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Maintenance Inspection and Rating

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49.1 Introduction

Before the 1960s, little emphasis was given to inspection and maintenance of bridges in the United States. After the 1967 tragic collapse of the Silver Bridge at Point Pleasant in West Virginia, national interest in the inspection and maintenance rose considerably. The U.S. Congress passed the Federal Highway Act of 1968 which resulted in the establishment of the National Bridge Inspection Standard (NBIS). The NBIS sets the national policy regarding bridge inspection procedure, inspection frequency, inspector qualifications, reporting format, and rating procedures. In addition to the establishment of NBIS, three manuals — FHWA Bridge Inspector's Training Manual 70 [1], AASHO Manual for Maintenance Inspection of Bridges [2], and FHWA Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges [3] — have been developed and updated [4–10] since the 1970s. These manuals along with the NBIS provide definitive guidelines for bridge inspection. Over the past three decades, the bridge inspection program evolved into one of the most-sophisticated bridge management systems. This chapter will focus only on the basic, fundamental requirements for maintenance inspection and rating.

49.2 Maintenance Documentation

Each bridge document needs to have items such as structure information, structural data and history, description on and below the structure, traffic information, load rating, condition and appraisal ratings, and inspection findings. The inspection findings should have the signature of the inspection team leader.

All states in the United States are encouraged, but not mandated, to use the codes and instructions given in the Recording and Coding Guide [8,9] while documenting the bridge inventory. In order to maintain the nation's bridge inventory, FHWA requests all state agencies to submit data on the Structure Inventory and Appraisal (SI&A) Sheet. The SI&A sheet is a tabulation of pertinent information about an individual bridge. The information on SI&A sheet is a valuable aid to establish maintenance and replacement priorities and to determine the maintenance cost of the nation's bridges.

49.3 Fundamentals of Bridge Inspection

49.3.1 Qualifications and Responsibilities of Bridge Inspectors

The primary purpose of bridge inspection is to maintain the public safety, confidence, and investment in bridges. Ensuring public safety and investment decision requires a comprehensive bridge inspection. To this end, a bridge inspector should be knowledgeable in material and structural behavior, bridge design, and typical construction practices. In addition, inspectors should be physically strong because the inspection sometimes requires climbing on rough, steep, and slippery terrain, working at heights, or working for days.

Some of the major responsibilities of a bridge inspector are as follows:

- Identifying minor problems that can be corrected before they develop into major repairs;
- Identifying bridge components that require repairs in order to avoid total replacement;
- Identifying unsafe conditions;
- Preparing accurate inspection records, documents, and recommendation of corrective actions; and
- Providing bridge inspection program support.

In the United States, NBIS requires a field leader for highway bridge inspection teams. The field team leader should be either a professional engineer or a state certified bridge inspector, or a Level III bridge inspector certified through the National Institute for Certification of Engineering Technologies. It is the responsibility of the inspection team leader to decide the capability of individual team members and delegate their responsibilities accordingly. In addition, the team leader is responsible for the safety of the inspection team and establishing the frequency of bridge inspections.

49.3.2 Frequency of Inspection

NBIS requires that each bridge that is opened to public be inspected at regular intervals not exceeding 2 years. The underwater components that cannot be visually evaluated during periods of low flow or examined by feel for their physical conditions should be inspected at an interval not exceeding 5 years.

The frequency, scope, and depth of the inspection of bridges generally depend on several parameters such as age, traffic characteristics, state of maintenance, fatigue-prone details, weight limit posting level, and known deficiencies. Bridge owners may establish the specific frequency of inspection based on the above factors.

49.3.3 Tools for Inspection

In order to perform an accurate and comprehensive inspection, proper tools must be available. As a minimum, an inspector needs to have a 2-m (6-ft) pocket tape, a 30-m (100-ft) tape, a chipping hammer, scrapers, flat-bladed screwdriver, pocketknife, wire brush, field marking crayon, flashlight, plumb bob, binoculars, thermometer, tool belt with tool pouch, and a carrying bag. Other useful tools are a shovel, vernier or jaw-type calipers, lighted magnifying glass, inspection mirrors, dye penetrant, 1-m (4-ft) carpenter's level, optical crack gauge, paint film gauge, and first-aid kits. Additional special inspection tools are survey, nondestructive testing, and underwater inspection equipment.

Inspection of a bridge prompts several unique challenges to bridge inspectors. One of the challenges to inspectors is the accessibility of bridge components. Most smaller bridges can be accessed from below without great effort, but larger bridges need the assistance of accessing equipment and vehicles. Common access equipment are ladders, rigging, boats or barges, floats, and scaffolds. Common access vehicles are manlifts, snoopers, aerial buckets, and traffic protection devices. Whenever possible, it is recommended to access the bridge from below since this eliminates the need for traffic control on the bridge. Setting up traffic control may create several problems, such as inconvenience to the public, inspection cost, and safety of the public and inspectors.

49.3.4 Safety during Inspection

During the bridge inspection, the safety of inspectors and of the public using the bridge or passing beneath the bridge should be given utmost importance. Any accident can cause pain, suffering, permanent disability, family hardship, and even death. Thus, during the inspection, inspectors are encouraged to follow the standard safety guidelines strictly.

The inspection team leader is responsible for creating a safe environment for inspectors and the public. Inspectors are always encouraged to work in pairs. As a minimum, inspectors must wear safety vests, hard hats, work gloves, steel-toed boots, long-sleeved shirts, and long pants to ensure their personal safety. Other safety equipment are safety goggles, life jackets, respirator, gloves, and safety belt. A few other miscellaneous safety items include walkie-talkies, carbon monoxide detectors, and handheld radios.

Field clothes should be appropriate for the climate and the surroundings of the inspection location. When working in a wooded area, appropriate clothing should be worn to protect against poisonous plants, snakes, and disease-carrying ticks. Inspectors should also keep a watchful eye for potential hazardous environments around the inspection location. When entering a closed bridge box cells, air needs to be checked for the presence of oxygen and toxic or explosive gases. In addition, care should be taken when using existing access ladders and walkways since the ladder rungs may be rusted or broken. When access vehicles such as snoopers, booms, or rigging are used, the safe use of this equipment should be reviewed before the start of work.

49.3.5 Reports of Inspection

Inspection reports are required to establish and maintain a bridge history file. These reports are useful in identifying and assessing the repair requirements and maintenance needs of bridges. NBIS requires that the findings and results of a bridge inspection be recorded on standard inspection forms. Actual field notes and numerical conditions and appraisal ratings should be included in the standard inspection form. It is also important to recognize that these inspection reports are legal documents and could be used in future litigation.

Descriptions in the inspection reports should be specific, detailed, quantitative, and complete. Narrative descriptions of all signs of distress, failure, or defects with sufficient accuracy should be noted so that another inspector can make a comparison of condition or rate of disintegration in the future. One example of a poor description is, “Deck is in poor condition.” A better description would be, “Deck is in poor condition with several medium to large cracks and numerous spalls.” The seriousness and the amount of all deficiencies must be clearly stated in an inspection report.

In addition to inspection findings about the various bridge components, other important items to be included in the report are any load, speed, or traffic restrictions on the bridge; unusual loadings; high water marks; clearance diagram; channel profile; and work or repairs done to the bridge since the last inspection.

When some improvement or maintenance work alters the dimensions of the structure, new dimensions should be obtained and reported. When the structure plans are not in the history file, it may be necessary to prepare plans using field measurements. These measurements will later be used to perform the rating analysis of the structure.

Photographs and sketches are the most effective ways of describing a defect or the condition of structural elements. It is therefore recommended to include sketches and/or photographs to describe or illustrate a defect in a structural element. At least two photographs for each bridge for the record are recommended.

Other tips on photographs are

- Place some recognizable items that will allow the reviewer to visualize the scale of the detail;
- Include plumb bob to show the vertical line; and
- Include surrounding details so one could relate other details with the specific detail.

After inspecting a bridge, the inspector should come to a reasonable conclusion. When the inspector cannot interpret the inspection findings and determine the cause of a specific finding (defect), the advice of more-experienced personnel should be sought. Based on the conclusion, the inspector may need to make a practical recommendation to correct or preclude bridge defects or deficiencies. All instructions for maintenance work, stress analysis, posting, further inspection, and repairs should be included in the recommendation. Whenever recommendations call for bridge repairs, the inspector must carefully describe the type of repairs, the scope of the work, and an estimate of the quantity of materials.

49.4 Inspection Guidelines

49.4.1 Timber Members

Common damage in timber members is caused by fungi, parasites, and chemical attack. Deterioration of timber can also be caused by fire, impact or collisions, abrasion or mechanical wear, overstress, and weathering or warping.

Timber members can be inspected by both visual and physical examination. Visual examination can detect the following: fungus decay, damage by parasites, excessive deflection, checks, splits, shakes, and loose connections. Once the damages are detected visually, the inspector should investigate the extent of each damage and properly document them in the inspection report. Deterioration of timber can also be detected using sounding methods — a nondestructive testing method. Tapping on the outside surface of the member with a hammer detects hollow areas, indicating internal decay. There are a few advanced nondestructive and destructive techniques available. Two of the commonly used destructive tests are boring or drilling and probing. And, two of the nondestructive tests are Pol-Tek and ultrasonic testing. The Pol-Tek method is used to detect low-density regions and ultrasonic testing is used to measure crack and flaw size.

49.4.2 Concrete Members

Common concrete member defects include cracking, scaling, delamination, spalling, efflorescence, popouts, wear or abrasion, collision damage, scour, and overload. Brief descriptions of common damages are given in this section.

Cracking in concrete is usually large enough to be seen with the naked eye, but it is recommended to use a crack gauge to measure and classify the cracks. Cracks are classified as hairline, medium, or wide cracks. Hairline cracks cannot be measured by simple means such as pocket ruler, but simple means can be used for the medium and wide cracks. Hairline cracks are usually insignificant to the capacity of the structure, but it is advisable to document them. Medium and wide cracks are significant to the structural capacity and should be recorded and monitored in the inspection reports. Cracks can also be grouped into two types: structural cracks and nonstructural cracks. Structural cracks are caused by the dead- and live-load stresses. Structural cracks need immediate attention, since they affect the safety of the bridge. Nonstructural cracks are usually caused by thermal expansion and shrinkage of the concrete. These cracks are insignificant to the capacity, but these cracks may lead to serious maintenance problems. For example, thermal cracks in a deck surface may allow water to enter the deck concrete and corrode the reinforcing steel.

