit is 20,000 pounds.

Tandem axle weight is the total weight transmitted to the road by two or more consecutive axles whose centers may be included between parallel vertical planes spaced more than 40, but not more than 96 inches apart, extending across the full width of the vehicle. The Federal Interstate Highway tandem axle weight limit is 34,000 pounds.

The Federal Gross Weight Formula is not required in states which have retained the following maximum weight limits for Federal-aid bridges: 18,000 pounds -- single axle, 32,000 pounds -- tandem axles, and 73,280 pounds -- gross weight.

Comprehensive studies are currently being conducted on the Vehicle Gross Weight Formula to determine issues such as: 1) Elimination of the existing grandfather provisions which allow higher axle loads and gross vehicle weights than the 20,000-pound single axle load limit, and 80,000-pound gross vehicle weight limit maximums authorized by the Federal-aid Highway Amendments of 1974 including permits for divisible loads and statutory specification 2) Analysis of alternative methods for determining the gross vehicle weight limit and axle loadings for all types of motor carrier vehicles 3) Analysis of the bridge formula in view of current vehicle configurations, pavement and bridge stresses in accord with the 1986 design and construction practices, and existing bridges on and off the Interstate system 4) Establishment of uniform 'reasonable access' provisions in the national network for combination vehicles established by the Surface Transportation Assistant Act of 1982 5) Recommendations for appropriate treatment for specialized hauling vehicles not complying with existing federal bridge formulas.

HOW OTHER LOADINGS AFFECT BRIDGE DESIGN

Bridges are also subject to other loading forces such as impact, earthquake, wind, thermal change, uplift, stream currents, and buoyancy. As examples, impact, thermal change, earthquake, and wind loadings are briefly discussed here.

Impact is the magnification of the load being applied dynamically. To account for impact, live load stresses produced by H- or HS- loadings are increased by a calculated percentage during design.

Thermal Change causes member expansion with a temperature increase or contraction with a decrease. If expansion or contraction is restrained, internal forces are induced. To avoid these forces, some members such as girders are supported on bearings that allow free longitudinal expansion of the bridge.

Earthquake motions generate inertia forces within the bridge structure. For design purposes, these effects are often represented by applying equivalent static loadings. Horizontal motions during earthquakes must be limited by design features which allow thermal movement but prevent extreme movement that would cause collapse. The possibility of differential lateral and vertical foundation movement due to shifting earth masses must also be considered.

Wind loadings are incorporated into bridge design by applying loads per square foot of exposed area. The exposed area is the sum of the areas of all members, including the floor system and railings, that are perpendicular to the wind forces. For design purposes, wind velocities of 100 mph are used. Higher velocities may be appropriate at some sites. The applied loads vary with the configuration of the superstructure and the substructure. Predetermined loads are available from tables in the AASHTO Standard Specifications for Highway Bridges.

HOW LOADS AFFECT IMPORTANT BRIDGE MEMBERS

The strength of structural bridge members is influenced by stresses, the effects induced by various forces. Stress is defined as the load per cross section unit area. The traditional units of stress are stated in pounds per square inch (psi) or kgs per sq. in. (ksi) where a k, p equate 1000 lbs. In SI system units, stress is expressed in Pascalo (Pa) or Newtons/sq. meter (N/m^2).

Different forces or loads (Figure 3.8) produce the following stresses which can be determined by engineering calculation:

Tension (Figure 3.9) elongates the bridge member. Tension is primarily induced by axial forces.

Compression (Figure 3.9) shortens the bridge member. Like tension, compression is primarily caused by axial forces.

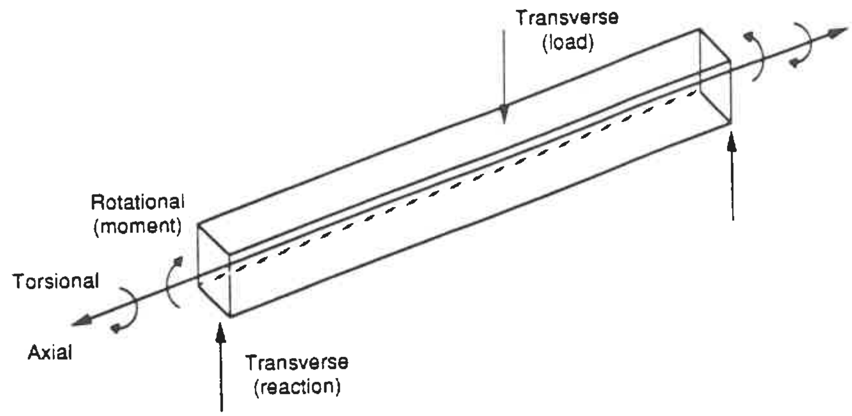


Figure 3.8 Loads Acting on Members

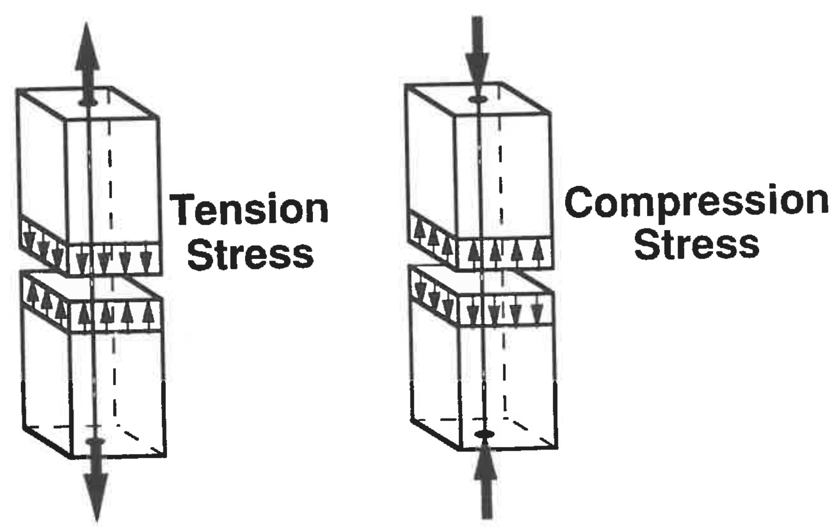


Figure 3.9 Tension and Compression Members

Bending (Figure 3.10) deforms the bridge member into a curved shape and is caused by a rotational force or moment, which is generally produced by a gravity load. Bending stresses are also called flexural stresses.

Shear (Figure 3.11) is a scissor-like force action transversely within members. Shear is induced by transverse, or crosswise, forces.

The Stress-Strain Relationship

Stress produces strain, a change in length per unit length. Figure 3.12 shows the relationships between stress and strain for concrete and steel, two of the most commonly used bridge materials. Other stress-strain relationships can be similarly represented. Figure 3.12 reveals the following important stress-strain characteristics of steel:

The elastic limit is that amount of stress at which deformation begins to increase at a faster rate than the increments of the applied load.

The elastic region is the range before the elastic limit in which deformation is directly proportional to stress. Within this range the bridge member will return to its original length and shape when the load is removed.

The permanent set is the permanent lengthening or deformation caused when the elastic limit is exceeded.

The modulus of elasticity is the ratio of unit stress to unit strain in the elastic region. It is a measure of the degree of stiffness of the material.

The yield point is the stress level at or beyond the elastic limit at which there is a significant increase in deformation without an increase in stress. As plotted on the curve in

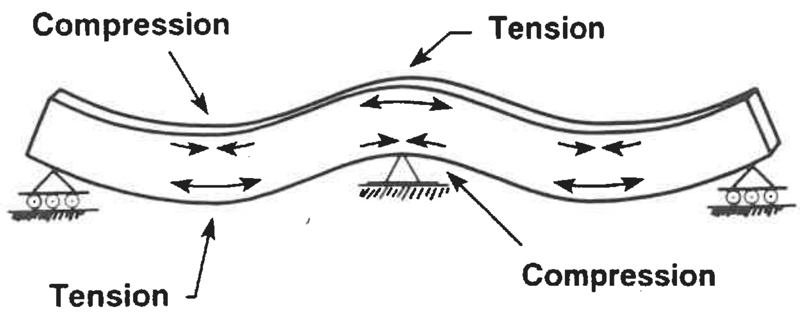


Figure 3.10 Bending of a Continuous Beam

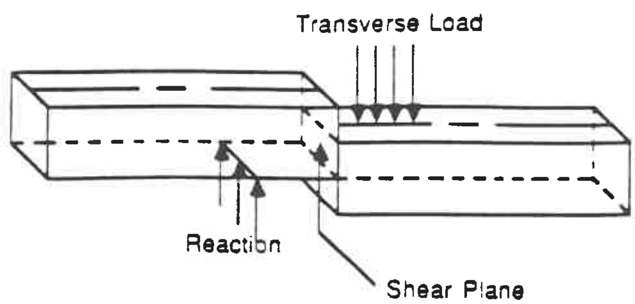


Figure 3.11 Shear Stresses

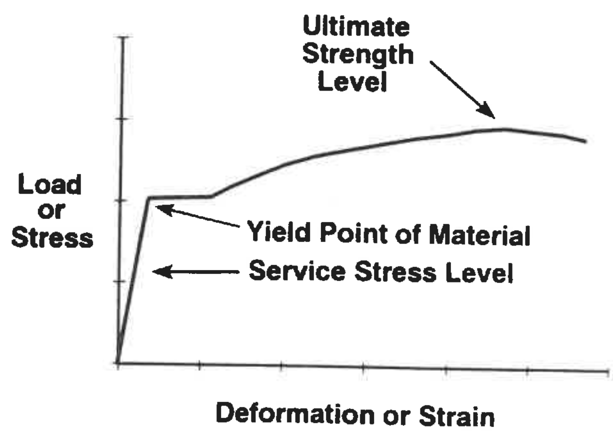


Figure 3.12 Stress vs. Strain Diagram

Figure 3.12, the yield region is almost horizontal for a short distance.

The ultimate strength is the amount of stress occurring at or just before rupture.

The Effects of Stress on Tension Members

Tension members (Figure 3.13) are bridge members normally subjected to tension, such as some truss members, cables, and eyebars. Most bridge tension members are constructed of steel because concrete has very weak tensile strength. Concrete tensile strength is only about 10% of its compressive strength. Therefore, when concrete members are subjected to any kind of tensile stresses, internal steel reinforcement is used to carry these tensile stresses.

Tension members and tension zones in bending members are susceptible to fatigue failure. Cracks are of concern in these members because tension stresses try to pull the member apart making cracks grow and eventually leading to failure. Some of these members are called fracture critical members (FCM) and they will cause a sudden collapse of part or all of the bridge if they fail. Suspension cables, hangers, some truss members, single box girder, tie girders of tied arches, cross girders and steel pier caps supporting the end reactions of two longitudinal spans are fracture critical.

The major concern in evaluating tension members is selecting a section with sufficient area to carry the load without exceeding the allowable tensile stress. To determine the allowable tensile strength for bolted or riveted connections in structural steel, net section area must also be considered.

A member with holes or openings for rivets or bolts will be weaker in tension than a solid member. Therefore, it is necessary to consider the actual effective area of the member, subtracting holes, when evaluating tension members.

The Effects of Stress on Compression Members

Compression members (Figure 3.13) are those subjected primarily to compressive forces. Compression members include piles, columns, arches, piers, and bents and can be constructed of concrete, steel, timber, or masonry.

