

Florida Department of Transportation

New Directions for Florida Post-Tensioned Bridges



Volume 10 A: Load Rating Post-Tensioned Concrete Segmental Bridges

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Preface

As a result of recent findings of corrosion of prestressing steel in post-tensioned bridges, the Florida Department of Transportation has changed policies and procedures to ensure the long-term durability of post-tensioning tendons. The background to these revised policies and procedures was presented in the study entitled, *New Directions for Florida Post-Tensioned Bridges*. The study has been presented in several volumes, with each volume focusing on a different aspect of post-tensioning or bridge type.

Volume 1: Post-Tensioning in Florida Bridges presents a history of post-tensioning in Florida along with the different types of post-tensioned bridges typically built in Florida. This volume also reviews the critical nature of different types of post-tensioning tendons and details a new five-part strategy for improving the durability of post-tensioned bridges.

Volumes 2 through 8: Design and Construction Inspection of various types of post-tensioned bridges applies the five-part strategy of Volume 1 to bridges in Florida. Items such as materials for enhanced post-tensioning systems, plan sheet requirements, grouting, and detailing practices for watertight bridges and multi-layered anchor protection are presented in detail. The various types of inspection necessary to accomplish the purposes of the five-part strategy are presented from the perspective of Construction Engineering Inspection. Detailed checklists of critical items or activities are included.

Volume 9: Condition Inspection and Maintenance of Florida Post-Tensioned Bridges addresses the specifics of ensuring the long-term durability of tendons in existing and newly constructed bridges. The types of inspections and testing procedures available for condition assessments are reviewed, and a protocol of remedies are presented for various symptoms found.

Volume 10 A: Load Rating Post-Tensioned Concrete Segmental Bridges in Florida provides recommendations for meeting AASHTO LRFR load rating requirements as they pertain to precast and cast-in-place, large box-section, segmental bridges.

Volume 10 B: Load Rating Post-Tensioned Concrete Beam Bridges in Florida provides recommendations for meeting AASHTO LRFR load rating requirements as they pertain to precast and cast-in-place beam-type bridges. This includes AASHTO I-beams, Bulb-T girders, spliced I-girders, Florida U-beams and similar structures.

Disclaimer

The information presented in this Volume represents research and development with regard to improving the durability of post-tensioned tendons; thereby, post-tensioned bridges in Florida. This information will assist the Florida Department of Transportation in modifying current policies and procedures with respect to post-tensioned bridges. The accuracy, completeness, and correctness of the information contained herein, for purposes other than for this express intent, are not ensured.

**Volume 10 A – Load Rating Post-Tensioned
Concrete Segmental Bridges**

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Disclaimer

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Chapter 1 – Introduction

Load rating post-tensioned concrete segmental bridges has historically presented difficulties for Owners and Engineers. These bridges are designed at service load limits. Precast segmental bridges are post-tensioned to keep joints closed under the design service loads. Post-tensioning in cast-in-place segmental bridges is proportioned to keep the concrete within allowable tensile limits. With service requirements satisfied, the bridges are verified at ultimate load levels in accordance with an applicable design specification.

Current Federal Highway Administration preferred policy is to load rate all “on-system” bridges at inventory and operating levels at ultimate load limits. This policy is normally met using the load factor principles of the *AASHTO Standard (LFD) Specifications* (Ref. 1.1). These requirements also call for service load checks to be performed in conjunction with the ultimate ratings. The Florida Department of Transportation (FDOT) provides additional guidance for load rating bridges in Florida in *“Bridge Load Rating, Permitting and Posting Manual”* (Ref. 1.2). Inventory ratings (design vehicle) differ from operating ratings (design vehicle, FDOT legal loads and permit vehicles) by the use of either different load factors at ultimate limits or different allowable stresses at service load limits.

Inventory ratings provide a measure of the adequacy of the bridge with regard to current design guidelines. Operating ratings acknowledge conservatism in design and provide bridge owners with flexibility in establishing operational capacities.

Difficulties in load rating segmental bridges (especially precast segmental bridges) arise because, regardless of whether the bridge is being load rated at inventory or operating level, serviceability requirements typically govern. Magnifying allowable stresses has no impact on the capacity of bridges where permissible tensile stresses are zero (Ref. 1.3). The result is that there is typically no difference between inventory and operating levels. Consequently, owners do not benefit from the flexibility that is afforded to other bridge types that do not have discrete joints. With this document, such circumstances should no longer limit the statewide system.