Scaling is the gradual and continuing loss of surface mortar and aggregate over an area. Scaling is classified into four categories: light, medium, heavy, and severe.

Delamination occurs when layers of concrete separate at or near the level of the top or outermost layer of reinforcing steel. The major cause of delamination is the expansion or the corrosion of reinforcing steel due to the intrusion of chlorides or salts. Delaminated areas give off a hollow sound when tapped with a hammer. When a delaminated area completely separates from the member, a roughly circular or oval depression, which is termed as spall, will be formed in the concrete.

The inspection of concrete should include both visual and physical examination. Two of the primary deteriorations noted by visual inspections are cracks and rust stains. An inspector should recognize the fact that not all cracks are of equal importance. For example, a crack in a prestressed concrete girder beam, which allows water to enter the beam, is much more serious than a vertical crack in the backwall. A rust stain on the concrete members is one of the signs of corroding reinforcing steel in the concrete member. Corroded reinforcing steel produces loss of strength within concrete due to reduced reinforced steel section, and loss of bond between concrete and reinforcing steel. The length, direction, location, and extent of the cracks and rust stains should be measured and reported in the inspection notes.

Some common types of physical examination are hammer sounding and chain drag. Hammer sounding is used to detect areas of unsound concrete and usually used to detect delaminations. Tapping the surfaces of a concrete member with a hammer produces a resonant sound that can be used to indicate concrete integrity. Areas of delamination can be determined by listening for hollow sounds. The hammer sounding method is impractical for the evaluation of larger surface areas. For larger surface areas, chain drag can be used to evaluate the integrity of the concrete with reasonable accuracy. Chain drag surveys of decks are not totally accurate, but they are quick and inexpensive.

There are other advanced techniques — destructive and nondestructive — available for concrete inspection. Core sampling is one of the destructive techniques of concrete inspection. Some of the nondestructive inspection techniques are

- Delamination detection machinery to identify the delaminated deck surface;
- Copper sulfate electrode, nuclear methods to determine corrosion activity;
- Ground-penetrating radar, infrared thermography to detect deck deterioration;
- Pachometer to determine the position of reinforcement; and
- Rebound and penetration method to predict concrete strength.

49.4.3 Steel and Iron Members

Common steel and iron member defects include corrosion, cracks, collision damage, and overstress. Cracks usually initiate at the connection detail, at the termination end of a weld, or at a corroded location of a member and then propagate across the section until the member fractures. Since all of the cracks may lead to failure, bridge inspectors need to look at each and every one of these potential crack locations carefully. Dirt and debris usually form on the steel surface and shield the defects on the steel surface from the naked eye. Thus, the inspector should remove all dirt and debris from the metal surface, especially from the surface of fracture-critical details, during the inspection of defects.

The most recognizable type of steel deterioration is corrosion. The cause, location, and extent of the corrosion need to be recorded. This information can be used for rating analysis of the member and for taking preventive measures to minimize further deterioration. Section loss due to corrosion can be reported as a percentage of the original cross section of a component. The corrosion section loss is calculated by multiplying the width of the member and the depth of the defect. The depth of the defect can be measured using a straightedge ruler or caliper.

One of the important types of damage in steel members is fatigue cracking. Fatigue cracks develop in bridge structures due to repeated loadings. Since this type of cracking can lead to sudden and catastrophic failure, the bridge inspector should identify fatigue-prone details and should perform a thorough inspection of these details. For painted structures, breaks in the paint accompanied by rust staining indicate the possible existence of a fatigue crack. If a crack is suspected, the area should be cleaned and given a close-up visual inspection. Additionally, further testing such as dye penetrant can be done to identify the crack and to determine its extent. If fatigue cracks are discovered, inspection of all similar fatigue details is recommended.

Other types of damage may occur due to overstress, vehicular collision, and fire. Symptoms of damage due to overstress are inelastic elongation (yielding) or decrease in cross section (necking) in tension members, and buckling in compression members. The causes of the overstress should be investigated. The overstress of a member could be the result of several factors such as loss of composite action, loss of bracing, loss of proper load-carrying path, and failure or settlement of bearing details.

Damage due to vehicular collision includes section loss, cracking, and shape distortion. These types of damage should be carefully documented and repair work process should be initiated. Until the repair work is completed, restriction of vehicular traffic based on the rating analysis results is recommended.

Similar to timber and concrete members, there are advanced destructive and nondestructive techniques available for steel inspection. Some of the nondestructive techniques used in steel bridges are

- Acoustic emissions testing to identify growing cracks;
- Computer tomography to render the interior defects;
- Dye penetrant to define the size of the surface flaws; and
- Ultrasonic testing to detect cracks in flat and smooth members.

49.4.4 Fracture-Critical Members

Fracture-critical members (FCM) or member components are defined as tension components of members whose failure would be expected to result in collapse of a portion of a bridge or an entire bridge [7,8]. A redundant steel bridge that has multiple load-carrying mechanisms is seldom categorized as a fracture-critical bridge.

Since the failure to locate defects on FCMs in a timely manner may lead to catastrophic failure of a bridge, it is important to ensure that FCMs are inspected thoroughly. Hands-on involvement of the team leader is necessary to maintain the proper level of inspection and to make independent checks of condition appraisals. In addition, adequate time to conduct a thorough inspection should be allocated by the team leader. Serious problems in FCMs must be addressed immediately by restricting traffic on the bridge and repairing the defects under an emergency contract. Less serious problems requiring repairs or retrofit should be placed on the programmed repair work so that they will be incorporated into the maintenance schedule.

Bridge inspectors need to identify the FCMs using the guidelines provided in the Inspection of Fracture Critical Bridge Members [7,8]. There are several vulnerable fracture-critical locations in a bridge. Some of the obvious locations are field welds, nonuniform welds, welds with unusual profile, and intermittent welds along the girder. Other possible locations are insert plate termination points, floor beam to girder connections, diaphragm connection plates, web stiffeners, areas that are vulnerable to corrosion, intersecting weld location, sudden change in cross section, and coped sections. Detailed descriptions of each of these fracture-critical details are listed in the Inspection of Fracture Critical Bridge Members [7,8]. Once the FCM is identified in a bridge structure, information such as location, member components, likelihood to have fatigue- or corrosion related damage, needs to be gathered. The information gathered on the member should become a permanent record and the condition of the member should be updated on every subsequent inspection.

FCMs can be inspected by both visual and physical examination. During the visual inspection, the inspector performs a close-up, hands-on inspection using standard, readily available tools. During the physical examination, the inspector uses the most-sophisticated nondestructive testing methods. Some of the FCMs may have details that are susceptible to fatigue cracking and others may be in poor condition due to corrosion. The inspection procedures of corrosion- and fatigue-prone members are described in Section 49.4.3.

49.4.5 Scour-Critical Bridges

Bridges spanning over waterways, especially rivers and streams, sometimes provide major maintenance challenges. These bridges are susceptible to scour of the riverbed. When the scoured riverbed elevation falls below the top of the footing, the bridge is referred to as scour critical.

The rivers, whether small or large, could significantly change their size over the period of the lifetime of a bridge. A riverbed could be altered in several ways and thereby jeopardize the stability of the bridges. A few of the possible types of riverbed alterations are scour, hydraulic opening, channel misalignment, and bank erosion. Scour around the bridge substructures poses potential structural stability concerns. Scour at bridges depends on the hydraulic features upstream and downstream, riverbed sediments, substructure section profile, shoreline vegetation, flow velocities, and potential debris. The estimation of the overall scour depth will be used to identify scour-prone and scour-critical bridges. Guidance for the scour evaluation process is provided in Evaluating Scour at Bridges [11].

A typical scour evaluation process falls into two phases: inventory phase and evaluation phase. The main goal of the inventory phase is to identify those bridges that are vulnerable to scour (scour-prone bridges). Evaluation during this phase is made using the available bridge records, inspection records, history of the bridge, original stream location, evidence of scour, deposition of debris, geology, and general stability of the streambed. Once the scour-prone bridges are identified, the evaluation phase needs to be performed. The scour evaluation phase requires in-depth field review to generate data for estimation of the hydraulics and scour depth. The procedure of scour estimation is outlined in Evaluating Scour at Bridges [11]. The scour depths are then compared with the existing foundation condition. When the scour depth is above the top of the footing, the bridge would require no action. However, when the scour depth is within the limits of the footing or piles, a structural stability analysis is needed. If the scour depth is below the pile tips or spread footing base, monitoring of the bridge is required. These results obtained from the scour evaluation process are entered into the bridge inventory.

49.4.6 Underwater Components

Underwater components are mostly substructure members. Since the accessibility of these members is difficult, special equipment is necessary to inspect these underwater components. Also, visibility during the underwater inspection is generally poor, and therefore a thorough inspection of the members will not be possible. Underwater inspection is classified as visual (Level 1), detailed (Level 2), and comprehensive (Level 3) to specify the level of effort of inspection. Details of these various levels of inspection are discussed in the Manual for Maintenance Inspection of Bridges [2] and Evaluating Scour at Bridges [11].

Underwater steel structure components are susceptible to corrosion, especially in the low to high water zone. Some of the defects observed in underwater timber piles are splitting, decay or rot, marine borers, decay of timber at connections, and corrosion of connectors. It is important to recognize that the timber piles may appear sound on the outside shell but be severely damaged inside. Some of the most common defects in underwater concrete piles are cracking, spalls, exposed reinforcing, sulfate attack, honeycombing, and scaling. When cracking, spalls, and exposed reinforcing are detected, structural analysis may be required to ensure the safety of the bridge.

49.4.7 Decks

The materials typically used in the bridge structures are concrete, timber, and steel. Sections 49.4.1 to 49.4.3 discuss some of the defects associated with each of these materials. In this section, the damage most likely to occur in bridge decks is discussed.