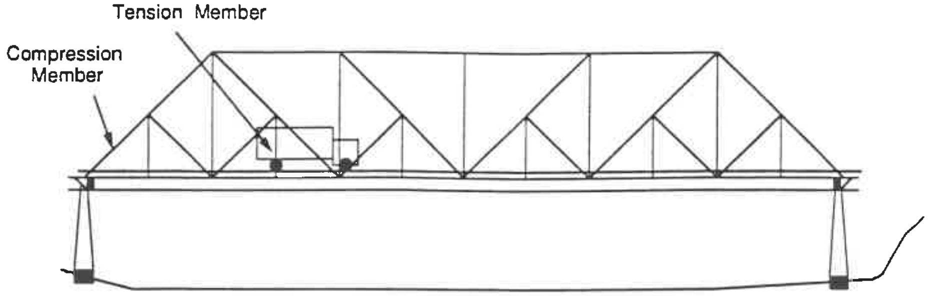


Figure 3.13 Example Tension and Compression Members

Compression members are susceptible to buckling (Figure 3.14) which is when the member bends suddenly out of its plane decreasing its load carrying capacity. Because of buckling, allowable stresses in compressive members are often lower than in tension members. Slender and long unbraced members are more susceptible to buckling.

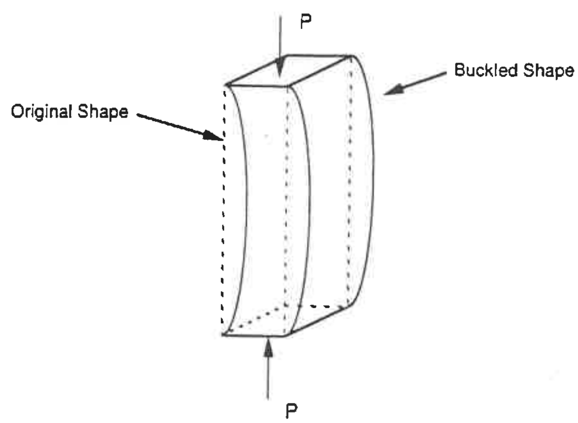


Figure 3.14 Compression Buckling

The Effects of Stress on Shear Critical Members

Shear critical members are subjected to transverse scissor-like forces. Beams, girders, and stringers can be shear critical members under certain circumstances where bridge elements require deep sections and short spans such as for cap beams. Shear critical members can be constructed of steel, concrete or timber.

When a transverse load is applied to a bridge member, both horizontal and vertical shear stresses of equal magnitude will occur (Figure 3.11). The most critical shear sections are near the supports of the member where large shear forces accumulate. Shear stresses may cause cracks to grow and eventually lead to failure of the member.

The Effects of Stress on Flexural Critical Members

Flexural members are subjected to bending forces. Typical flexural members are beams, girders, or other main supporting members; these can be constructed of steel, concrete, or timber.

Stresses are greater at top and bottom beam surfaces and decrease linearly to zero at the neutral axis. Thus, a simple supported beam will have compressive stresses above the neutral axis and tensile stresses below the neutral axis, produced by a positive moment (Figure 3.10). If the beam requires intermediate supports, in addition to the positive moment, there will also be a negative moment (Figure 3.10) causing tension to the upper and compression to the lower parts of the beam at and near the intermediate supports. It is necessary to use steel elements or reinforcement in sections affected by tension because concrete has a low tensile carrying capacity.

BRIDGE MEMBER TO MEMBER CONNECTIONS

Several types of connections exist in bridge construction to link steel, concrete, and wood structural members.

Many connections are accomplished by direct bearing of one member on top of another. This approach is simple and highly reliable. However, many other connections must rely on more complex modes of load transfer involving tension or shear.

The design of those connections is in many ways more difficult than the design of the structural members because the behavior of the connections is often so complex that it defies exact mathematical formulation. Consequently, connection design has sometimes been inadequate, resulting in weak structural links and in the decreased ability of the overall structure to withstand the loads imposed.

Connection elements and types include:

- bolts (Figure 3.15)
- rivets (Figure 3.16)
- pins (Figure 3.17)
- welds (Figure 3.18)

Connection fastening usually involves bolts, rivets, pins, welds, studs, and wedge or mechanical anchors. These are used in conjunction with:

- hangers
- eyebars
- quart plates
- splice plates
- clip angles
- cover plates
- cables and sockets
- split rings
- shear plates
- spike grids
- rebar mechanical couplers
- post-tensing anchors
- etc.

to produce a variety on connections as illustrated in Figures 3.15-3.20.

Generally, the welded connection is simpler and more compact because it avoids gusset plates, framing elements, and the holes drilled for rivets or bolts. However, welded connections must be designed and executed carefully to avoid fatigue problems.

FATIGUE

Fatigue is the process of progressive permanent internal structural damage in a material subjected to repetitive cyclic stresses. If this cumulative damage is significant, it may lead to complete fracture, as was dramatically illustrated by the 1967 collapse of the Silver Bridge between Ohio and Point Pleasant, West Virginia.

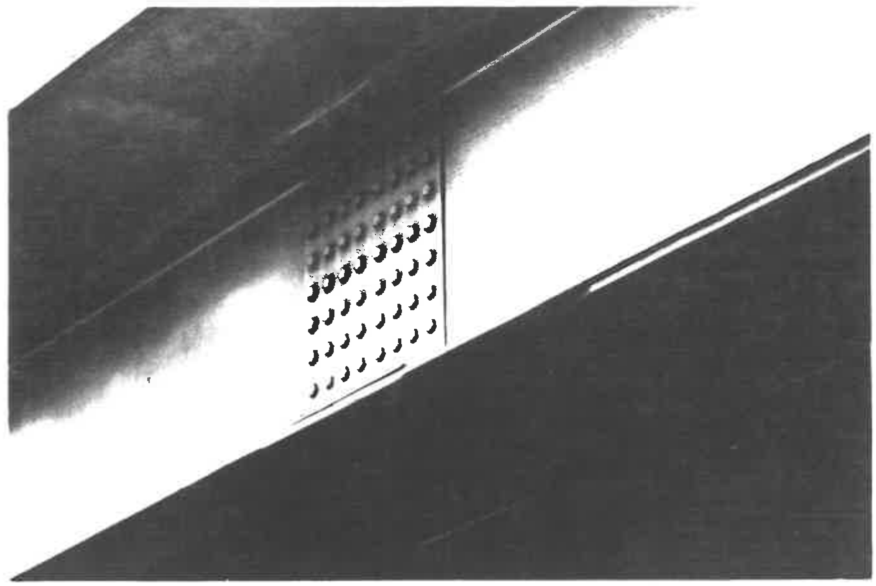


Figure 3.15 Bolted Connection

Figure 3.16 Riveted Connection

Presented here is a discussion of static and fatigue strengths, elements influencing fatigue, the static and fatigue behavior of different bridge materials, and specific fatigue problems illustrated by actual case bridges. Finally, some corrective measures are recommended to reduce fatigue.

Static Strength

Static strength is the maximum load capacity of a material under static loading, those loads which remain essentially constant with time. Static strength influences bridge design as the critical condition of the worst possible combination of loads that can occur simultaneously during the life of the structure.

Fatigue Strength

Fatigue strength is the stress at which the material fails by fatigue. Bridge fatigue strength is influenced by load history, material properties, environmental conditions, and by the stress range, rate, and eccentricity of loading.

Fatigue Loading

Fatigue loading is repetitive loading, or loads that vary with time. This sequence of variable load repetitions may cause bridge failure after a certain number of cycles even though the static strength has not been exceeded. An increase in fatigue loading will decrease the structural life of a bridge, and a decrease in fatigue loading will increase the life of the structure. Therefore, the effects of fatigue loading are measured by the remaining safe fatigue life of the bridge.

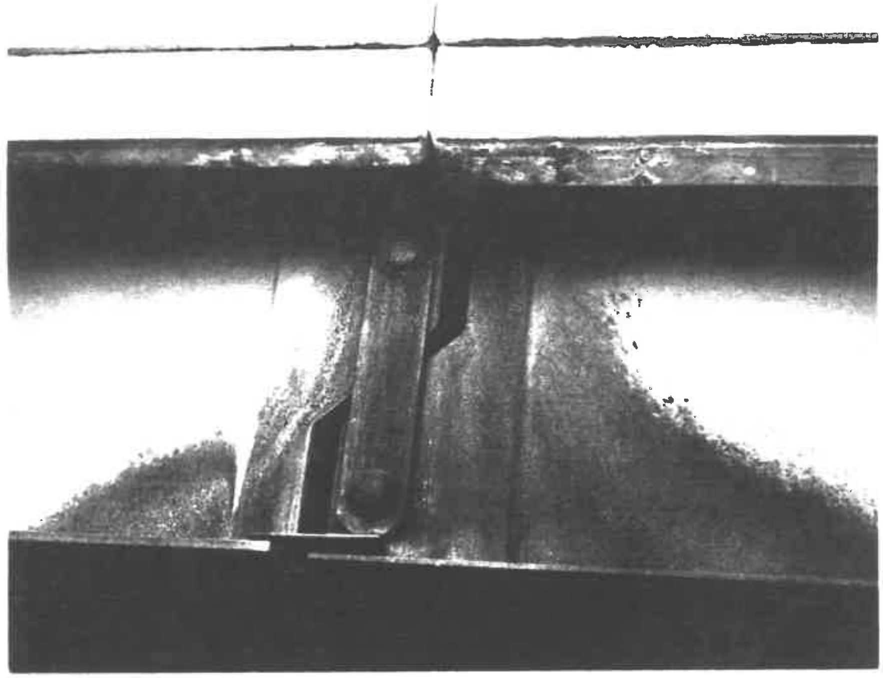


Figure 3.17 Pin and Hanger Connection

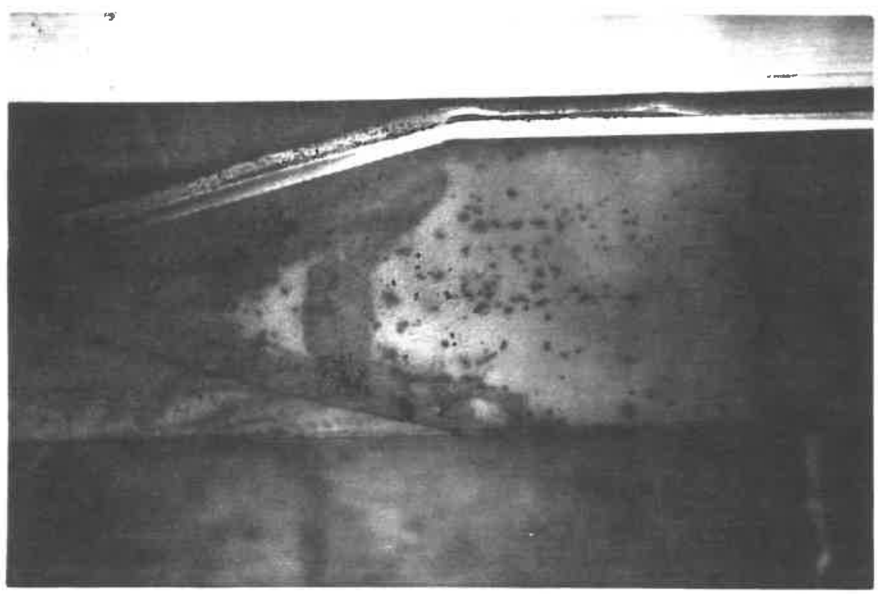


Figure 3.18 Welded Connection

Stress Range

The stress range (Figure 3.19) is the algebraic difference between the maximum and minimum stresses calculated in the analysis of a particular bridge element. This range of cyclic stresses induced by variable loads is one of the major parameters affecting fatigue crack propagation.