Introduction of *AASHTO Load and Resistance Factor Design* (Ref. 1.4) has led to many changes in the verification of segmental bridges at ultimate load (strength) limits. Except for increases in live loads, the LRFD design requirements for segmental bridges at service load limits are not otherwise significantly different from LFD requirements. As a result, LRFD has not led to a significant change, or resolved any issues, for load rating segmental bridges.

However, AASHTO has adopted the final results of the National Cooperative Highway Research Program (NCHRP) Report Number 12-46, entitled “Manual for Condition Evaluation for Load and Resistance Factor Rating of Highway Bridges” (Ref. 1.5), as a guide specification – hereinafter referred to as “LRFR”. NCHRP Report Number 12-46 draws upon important information presented in NCHRP Report Number 406, “Redundancy in Highway Bridge Superstructures” (Ref. 1.6) and NCHRP Report Number 454, “Calibration of Load Factors for LRFR Bridge Evaluation” (Ref. 1.7).

The intent of LRFR is to provide a load rating methodology consistent with LRFD and to incorporate operational flexibility by establishing target reliability indices for load ratings different from those established for design. The current version of LRFR does not provide specific

guidance for load rating bridges that are governed by service performance at discrete joints. LRFR does, however, provide a reliability based framework that is established upon important concepts of internal redundancy that may be extended to address these types of bridges.

With these thoughts in mind, the Florida Department of Transportation has tasked Corven Engineering, Inc. to produce recommendations for load rating post-tensioned concrete segmental bridges that are consistent with LRFR. Documents important to this task are presented in References 1.1 through 1.7. These documents, their historical background and development, along with the experience of Corven Engineering, Inc. in the design and load rating of segmental bridges, form the basis for completing this task.

References:

- 1.1 AASHTO (1996 and interims). *Standard (LFD) Specifications for Highway Bridges*, 16th Edition, American Association of State Highway and Transportation Officials, Washington, DC.
- 1.2 *Bridge Load Rating, Permitting and Posting Manual*, Florida Department of Transportation (FDOT), March 1995.
- 1.3 AASHTO (1999). *Guide Specifications for Design and Construction of Segmental Concrete Bridges*, 2nd Edition, American Association of State Highway and Transportation Officials, Washington, DC.
- 1.4 *AASHTO LRFD Bridge Design Specifications* (1998), Customary U.S. Units, 2nd Edition, American Association of State Highway and Transportation Officials, Washington, DC.
- 1.5 National Cooperative Highway Research Program, Report Number 12-46, "Manual for Condition Evaluation for Load and Resistance Factor Rating of Highway Bridges," (Adopted as a guide specification – "AASHTO LRFR").
- 1.6 National Cooperative Highway Research Program, Report Number 406, "Redundancy in Highway Bridge Superstructures," Transportation Research Board, 1998 by Michel Ghosn and Fred Moses, Department of Civil Engineering, City College of the City University of New York, NY.
- 1.7 National Cooperative Highway Research Program, Report Number 454, "Calibration of Load Factors for LRFR Bridge Evaluation," Transportation Research Board, 2001 by Fred Moses.

Chapter 2 – Load Rating Philosophy

2.1 General

The purpose of these recommendations is to allow the FDOT to establish uniform procedures for the load rating of Segmental Bridges. Uniformity has been accomplished by developing minimum prescriptive procedures that use proven analytical methods which are appropriately conservative. More refined analytical methods will continue to be used in designing new bridges. However, while performing load ratings, these refined techniques will be used as tools for posting avoidance and processing of permit loads.

The recommendations presented in this volume take into consideration the “New Directions for Florida Post-Tensioned Bridges,” implemented by the FDOT in 2002. Advantage is taken of improvements afforded to new bridges built to these recommendations. Also, certain allowable stress levels have been introduced to minimize potential cracking, opening of precast joints and breach of corrosion protection to post-tensioning in order to enhance durability. When appropriate, this Volume also includes recommendations for changing design practice in order to meet future load rating goals.

2.2 Load and Resistance Factor Rating (LRFR)

Load rating methodology in the United States has changed along with design trends (i.e. the change from Service Load Design to Load Factor Design). Most recently, engineering judgment was applied to live load factors to acknowledge differences in factors of safety for “Design” as compared to “Operating Ratings”. The existing FDOT load rating policy is a good example of this philosophy. Though verified by experience, this approach was not based on the application of a systematic, uniformly applied methodology. The result is that the relative merits of different bridge types and structural configurations on load ratings could not be realized.

The adoption of Load and Resistance Factor Design (LRFD) as the basis for a new AASHTO code was an important move towards achieving uniform reliability and equitable consideration of common bridge types. Development of LRFD included extensive reviews of existing bridges using structural reliability theory to determine the range of reliabilities inherent to traditionally designed bridges. This work culminated in the development of calibrated load and resistance factors at the Strength Limit State (NCHRP Report 454). NCHRP Report 406 extended the work to include the effects of redundancy in highway structures.