Common defects in steel decks are cracked welds, broken fasteners, corrosion, and broken connections. In a corrugated steel flooring system, section loss due to corrosion may affect the load-carrying capacity of the deck and thus the actual amount of remaining materials needs to be evaluated and documented.

Common defects in timber decks are crushing of the timber deck at the supporting floor system, flexure damages such as splitting, sagging, and cracks in tension areas, and decay of the deck due to biological organisms, especially in the areas exposed to drainage.

Common defects in concrete decks are wear, scaling, delamination, spalls, longitudinal flexure cracks, transverse flexure cracks in the negative moment regions, corrosion of the deck rebars, cracks due to reactive aggregates, and damage due to chemical contamination. The importance of a crack varies with the type of concrete deck. A large to medium crack in a noncomposite deck may not affect the load-carrying capacity of the main load-carrying member. On the other hand, several cracks in a composite deck will affect the structural capacity. Thus, an inspector must be able to identify the functions of the deck while inspecting it.

Sometimes a layer of asphalt concrete (AC) overlay will be placed to provide a smooth driving and wearing surface. Extra care is needed during the inspection, because AC overlay prevents the inspector's ability to inspect the top surface of the deck visually for damage.

49.4.8 Joint Seals

Damage to the joint seals is caused by vehicle impact, extreme temperature, and accumulation of dirt and debris. Damage from debris and vehicles such as snowplows could cause the joint seals to be torn, pulled out of anchorage, or removed altogether. Damage from extreme temperature could break the bond between the joint seal and deck and consequently result in pulling out the joint seal altogether.

The primary function of deck joints is to accommodate the expansion and contraction of the bridge superstructure. These deck joints also provide a smooth transition from the approach roadway to the bridge deck. Deck joints are placed at hinges between two decks of adjacent structures, and between the deck sections and abutment backwall. The joint seals used in the bridge industry can be divided into two groups: open joints and closed joints. Open joints allow water and debris to pass through the joints. Dripping water through open joints usually damages the bearing details. Closed joints do not allow water and debris to pass through them. A few of the closed joints are compression seal, poured joint seal, sliding plate joint, plank seal, sheet seal, and strip seal.

In the case of closed joints, damage to the joint seal material will cause the water to drip on the bearing seats and consequently damage the bearing. Accumulation of dirt and debris may prevent normal thermal expansion and contraction, which may in turn cause cracking in the deck, backwall, or both. Cracking in the deck may affect the ride quality of the bridge, may produce larger impact load from vehicles, and may reduce the live-load-carrying capacity of the bridge.

49.4.9 Bearings

Bearings used in bridge structures could be categorized into two groups: metal and elastomeric. Metal bearings sometimes become inoperable (sometimes referred as "frozen") due to corrosion, mechanical bindings, buildup of debris, or other interference. Frozen bearings may result in bending, buckling, and improper alignment of members. Other types of damage are missing fasteners, cracked welds, corrosion on the sliding surface, sole plate rests only on a portion of the masonry plate, and binding of lateral shear keys.

Damage in elastomeric bearing pads is excessive bulging, splitting or tearing, shearing, and failure of bond between sole and masonry plate. Excessive bulging indicates that the bearing might be too tall. When the pad is under excessive strain for a long period, the pad will experience shearing failure.

Inspectors need to assess the exact condition of the bearing details and to recommend corrective measures that allow the bearing details to function properly. Since the damage to the bearings will affect the other structural members as time passes, repair of bearing damage needs to be considered as a preventive measure.

49.5 Fundamentals of Bridge Rating

49.5.1 Introduction

Once a bridge is constructed, it becomes the property of the owner or agency. The evaluation or rating of existing bridges is a continuous activity of the agency to ensure the safety of the public. The evaluation provides necessary information to repair, rehabilitate, post, close, or replace the existing bridge.

In the United States, since highway bridges are designed for the AASHTO design vehicles, most U.S. engineers tend to believe that the bridge will have adequate capacity to handle the actual present traffic. This belief is generally true if the bridge was constructed and maintained as shown in the design plan. However, changes in a few details during the construction phase, failure to attain the recommended concrete strength, unexpected settlements of the foundation after construction, and unforeseen damage to a member could influence the capacity of the bridge. In addition, old bridges might have been designed for a lighter vehicle than is used at present, or a different design code. Also, the live-load-carrying capacity of the bridge structure may have altered as a result of deterioration, damage to its members, aging, added dead loads, settlement of bents, or modification to the structural member.

Sometimes, an industry would like to transport their heavy machinery from one location to another location. These vehicles would weigh much more than the design vehicles and thus the bridge owner may need to determine the current live-load-carrying capacity of the bridge. In the following sections, establishing the live load-carrying capacity and the bridge rating will be discussed.

49.5.2 Rating Principles

In general, the resistance of a structural member (R) should be greater than the demand (Q) as follows:

$$R \geq Q_d + Q_l + \sum_i Q_i \tag{49.1}$$

where Q_d is the effect of dead load, Q_l is the effect of live load, and Q_i is the effect of load i.

Eq. (49.1) applies to design as well as evaluation. In the bridge evaluation process, maximum allowable live load needs to be determined. After rearranging the above equation, the maximum allowable live load will become

$$Q_l \leq R - \left(Q_d + \sum_i Q_i \right) \tag{49.2}$$

Maintenance engineers always question whether a fully loaded vehicle (rating vehicle) can be allowed on the bridge and, if not, what portion of the rating vehicle could be allowed on a bridge. The portion of the rating vehicle will be given by the ratio between the available capacity for live-load effect and the effect of the rating vehicle. This ratio is called the rating factor (RF).

$$\text{RF} = \frac{\text{Available capacity for the live-load effect}}{\text{Rating vehicle load demand}} = \frac{R - \left(Q_d + \sum_i Q_i \right)}{Q_l} \tag{49.3}$$

When the rating factor equals or exceeds unity, the bridge is capable of carrying the rating vehicle. On the other hand, when the rating factor is less than unity the bridge may be overstressed while carrying the rating vehicle.

The capacity of a member is usually independent of the live-load demand. Thus, Eq. (49.3) is generally a linear expression. However, there are cases where the capacity of a member dependent on the live-load forces. For example, available moment capacity depends on the total axial load in biaxial bending members. In a biaxially loaded member, the Eq. (49.3) will be a second-order expression.

Thermal, wind, and hydraulic loads may be neglected in the evaluation process because the likelihood of occurrence of extreme values during the relatively short live-load loading is small. Thus, the effects of the dead and live loads are the only two loads considered in the evaluation process.

49.5.3 Rating Philosophies

During the structural evaluation process, the location and type of critical failure modes are first identified; Eq. (49.3) is then solved for each of these potential failures. Although the concept of evaluation is the same, the mathematical relationship of this basic equation for allowable stress design (ASD), load factor design (LFD), and Load and resistance factor design (LRFD) differs. Since the resistance and load effect can never be established with certainty, engineers use safety factors to give adequate assurance against failure. ASD includes safety factors in the form of allowable stresses of the material. LFD considers the safety factors in the form of load factors to account for the uncertainty of the loadings and resistance factors to account for the uncertainty of structural response. LRFD treats safety factors in the form of load and resistance factors that are based on the probability of the loadings and resistances.

For ASD, the rating factor expression Eq. (49.3) can be written as

$$\mathrm{RF} = \frac{R - \left(\sum D + \sum_i L_i (1+I) \right)}{L(1+I)} \tag{49.4}$$

For LFD, the rating factor expression Eq. (49.3) can be written as

$$\mathrm{RF} = \frac{\phi\, R_n - \sum \gamma_D D - \sum_{i=1}^{n} \gamma_{Li} L_i (1+I)}{\gamma_L L(1+I)} \tag{49.5}$$

For LFRD, the rating factor expression Eq. (49.3) can be written as

$$\mathrm{RF} = \frac{\phi\, R_n - \sum \gamma_D D - \sum_{i=1}^{n} \gamma_{Li} L_i (1+I)}{\gamma_L L(1+I)} \tag{49.6}$$

where R is the allowable stress of the member; ϕR_n is nominal resistance; D is the effect of dead loads; L_i is the live-load effect for load i other than the rating vehicle; L the nominal live-load effect of the rating vehicle; I is the impact factor for the live-load effect; γ_D, γ_{Li}, and γ_L are dead- and live-load factors, respectively.

Researchers are now addressing the LRFD method, and thus the LRFD approach may be revised in the near future. Since the LRFD method is being developed at this time, the LRFD method is not discussed further in this chapter.

In order to use the above equations (Eqs. 49.4 to 49.6) in determining the rating factors, one needs to estimate the effects of individual live-load vehicles. The effect of individual live-load vehicles on structural member could only be obtained by analyzing the bridge using a three-dimensional analysis. Thus, obtaining the rating factor using the above expressions is very difficult and time-consuming.

To simplify the above equations, it is assumed that similar rating vehicles will occupy all the possible lanes to produce the maximum effect on the structure. This assumption allows us to use the AASHTO live-load distribution factor approach to estimate the live-load demand and eliminate the need for the three-dimensional analysis.

And the simplified rating factor equations become as follows:

$$\text{For ASD:} \quad \mathrm{RF} = \frac{R - D}{L(1+I)} \tag{49.7}$$

$$\text{For LFD:} \quad \mathrm{RF} = \frac{\phi R_n - \gamma_D D}{\gamma_L L(1+I)} \tag{49.8}$$

$$\text{For LRFD:} \quad \mathrm{RF} = \frac{\phi\, R_n - \gamma_D D}{\gamma_L L(1+I)} \tag{49.9}$$

In the derivation of the above equations (Eqs. 49.7 to 49.9), it is assumed that the resistance of the member is independent of the loads. A few exceptions to this assumption are beam–column members and beams with high moment and shear. In a beam–column member, axial capacity or moment capacity depends on the applied moment or applied axial load on the member. Thus, as the live-load forces in the member increase, the capacity of the member would decrease. In other words, the numerator of the above equations (available live-load capacity) will drop as the live load increases. Thus, the rating factor will no longer be a constant value, and will be a function of live load.

49.5.4 Level of Ratings

There are two levels of rating for bridges: inventory and operating. The rating that reflects the absolute maximum permissible load that can be safely carried by the bridge is called an operating rating. The load that can be safely carried by a bridge for indefinite period is called an inventory rating.