The AASHTO Standard Specifications for Highway Bridges establishes an allowable stress range for those structures in which the failure of a single element could cause steel collapse. This allowable range varies from 2.5 ksi to 24 ksi (for over 2,000,000 cycles), depending upon the category of the detail. For bridges in which the single fracture in a member cannot cause collapse, the allowable stress range varies from 5 ksi to 24 ksi. A detailed table of allowable stress ranges can be found in the Standard.

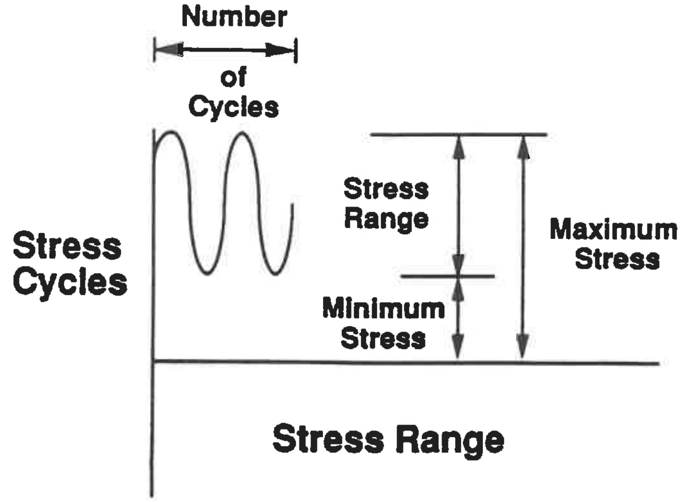


Figure 3.19 Stress Range

Fatigue Life

Fatigue life is the number (N) of stress cycles a member can support before it fails. The relationship between fatigue life and maximum stress range (S) is illustrated by the stress-fatigue life curves, known as S-N curves (Figure 3.20). As illustrated by these curves, fatigue life increases with a decrease in maximum stress up to a lower limit, where the fatigue life is no longer affected by a further decrease in stress. This lower stress limit is called the fatigue limit.

When evaluations indicate that the remaining safe fatigue life is inadequate it is recommended to either (a) restrict traffic on the bridge, (b) repair the bridge, or (c) establish periodic inspections.

The fatigue life of a bridge member can be affected by overstressing and understressing. <u>Overstressing</u> reduces fatigue life either through a large

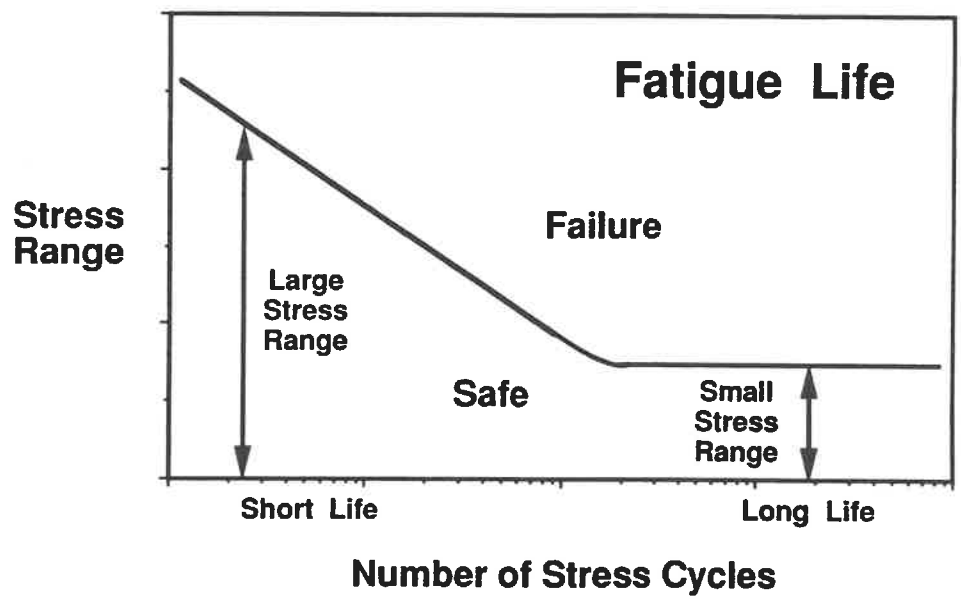


Figure 3.20 Fatigue Life

number of stress cycles slightly in excess of the fatigue limit or a small number of cycles greatly above the limit. Understressing may extend fatigue life through a large number of stress cycles slightly below the fatigue limit.

The Effects of Repeated Overstress and Cumulative Damage

Cumulative damage to a bridge member is caused by repeated, applied stresses and can result in complete bridge failure. This process of progressive permanent internal change in bridge material has three distinct steps:

1) Crack Initiation
2) Crack Propagation
3) Fracture

Cracks initiate from flaws in structural members and at structural details when subjected to a sufficient number of stress cycles. Although many bridge members and details have flaws, only those receiving high concentrations of stress have a short initiation life. A fatigue crack propagates across the section of the bridge member until it reaches the critical size causing fracture and failure (Figure 3.21).

How Fatigue Affects the Behavior of Bridge Materials

Concrete is not very susceptible to fatigue failure. However, due to inherent material variability repeated loads may cause inclined cracks in reinforced and prestressed beams at lower than expected loads; these repeated loads may also cause cracking in component materials altering the static and fatigue strength of the bridge element. Fatigue failure in concrete is generally caused by progressive internal microcracking

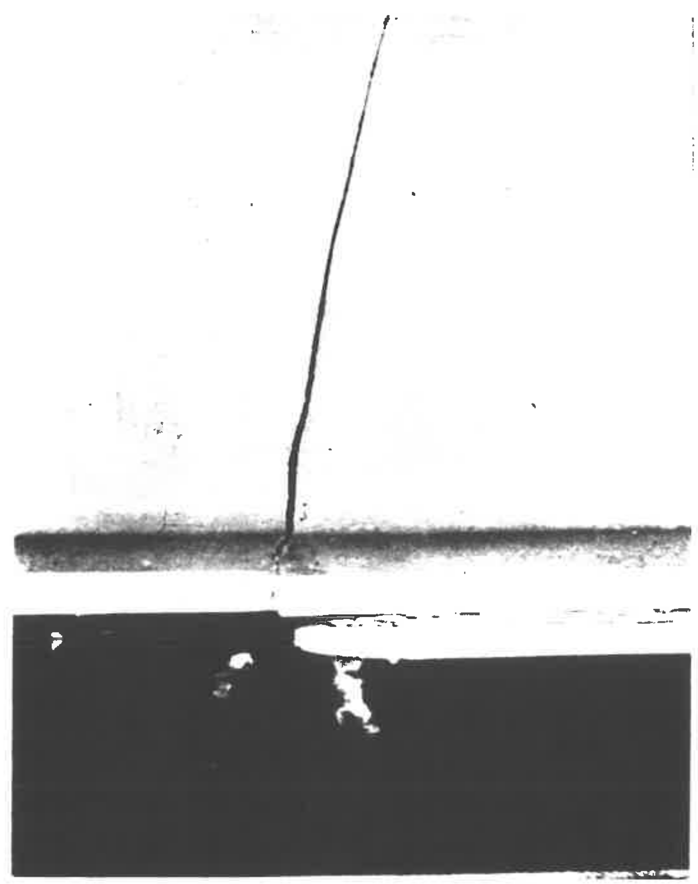


Figure 3.21 Fatigue Crack in Plate Girder

preceded by a large increase in longitudinal and transverse strains.

Steel, like other metals, is composed of random microparticles. When steel is subjected to cyclic stress variations, these particles tend to move slightly. Eventually these particles begin to lose strength, and then they fail. In the zone of failure, stress concentration develops with successive repetitions of stress causing failure of the complete section. This process is called progressive fracture.

The fatigue strength of steel varies according to composition, grain structure, heat treatment, mechanical working, surface finish, and abrupt changes in the cross section.

When fatigue occurs simultaneously with corrosion, corrosion fatigue takes place. Corrosion fatigue may reduce fatigue strength by as much as 50%. Under repeated stress, higher stress concentrations result from longer exposure to corroding agents such as oxygen, water, and sulfides.

The fatigue strength of steel is also affected by welding. Welds are subject to thermal shrinkage cracks, lack of fusion, porosity, and inclusions, all of which make welded metals more subject to fatigue.

Bridge Details That Are Susceptible to Fatigue Failure

Beam stiffeners are used to control buckling in very thin girders (Figure 3.22). However, stiffeners cause discontinuity with the bridge member, making it susceptible to fatigue cracks. When stiffeners are welded to the web, fatigue failure may be initiated at the termination of the stiffener to the web fillet weld. When stiffeners are welded to the tension flange, failure is usually initiated at the toe of the fillet weld on the flange.

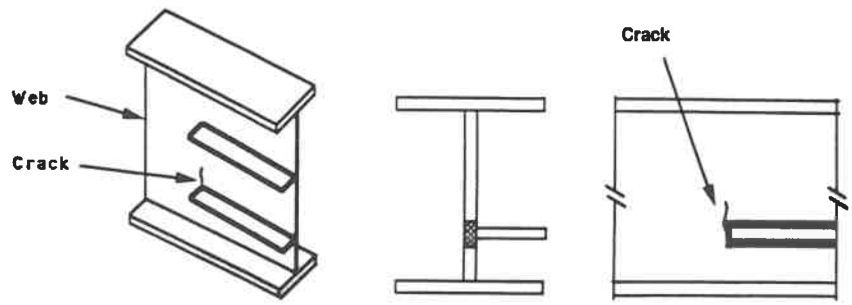


Figure 3.22 Beam Stiffeners

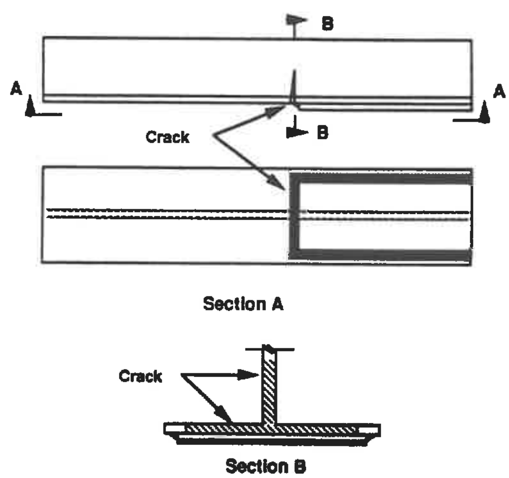


Figure 3.23 Cover Plate Details

Cross section changes are critical for fatigue damage. Sections should be changed gradually to reduce stress concentration. Fatigue cracks sometimes initiate and develop at the termination point of welded cover plates (Figure 3.23).

Stress raisers are discontinuities in steel members that create stress concentration and are therefore vulnerable to fatigue cracking due to the high stress range.

Bolted and Riveted joints (Figures 3.15 & 3.16) affect fatigue strength depending on the clamping force, grip length, type of fastener, fastener pattern, tension-shear ratio, and type of steel.