NCHRP Report 454 established that historical design practice produced bridges that correlated to reliability indices (β) of 3.5 for Design and Inventory Rating, and 2.5 for Operating Rating of design loads. These levels of reliability were adopted as target reliability indices for LRFD and LRFR. In keeping with a traditional format, these reliability indices were not incorporated directly into the codes. Rather, the reliability indices were incorporated by applying different live load factors at Inventory and Operating Rating levels. (For example, the HL93 loading is factored by 1.75 for the Inventory Level and by 1.35 for Operating Level at Strength Limit States).

LRFR also introduced a range of Live Load Factors (γ_L) for load rating for Legal and Permit

Loads at the Operating Level depending upon the average daily truck traffic (ADTT). The purpose was to allow the Owner to take advantage of conservatism in design to facilitate rating for more severe loads of less frequent occurrence. This was incorporated by modifying Live Load Factors for different levels of ADTT to reflect different levels of reliability.

Under LRFR, all types of bridges are to be load rated for:

Inventory Level ($\beta = 3.5$)

(1) Design Level Loads (e.g. HL93)

Operating Level ($\beta = 2.5$)

(1) Design Level Loads

(2) Legal Loads (AASHTO and/or FDOT defined legal loads)

(3) Permit Loads (overloads / FDOT Permit Vehicles)

LRFR, as adopted by AASHTO in October 2003 as a Guide Specification, currently does not specifically address long span, movable, curved steel bridges or, in particular, segmental construction – although FDOT developed and released recommendations for the latter in July 2003. LRFR does, however, address the rating of prestressed concrete girder bridges, although it focuses on rating at Strength Limit State.

2.3 LRFR Philosophy for Concrete Segmental Bridges

Although LRFR was calibrated based on structural reliability theory (NCHRP Report 406) to achieve minimum target reliabilities (β) for the Strength Limit State, consistent with historically achieved reliabilities, it does not directly offer a corresponding calibration for the Service Limit State. In addition, LRFR does not address service limits for bridges with discrete joints. These limitations require that recommendations be developed that usefully extend the scope of LRFR to the load rating of segmental bridges. The resulting recommendations will assist the FDOT in identifying when a bridge needs to be posted or strengthened and to facilitate decisions concerning overweight permit vehicles.

The LRFR approach to post-tensioned segmental construction should be based on a philosophy that strikes an appropriate balance between safety and economics, and respects traditional approaches to load rating while embracing LRFR concepts. With this principal as a guide, the philosophy recommended for a FDOT LRFR for segmental construction includes the following features:

- Rating procedures should accommodate all FDOT Legal and Permit Loads.
- Load ratings should achieve reliability levels consistent with other bridge types.
- Service and Strength limits should be adequately addressed.
- Loads should not induce permanent cracks that might breach the integrity of the corrosion protection to any post-tensioning.
- Benefits of recent developments to enhance durability should be recognized.
- Strategies for posting should be included.
- Possible options for strengthening should be identified.
- Rating Factors for all Design, Legal and Permit Loads at both the Inventory and Operating Levels shall not be less than 1.0.

2.4 Inventory and Operating Rating Levels

Current FDOT load rating policy provides definitions for Inventory and Operating Ratings:

Inventory Rating – The rating which represents the load level at which an existing structure can be utilized for an indefinite period of time.

Operating Rating – The rating which represents the absolute maximum permissible load level to which a structure may be subjected.

The FDOT load rating policy also provides service and strength verifications at both Inventory and Operating Rating Levels for prestressed concrete bridges. In practice, however, Inventory Ratings are only performed at Service Limit States and Operating Ratings only at Strength Limit States. This practice is justified for the majority of Florida bridges (pretensioned I-girder bridges) because of known conservatism in past design practice. The negative ramification of identifying Inventory Ratings with Service Limit States and Operating Ratings with Strength Limit States is that this mind-set is inadvertently carried over to load ratings of bridge types where both limit states are important at both rating levels.

The recommendations in this Volume recognize that for segmental bridges, Inventory and Operating Ratings are to be performed at both the Strength Limit State and Service Limit State.

Chapter 3 – Data Collection

The load rating profession shall collect relevant data from available sources of plans, construction records and maintenance inspection reports. These records should be verified by field inspection.