The life of a bridge depends on the fatigue life or serviceability limits of bridge materials. Higher frequent loading and unloading may affect the fatigue life or serviceability of a bridge component and thereby the life of the bridge. Thus, in order to maintain a bridge for an indefinite period, live-load-carrying capacity available for frequently passing vehicles needs to be estimated at service. This process is referred to as inventory rating.

Less frequent vehicles may not affect the fatigue life or serviceability of a bridge, and thus live-load-carrying capacity available for less frequent vehicles need not be estimated using serviceability criteria. In addition, since less frequent vehicles do not damage the bridge structure, bridge structures could be allowed to carry higher loads. This process is referred to as operating rating.

49.5.5 Structural Failure Modes

In the ASD approach, when a portion of a structural member is stressed beyond the allowable stress, the structure is considered failed. In addition, since any portion of the structural member material never reaches its yield, the deflections or vibrations will always be satisfied. Thus, the serviceability of a bridge is assured when the allowable stress method is used to check a bridge member. In other words, in the ASD approach, serviceability and strength criteria are satisfied automatically. The inventory and operating allowable stresses for various types of failure modes are given in the AASHTO Manual for Condition Evaluation of Bridges 1994 [12] (Rating Manual).

In the LFD approach, failure could occur at two different limit states: serviceability and strength. When the load on a member reaches the ultimate capacity of the member, the structure is considered failed at its ultimate strength limit state. When the structure reaches its maximum allowable serviceability limits, the structure is considered failed at its serviceability limit state. In LFD approach, satisfying one of the limit states will not automatically guarantee the satisfaction of the other limit state. Thus, both serviceability and strength criteria need to be checked in the LFD method. However, when the operating rating is estimated, the serviceability limits need not be checked.

49.6 Superstructure Rating Examples

In this section, several problems are illustrated to show the bridge rating procedures. In the following examples, AASHTO *Standard Specification for Highway Bridges,* 16th ed.[13] is referred to as Design Specifications and AASHTO *Manual for Condition Evaluation of Bridges* 1994 [12] is referred to as Rating Manual. All the notations used in these examples are defined in either the Design Specifications or the Rating Manual.

49.6.1 Simply Supported Timber Bridge

Given

Typical cross section of a 16-ft (4.88-m) long simple-span timber bridge is shown in Figure 49.1. 13.4 × 16 in. (101.6 × 406.4 mm) timber stringers are placed at 18 in. (457 mm) spacing. 4 × 12 in.

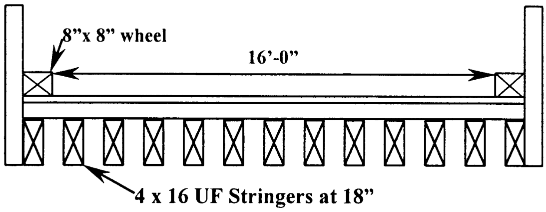

FIGURE 49.1 Typical cross section detail of simply supported timber bridge example.

(101.6 × 305 mm) timber planks are used as decking. 8 × 8 in. (203 × 203 mm) timber is used as wheel guard. Barrier rails (10 lb/ft or 0.1 N/mm) are placed at either side of the bridge. The traffic lane width of the bridge is 16 ft (4.88 m). Assume that the allowable stresses at operating level are as follows: F_b for stringer as 1600 psi (11 MPa) and F_v of stringer level as 115 psi (0.79 MPa).

Requirement
Determine the critical rating factors for interior stinger for HS20 vehicle using the ASD approach.

Solution
For this simply supported bridge, the critical locations for ratings will be the locations where shear and moments are higher.

According to Design Specifications Section 13.6.5.2, shear needs to be checked at a distance (s) $3d$ or $0.25L$ from the bearing location for vehicle live loads; thus,

$$s = 3d = 3 \times 16 \text{ in.}/12 = 4.0 \text{ ft or}$$

$$= 0.25L = 0.25 \times 16 \text{ ft} = 4.0 \text{ ft. Thus, } s \text{ is taken as } 4.0 \text{ ft } (1.22 \text{ m}).$$

Maximum dead- and live-load shear will occur at this point and thus in the following calculations, shear is estimated at this critical location.

1. Dead Load Calculations

Self-weight of the stringer $= 0.05 \times 4 \times 16 \times \dfrac{1}{144} = 0.022$ kips/ft

Weight of deck (using tributary area) $= 1.5 \times 4 \times 12 \times \dfrac{1}{144} \times 0.05 = 0.025$ kips/ft

Weight of 1.5 in. AC on the deck $= 1.5 \times \dfrac{1.5}{12} \times 0.144 = 0.027$ kips/ft

Barrier rail and curb $= \left(10 + 50 \times \dfrac{8 \times 8}{144}\right) \times \dfrac{2}{13 \times 1000} = 0.004$ kips/ft

Total uniform dead load on the stringer $= 0.022 + 0.025 + 0.027 + 0.004$

$= \underline{0.078 \text{ kips/ft}}$

Maximum dead load moment at midspan $= \dfrac{wl^2}{8} = \dfrac{0.078 \times 16^2}{8} = 2.5$ kip-ft (3390 N-m)

Maximum dead load shear at this critical point $= w \times (0.5L - s)$

$= 0.078 \times (0.5 \times 16 - 4)$

$= 0.31$ kips (1.38 kN)

2. **Live-Load Calculations**
   The travel width is less than 18 ft. Thus, according to Section 6.7.2.2 of the Rating Manual, this bridge needs to be rated for one traffic lane. From Designs Specifications Table 3.23.1,

$$\text{Number of wheels on the stringer} = \frac{S}{4} = \frac{1.5}{4} = 0.38$$

Maximum moment due to HS20 loading (Appendix A3, Rating Manual)

= (64) (0.38)
= 24.32 kip-ft (33,000 N-m)

In order to estimate the live-load shear, we need to estimate the shear due to undistributed and distributed HS20 loadings. (See Design Specifications 13.6.5.2 for definition of V_{LU} and V_{LD})

Shear due to undistributed HS20 loadings $= V_{LU}$ $= 16 \times 12/16 = 12$ kips (53.4 kN)

Shear due to distributed HS20 loading $= V_{LD}$ $= 16 \times 12/16 \times (0.38)$

$= 4.56$ kips (20.3 kN)

Thus, shear due to HS20 live load $= 0.5(0.6V_{LU} + V_{LD})$ $= 5.88$ kips (26.1 kN)

3. **Capacity Calculations**
   a. *Moment capacity at midspan*:
   Moment capacity of the timber stringer at *Operating level* =

$$F_b S_x = 1600 \times \frac{1}{6} \times 4 \times 16^2 \times \frac{1}{12{,}000} = 22.8 \text{ kip-ft } (30{,}900 \text{ N-m})$$

According to Section 6.6.2.7 of Rating Manual, the operating level stress of a timber stringer can be taken as 1.33 times the inventory level stress.
Thus, moment capacity of the timber stringer at *Inventory level*

= 22.8/1.33
= 17.1 kip-ft (23,200 N-m)

   b. *Shear capacity at support*:
   Shear capacity of the timber section (controlled by horizontal shear) = $(2/3)bdf_v$:

V_c at *operating level* = (2/3) × 4 × 16 × 115 psi × 1/1000 = 4.91 kips (21.8 kN)

V_c at *inventory level* = 14.91/1.33 = 3.69 kips (16.4 kN)

4. **Rating Calculations**

$$\text{Rating factor based on ASD method} = \text{RF} = \frac{R - D}{L(1+I)}$$

By substituting appropriate values, the rating factor can be determined.

a. *Based on moment at midspan*:

$$\text{Inventory rating factor } RF_{INV\text{-}MOM} = \frac{17.1 - 2.5}{24.32} = 0.600$$

$$\text{Operating rating factor } RF_{OPR\text{-}MOM} = \frac{22.8 - 2.5}{24.32} = 0.835$$

b. *Based on shear at the support*:

$$\text{Inventory rating factor } RF_{INV\text{-}SHE} = \frac{3.69 - 0.31}{5.88} = 0.575$$

$$\text{Operating rating factor } RF_{OPR\text{-}SHE} = \frac{4.91 - 0.31}{5.88} = 0.782$$

5. **Summary**

It is found that the critical rating factor is controlled by shear in the stringers. The critical inventory and operating rating of the bridge will be 0.575 and 0.782, respectively.

49.6.2 Simply Supported T-Beam Concrete Bridge

Given

A bridge, which was built in 1929, consists of three simple-span reinforced concrete T-beams on concrete bents and abutments. The span lengths are 16 ft (4.88 m), 50 ft (15.24 m), and 10 ft (3.05 m). Typical cross section and girder details are shown in Figure 49.2. General notes given in the plan indicate that f_c = 1000 psi (6.9 MPa) and f_s = 18,000 psi (124.1 MPa). Assume the weight of each barrier rail as 250 lb/ft (3.6 N/mm).

Requirement

Determine the critical rating factor of the interior girder of the second span (50 ft. or 15.24 m) for HS20 vehicles assuming no deterioration of materials occurred.