Pins and hangers are used on cantilever bridges to support suspended spans (Figure 3.17). Repeated overstress and thermal expansion will cause both pin and hanger to rotate, wearing them over time and causing cracks leading to failure.

Connections suffer fatigue cracking from secondary and displacement-induced cyclic stresses. This cracking can be found in the following connections:

a) stringer-floor beams
b) floor-beam webs
c) cross-frames
d) girder-floor beams

Welded connections are more likely to suffer critical cracking than riveted or bolted connections, especially where a weld toe exists in a high cyclic stress region. The ends of welded cover plates (Figure 3.18) are very susceptible to fatigue failure. Other details such as gusset plates have experienced fatigue cracking at weld terminations.

Specific Examples of Bridge Fatigue Problems

Fatigue can affect many bridge structural details, such as eyebars, pin and hangers, cover plates, connections, stiffeners, and welds. Some well known bridge fatigue failures are presented below.

1. The Route 157 Bridge in St. Clair County, Illinois was opened to traffic in 1945. In 1978 fractures were found in some of its hanger plates. Due to rust and corrosion, the lower pins had frozen, becoming rigid over time. In addition, traffic loads and thermal conditions had induced large in-plane bending stresses in the hanger plates, causing cracks to form and, finally, failure of the pin plates.

2. The Point Pleasant Bridge (also known as the Silver Bridge) between Ohio and West Virginia, was opened to traffic in 1928. In 1967 it collapsed without warning. The main cause of the collapse was found to have been the brittle fracture of two corrosion cracks located at one of the four eyebars used as main suspension members for this suspension span bridge.

3. The Yellow Mill Pond Bridge in Bridgeport, Connecticut was opened to traffic in 1958. In 1970, during routine inspection, a crack was discovered in one of its spans extending from the end of the cover plate of the beam through the flange. Subsequent inspections showed incipient cracks of varying magnitude along the cover plate welds. The racking actions of the attached primary members was the main contributor to this failure.

4. The Poplar Street Approach Bridges on the Mississippi River in East St. Louis, Illinois were opened to traffic in 1971. In 1973, during inspection, cracks were found in the web gaps between the ends of the floor beam connection plates and the girders (Figure 3.24). These cracks originated from out-of-plane bending stresses due to the out-of-plane

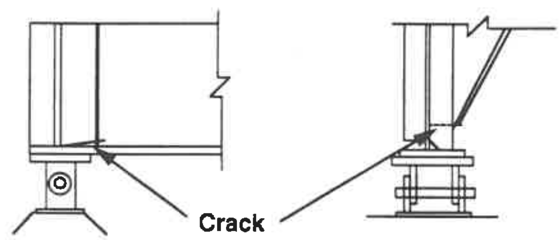


Figure 3.24 Cracks in Web Near End Support

displacement of the web in the gap region. Additional inspections in 1975 showed that cracks also existed in the main girders in the negative moment region.

5. The Quinnipiac River Bridge on I-91 near New Haven Connecticut was opened to traffic in 1964. In 1973 a large crack was discovered in the girder web of a suspended span. This failure is the example of cracking initiated from an unfused butt weld in a longitudinal stiffener-girder web interface, and enlarged from fatigue crack propagation. The girder was repaired by using bolt splice plates after removing the cracked segments.

6. The I-275 Bridge in Kenton County, Kentucky; the I-24 Bridge across the Ohio River in Kentucky; and the Ft. Duquesne Bridge in Pittsburgh had fatigue problems with box beams/girders (Figure 3.25) framed into columns. Such fatigue problems occur because joints are highly restrained.

7. The Lafayette Street Bridge over the Mississippi River at St. Paul, Minnesota was opened to traffic in 1968. In May 1975, a crack was discovered in one of its main girders. The crack had propagated from the web to the top flange. The bottom flange was fractured at the time. Examinations indicated that the fatigue crack growth started in the weld between the gusset plate and the transverse stiffener because of lack of fusion.

8. The Chicago Dan Ryan Rapid Transit Line was opened to traffic in 1969. It is an elevated railway that is used on an average day by 467 trains in both directions. In 1978 a crack was discovered in one of its steel box girder bents. Subsequent inspections found this type of cracks in other two bents. Studies indicated that the fractures were caused by fatigue cracking. The cracks

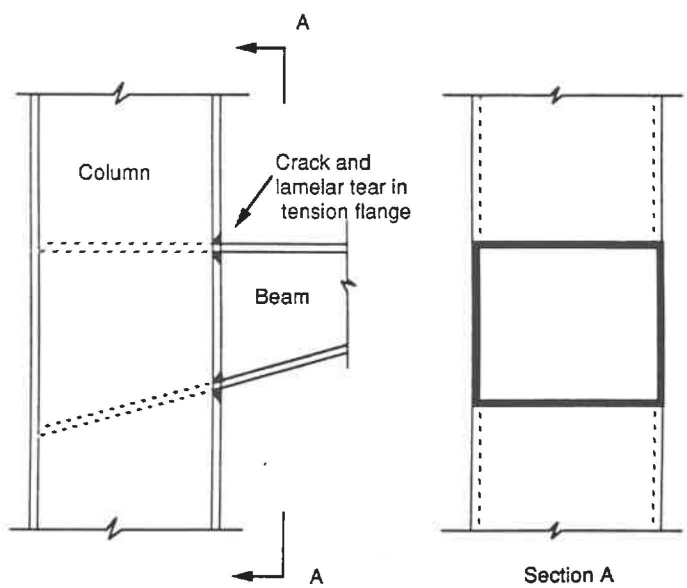


Figure 3.25 Box Beam/Girder Framing Into Column

originated at the intersections of the stringers and the box girders bents and were due to poor quality of the welds which created high stress concentration.

9. The Cuyahoga River Bridges are two bridges on Interstate I-480 near Independence, Ohio. In April, 1973 while the structure was being erected, a crack was found in the web of a girder at the intersection with a stiffener (Figure 3.26). Other girders were inspected and cracks were found. Cracks were due to fatigue caused by the relative movement between the end of the stiffeners and the web of the girder. The movement was induced either during transportation of the members to the construction site or by wind-induced motion during storage on the ground.

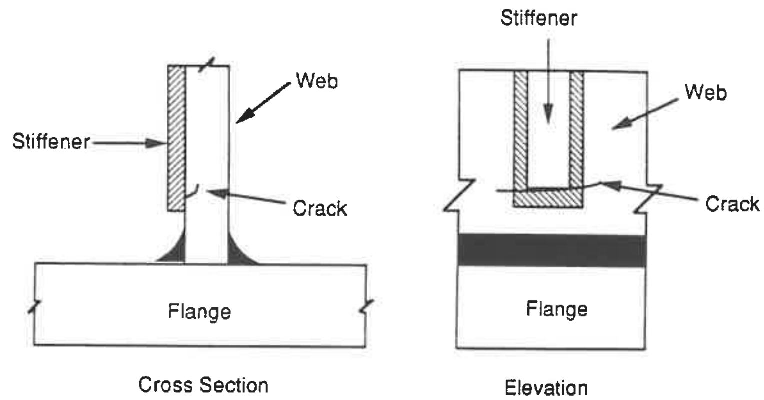


Figure 3.26 Crack Growth at End of Stiffener Into Weld and Web

SECTION 4
BRIDGE POSTING AND OVERLIMIT PERMITS

The weight limits that can be carried safely by a bridge are controlled by load posting and through weigh stations. Posting notifies drivers what loads can be safely carried by a particular bridge (Figure 4.1). Weigh stations are used to regulate the maximum permissible weight of trucks on selected roadways to the legal limits for each truck configuration.

To ensure safety, it is critical to post signage indicating limits at bridges that cannot safely support the legal load of vehicles or that have geometric deficiencies. Fatigue damage can also be minimized by proper load capacity evaluation and posting.

Many older existing bridges can no longer carry the weight of today's larger vehicles. The load limits may be a result of either a structural deterioration or an original design that is insufficient for today's needs. Until these bridges can be rehabilitated, traffic on them is regulated by posting weight limits that the bridges can safely support in their present condition. Additional postings assure bridge safety with regard to vehicle clearances for width and height (Figures 4.2 & 4.3).

This section reviews current bridge posting practices, the process for evaluating the bridge capacity, weight restrictions, and the overlimit permitting process.

BRIDGE POSTING

Before being posted, bridges are evaluated to determine the live loads they can safely carry. An accurate inspection must include a detailed structural evaluation to determine the physical condition of the bridge in question.

Figure 4.1 Load Posted Single Lane Truss Bridge

Figure 4.2 Narrow Bridge Warning Sign

Several alternatives are available for consideration if the bridge evaluation recommends posting.

The Bridge Posting Process

For a bridge found to be structurally inadequate for the legal loads on the roadway served, immediate weight limit posting is the only alternative. However, repairs or other restrictions may provide some relief. One alternative is making minor repairs, such as reinforcing a weak component. If feasible, such repairs can usually be made quickly and economically and require posting the bridge for only a temporary period. Another alternative is regulating the speed or volume of traffic through speed limits or lane restrictions. This alternative may allow higher individual vehicle loads; however, the effects on traffic may be otherwise unacceptable.

Recommended standard signing for posting is included in the *Manual of Uniform Traffic Control Devices* (MUTCD). However, bridge posting procedures vary among different jurisdictions. Most state and local governments use single gross weight limit word message signs. Example signs used to post load capacities are illustrated in Figure 4.4. The posting may describe different limits for different truck configurations. All signs must be placed so that the weight or size restrictions can be easily seen by drivers traveling in either direction. Advance warning is desirable to allow the driver to detour without back-tracking (Figure 4.5).

A posted bridge should be inspected periodically to detect any further signs of deterioration which could affect the structural strength of the bridge.

The enforcement of posted restrictions is just as important as the posting itself. Many bridge failures

Figure 4.3 Height Posting Sign

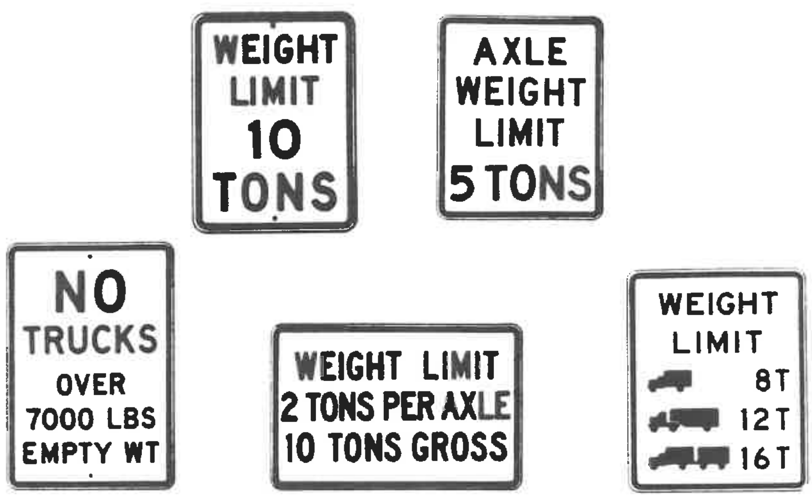


Figure 4.4 Examples of Load Posting Signs

have occurred due to overweight vehicles using posted bridges (Figure 4.6). However, it is sometimes necessary to allow overweight vehicles to use posted bridges for emergency purposes under strictly controlled conditions. When this is allowed, the loads must still be limited to avoid permanent deformation or damage and the cumulative fatigue effects must also be considered.