3.1 Existing Plans

Existing plans typically consist of Design Plans, Shop Drawings, and As-Built Drawings. The Design Plans are those created by the Engineer of Record (EOR) for bid purposes. The Design Plans for post-tensioned concrete segmental bridges should contain the design criteria, material properties, assumed loads, bridge geometry, cross-section data and post-tensioning layouts. These plans would also have been developed by the EOR based on other important assumptions, such as the age of the concrete segments at erection, sequence and timing of both casting and erection, method of erection, erection equipment loads, and temporary support conditions.

Because specific assumptions made by the EOR may not be those preferred by the Contractor, it is customary to allow a certain amount of flexibility with regard to the Contractor's means and methods. Standard and supplemental specifications typically give direction as to the extent of the changes that the Contractor can make. Typical changes made during construction may include segment lengths, changes in thickness of webs or slabs, repositioning or re-sizing of mild reinforcing, and transverse and longitudinal post-tensioning layouts and details.

Contractor changes are most typically implemented through the Shop Drawing process. Changes may affect structural capacity and as such should be compared to the Design Plans prior to load rating the bridge.

Occasionally, major changes to a bridge design may have been made through a Value Engineering Change Proposal (VECP) during construction. For an existing segmental bridge, this might have involved changing span lengths, segment cross section or construction method. In this case, it is essential to obtain the approved VECP plans used for construction before load rating the bridge. For a VECP under consideration, evaluations shall include the effect on the load rating.

Another source of existing plans is the As-Built Drawings. These drawings are typically prepared by the EOR and are generally an update of the Design Plans to include the changes made during construction.

It is recommended that a walk-through inspection be conducted prior to load rating a concrete segmental bridge. The inspection should focus on the accuracy of the information shown in the available Existing Plans. The extent of the inspection should be based on which Existing Plans are available for review.

3.2 Construction Records

The construction of a segmental bridge is typically documented in two project specific manuals,

the “Casting Manual” (sometimes referred to as the “Geometry Control Manual” or “Casting Yard Manual”) and the “Erection Manual”. The “Casting Manual” documents the geometry control procedures used in making precast, match-cast segments in the casting yard. The geometry control records, if available, should contain casting dates and concrete strengths of the segments.

The “Erection Manual” typically lists, in step-by-step detail, the sequence in which each segment or span should have been erected and post-tensioned. The detailed sequence should include moving of erection equipment, temporary supports or loads. If available, this manual should provide much of the information necessary for re-creating a structural analysis using a time-dependent computer program.

Information that may be provided in the Casting and Erection Manuals includes:

- Casting date for each segment.
- Erection date for each segment.
- Concrete strength for each segment or closure joint.
- Concrete density or weights of actual segments.
- Dates of casting closure joints in span-by-span and cantilever construction.
- Dates of introduction and removal of temporary supports.
- Magnitude and location of erection equipment loads.
- Dates of placing or removing erection equipment loads.
- Magnitude and location of other temporary erection loads (counterweights).
- Dates of placing or removing other temporary erection loads.
- Dates of stressing and magnitude of jacking force for each permanent post-tensioning strand or bar tendon.
- Dates of stressing and magnitude of jacking force for each temporary post-tensioning bar or strand tendon.
- Dates of de-tensioning temporary post-tensioning bar or strand tendons.

Actual Concrete Strength:

Whenever possible, the actual concrete strength of segments should be used when load rating a concrete segmental bridge. In the longitudinal direction, the ratings of most segmental bridges at the Service Limit State are controlled by limiting the tensile stresses at the match-cast joints to zero. In these instances nothing is gained by considering actual concrete strengths. However, when a Service Limit State rating is controlled by compression in slabs, principal tension in the webs, or by transverse tension in the top slab, the use of actual concrete strengths will improve load rating results.

Construction records should be reviewed prior to load rating in order to establish the actual mean concrete strength of the segments at the time of construction. If the concrete strength cannot be determined from field records, or if ratings are inappropriately controlled by the field strengths, then it is recommended that current concrete strengths of the bridge be ascertained. This can be accomplished by using appropriate non-destructive field techniques or by testing cores removed from the bridge. Alternatively, a conservative allowance for strength may be made based upon historical local knowledge of concrete production, strength and maturity.

3.3 Maintenance Inspection Reports

Bridge Maintenance Inspection Reports should be examined prior to load rating to determine if there has been any deterioration or damage that would change the capacity of the bridge. The National Bridge Inventory (NBI) condition rating value for the superstructure should be noted. The inspection reports should be reviewed for comments that might indicate corrosion or damage to post-tensioning tendons. Typical comments may include phrases such as “rust stains present”, “efflorescence seeping from anchorage pourbacks” or “leaks at segment joints”. The presence of these comments in the inspection report may lead to additional inspections to ascertain if any loss of strands has occurred that will reduce the load capacity and load rating. (Refer to Volume 9, “Condition Inspection and Maintenance of Florida Post-Tensioned Bridges,” for further information).