Solution

1. **Dead-Load Calculations**

Self-weight of the girder = (3.5) (1.333) (0.15) = 0.700 kips/ft

(4 × 4 in.) Fillets between girder and slab = 2(1/2) (4/12) (4/12) (0.15) = 0.017 kips/ft

Slab weight (based on tributary area) = (6.667)(8/12) (0.15) = 0.667 kips/ft

Contribution from barrier rail (equally distributed among girders) = 2 (0.25/3) = 0.167 kips/ft

$$\text{Thus, total uniform load on the interior girder} = \underline{1.551 \text{ kips/ft}} \; (22.6 \text{ N/mm})$$

$$\text{Dead-load moment at midspan} = \underline{484.6 \text{ kips/ft} \; (0.657 \text{ MN/m})}$$

$$\text{Dead load shear at a distance } d \text{ from support} = \underline{32.31 \text{ kips} \; (143.7 \text{ kN})}$$

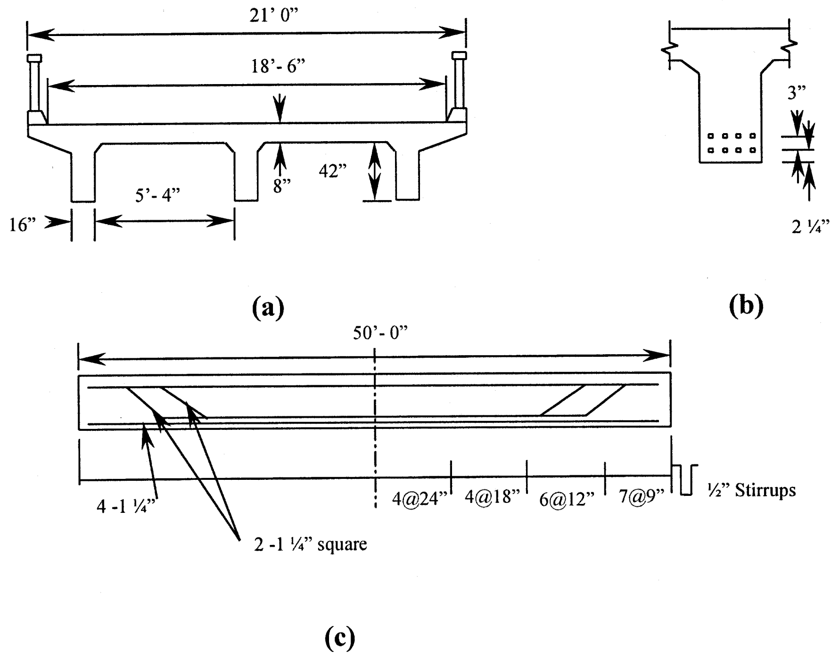

FIGURE 49.2 Details of simply supported T-beam concrete bridge example. (a) Typical cross section; (b) reinforcement locations; (c) T-beam girder details.

2. **Live-Load Calculations**
   The traffic lane width of this bridge is 18.5 ft. According to Design Specifications, any bridge with a minimum traffic lane width of 18 ft needs to carry two lanes. Hence, the number of live-load wheels will be based on two traffic lanes. From Table 3.23.1A of Design Specifications for two traffic lanes for T-beams is given by $S/6.0$

   Number of live-load wheel line = 6.667/6.0 = $\underline{1.111}$

   AASHTO standard impact factor for moment = 50/(125 + 50) = $\underline{0.286}$

   AASHTO standard impact factor for shear at support = 50/(125 + 50) = $\underline{0.286}$

   The live-load moments and shear tables listed in the Rating Manual are used to determine the live-load demand.

Maximum HS20 moment for 50 ft span without impact/wheel line = 298.0 kips-ft

Thus, HS20 moment with impact at midspan = (1.286) (1.111) (298.0)

$= \underline{425.7 \text{ kips-ft } (0.58 \text{ MN-m})}$

Maximum HS20 shear at a distance d from the support/wheel line = 28.32 kips

Thus, maximum HS20 shear = (1.286) (1.111) (28.32) $= \underline{40.46 \text{ kips } (180.0 \text{ kN})}$

3. **Capacity Calculations**

Strengths of concrete and rebars are first determined (see Rating Manual Section 6.6.2.3):

$$f_c' = \frac{f_c}{0.4} \quad \text{thus} \quad f_c' = 2500 \text{ psi}$$

and f_s = 18,000 psi and thus f_y = 33,000 psi.

a. *Moment capacity at midspan*:

Total area of the steel (note these bars are 1¼ square bars) = (8) (1.25) (1.25) = 12.5 in.^{2}

Centroid of the rebars from top deck = 42 + 8 – 3.75 = 46.250 in.

Effective width of the deck b_{eff} = minimum of $12t_s + b_w$ = 112 in.

Span/4 = 150 in.

Spacing = 80 in. (Controls)

Uniform stress block depth = $a = \dfrac{A_s f_y}{0.85 f_c' b_{\text{eff}}} = 2.426$ in. $< t_s = 8$ in.

$$M_u = \phi A_s f_y \left(d - \frac{a}{2}\right) = 0.9 \times 12.5 \times 33 \left(46.25 - \frac{2.426}{2}\right) \times \left(\frac{1}{12}\right) = 1393.3 \text{ kips-ft (1.88 MN-m)}$$

b. *Shear capacity at support*:

According to AASHTO specification, shear at a distance *d* (50 in.) from the support needs to be designed. Thus, the girder is rated at a distance *d* from the support. From the girder details, it is estimated that ½ in. ϕ stirrups were placed at a spacing of 12 in. and two 1¼ square bars were bent up. The effects of these bent-up bars are ignored in the shear capacity calculations.

Shear capacity due to concrete section:

$$V_c = 2\sqrt{f_c'}\, b_w d = 2\sqrt{2500} \times 16 \times 46.25 \times \left(\frac{1}{1000}\right) = 74 \text{ kips (329 kN)}$$

Shear capacity due to shear reinforcement:

$$V_s = 2A_v \frac{F_y d_s}{S} = 2 \times 0.20 \times \frac{33 \times 46.25}{12} = 50.88 \text{ kips (226 kN)}$$

Total shear capacity:

$$V_u = \phi (V_s + V_c) = 0.85\,(74.0 + 50.88) = 106.2 \text{ kips (472 kN)}$$

4. **Rating Calculations**

$$\text{Rating factor} = \frac{\phi R_n - \gamma_D D}{\gamma_L \beta_L L(1+I)}$$

TABLE 49.1 Rating Calculations of Simply Supported T-Beam Concrete Bridge Example

Location	Description	Inventory Rating	Operating Rating
Midspan	Moment	$\frac{1393.3 - 1.3 \times 484.6}{1.3 \times 1.67 \times 425.7} = 0.825$	$\frac{1393.3 - 1.3 \times 484.6}{1.3 \times 425.7} = 1.38$
At support	Shear	$\frac{106.2 - 1.3 \times 32.31}{1.3 \times 1.67 \times 40.46} = 0.731$	$\frac{106.2 - 1.3 \times 32.31}{1.3 \times 40.46} = 1.22$

According to Rating Manual, γ_D is 1.3, γ_L is 1.3, and β_L is 1.67 and 1.0 for inventory and operating factors, respectively. By substituting these values and appropriate load effect values, the moment and shear rating could be estimated. The calculations and results are given in Table 49.1.

5. **Summary**
   Critical rating of the interior girder will then be 0.731 at inventory level and 1.22 at operating rating level for HS20 vehicle.

49.6.3 Two-Span Continuous Steel Girder Bridge

Given
Typical section of a two-span continuous steel girder bridge, which was built in 1967, is shown in Figure 49.3a. Steel girder profile is given in Figure 49.3b. The general plan states that f_s = 20,000 psi (137.9 MPa) and f_c = 1200 psi (8.28 MPa). Assume that (a) each barrier rail weighs 250 lb/ft (3.6 N/mm); (b) girders were not temporarily supported during the concrete pour; (c) girder is composite for live loads; (d) girder is braced every 15 ft and the weight of bracing per girder is 330 lb.

Requirement
Determine the rating factors of interior girders using ASD method.

Solution

1. **Dead Load Calculations**

Deck weight (tributary area approach) = (6.625/12) (6.625) (0.15) = 0.549 kips/ft

Average uniform self-weight for the analysis = 1431 kips/90 ft = 0.159 kips/ft

Average diaphragm load (uniformly distributed) = (0.33) (4/90) = 0.015 kips/ft

$$\text{Thus, total uniform dead load on the girder} = \underline{0.723 \text{ kips/ft} \, (10.5 \text{ N/mm})}$$

Barrier rail load (equally distributed among all girders) = (2)(250)/14 = 0.0358 kips/ft

$$\text{Thus, total additional dead load on the girder} = \underline{0.0358 \text{ kips/ft}} (0.56 \text{ N/mm})$$

2. **Live Load Calculations**

$$\text{Number of wheels per girder (for two or more lanes)} = S/5.5 = 6.625/5.5 = \underline{1.206}$$

Analysis Results: Analysis is done using two-dimensional program and the moments and shears at critical locations are listed in the Table 49.2.

Section properties at 0.4th and 1.0th points are estimated and the results are given in Table 49.3.

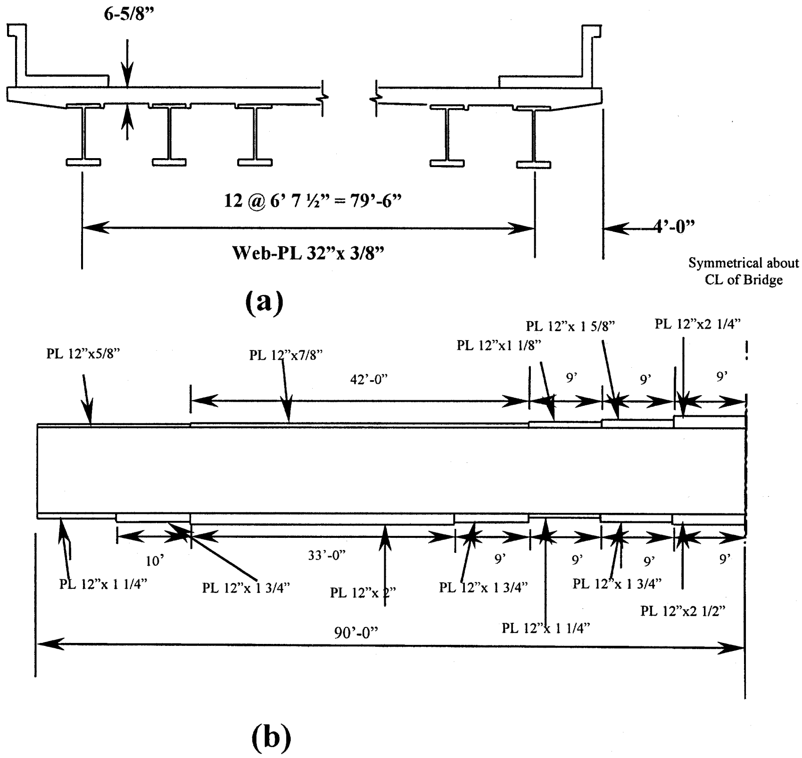

FIGURE 49.3 Details of two-span continuous steel girder bridge example. (a) Typical section; (b) girder elevation.