The Current Status of Bridge Posting

Overall compliance with National Bridge Inspection Standards (NBIS) for posting is good. However, some noncompliance exists, primarily in local districts. Some bridge owning agencies have unposted bridges with load ratings lower than those which can safely support legal highway loads.

It should be noted that the federal government does not have the jurisdictional authority to require compliance with the NBIS. The incentive for states and local governments to comply is stimulated by the ability of the federal government to withhold funds appropriated for bridge and highway construction. However, even making funding available for repair and rehabilitation has not always led to posting compliance due to local level political interests that sometimes oppose bridge posting. In addition, many local governments have not allocated sufficient funds to engage professional engineers to inspect and rate their bridges.

When local governments cannot perform the necessary inspections and ratings, the states must take over this responsibility. However, some state governments are prevented by law from giving this type of assistance to local agencies and also lack the jurisdictional authority to require them to comply with the NBIS. These states, unfortunately, cannot force local governments to post their bridges. This

Figure 4.5 Height Restriction Warning

Figure 4.6 Bridge Collapse Due to Overloading

places the public at risk, results in added damage to bridges, and reduces the service life of the local government investment in the bridge.

Bridge Posting Statistics

Permanent bridge posting does not appear to be a problem on the Interstate highway system. The Interstate system is relatively new and was constructed to high design standards. Additionally, the system is so important to the economy and security of the nation that any deficiencies are quickly detected and corrected, requiring only temporary posting.

According to National Bridge Inventory data from June 30, 1992, there are 575,583 highway bridges in the United States. Of these bridges, 118,408 are posted, but an additional 16,623 bridges should be posted and are not (Figure 4.7). Most of the bridges needing posting are on local systems.

Bridges are closed when they become too deficient to benefit from weight-limit posting. Bridges that are closed to all traffic may be closed temporarily for repairs or closed permanently (Figure 4.8).

The failure to post a deficient bridge will increase the probability of a collapse under vehicular loading. One study indicated that local roadway bridges fail much more frequently than bridges on the national highway system. Furthermore, almost 50% of the failed bridges were not posted. Data collected on bridge failures has shown that steel and timber bridges have a higher chance of sudden failure than reinforced concrete bridges. Reinforced concrete bridges tend to show signs of distress before collapse. Prestressed concrete bridges, which are relatively recent in design and construction, have not yet been reported to collapse in significant numbers.

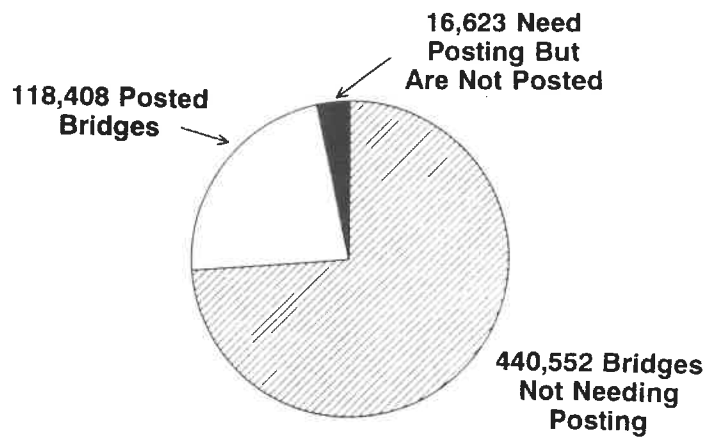


Figure 4.7 Nationwide Load Posting Status

Figure 4.8 Bridge Closed for Repairs

EVALUATING THE CAPACITY OF AN EXISTING BRIDGE

The specifications used to evaluate the capacities of existing bridges determine their maximum safe live loads. These specifications provide a basis for computing the maximum allowable loads based upon actual material quality, when bridge members are acting normally, and when deductions in size or area have been made for deteriorated members and components. The specifications are used with the assumption that the bridges are subject to inspection as often as structural conditions require, and that the investigating engineers will exercise sound judgement in determining any necessary increased safety factors.

Loadings

The capacities of existing bridges (Figure 4.9) should be determined in accordance with the current AASHTO Standard Specifications for Highway Bridges.

Analysis of the capacity of an existing bridge begins by calculating the loadings discussed in Section 3. Failure to include these loadings in this determination may reduce the life of the bridge or may even result in bridge failure. The strength evaluation requirements specify the loading conditions which must be examined.

Load Rating Methods

Bridge evaluation should include a complete analysis of the load capacity of the structure, the degree to which weight restrictions can be enforced, and the applicable laws and regulations within the jurisdiction of the bridge.

Figure 4.9 Posting Sign

Bridge load capacity is usually evaluated by one of two methods, the allowable stress method or the load factor method. The evaluation should properly account for the strength of the materials in the structure in its current state. The material strength is only considered to be equivalent to the strength at the time or original construction if the material remains sound. The effective area of any member used in calculations should be the gross area less the proportion which has deteriorated due to decay or corrosion. These conditions should be determined by engineering evaluation in the field.

1. The Allowable Stress Method limits the stresses induced by the actual loadings to the strength of the material divided by a *factor of safety*. The factor of safety varies with the type of stress and the reliability of different materials. It assures that under routine use, the yield or rupture levels of stress will not be approached.

2. The Load Factor Method considers the reliable ultimate strength of the structure and ensures that this strength is not exceeded by the combination of loadings, each multiplied by *load factor for safety*. The load factors vary with the type of load and the materials involved.

Both methods include evaluations under service actual loads to control deflection and consider fatigue.

Both methods lead to safe evaluations when applied properly. The load factor method considers the reliability and the reserve capacity of some materials which is enhanced in certain situations. Thus, the load factor method sometimes results in a higher safe load capacity determination.

Evaluation Ratings

Inventory and operating ratings, the results of bridge evaluation, indicate the magnitude and frequency, with which bridges can safely support loads.

To establish these ratings and the safe load carrying capacity, the live load used must be a standard AASHTO "H" or "HS" loading or one of the conventional vehicle types (see Section 3).

The operating rating, also known as the first or upper load level, is the maximum permissible live load, when combined with the effect from all other loads, to which the bridge may be subjected.

The inventory rating, also known as the second or lower load level, is the live load, when combined with all other loads, that can be safely supported by an existing structure for an indefinite period of time. This rating is calculated from various load-strength relationships. Avoiding the long-term effects of fatigue and recognition of possible deterioration are implicit in determining this load.

Posting policy varies from state to state. Some states post bridges based on their operating rating, some post bridges based upon their inventory rating.

A concrete bridge does not need to be posted for a weight limit if it shows no distress after carrying normal traffic for an appreciable length of time. As stated previously, the concrete bridge will generally show warning signs of distress before a complete collapse. This rule also applies to bridges for which reinforcement details are not known and for which the computed stresses exceed the allowable stresses. However, these bridges should be inspected frequently for signs of distress.

WEIGHT RESTRICTIONS

The Federal-aid Highway Act of 1975 set weight limits on the Interstate Highway System for single- and tandem-axle vehicles and for gross weight. Outside the interstate system, however, size and weight regulations have traditionally been the prerogative of the individual states.

The diversity of the regulations of individual states has created inconsistent controls which have long been a problem for anyone needing to move large, heavy loads across the nation's highways. In addition, permits and enforcement within individual states are sometimes not administered by the same state agencies, further complicating the task of the trucking industry.

Legal dimensions and weights, for example, vary dramatically from state to state. Some states make allowances beyond their specified legal limits to account for weighing scale inaccuracies. These additional weight allowances are known as statutory or gross weight tolerances. Most, but not all, states grant extensions of their legal limits to agriculture, the military, local interest industries, and emergency vehicles. Details of the individual state's requirements change periodically. Local governments in each state can obtain the legal limits and special provisions from their state department of transportation.

Weight Restrictions on Federal-aid Interstate Highways

The Federal-aid Highway Act of 1975 allows the following limits on the Interstate Highway System:

Single axles - 20,000 lbs. (2,100 kg.)
Tandem axles - 34,000 lbs. (15,400 kg.)
Gross weight - 80,000 lbs. (36,300 kg.)
Width - up to 8 ft. (2.4 m.)
Height - up to 13.5 ft. (4.1m)

However a clause in the 1975 Act provides for greater weights and widths if they were permitted before an earlier, 1956 Federal-Aid Highway Act.

State Weight and Dimension Restrictions

Each state has defined its own legal allowable limits for height, width, length, and weight for specific types of vehicles. Examples of these limits are presented in Table 4.1.

Sometimes limits differ among highway classes within a state. About half of the states establish higher limits for some highway classifications or specific routes such as toll roads.

Some northern states allow heavier loads in the winter in order to use the extra strength of frozen pavement subbase. Some states decrease weight limits in the spring to protect pavement from excessive damage during the spring thaw.

Most states grant some extensions of legal limits to the military and to agriculture or other industries of local interest. The loads granted extension may be either divisible or indivisible, depending upon the discretion of the individual states. The most common exceptions are extra width for farm implements and length extensions for indivisible loads such as pipes, poles, pilings, and structural members. Also, access for overweight emergency vehicles is allowed under strictly controlled conditions.

	Usual Limit	Range Including Most State Limits
Length		
Single	40 ft	35 - 55 ft
Combination	65 ft	50 - 85 ft
Width	8 ft	8 - 8.5 ft
Height	13.5 ft	13.5 - 14 ft
Axle Weights Limits		
Single	20000 lbs	18000 - 22400 lbs
Tandem	34000 lbs	
Gross Vehicle Weight		
Five Axles	80000 lbs	73280 - 101000 lbs
Other Combinations	80000 lbs	73000 - 154000 lbs

Table 4.1 Example Legal Dimensions and Weights

Permit Operations

Statutory weight and size limits specify the maximum allowable weight and dimensions of motor vehicles on the highway system within an individual state. When an indivisible piece of cargo exceeds the specified limits in weight, length, width, or height and the highway is the most practicable method of transportation, a permit must be obtained before shipping begins. In addition, several states also grant permits for reducible loads of significant economic importance.

The primary objectives of permit operations are to control the shipment of overlimits and not readily dismantled cargo so that (1) the structural integrity of the highway system may be protected and (2) such shipments do not create traffic safety hazards or undue delays for motorists. While all states maintain the same objectives, many problems exist with the system of permit operations, primarily because individual states vary considerably in their permit practices.

Permit limits have been established in an attempt to protect bridges from increased structural deterioration, particularly fatigue, caused by overlimit loads. Routine overlimit regulations have been established for every state, but these regulations vary widely. In an effort to standardize the permit policies of all states, a proposed policy statement on uniform overweight permits has been issued by the Specialized Carriers Conference.

Overlimit Permit Types

A shipment may be oversized, overweight, or both. Oversized loads are overlength, overwidth, overheight, or any combination of the three (Figure 4.10). Overweight loads are either over the gross weight limit or overweight on a specific axle.

Figure 4.10 Wide Load Warning Sign

In general, there are two types of overlimit permits: single-trip permits and multiple-trip permits:

1. Single-trip permits are good for a single, one-way or round trip as specified in the laws of each state. Most single-trip permits are valid for less than one week, depending on the state of issuance. A few states issue single-trip permits for longer periods, up to 30 days. Over one-half of the states grant extensions if a shipment cannot be made due to unforeseen circumstances such as bad weather; however, the request for extension must be made while the permit is still valid.