One additional source of information that is usually kept with the Maintenance Inspection Reports is plans detailing repair of the bridge. These plans should be reviewed to determine their impact on the load-carrying capacity of the structure.

Chapter 4 – Analysis Requirements

Prescriptive procedures presented in this Chapter help to promote uniformity in the load ratings of post-tensioned concrete segmental bridges in Florida. These prescriptive procedures use proven analytical methods which are appropriately conservative considering the lower target reliability (β) of 2.5 for Operating Ratings. More refined analytical methods may be used as tools for posting avoidance of the existing inventory and processing of permit loads (See Chapter 9). Boundary conditions must be carefully reviewed with the Department when utilizing these refined analytical methods.

4.1 Longitudinal Analysis

The superstructure of a typical concrete segmental bridge is usually comprised of a single-cell, two-web box section. Two-cell box girders with three webs are also used in segmental construction, but are not as common. Span lengths for these bridges typically range from 100 feet to 400 feet. Segmental bridges are erected using a phased construction process, involving many different intermediate statical schemes before the structure is completed. The appropriate prediction of stresses in the completed bridge requires that the longitudinal superstructure analysis be modeled in the same manner and sequence in which the bridge was erected (Ref. 4.1, 4.2).

Longitudinal construction analyses for FDOT LRFD load ratings should be performed using a two-dimensional plane frame computer program. The program should include time-dependent material properties and phased construction modeling capability.

Three degree of freedom nodes (horizontal displacement, vertical displacement and in-plane rotation) define the geometry of the structure and should be located at the segment joints, over support locations within segments, and at other locations that would facilitate gathering results needed for load rating. Beam elements characterized by a first-order, 6-by-6 stiffness matrix should connect the nodes. Stiffness coefficients for the beam elements should be defined by the member length and box girder superstructure cross-section properties. In addition to the superstructure definition, the stiffness of bridge bearings, columns and foundations should be appropriately included in the longitudinal analysis.

The two-dimensional construction analysis computer program should have the ability to define temporary and permanent post-tensioning in accordance with Existing Plan information. The program should have the capability of stressing tendons, computing friction and anchor set losses, and adjusting tendon force with time as a function of the relaxation of the prestressing steel and creep and shrinkage of the concrete.

The phased construction analysis should include as much information as possible related to the construction of the bridge (See Chapter 3). When the exact dates of key activities are not available, it is recommended that the model follow the general sequence of construction and that missing dates be approximated. If no casting and erections dates are available, conservative assumptions with regard to time should be made:

Balanced Cantilever – Assume the age of the segments at erection is 28 days.

	Assume 2 segments are placed at both ends of a cantilever each day. Assume continuity between spans takes 1 week.
Span-by-Span –	Assume the age of the segments at erection is 28 days. Assume 2 spans are erected each week.

Torsional forces resulting from eccentric loads and bridge curvature may be computed separately and included with the results of the two-dimensional time-dependent computer analysis model as necessary (Ref. 4.4, 4.5). Three-dimensional structural analysis programs may be used as an alternative to manual combinations of two-dimensional and torsional analyses, provided that the program contain time-dependent and phased construction capability and that only beam elements are used to model the bridge.

Live load bending moments, axial forces and shears should be generated for the Design, Legal and Permit loads at controlling locations along the bridge using a two-dimensional structural analysis program. The program should generate influence lines for the various load effects and “move” the respective loads to determine maximum effects.

4.2 Transverse Analysis

Transverse analyses for FDOT LRFD load ratings of the top slabs of concrete segmental bridges should be performed using a two-dimensional plane frame computer program. The program should include time-dependent material properties and the ability to model all aspects of transverse post-tensioning tendons in the top slab of the box girder superstructure.

A foot long plane frame section of the box girder superstructure should be analyzed. Nodes should be placed at the center of member thicknesses, the extremities of the top slab, intersections of the slabs and webs, and other locations where member thicknesses change. Beam elements should be used to connect the nodes. Stiffness coefficients for the beam elements should be defined by the member length and width. The frame should be supported at the nodes at the intersections of the webs with the bottom slabs.

Loads that are uniformly distributed along the length of the span should be applied directly to the one foot long plane frame. These loads include self weight, barrier rails, and wearing surfaces. Post-tensioning tendons should be modeled in accordance with the relevant Existing Plans and stressed to a force equal to the total tendon force divided by the longitudinal spacing of the tendons.