TABLE 49.2 Load Demands at 0.4th and 1.0th Point for Steel Girder Bridge Example

Description	0.4th Point (36.1 ft)	At Support (90 ft)
Dead load moments in kip-ft	410.0	–732.0
Dead load shear in kips	–1.7	–40.7
Additional dead load moment in kip-ft	22.0	–39.0
Additional dead load shear in kips	–0.1	–2.2
HS20 maximum positive moment in kip-ft	807.0	0.0
HS20 maximum negative moment in kip-ft	–177.0	–714.0
HS20 max. positive shear force in kips	21.8	0.0
HS20 max. negative shear force in kips	–19.3	–49.9

TABLE 49.3 Section Properties of Girder Sections for Steel Girder Bridge Example

	I_{gg} (in.^{4})	Y_b (in.)	Y_t (in.)	S_{xb} (in.^{3})	S_{xt} (in.^{3})
			Section at 0.4th Point		
For dead loads	9,613.9	12.94	21.94	743.11	438.24
For additonal dead loads	17,406.5	19.69	15.19	884.18	1,146.10
For live loads	24,782.7	26.02	8.86	952.59	2,797.50
			Section at 1.0th Point		
For dead loads	17,852.1	17.70	19.05	1,008.50	937.20
For additional dead loads	17,852.1	17.70	19.05	1,008.50	937.20
For live loads	17,852.1	17.70	19.05	1,008.50	937.20

3. Allowable Stress Calculations

Strengths of concrete and rebars are first determined (see Rating Manual Section 6.6.2.3):

$$f_c' = \frac{f_c}{0.4} \quad \text{and, thus} \quad f_c' = 3000 \text{ psi}$$

and f_s = 20,000 psi and thus F_y = 36,000 psi.

a. *Compression and tensile stresses at 0.4th point*:
   Note that the section is fully braced at this location.
   i. Allowable compressive stress at inventory level = 0.55 F_y = 20 ksi (137.9 MPa)
   ii. Allowable compressive stress at operating level = 0.75 F_y = 27 ksi (186.2 MPa)
   iii. Allowable tensile stress at inventory level = 0.55 F_y = 20 ksi (137.9 MPa)

b. *Compression and tensile stresses at 1.0th point*:
   i. allowable tensile stress at inventory level = 0.55 F_y = 20 ksi (137.9 MPa)
   ii. allowable compressive stress at inventory level: Girder is braced 15 ft away from the support and thus L_b = 15 × 12 = 180 in. It can be shown that S_{xc} = 1008.3 in^3; d = 36.75 in.; J = 108.63 in.^{4}; I_{yc} = 360 in.^{4}
   Then allowable stress at inventory level (Table 6.6.2.1-1 of Rating Manual):

$$F_b = \frac{91\times10^6 C_b}{1.82\times S_{xc}}\left(\frac{I_{yc}}{L_b}\right)\sqrt{0.772\left(\frac{J}{I_{yc}}\right)+9.87\left(\frac{d}{L_b}\right)^2}$$

$$= \frac{91\times10^6 (1.00)}{1.82\times1008.3}\left(\frac{360}{180}\right)\sqrt{0.772\left(\frac{108.63}{360}\right)+9.87\left(\frac{36.75}{180}\right)^2}\left(\frac{1}{1000}\right)$$

$$= 79.5 > 0.55F_y = 20 \text{ ksi (Note that } C_b \text{ is conservatively assumed as 1.0.)}$$

   Thus, F_b = 20 ksi (137.9 MPa)
   iii. Allowable compressive stress at operating level: The allowable stress at operating level is given:

$$F_b = \frac{91\times10^6 C_b}{1.34\times S_{xc}}\left(\frac{I_{yc}}{L_b}\right)\sqrt{0.772\left(\frac{J}{I_{yc}}\right)+9.87\left(\frac{d}{L_b}\right)^2}$$

(Table 6.6.2.1-2, Rating Manual)

$$= 108.0 > 0.75\ F_y = 27 \text{ ksi}$$

   Thus, F_b = 27 ksi (186.2 MPa)

c. *Allowable inventory shear stresses at 0.4th and 1.0th point*:

$$D/t_w = 32/0.375 = 85.33$$

   Girder is unstiffened and thus k = 5;

$$\frac{6000\sqrt{k}}{\sqrt{F_y}} = 70.71 < D/t_w < \frac{7500\sqrt{k}}{\sqrt{F_y}} = 88.3$$

TABLE 49.4 Estimated Stress Demands for Steel Girder Bridge Example

Load Description	At 0.4th Point	At 1.0th Point	Fiber Location
DL moment	–6.62	8.71	At bottom fiber
ADL moment	–0.30	0.463	At bottom fiber
LL + *I* moment	–10.16	8.49	At bottom fiber
DL moment	11.23	–9.37	At top fiber
ADL moment	0.23	–0.49	At top fiber
LL + *I* moment	3.46	–9.14	At top fiber
DL shear	0.129	2.95	Shear stress
ADL shear	0.007	0.15	Shear stress
LL + *I* shear	1.667	3.62	Shear stress

Thus,

$$C = \frac{6000\sqrt{k}}{\left(\frac{D}{t_w}\right)\sqrt{F_y}} = 0.828$$

$$F_v = \frac{F_y}{3}\left(C + \frac{0.87(1-C)}{\sqrt{1+\left(\frac{d_o}{D}\right)^2}}\right) = 11.76 \text{ ksi } (81.1 \text{ MPa})$$

d. *Allowable operating shear stresses at 0.4th and 1.0th point*:

$$F_v = 0.45F_y\left(C + \frac{0.87(1-C)}{\sqrt{1+\left(\frac{d_o}{D}\right)^2}}\right) = 15.88 \text{ ksi } (109.5 \text{ MPa})$$

4. **Load Stress Calculations**
   Bending stress calculations are made using appropriate section modulus and moments. Results are reported in Table 49.4. The sign convention used in Table 49.4 is as follows: compressive stress is positive and tensile stress is negative. Also, estimated shear stresses are given in Table 49.4.

5. **Rating Calculations**
   The rating factor in ASD approach is given by

$$\frac{R-D}{L(1+I)}$$

   and the rating calculations are made and given in Table 49.5.

6. **Summary**
   The critical rating factor of the girder is controlled by tensile stress on the top fiber at the 1.0th point. The critical inventory and operating rating factors are 1.11 and 1.87, respectively.

TABLE 49.5 Rating Calculations Using ASD Method for Steel Girder Bridge Example

Location	Description	Inventory Rating	Operating Rating
0.4th point	Shear	$\frac{11.76-(0.129+0.007)}{1.667}=6.97$	$\frac{15.88-(0.129+0.007)}{1.667}=9.44$
	Stress at top fiber	$\frac{20-(11.23+0.23)}{3.46}=2.46$	$\frac{27-(11.23+0.23)}{3.46}=4.49$
	Stress at bottom fiber	$\frac{20-(6.62+0.30)}{10.16}=1.28$	$\frac{27-(6.62+0.30)}{10.16}=1.97$
1.0th point	Shear	$\frac{11.76-(2.95+0.15)}{3.62}=2.39$	$\frac{15.88-(2.95+0.15)}{3.62}=3.53$
	Stress at top fiber	$\frac{20-(9.37+0.50)}{9.14}=1.11$	$\frac{27-(9.37+0.50)}{9.14}=1.87$
	Stress at bottom fiber	$\frac{20-(8.71+0.463)}{8.49}=1.28$	$\frac{27-(8.71+0.463)}{8.49}=2.10$

49.6.4 Two-Span Continuous Prestressed, Precast Concrete Box Beam Bridge

Given

Typical section and elevation of three continuous-span precast, prestressed box-girder bridge is shown in Figure 49.4. The span length of each span is 120 ft (36.5 m), 133 ft (40.6 m), and 121 ft (36.9 m). Total width of the bridge is 82 ft (25 m) and a number of precast, prestressed box girders are placed at a spacing of 10 ft (3.1 m). The cross section of the box beam and the tendon profile of the girder are shown in Figure 49.4. Each barrier rail weighs 1268 lb/ft (18.5 N/mm). Information gathered from the plans is (a) f_c' of the girder and slab is 5500 and 3500 psi, respectively; (b) working force (total force remaining after losses including creep) = 2020 kips; (c) x at midspan = 9 in. Assume that (1) the bridge was made continuous for live loading; (2) no temporary supports were used during the erection of the precast box beams; (3) properties of the precast box are area = 1375 in^2; moment of inertia = 30.84 ft^4; Y_t = 28.58 in.; Y_b = 34.4 in.; (4) F_y of reinforcing steel is 40 ksi.

Requirement

Rate the interior girder of Span 2 for HS20 vehicle.

Solution

1. **Dead Load Calculations**

Self-weight of the box beam = (1375/144) (0.15) = 1.43 kips/ft

Weight of Slab (tributary area approach) = (6.75/12) (10) (0.15) = 0.85 kips/ft

Total dead weight on the box beam = $\underline{2.28\ \text{kips/ft}\ (33.2\ \text{N/mm})}$

Contribution of barrier rail on box beam = 2(1.268/8) = 0.318 kips/ft

Thus, total additional dead load on the box beam = $\underline{0.318\ \text{kips/ft}\ (4.6\ \text{N/mm})}$

Girder is simply supported for dead loads; thus maximum dead load moment = (2.28) (133^2/8)

$= \underline{4926\ \text{kips/ft}\,(6.68\ \text{MN/m})}$

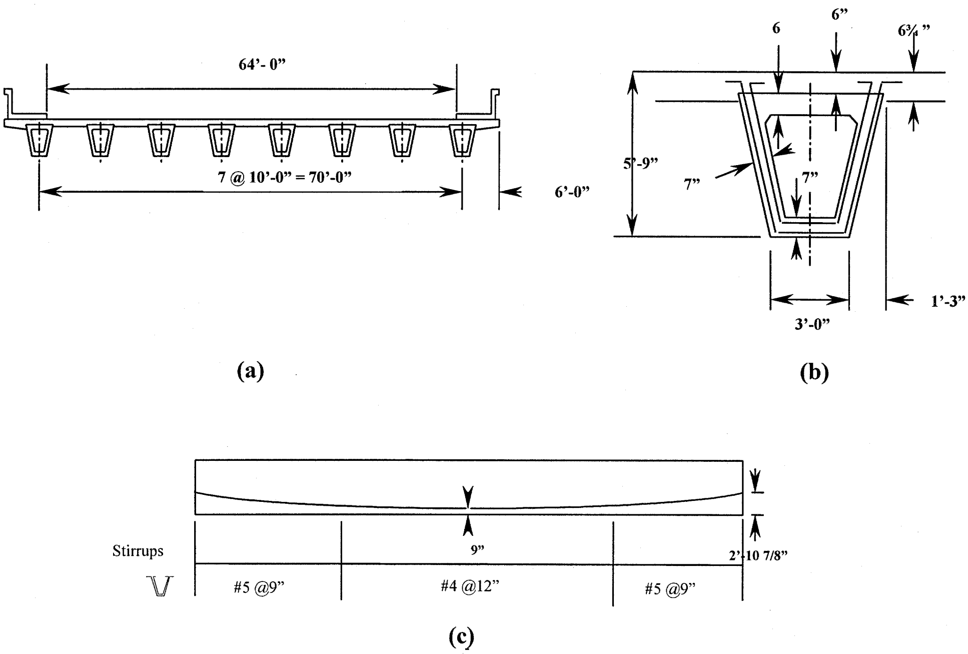

FIGURE 49.4 Details of two-span continuous prestressed box beam bridge example. (a) Typical section; (b) beam section details; (c) prestressing tendon profile.