   Most states have adopted standard procedures to expedite processing single-trip permit applications. A set of routine limits is usually selected. If these limits are exceeded, an engineer or professional must be consulted. A routine limit can be defined as a size or weight limit beyond which any or all of the following are needed: bridge division check, escort, extensive route analysis, approval and/or issuance from headquarters only. This definition of routine limits varies among the individual states.

2. Multiple-trip permits last from two weeks to one year. These permits are generally issued to manufacturers, contractors, and others who frequently need to ship the same type of overlimit loads. Most states have ceilings on the number of permits they will issue in a given year. If an overlimit shipment exceeds multiple-trip permit limits, a single-trip permit is needed.

Most states grant permits only to non-reducible loads. However, some states allow overlimits on divisible loads for economic reasons. These divisible loads, which must be moved by permit, include grain, crops, forest products, fuel, and concrete. As a result, several movements may be permitted under a single-trip permit.

Permit Controls

States designate the specific routes on state highways over which the cargo with an overlimit permit can move. These routes are selected after considering such things as geometric characteristics of highway options, structural characteristics of the bridges, and the general condition of the environment. Route maps are sometimes distributed with either single- or multiple-trip permits and may be used to specify in writing the routes that must be followed.

Permit issuance characteristics vary widely among the individual states. For example, a state with lower size and weight limits usually issues more permits than states allowing similar shipments without permits. States experiencing rapid growth usually have more permits issued for construction equipment. Permit statistics are also influenced by the degree of enforcement in individual states: states with less enforcement have more truckers who avoid the red tape of obtaining a permit. A definite correlation exists between the effectiveness of size and weight enforcement and success of the permitting process in achieving the goals of state regulations.

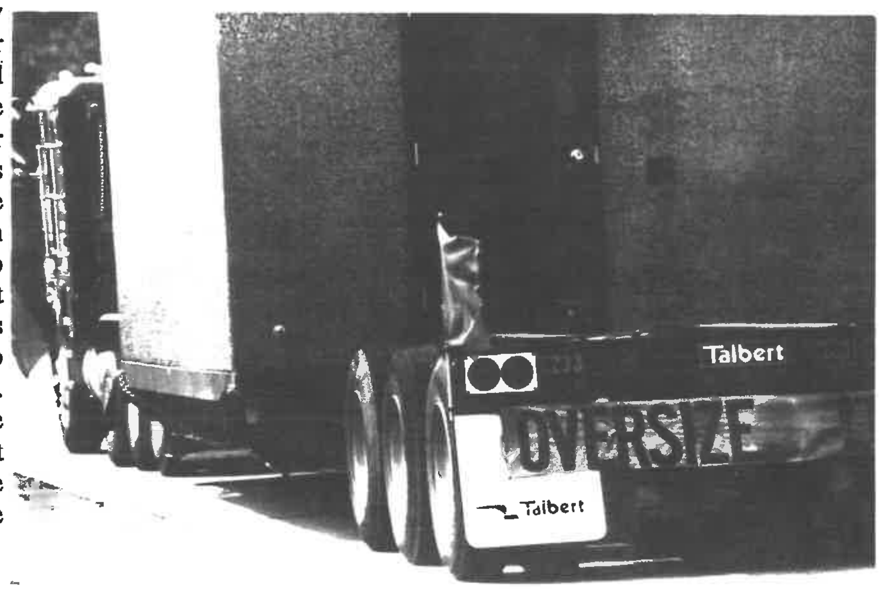


Figure 4.11 Flag on Oversized Vehicle

Accessory Requirements

Most states require permit vehicles to warn motorists of the overlimit load. Such warnings appear as flags, signs, and escort vehicles.

Red Flags are required on oversized loads in all but nine states (Figure 4.11). However, flag size requirements vary from state to state. The numbers of flags range from two to a variable requirement for flags at all corners, extremities, protrusions, and overhangs.

Signs are required by most states to warn motorists of the nature of the overlimit load. A few states also require warning signs on escort vehicles.

Sign standards vary considerably among individual states in terms of color and size. Warning sign wording is usually OVERSIZE/WIDE/LONG LOAD, depending upon the situation. Three states have adopted the wording CAUTION OVERSIZE/WIDE LOAD. One state uses DANGER OVERSIZE LOAD.

Escort vehicles are required by all states when the overlimit size or weight exceeds a specified limit. Some states have different escort requirements for overlimit loads traveling on two- or four-lane highways. These states are more likely to require escorts on two-lane highways than on four-lane highways.

Requirements for escort vehicle accessories vary widely among the states:

1. amber flashing light;
2. flags;
3. two-way radios between overlimit and escort vehicles; and
4. warning signs.

Overlimit Permit Time Restrictions

Each state may restrict or prohibit overlimit permit operations during specific periods. These include:

- Time of day restrictions;
- Weekend restrictions;
- Holiday restrictions;
- Weather restrictions; and
- Seasonal restrictions.

INTERACTION OF EVALUATION, POSTING, AND PERMITS

Both the safety and ultimate benefit to the public of an existing bridge is greatly effected by proper evaluation and posting of the bridge to control of overlimit loads (Figure 4.12). If the bridge is not properly evaluated and posted, the bridge may be subjected to loads which can

- cause failure of the bridge and accompanying accidents;
- cause direct damage because of overstress resulting in repair cost; or
- cause fatigue damage which will shorten the life of the bridge.

If overlimit permits are issued too frequently or without adequate knowledge of the bridge capacity, similar damage can occur. Thus, it is in the interest of both the public and the bridge owning agency to have bridges properly evaluated and posted. These activities are also an integral part of a Bridge Management System.

Figure 4.12. Low Capacity Weight Limit Sign

SECTION 5

BRIDGE MANAGEMENT: MAINTAINING AND IMPROVING A BRIDGE SYSTEM

In order to practice sound bridge management, it is necessary to:

- Inventory and inspect bridges;
- Evaluate deficiencies;
- Evaluate priorities;
- Select and program projects; and
- Maintain and improve bridges.

The earlier sections of this manual have described the inspection, inventory, and evaluation of individual bridges. However, to compare alternatives and evaluate priorities across a network of bridges, additional rating methods are needed.

This final section discusses the ratings used to establish federal funding eligibility, improvement options, methods of evaluation and prioritization, and funding options. Included is an overview of the Federal Highway Bridge Replacement and Rehabilitation Program (HBRRP) and the significance of the Intermodal Surface Transportation Efficiency Act of 1992 (ISTEA) in the bridge management process.

Finally, comprehensive bridge management systems are described. These systems can estimate present and future bridge needs, set project priorities, and determine how to use limited funds (Figure 5.1) most efficiently.

Figure 5.1 New Concrete Bridge Deck and Rails

FEDERAL EVALUATION OF BRIDGE SUFFICIENCY

The Federal Highway Administration has established evaluation procedures for assessing the overall state of the nation's bridges, determining eligibility for funding of individual bridges, and apportioning funds available to the various states. The evaluation process includes a determination of a Sufficiency Rating for each bridge and whether it is Structurally Deficient and/or Functionally Obsolete.

The Sufficiency Rating

The sufficiency rating is an overall summary evaluation of how well a bridge meets needs and provides safety. It is a numerical value ranging from 0 to 100. The lower the rating numerically, the poorer the bridge is judged to be in meeting overall needs. Four summary factors have been formulated into the sufficiency rating formula:

$$SR = S_1 + S_2 + S_3 - S_4$$

where:

S_1 = structural adequacy and safety (<55%), as indicated by the bridge inventory rating plus evaluations of the superstructure, substructure, and culverts.

S_2 = serviceability and functional obsolescence (<30%), as indicated by evaluations of the approach roadway alignment, underclearance, deck condition, deck geometry, ADT, the number of lanes on the structure, approach roadway width, main structure type, vertical clearance over deck, bridge roadway width, structural evaluation, waterway adequacy, and defense highway designation.

S_3 = essentiality for public use (<15%), as indicated by the ADT, detour length, and defense highway designation.

S_4 = special reductions (<13%), used only if $S_1 + S_2 + S_3 > 50$.

Sufficiency variables are calculated by subtracting the following associated reduction factors from their corresponding maximum weights:

$S_1 = 55 - F_1$

$S_2 = 30 - F_2$

$S_3 = 15 - F_3$

More information on determining the Sufficiency Rating (SR), including detailed equations for calculating each reduction factor from the inspection data, appears in the FHWA "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges."

Deficiency and Obsolescence Classifications

Bridge condition classifications determine whether a bridge is deficient. The FHWA defines bridge deficiency to include various inadequacies and conditions (see Section 2):

1. A bridge is Structurally Deficient if either its deck, superstructure, substructure, or culvert has deteriorated to the point of marginal conditions for major rehabilitation or if its appraisal rating indicates intolerable conditions requiring replacement.

2. A bridge is Functionally Obsolete if its deck geometry, load capacity, underclearance, or approach roadway alignment no longer meet the usual criteria for the roadway served.

IMPROVEMENT ELIGIBILITY CRITERIA AND OPTIONS

About 95% of all deficient bridges are eligible for replacement or rehabilitation under the Federal bridge program to the extent that funds are available. To determine whether a bridge should be repaired or replaced, the first steps should be a complete inspection and evaluation of the structure. Included in this evaluation should be the structure inventory and appraisal, the sufficiency rating, and the condition classification of the bridge.

Bridges that are either Structurally Deficient or Functionally Obsolete are eligible for consideration for matching funds under the Federal Bridge Program. Those with a Sufficiency Rating of below 50 may qualify for replacement or rehabilitation. Those with a Sufficiency rating between 50 and 80 are eligible for rehabilitation only.

Approximately 60% of the nation's deficient bridges have sufficiency ratings below 50 and thus qualify either rehabilitation or replacement. An additional 35% have sufficiency ratings between 50 and 80 and are eligible for rehabilitation only. However, in a few instances, the FHWA has approved replacing some of these bridges, at the request of the individual states, when cost comparisons indicate that it would be more cost effective in the long run to replace these structures.

The decision to rehabilitate or replace a bridge is usually critical to its roadway system. However, equally important options include the method(s) of rehabilitation and the materials and structural components of the replacement bridge.

Because of such constraints such as traffic during construction or relocation/replacement costs, rehabilitation is an important alternative for many bridges. If the bridge is a major structure located in a large city or urban area, rehabilitation will often be a cost-effective choice.

Maintenance Options

Several levels of bridge maintenance are practiced, depending upon ADT, local conditions, age of the structure, etc:

1. Routine maintenance consists of spot painting, removing debris, cleaning the drainage system, tightening loose bolts, and cleaning the joints.

2. The next level of maintenance includes complete repainting, repairing potholes, filling cracks, adjusting bearings, and sealing concrete surfaces.

3. The third level of maintenance approaches rehabilitation. It includes replacement of bearings and joints, fatigue crack repair, waterway adjustment, readjustment of forces, such as in cables, and other specialized activities.

Although preventive maintenance is critical to extend bridge life, these activities in themselves do not remove major deficiencies. Thus, preventive maintenance, although essential, does not qualify for special funding programs. Bridge owning agencies should adopt maintenance levels-of-service that specify the desired condition to be maintained for each bridge component, and then budget adequate funds for this purpose. Rehabilitation or replacement options which adequately remove deficiencies are eligible for funding consideration.