Concentrated loads, such as wheel loads from live loads, must not be placed directly on the plane frame model of the box girder superstructure. Some method of considering the distribution of these concentrated forces in the longitudinal direction must be used in order to appropriately develop frame bending moments on a per foot basis. One acceptable method for determining the local effect of wheel loads relies on the use of influence surfaces. Those most commonly used were developed by Pucher (Ref. 4.7) for slabs of constant thickness and Homberg (Ref. 4.8) for selected variable thickness slabs. These influence surfaces are perfectly fixed along the transverse slab edges and are infinite in length in the longitudinal direction. Concentrated loads are “placed” on the influence surfaces and peak values for transverse moments on a per foot basis are computed.

Three sections should be checked at both the Service Limit State and Strength Limit State for load rating the top slab of the box girder, these are:

- At the root of the cantilever wing.
- At each interior face of the web.
- At mid-span of interior slab(s).
- Or any other taper points in the cantilever.

Influence surfaces may be used to directly compute the negative flexural moment at the root of the cantilever. Judgment is sometimes required to select the critical section if linear or circular fillets exist at transitions between the top slab and webs.

Fixed-end negative moments at the inside face of the web should be computed from an appropriate influence surface. These moments must, however, be reduced in proportion to the relative flexural stiffnesses of the top slab and webs. Fixed-end negative moments are typically applied as external moments to the intersection of the webs and top slabs in the two-dimensional frame model of the one foot long section. The resulting internal forces in the top slab represent the redistributed portion of the top slab fixed-end moment. Corresponding negative moments from the cantilever wing should be distributed between the top slab and webs and added to the negative moment at the inside face of the web as required by the loading pattern that produces maximum negative moment results.

Positive moments at mid-span of top slab are found by first using an influence surface to determine the positive moment at this location assuming the slab is fixed along its edges. Negative moment influence charts should be used to determine the corresponding fixed-end moments at the face of the webs. The fixed-end negative moments should be redistributed as described above. The average of the change in slab end moments represents the additional positive moment to be added to the positive moment at mid-span determined from the fixed slab condition.

For variable depth balanced cantilever bridges, it is recommended that the cross section selected for transverse load rating be that at mid-span. It is understood for a variable depth box girder bridge the relative flexural stiffness of the top slab and webs varies along the length of the span. The use of the shallow girder depth at mid-span leads to use of more conservative negative moment ratings at the inside face of the webs. Results will be less conservative for the positive transverse flexure at the mid-span of the slab. However, this approach is warranted to ensure the adequacy of cover concrete required as an essential part of the corrosion protection for cantilever and transverse post-tensioning.

An acceptable alternative to the use of influence surfaces to determine fixed-end and in-span moments from concentrated loads on the top slab is using a suitable three-dimensional grillage or finite element model (Ref. 4.9, 4.10, 4.11, 4.12). Fixed-end moments should be distributed around the frame in the same manner as those found using influence surfaces.

For each of the above analysis methods, concentrated wheel loads may be represented as a force distributed over a tire contact surface in accordance with LRFD 3.6.1.2.5 and dispersed at 45 degrees to the mid-depth of the slab.

During design, slab moments transmitted to the webs are combined with effects from longitudinal shear and torsion to determine reinforcing requirements in the webs. However, for load rating purposes, this need not be done providing that such analysis is performed if there is evidence of distress in the structure. In which case, an explanation of the observed behavior should be sought and an appropriate evaluation of capacity should be made.

The use of a three-dimensional grillage or finite element analysis that models the entire superstructure may only be used as a means of enhancing rating as a Posting Avoidance method (Chapter 9).

4.3 Analysis of Local Details

Concrete segmental bridges contain several important local details that must be appropriately designed so that the major bridge components (webs and slabs) can support anticipated loadings. Although forces in these details can vary as a function of the applied live loads, it is recommended that these details not be included in the load rating. Rather, the capacities of such details should be checked only for critical ratings and then only if there is evidence of distress in the bridge.

Important local details in segmental concrete bridges include dapped hinges within a span, the interaction of transverse web flexure and longitudinal shear, diaphragms, and transverse beams that support expansion joints. The behavior of these details and the forces to which they are subjected are well documented (Ref. 4.13, 4.14 and 4.15). Analysis methods and design procedures for these local details have been established following significant research and testing (Ref. 4.16, 4.17, 4.18 and 4.19).