2. **Live Load Calculations**
   According to Article 3.28 of Design Specifications, distribution factor (DF) for interior spread box beam is given by

$$\mathrm{DF} = \left(\frac{2\,N_L}{N_B}\right) + k\left(\frac{S}{L}\right)$$

where N_L = number of traffic lanes = 64/12 = 5 (no fractions); N_B = Number of beams = 8; S = girder spacing = 8 ft; L = span length = 133 ft; W = roadway width = 64 ft

$$k = 0.07\ W - N_L\ (0.10\ N_L - 0.26) - 0.2\ N_B - 0.12 = 1.56$$

$$\text{Thus, DF} = \left(\frac{2 \times 5}{8}\right) + 1.56\left(\frac{10}{133}\right) = 1.37 \text{ wheels}$$

3. **Demands on the Girder**
   Load demands are estimated using a two-dimensional analysis, and a summary is given in Table 49.6.

4. **Section Property Calculations**
   In order to estimate the stresses on the prestress box beam, the section properties for composite girder need to be estimated. Calculations of the composite girder properties are done separately and the final results are listed here in Table 49.7.

TABLE 49.6 Load Demands for Prestressed Precast Box Beam Bridge Example

Description	0.5L	At Bent 2	At Bent 3
Dead load moment (kip/ft)	4224	0	0
Additional dead load moment (kip/ft)	194	–506	–513
HS20 moment with impact (kip/ft)	1142	–1313	–1322
Dead load shear (kips)	0.0	153.6	–153.6
Additional dead load shear (kips)	0.0	21.1	–21.2
HS20 positive shear (moment)[a] (kips)	24.8 [1104]	61.1 [–974]	7.1 [127]
HS20 negative shear (moment)[a] (kips)	–24.8 [1104]	–7.1 [131]	–61.2 [–980]

[a] Values within brackets indicate the moment corresponds to the reported shear.

TABLE 49.7 Section Properties for Prestressed, Precast Box Beam Bridge Example

Description	Area (in.^{2})	Moment of Inertia (ft^4)	Y Bottom of Girder (in.)	Y Top of Girder (in.)	Y Top of Slab (in.)
For dead loads	1375	30.84	34.42	28.58	NA
For additional dead loads	1578	39.22	38.55	24.45	30.45
For live loads	1984	50.75	44.23	18.77	24.77

TABLE 49.8 Stresses at Midspan for Prestressed, Precast Box Beam Bridge Example

Location = Midspan	Stresses in the Box Beam (psi)			
Load Description	At Top Concrete Fiber	At Bottom Concrete Fiber	At Centroid of Composite Box Beam Concrete Fiber	At Prestress Tendon
Dead load (self + slab)	2265	–2728	777	20.15
Prestress P_{eff} = 2020 kips e = 25.42 in.	–1615	3443	–108	147.1
Additional dead (barrier)	70	–110	16	0.845
Live load	244	–575	0	4.59
Live load moment for shear	236	–556	0	4.43

TABLE 49.9 Stresses at Bent 2 for Prestressed, Precast Box Beam Bridge Example

Location = Bent 2	Stresses in the Box Beam (psi)				
Load Description	At Top Concrete Fiber	At Bottom Concrete Fiber	At Centroid of Composite Box Beam Concrete Fiber	At Top of Slab Fiber	At Prestress Tendon
Dead load (self + slab)	0	0	0	0	0
Prestress P_{eff} = 2020 kips e = 12 in.	680	680	680	0	167.5
Additional dead (barrier)	–183	288	–4	–228	–0.3
Live load	–281	662	0	–371	–1.47
Live load moment for positive shear	–208	491	0	–274	–1.08

5. **Stress Calculations**

Stresses at different fiber locations are calculated using

$$\left(\frac{P}{A}\right) + \left(\frac{M\,c}{I}\right)$$

expression. The summary of the results at midspan and at Bent 2 locations is given in Tables 49.8 and Table 49.9, respectively.

6. **Capacity Calculations**

a. *Moment capacity at midspan*:

The actual area of steel could only be obtained from the shop plans. Since the shop plans are not readily available, the following approach is used. Assume the total loss including the creep loss = 35 ksi (241.3 MPa).

$$\text{Thus, the area of prestressing steel} = \frac{\text{Working force}}{0.75 \times 270 - 35} = \frac{2020}{167.5} = 12.06 \text{ in.}^2 \;\; (7781 \text{ mm}^2)$$

b_{eff} = 120 in.; t_s = 6.75 in.; d_p = (5.75)(12) – 9 in. = 60 in.; b_w = 14 in.

$$\rho^* = \frac{A_s^*}{bd} = \frac{12.06}{120 \times 60} = 0.001675$$

$$f_{su}^* = f_s\left(1 - \frac{0.5\rho^* f_s'}{f_c'}\right) = 270\left(1 - \frac{0.5 \times 0.001675 \times 270}{5.5}\right) = 258.9 \text{ ksi} \;\; (1785 \text{ MPa})$$

$$\text{Neutral axis location} = 1.4\, d\rho^* \frac{f_{su}^*}{f_c'} = 1.4 \times 60 \times 0.001675 \times \frac{258.9}{5.5} = 6.62 \text{ in.} < t_s = 6.75 \text{ in.}$$

Since the neutral axis falls within the slab, this girder can be treated as a rectangular section for moment capacity calculations.

$$R = \phi\, M_n = \phi\, A_s^*\, f_{su}^*\, d\left(1 - 0.6\,\rho\,\frac{f_{su}^*}{f_c'}\right) \quad \text{and} \quad \phi = 1.00$$

$$= \underline{14873.1 \text{ kips/ft } (20.17 \text{ MN/m})}$$

b. *Moment capacity at the face of the support*:

15 #11 bars are used at top of the bent; thus, the total area of steel = (15)(1.56) = 23.4 in.^{2} Depth of the reinforcing steel from the top of compression fiber = 69 – 1.5 – 1.41/2 = 66.795 in. (1696.6 mm). F_y = 60 ksi. Resistance reduction factor ϕ = 0.90. Then, the moment capacity

$$\phi\, M_n = \underline{6547.2 \text{ kip/ft } (8.88 \text{ MN/m})} \;\; \text{(based on T section)}$$

c. *Shear capacity at midspan*:

Design Specification's Section 9.20 addresses the shear capacity of a section. Shear capacity depends on the cracking moment of the section. When the live load causes tension at bottom fiber, cracking moment is to be calculated based on the bottom fiber stress. On the other hand, when the live load causes tension at the top fiber of the beam, cracking moment is to be calculated based on the top fiber stress.

At midspan location, the moment reported with the maximum live-load shear is positive. Positive moments will induce tension at the bottom fiber and thus cracking moment is to be based on the stress at bottom fiber.

TABLE 49.10 Rating Calculations Prestressed, Precast Box Beam Bridge Example

Location	Description	Inventory Rating	Operating Rating
Midspan	Maximum moment	$\frac{14873.1-1.3\times(4224+194)}{1.3\times1.67\times1142} = 3.69$	$\frac{14873.1-1.3\times(4224+194)}{1.3\times1142} = 6.16$
	Maximum shear	$\frac{179-1.3\times(0+0)}{1.3\times1.67\times24.8} = 3.33$	$\frac{179-1.3\times(0+0)}{1.3\times24.8} = 5.56$
Bent 2	Maximum moment	$\frac{6544.2-1.3\times(0+506)}{1.3\times1.67\times1313} = 2.06$	$\frac{6544.2-1.3\times(0+506)}{1.3\times1313} = 3.45$
	Maximum shear	$\frac{766-1.3\times(153.6+21.1)}{1.3\times1.67\times61.1} = 4.07$	$\frac{766-1.3\times(153.6+21.1)}{1.3\times61.1} = 6.80$

f'_c = 5500 psi and from Table 49.10; f_{pe} at midspan bottom fiber = 3443 psi

f_d at bottom fiber = –2728 – 110 = –2838 psi; f_{pc} at centroid = 777 – 108 + 16 = 685 psi

$$M_{cr} = \frac{I}{Y_t}\left(6\sqrt{f'_c} + f_{pe} - f_d\right) = \frac{50.75\times12^4}{44.23}\left(6\sqrt{5500}+3443-2838\right)\left(\frac{1}{12,000}\right) = 2081 \text{ kips/ft}$$

Factored total moment:

$$M_{\max} = 1.3\, M_D + (1.3)(1.67)\, M_{LL+I}$$

$$= 1.3\,(4224 + 194) + 2.167\,(1104) = 8136 \text{ kips-ft}$$

Factored total shear:

$$V_i = 1.3\,(0 + 0) + 2.167\,(24.8) = 53.7 \text{ kips}$$

V_d = 0 kips; b_w = 14 in.; d = 60 in.; f_{pc} = 685 psi

$$V_{ci} = 0.6\sqrt{f'_c}\,b_w d + V_d + \frac{V_i M_{cr}}{M_{\max}} = 0.6\sqrt{5500}\times14\times60\times\left(\frac{1}{1000}\right)+0+\frac{53.7\times2081}{8136}$$