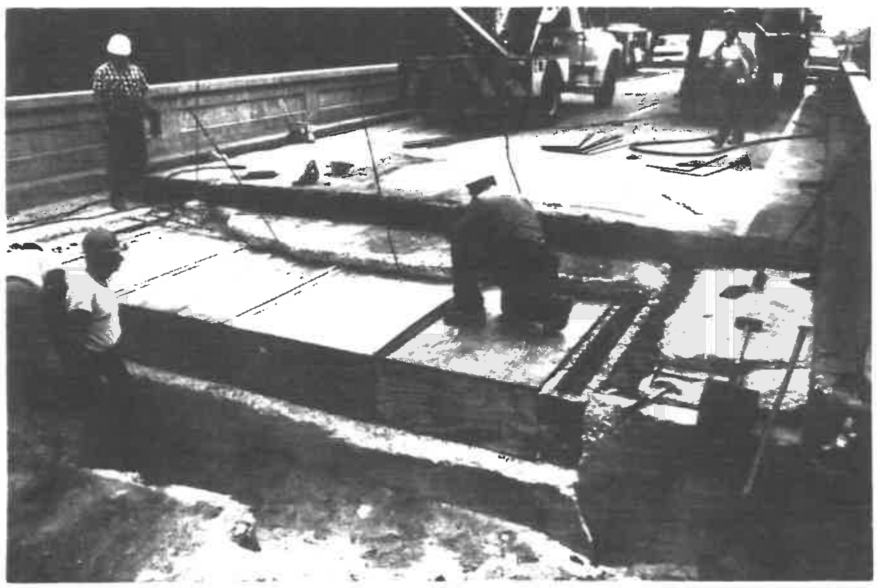


Figure 5.2 Rehabilitation of Low Volume Roadway Bridge

Rehabilitation Options

Once the decision has been made to repair a bridge rather than replace it, the most appropriate rehabilitation method must be selected (Figure 5.2). Management personnel making this selection must consider, in addition to technical data, the following:

a. Bridge location and importance in

the highway system;

b. ADT;

c. Type, size, and geometry of the structure;

d. Nature and extent of the deterioration;

e. Load carrying capacity;

f. Skills required to perform repairs;

g. Special equipment required to perform repairs; and

h. Repair costs and available budget.

Bridges are usually rehabilitated by increasing the load carrying capacity, improving the bridge geometry, correcting mechanical deficiencies and drainage or scour problems, or by making other miscellaneous repairs.

Increasing the load carrying capacity is achieved with one or more of the following practices:

1. Strengthening a critical member by adding additional material to the member itself or by replacing it with a new member.

2. Adding new supports or members to help carry loads.

3. Reducing the dead load on the bridge, thus providing additional live load capacity. The dead load can be reduced by removing the existing deck and providing a new, lighter one.

4. Changing the structure to increase the live load capacity of the bridge.

Improving bridge geometry includes increasing vertical clearance, widening the roadway, and/or improving the approach horizontal and vertical alignment (Figure 5.3).

Correcting mechanical deficiencies usually includes replacing defective parts or cleaning and adjusting them to their proper positions. These parts are

Figure 5.3 Bridge and Widened Roadway

bridge bearings, expansion joints, hangers, or other parts which permit the expansion and contraction of the structure.

Correcting drainage problems should include:

1. Any necessary measures to prevent deck wash from reaching the superstructure and substructure members and the bearings.
2. Scuppers which are large enough and properly spaced to minimize clogging from roadway deposits.
3. Carefully detailed discharge points to prevent erosion.
4. Determining that all drainage system parts are either galvanized or made of weathering steel.
5. Maintenance cleaning of collection pipes and downspouts.

Correcting scour problems (Fig. 5.4) is usually accomplished by one of the following procedures:

a. Providing riprap at piers and abutments;
b. Constructing guides banks (spur dikes);
c. Constructing channel improvements;
d. Strengthening the bridge foundations;
e. Constructing sills or drop structures; and
f. Constructing relief bridges or lengthening existing bridges.

Making miscellaneous repairs includes:

a. Replacing inadequate bridge railings;
b. Altering parapet and railing ends of through girders and through trusses; and
c. Making any necessary repairs to the approach roadway and bridge deck to improve riding quality.

Figure 5.4 Advanced Scour at Abutment

Any deck rehabilitation should include a sealing system to prevent or reduce the further buildup of chlorides.

Replacement Options

Bridge improvement decisions do not end with the decision to replace a structure. There are numerous replacement options to choose from. An appropriate replacement bridge can only be selected after giving due attention to the traffic volume and resulting, required capacity of the new structure; the capabilities of skilled contractors; and the cost and availability of different structural materials, forms, and equipment.

A replacement bridge should provide the best combination of construction and maintenance costs. For replacement bridges serving relatively low traffic volumes in moderate environments, the optimum objective is often to minimize construction costs. For bridges serving higher traffic volumes in more severe environments, the optimum objective is often to minimize maintenance costs. In either situation the decision should be based on the lowest life-cycle cost option.

HBRRP: THE PRIMARY FUNDING OPTION

The primary source of bridge improvement funds is the Federal Highway Bridge Replacement and Rehabilitation Program (HBRRP). The types of projects eligible for funding and funding limitations are discussed here, as well as how to participate in the HBRRP.

The Highway Bridge Replacement and Rehabilitation Program (HBRRP)

In 1978, Congress extended the Special Bridge Replacement Program of the Surface Transportation Assistance Act (STAA) into what is now known as the Highway Bridge Replacement and Rehabilitation Program. It is important to note that the STAA also extended inventory and inspection requirements to bridges that are off the Federal-aid system.

Funding categories are as follows:

1. Apportioned funds, distributed to the states according to bridge needs. Needs are determined by placing deficient bridges into one of four funding categories, depending upon whether they should be replaced or rehabilitated and are on or off the National Highway System.

   Funds apportioned to each state equal the percentage obtained by dividing the total square foot cost of deficient bridges in each category by the total deficient bridge costs in all states. The maximum share awarded to any state is 10% of the total funds; the minimum is 0.25%. Bridges that are off the National Highway System are awarded at least 10%, but not more than 35%, of the bridge funds apportioned to each state.

2. Discretionary funds, are allocated by the Secretary of the Department of Transportation for bridge replacement or rehabilitation costing over $10 million or more than twice a state's annual apportioned funds. A rating formula mandated by the Congress gives funding priority to discretionary bridge projects already in process. This rating process also establishes eligibility for candidate bridges not previously selected, but having ratings of 100 or less.

<u>Project eligibility</u> is open to any deficient highway bridge on a public road. Projects eligible for program funds include:

1. Bridge replacement with a structure meeting current standards;
2. Rehabilitation that removes a bridge from Functionally Obsolete or Structurally Deficient status and increases the Sufficency Rating to above 80;
3. Preliminary engineering, right-of-way, and utilities projects;
4. Inventory, inspection, and rating of certain bridges; and
5. Inventories for bridges with historical significance.

When rehabilitation activities are undertaken, they should be designed to assure that the improvement will eliminate the deficiencies. Once Federal funds are used for a specific bridge, it is not eligible for further funding for 10 years.

<u>HBRRP participation procedures</u> are relatively uncomplicated. First, a state or local government conducts bridge inspections according to the National Bridge Inspection Standards (NBIS). Inspection data are then submitted by the state through the FHWA division office in each state to FHWA headquarters in Washington, D.C.

Next, the FHWA evaluates state inspection data in the following manner:

1. A sufficiency rating, using a formula developed by AASHTO and FHWA, is assigned to each inventoried bridge.
2. The sufficiency rating is used to establish eligibility and priority for work to be done on structurally deficient and functionally obsolete bridges.
3. A bridge with a sufficiency rating

of 80 or less is eligible for rehabilitation. If the sufficiency rating is less than 50, the bridge is also eligible for replacement.

Following evaluation, the FHWA provides a selection list to each state. This list includes both off-system and on-system bridges eligible for improvement.

The state and local governments then indicate the bridge projects they select from the list by assigning them priority numbers. At this time, the state and local jurisdictions must also indicate how they wish their share of the apportioned funds to be distributed among the individual bridges to be improved. The requested distribution must assign between 15% and 35% of these funds to off-system bridge improvement (Figure 5.5).

Finally, the states submit their bridge improvement priorities and fund distribution requests to the FHWA for review and approval.

Limitations on HBRRP funds include the costs of such improvements as long approach fills, causeways, connecting roadways, interchanges, ramps, and other expensive earth structures.

OTHER FUNDING OPTIONS

Additional bridge improvements may be financed with general and specific Federal funds and with revenue from state and local governments.

Federal-aid Highway Funds

There are additional, non-HBRRP, Federal funds available for Interstate, primary, secondary, and urban bridge projects. These funds include revenue sharing and disaster relief funds. It has been reported that the states use over 50% of these funds for rehabilitation.

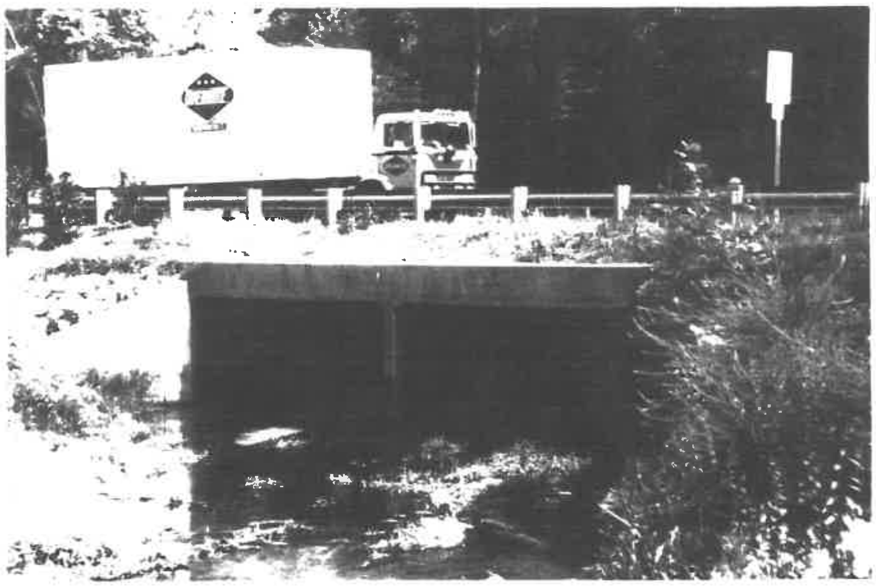


Figure 5.5 Local Road Bridge Replaced by Culvert

State and Local Funds

Many state and local governments match Federal-aid funds with their own funds. In some cases, these governments finance bridge replacement or repair entirely with their own funds derived from general revenue, highway user taxes, and bond issues.

PROBLEMS WITH SELECTION PROCEDURES

The FHWA and state procedures to establish priorities and select projects have not always insured that the bridges most in need are the ones replaced or rehabilitated.

Problems with eligibility criteria result in funding for some deficient bridges with high sufficiency ratings. Some examples are:

1. Bridges with a legislative history of needing improvement are often given high priorities for discretionary funds even though these bridges have high sufficiency ratings and are in relatively good condition.
2. Two bridges in the same structural condition may have substantially different sufficiency ratings because of other factors, such as the ADT. The sufficiency rating of a bridge can be reduced by as much as 15% for a high volume of average daily traffic.
3. For sufficiency ratings above 50, the SR formula provides for an additional reduction of up to 13% for long detours and substandard guardrails and bridge railings. This provision qualifies some bridges for replacement that would otherwise only be eligible for rehabilitation.
4. All bridges, whether they have SR's of 0 or 99, receive equal weight in the apportionment

factors considered even though their improvement needs and costs vary substantially.