4.3.1 Dapped Hinges within a Span

Forces acting on dapped hinges within a span should be determined as a part of the time-dependent construction analysis. Maximum live load reactions should also be appropriately determined. When these forces are known, local analyses should be performed to develop the hinge forces into the major bridge components. Two-dimensional strut-and-tie models, in both the vertical and horizontal planes, may be established to account for the effects of lateral as well as longitudinal eccentricities of bearings to the webs and slabs. If torsional effects are significant, a three-dimensional strut-and-tie analysis (space truss) may be required. An alternate approach would be to develop three-dimensional finite element models to analyze the flow of forces.

4.3.2 Interaction of Transverse Web Flexure and Longitudinal Shear

Self weight, superimposed dead loads and live loads produce both shear forces and transverse bending moments in the webs of box girder superstructures. Stresses from shear forces are greatest at the neutral axis of the box girder. Bending moments in the webs can produce tensile stresses on either face of the webs and are greatest at the top of the web.

Significant effort should have been expended during design to select the appropriate amount and distribution of reinforcing on each face of the webs. For typical box girders, the loadings that produce the maximum effects for shear and transverse flexure are not concurrent. As a result, historical practice has been to compute the amount of reinforcing required on each face

of the webs independently under the action of both shear forces and transverse moments. The total amount of reinforcing (A_s) placed on the face of a web is then chosen as the greatest of:

$$A_s = 1.0(A_v) + 0.5(A_f)$$

$$A_s = 0.5(A_v) + 1.0(A_f)$$

$$A_s = 0.7(A_v + A_f)$$

Where: A_v = Area of reinforcement required for longitudinal shear and torsion.
 A_f = Area of reinforcement required for transverse flexure.

4.3.3 Diaphragms at Interior Pier and Expansion Joint Segments

Two important functions of diaphragms in interior and expansion joint pier segments are to transfer shear forces in the webs to the bearings and provide torsional stability to the box girder. Diaphragms are typically detailed to permit appropriate strut-and-tie modeling of the flow of forces in the event that verification of these elements is required.

4.3.4 Transverse Beams to Support Expansion Joints

Transverse beams or thickened seats to support expansion joint devices are usually a contiguous part of the expansion joint segment and are integral with the deck slab and diaphragm. Typically a block-out or recess to accommodate embedded anchors of the expansion joint device itself would have been provided during construction and later filled with concrete. The recess often contains transverse post-tensioning tendons.

This transverse beam acts integrally with the segment and the diaphragm. The cantilever wing may be analyzed as a beam, assuming an appropriate portion of the top flange participates. The section on the interior between the webs should be analyzed in conjunction with the diaphragm action and forces. Local corbel action that the transverse beam provides to the expansion joint device itself should be considered.

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Chapter 5 – Material Properties

Material properties presented in the Chapter are prescriptive to promote uniformity in the load ratings of FDOT concrete bridges.

5.1 Modulus of Elasticity of Concrete

It is recommended that the Modulus of Elasticity of Concrete at 28 days, $E_c(28)$, be as directed in the Structures Design Guidelines:

- For concrete made with aggregate other than Florida Limerock:

$$E_{c28} = w_c^{1.5} \times 33\sqrt{f'_c}$$

- For concrete made with aggregate of Florida Limerock:

$$E_{c28} = 0.9(w_c^{1.5} \times 33\sqrt{f'_c})$$

Where: w_c = unit weight of concrete used in design (pcf) or load rating.

f'_c = nominal strength of concrete at 28 days (psi). The 28 day strength should be used regardless of actual age and strength of concrete at time of performing load rating evaluation.

For variation of the modulus of elasticity with age of concrete, see Table 5.1

5.2 Creep and Shrinkage of Concrete

It is recommended that the FDOT recognize the use of three models for predicting creep and shrinkage characteristics of concrete. The three models are those presented in CEB-FIP 1978 (1983), CEB-FIP 1990 and ACI 209 (Ref. 5.1, 5.2 and 5.3). In applying these creep and shrinkage models to bridges in Florida, the relative humidity should be taken as 75%.

5.2.1 Creep and Shrinkage According to CEB-FIP 1978 (1983)

The majority of concrete segmental bridges in Florida were designed in accordance with the creep and shrinkage models presented in Appendix E of CEB-FIP 1978 Code. The 1978 edition of the CEB-FIP presented the creep and shrinkage parameters in tabular form. The 1983 revision to this document presented the tabular information in equation form. Key formulae and values from the 1978 edition of CEB-FIP are presented below for convenience and may also be used for concrete beam bridges.

a. *Notional Thickness*

The notional thickness, h_0 is given by:
$$h_0 = \lambda \left(\frac{2A_c}{\mu} \right)$$

Where:

- $\lambda = 2.0$ (See Table 5.2 for 75% relative humidity).
- $A_c =$ area of concrete section.
- $\mu =$ perimeter in contact with the atmosphere.