$= 51.2$ kips (227.7 kN) (Controls — since smaller than V_{cw})

$$V_{cw} = \left(3.5\sqrt{f'_c}+0.3 f_{pc}\right) b_w d + V_p = \left(3.5\sqrt{5500}+0.3\times685\right)14\times60\times\left(\frac{1}{1000}\right)+0$$

$= 390$ kips (1734 kN)

V_c = 51.2 kips (227.7 kN) (smaller of V_{ci} and V_{cw})

$$V_s = 2A_v\frac{F_y d_s}{S} = 4\times0.20\times\frac{40\times60}{12} = 160 \text{ kips (711.7 kN)}$$

Shear capacity at midspan:

$$V_u = \phi\,(V_c + V_s) = 0.85\,(51.2 + 160) = 179 \text{ kips (796.1 kN)}$$

d *Shear capacity at the face of support at Bent 2*:
Negative shear reported at this location is so small and thus rating will not be controlled by the negative shear at Bent 2. Moment reported with the positive shear is negative, and thus, the following calculations are based on the stress at top fiber. From the Table 49.11, f_d at top of slab fiber = –228 psi, and f_{pe} at support top of slab fiber (slab poured after prestressing) = 0 psi.

$$M_{cr} = \frac{I}{Y_t}\left(6\sqrt{f_c'} + f_{pe} - f_d\right) = \frac{50.75\times 12^4}{44.23}\left(6\sqrt{3500} + 0 - 228\right) = 252 \text{ kips/ft}$$

$V_d = 153.6 + 21.1 = 174.7$ kips; $b_w = 14$ in.; $d = 69 - 1.5 - 1.41/2 = 66.795$ in.; $f_{pc} = 676$ psi

Factored total moment:

$$M_{max} = 1.3\, M_D + (1.3)(1.67)\, M_{LL+I}$$
$$= 1.3(0 + -506) + 2.167(-974) = -2769 \text{ kips-ft}$$

Factored total shear:

$$V_i = 1.3 \times (153.6 + 21.1) + 2.167 \times (61.1) = 360 \text{ kips}$$

$$V_{ci} = 0.6\sqrt{5500}\times 14\times 66.795\left(\frac{1}{1000}\right) + 0 + \frac{360\times 251.7}{2769} = 74.3 \text{ kips}$$

$$V_{cw} = \left(3.5\sqrt{f_c'} + 0.3 f_{pc}\right) b_w d + V_p = \left(3.5\sqrt{5500} + 0.3\times 676\right)14\times 66.795\left(\frac{1}{1000}\right) + 0 = 432 \text{ kips}$$

V_c = 74.3 kips (330.4 kN) (smaller of the V_{cw} and V_{ci})

$$V_s = 2A_v\frac{F_y d_s}{S} = 4\times 0.31\times\frac{60\times 66.695}{6} = 827 \text{ kips (3678.5 kN)}$$

Shear capacity at Bent 2:

$$V_u = \phi\,(V_c + V_s) = 0.85\,(74.3 + 827) = 766 \text{ kips (3408 kN)}$$

7. Rating Calculations

As discussed in Section 49.5.4, the rating calculations for load factor method need to be done using strength and serviceability limit states. Serviceability level rating needs not be done at the operating level.

a. *Rating calculations based on serviceability limit state*:
Serviceability conditions are listed in AASHTO Design Specification Sections 9.15.1 and 9.15.2.2. These conditions are duplicated in the Rating Manual.

i. Using the compressive stress under all load combination:

The general expression will be $RF_{INV\text{-}COMALL} = \dfrac{0.6 f_c' - f_d - f_p + f_s}{f_l}$

At midspan $RF_{INV\text{-}COMALL} = \dfrac{0.6\times 5500 - (2265+70) - (-1615) + 0}{244} = 10.57$

At Bent 2 support $\mathrm{RF}_{\text{INV-COMALL}} = \dfrac{0.6 \times 5500 - (0 + 288) - 680 + 0}{662} = 3.52$

ii. Using the compressive stress of live load, half the prestressing and permanent dead load:

$$\text{The general expression will be } RF_{INV\text{-}COMLIVE} = \frac{0.4 f_c' - f_d - 0.5 f_p + 0.5 f_s}{f_l}$$

$$\text{At midspan } RF_{INV\text{-}COMLIVE} = \frac{0.4 \times 5500 - (2265 + 70) - 0.5(-1615) + 0.5(0)}{244} = 2.76$$

$$\text{At Bent 2 support } RF_{INV\text{-}COMLIVE} = \frac{0.4 \times 5500 - (0 + 288) - 0.5(680) + 0.5(0)}{662} = 2.37$$

iii. Using the allowable tension in concrete:

$$\text{The general expression will be } RF_{INV\text{-}CONTEN} = \frac{6\sqrt{f_c'} - f_d - f_p - f_s}{f_l}$$

$$\text{At midspan } RF_{INV\text{-}CONTEN} = \frac{6\sqrt{5500} - (2728 + 110) - (-3443) - 0}{575} = 1.826$$

$$\text{At Bent 2 support } RF_{INV\text{-}CONTEN} = \frac{6\sqrt{5500} - (0 + 183) - (-680) - 0}{281} = 3.352$$

iv. Using the allowable prestressing steel tension at service level:

$$\text{The general expression will be } RF_{INV\text{-}PRETEN} = \frac{0.8 f_y^* - f_d - f_p - f_s}{f_l}$$

$$\text{At midspan } RF_{INV\text{-}PRETEN} = \frac{0.8 \times 270 - 20.99 - (147.1) - 0}{4.59} = 10.43$$

$$\text{At Bent 2 support } RF_{INV\text{-}PRETEN} = \frac{0.8 \times 270 - (-3.08) - 167.5 - 0}{1.468} = 30.94$$

b. Rating calculations based on strength limit state:

$$\text{The general expression for Rating factor} = \frac{\phi R_n - \gamma_D D}{\gamma_L \beta_L L(1 + I)}$$

According to AASHTO Rating Manual, γ_D is 1.3, γ_L is 1.3, and β_L is 1.67 and 1.0 for inventory and operating factor, respectively. Rating calculations are made and given in Table 49.10.

8. **Summary**
   The critical inventory rating of the interior girder is controlled by the tensile stress on concrete at midspan location. The critical operating rating of the girder is controlled by moment at Bent 2 location.

49.6.5 Bridges without Plans

There are some old bridges in service without plans. Establishing safe live-load-carrying capacity is essential to have a complete bridge document. When an inspector comes across a bridge without plans, sufficient field physical dimensions of each member and overall bridge geometry should be taken and recorded. In addition, information such as design year, design vehicle, designer, live-load history, and field condition of the bridge needs to be collected and recorded. This information will be very helpful to determine the safe live-load-carrying capacity. Also, bridge inspectors need to establish the material strength either using the design year or coupon testing.

Design vehicle information could be established based on the designer (state or local agency) and the design year. For example, all state bridges have been designed using the HS20 vehicle since 1944 and all local agency bridges have been designed using the H15 vehicle since 1950.

In steel girder bridges, section properties of the members could be determined based on the field dimensions. During the estimation of the moment capacity, it is recommended to assume that the steel girders are noncomposite with the slab unless substantial evidence is gathered to prove otherwise.

In concrete girder bridges, field dimensions help to estimate the dead loads on the girders. Since the area of reinforcing steels is not known or is difficult to establish, determining the safe live load poses challenges to bridge owners. The live-load history and field condition of a bridge could be used to establish the safe load capacity of the bridge. For example, if a particular bridge has been carrying several heavy vehicles for years without damaging the bridge, this bridge could be left open for all legal vehicles.

49.7 Posting of Bridges

Bridge inspection and the strength evaluation process are two integral parts of bridge posting. The purpose of bridge inspection is to obtain the information that is necessary to evaluate the bridge capacity and the adequacy of the bridge properly. When a bridge is found to have inadequate capacity for legal vehicles, engineers need to look at several alternatives prior to closing the bridge to the public. Some of the possible alternatives are imposing speed limits, reducing vehicular traffic, limiting or posting for vehicle weight, restricting the vehicles to certain lanes, recommending possible small repairs to alleviate the problem. In addition, when the evaluations show that the structure is marginally inadequate, frequent inspections to monitor the physical condition of the bridge and traffic flow may be recommended.

Standard evaluation methods described in the Section 49.5 may be overly conservative. When a more accurate answer is required, a more-detailed analysis, such as three-dimensional analysis or physical load testing can be performed.

The weight and axle configuration of vehicles allowed to use highways without special permits is governed by the statutory law. Thus, the traffic live loads used for posting purposes should be representative of the actual vehicles using the bridge. The representative vehicles vary with each state in the United States. Several states use the three hypothetical legal vehicle configurations given in the Rating Manual [12]. Whereas a few states use their own specially developed legal truck configurations, AASHTO H or HS design trucks, or some combination of truck types. NBIS requires that posting of a bridge must be done when the operating rating for three hypothetical legal vehicles listed in the Rating Manual [12] is less than unity. Furthermore, the NBIS requirement allows the bridge owner to post a bridge for weight limits between inventory and operating level. Because of this flexible NBIS requirement, there is a considerable variation in posting practices among various state and local jurisdictions.

Although engineers may recommend one or a combination of the alternatives described above, it is the owner, not the engineer, who ultimately makes the decision. Many times, bridges are posted for reasons other than structural evaluation, such as posting at a lower weight level to limit vehicular or truck traffic, posting at a higher weight level when the owner believes a lower posting would not be prudent and is willing to accept a higher level of risk. Weight limit posting may cause inconvenience and hardship to the public. In order to reduce inconvenience to the public, the owner needs to look at the weight limit posting as a last resort. In addition, it is sometimes in the public interest to allow certain overweight vehicles such as firefighting equipment and snow removal equipment on a posted bridge. This is usually done through the use of special permits.

References

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