Problems with outdated and incomplete data also result in some allocations being awarded to less deserving bridge projects. Thus, it is necessary that the local and state governments report all inspection data promptly.

BENEFITS OF A BRIDGE MANAGEMENT SYSTEM

Of the estimated $91 billion needed to improve today's deficient bridges, only approximately $5 billion has been available each year for this task. Consequently, many states have been developing or seeking effective Bridge Management Systems (BMS) in order to use their limited funds to the greatest advantage.

The Intermodal Surface Transportation Efficiency Act of 1992, known as ISTEA, now mandates each state to implement a Bridge Management System as one part of an overall transportation management and monitoring system.

Each state is to maintain a data base that contains the BMS data for all bridges on and off Federal-aid highways, except those that are Federally owned. Each state must implement network analysis procedures (Table 5.1) that are capable of analyzing data for all bridges in the inventory or in any subset.

Subsets of data maintained and analyzed by each state include the inventories of each Municipal Planning Organization (MPO) jurisdiction and each local agency jurisdiction. Local bridge owners may supplement the State BMS with a locally operated system that is tailored to their particular needs.

Table 5.1 State BMS Procedure Requirements

The BMS must have analysis procedures with the ability to:

1. Predict the deterioration of bridge elements with and without intervening actions.
2. Identify feasible actions to improve bridge condition, safety, and serviceability.
3. Estimate the cost of actions.
4. Estimate expected user cost savings for safety and serviceability improvements.
5. Determine least-cost maintenance, repair, and rehabilitation strategies for bridge elements using life-cycle cost analysis or a comparable procedure.
6. Perform multi-period optimization.
7. Use feedback from actions taken to update prediction and cost models.
8. Generate summaries and reports as needed for the planning and programming process.

The Basic Functions of a Bridge Management System

A Bridge Management System is any set of engineering and management functions used to estimate present and future bridge needs, set project priorities, and analyze the economic goals and limitations of bridge systems.

BMS objectives are to:

1. Organize and conduct all necessary activities to manage a successful bridge program.
2. Provide decision-makers with the best solutions from the possible cost-effective alternatives for every necessary action.
3. Achieve desired service levels with allocated funds and identify future funding needs.
4. Eliminate, if possible, all existing needs at the beginning of each year.
5. Define bridge conditions.
6. Identify, assign priorities, and recommend bridges for improvement.
7. Allocate funds for maintenance, rehabilitation, and replacement.
8. Analyze the economic conditions of each bridge and of the entire bridge system.
9. Maintain a data base of appropriate bridge information.

General BMS functions include inventorying bridge inspection data, evaluating bridge priorities, and selecting and programming bridge projects. These functions include needs predictions and evaluations of bridge improvement alternatives.

Needs predictions are essential to bridge managers who must plan funding requirements in advance in order to provide the lead time necessary for the legislative process.

Predicting future needs includes

formulating theoretical, empirical, and mechanical models and applying these to principles of sound engineering judgement.

Economic bridge improvement evaluations are calculated as costs and benefits for both highway agencies and highway users.

a) Agency costs and benefits represent long-term investments in the maintenance and rehabilitation of any bridge. Improvement expenditures are considered net beneficial if they lower discounted future costs by more than the expenditures themselves.

   Agency benefits are the present worth of the future cost savings of expenditures which extend bridge service life, thus postponing bridge replacement.

b) User costs and benefits result from functional bridge deficiencies and the actions taken to reduce them. Functional deficiencies such as load posting or clearance restrictions cause higher vehicle operating costs, lost travel time, and higher accident rates. User benefits are calculated by subtracting the user costs after improvement from the user costs before improvement.

Multi-period optimization is a process in which mathematical computer algorithms in the BMS recommend the optimum set of projects matching the constraints of the funds available. The objective of the optimization, in both near term and future years, is to select the alternatives resulting in maximum improvement of the bridge network and maximum benefit to the user at lowest life-cycle cost. The modeling procedures account for future traffic growth and element deterioration, and facilitates analyses of the effects of alternative policies, budgets and operational practices on the future conditions and long term serviceability of the bridge inventory.

Summary

In summary, the objective of a Bridge Management System should be to optimize the efficient use of funds available by minimizing total life-cycle costs of the system to the ultimate owner, the user/taxpayer, while concurrently providing levels-of-service for load capacity and geometry which will enable our bridge system to safely meet the public's needs. Proper bridge evaluation and posting are an integral and critical part of this process.

REFERENCES

1. American Association of State Highway and Transportation Officials, "Guidelines for Bridge Management Systems," Washington, DC, 1993.

2. American Association of State Highway and Transportation Officials, "Manual for Maintenance Inspection of Bridges 1983," Washington, DC, 1984 with subsequent interim revisions.

3. American Association of State Highway and Transportation Officials, "AASHTO Manual for Bridge Maintenance 1987," Washington, DC, 1987.

4. American Association of State Highway and Transportation Officials, "Standard Specifications for Highway Bridges," Washington, DC, 1989 with subsequent interim revisions.

5. American Association of State Highway and Transportation Officials, "Guide Specification for Fatigue Evaluation of Existing Steel Bridges," Washington, DC, 1990.

6. American Association of State Highway and Transportation Officials, "Guide Specification for Strength Evaluation of Existing Steel and Concrete Bridges," Washington, DC, 1989.

7. American Association of State Highway and Transportation Officials, "Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members," Washington, DC, 1978.

8. Berger, Byrd, Tallamy, McDonald, and Gordon, "Extending the Service Life of Existing Bridges," Transportation Research Record 664, 1978.

9. "Bridgit Bridge Management System-Technical Manual," NCHRP Project 12-28(2)A, National Engineering Technology Corporation, August 1993.

10. Chen, C-J., and Johnston, D.W., "Bridge Management Under a Level of Service Concept Providing Optimum Improvement Action, Time, and Budget Prediction," FHWA/NC/88-004, Center for Transportation Engineering Studies, Department of Civil Engineering, North Carolina State University, 383 pgs., September 1987.

11. Farid, F., Johnston, D.W., Chen, C-J., Laverde, M.A., and Rihani, B.S., "Feasibility of Incremental Benefit-Cost Analysis for Optimal Allocation of Limited Budgets to Maintenance, Rehabilitation, and Replacement of Bridges," Report No. FHWA-DP-71-02, Center for Transportation Engineering Studies, Department of Civil Engineering, North Carolina State University, 203 pgs., February 1988.

12. Federal Highway Administration, "Bridge Inspector's Training Manual, FHWA, Washington, DC, 1979.

13. Federal Highway Administration, "Manual on Uniform Traffic Control Devices," FHWA, Washington, DC, 1988.

14. Federal Highway Administration, "Recording and Coding Guide for the Structure Inventory and Appraisal for the Nation's Bridges," FHWA, Washington, DC, December 1988.

15. Federal Highway Administration, "The Status of the Nation's Highways & Bridges: Conditions and Performance and Highway Bridge Replacement and Rehabilitation Program," FHWA, Washington, DC 1989.

16. Galambos, C.F., "Bridge Design, Maintenance and Management," Public Roads, vol. 50, No. 4, pp. 109-115, March 1987.

17. Hudson, S.W., Carmichael, R.F., Moser, L.O., Hudson, W.R., and Wilkes, W., "Bridge Management Systems," NCHRP Report No. 300, Transportation Research Board, Washington, DC, 1987.

18. Imbsen, R.A., "Bridge Weight-Limit Posting Practice," NCHRP Report No. 108, Transportation Research Board, Washington, DC, 1984.

19. Johnston, D.W., and Zia, P., "A Level of Service System for Bridge Evaluation," Transportation Research Record 962, Transportation Research Board, Washington, DC, 1984.

20. Lamberton, H.C., and Sainz, A.J., "Underwater Inspection and Repair of Bridge Substructures," NCHRP Report No. 88, Transportation Research Board, Washington, DC, 1981.

21. Manning, and Ryell, J., "Decision Criteria for the Rehabilitation of Concrete Bridge Decks," Transportation Research Record 762, Transportation Research Board, Washington, DC, 19__.

22. Mitsuru, S., and Sinha, K.C., "Review of Current Practices of Bridge Management at the State Level," Transportation Research Record 1113, Transportation Research Board, Washington, DC, p. 1-8, 1987.

23. Moses, F., Schilling, C.G., and Raju, K.S., "Fatigue Evaluation Procedures for Steel Bridges," NCHRP Report No. 299, Transportation Research Board, Washington, DC, 1987.

24. "Motor Vehicle Size and Weight Regulations, Enforcement, and Permit Operations, NCHRP Report No. 68, Transportation Research Board, Washington, DC, 1980.

25. O'Connor, D.S. and Hyman, W.A., "Bridge Management Systems," FHWA-DP-71-01R, Demonstration Projects Division, Federal Highway Administration, Washington, DC, October 1989.

26. Park, S.H., Bridge Rehabilitation and Replacement (Bridge Repair Practice), S.H. Park, Trenton, NJ, 1984.

27. "Pontis-User's Manual," Version 2.0, FHWA-SA-93-083, Federal Highway Administration, Office of Technology Applications, Washington, DC, December 1993.

28. "Proposed Rules," Federal Register, Vol. 56, No. 107, June 4, 1991.

29. "Rehabilitation of Existing Bridges Workshop," Participant Notebook, U.S. Department of Federal Highway Administration, Prepared by Byrd, Tallamy, McDonald and Lewis, Virginia, 1984.

30. "Rules and Regulations," Federal Register, Vol. 53, No. 166, August 26, 1988.

31. Saito, M., and Sinha, K.C., "Review of Current Practices of Bridge Management at the State Level," Transportation Research Record 1113, Transportation Research Board, Washington, DC, 1987.

32. Secretary of Transportation, "Highway Bridge Replacement and Rehabilitation Program: Eleventh Report to the United States Congress," April 1993.

33. Shanafelt, G.O., and Horn, W.B., "Guidelines for Evaluation and Repair of Prestressed Concrete Bridge Members," NCHRP Report 280, Transportation Research Board, Washington, DC, 1985.

34. United States Comptroller General, "Better Targeting of Federal Funds Needed to Eliminate Unsafe Bridges," Report to the honorable James R. Sasser, United States Senate, U.S. General Accounting Office, August 11, 1981.

35. United States Congress, "Surface Transportation and Uniform Relocation Assistance Act of 1987."

SOURCES OF ADDITIONAL INFORMATION

The following organizations publish information, manuals, and/or standards related to bridge inspection, evaluation, posting, and management. Publication catalogs may be obtained through to addresses given.

American Association of State Highway and Transportation Officials
444 North Capital Street, N.W., Suite 249
Washington, DC 20001
Tel. (202) 624-5800
Fax: (202) 624-5806

Transportation Research Board
National Research Council
2101 Constitution Avenue, N.W.
Washington, DC 20418
Tel. (202) 334-3214
Fax: (202) 334-2519

Office of Technology Applications
Federal Highway Administration
400 7th Street, S.W.
Washington, DC 20590
Tel.
Fax:

Innovation Through Partnerships

Office of Technology Applications
Federal Highway Administration
400 Seventh Street, SW.
Washington, D.C. 20590

Publication No. FHWA-SA-93-077