For concrete made with Florida Limerock aggregate, the notional thickness should be taken as 70% of the values found from the equation above.

b. Creep

The strain due to the creep deformations under constant stress is defined as:

$$\varepsilon_c(t, t_0) = \left(\frac{\sigma_0}{E_{c28}} \right) \times \phi(t, t_0)$$

Where:

$\varepsilon_c(t, t_0)$ denotes the creep strain at time (t) under a constant stress, σ_0 applied at time t_0 .

E_{c28} is the longitudinal modulus of elasticity at 28 days (Section 5.1).

The total strain at the instant (t) under a constant stress (initial strain at the instant t_0 plus creep deformation) is given by:

$$\varepsilon_{tot}(t, t_0) = \sigma_0 \left(\frac{1}{E_c(t_0)} + \frac{\phi(t, t_0)}{E_{c28}} \right)$$

Where $E_c(t_0)$ is the initial value of the longitudinal modulus of elasticity at age t_0

The term:

$$\phi_{tot}(t, t_0) = \frac{1}{E_c(t_0)} + \frac{\phi(t, t_0)}{E_{c28}}$$

is called the "Creep Function."

The term,

$$\phi(t, t_0) = \beta_a(t_0) + \phi_d \beta_d(t - t_0) + \phi_f [\beta_f(t) - \beta_f(t_0)]$$

is the "creep coefficient" and incorporates various aspects of creep development with time depending upon the age of the concrete, environment (humidity), and notional thickness, where:

$\beta_a(t_0) / E_{c28}$ represents the irreversible part of the deformation developed during the first few days after loading.

$\phi_d \beta_d (t-t_0) / E_{C28}$ represents the recoverable part of the delayed deformation (delayed elasticity) assumed to be independent of aging in its development and is defined by a constant value of coefficient, ϕ_d .

$(\phi_f [\beta_f(t) - \beta_f(t_0)]) / E_{C28}$ represents the irreversible delayed deformation (flow) and is very much affected by the age at which loading commences.

In the above expressions:

$\phi_d = 0.4 =$ delayed modulus of elasticity.

$\phi_f = \phi_{f1} \times \phi_{f2} =$ flow coefficient.
 $\phi_{f1} = 1.83$ (See Table 5.2 for 75% relative humidity).
 ϕ_{f2} depends upon the notional thickness (Table 5.3).

$\beta_d =$ a function corresponding to the development with time of the delayed elastic strain (Table 5.4).

$\beta_f =$ a function corresponding to the development with time of the delayed plasticity depending upon the notional thickness (Table 5.5).

$t =$ denotes the age of the concrete in days at the time considered, corrected for temperature (below).

$t_0 =$ is the age of the concrete in days at the time of loading, corrected for temperature (below).

and:

$$\beta_a(t_0) = 0.8 \left(1 - \frac{f_c(t_0)}{f_c(\infty)} \right)$$

For β_a , the term " $f_c(t_0)/f_c(\infty)$ " represents the variation of the strength of the concrete with age. The value, as a ratio of the strength at time infinity, may be taken from Table 5.1.

c. Shrinkage

The strain due to shrinkage which develops in an interval of time $(t - t_0)$ is given by:

$$\varepsilon_s(t, t_0) = \varepsilon_{s0} [\beta_s(t) - \beta_s(t_0)]$$

Where:

$\varepsilon_{s0} = \varepsilon_{s1} \times \varepsilon_{s2}$
 $\varepsilon_{s1} = -0.00027$ (See Table 5.2 for 75% relative humidity).
 ε_{s2} = is that part of the development of shrinkage with time that depends upon the notional thickness (h_0) (Table 5.3).
 $\beta_s =$ function corresponding to the change of shrinkage with time and depends

upon the notional thickness, h_0 (Table 5.6).

t = denotes the age of the concrete in days at the time considered, corrected for temperature (below).

t_0 = is the age of the concrete in days at the time from which the influence of the shrinkage is considered.

For concrete made with Florida Limerock, the term " ϵ_{s2} " and the function " β_s " shall be taken for a notional thickness of 70% of the computed value.

5.2.2 Creep and Shrinkage According to CEB-FIP 1990

The 1990 version of the CEB-FIP Code contains formulations slightly different to those of 1983. Parametric studies show that, relative to the 1983 Code, the 1990 version underestimates creep by approximately 16 to 18% and underestimates shrinkage by 10 to 13%. Since one significant effect of creep and shrinkage is to reduce the effective post-tensioning, it remained customary and conservative practice within certain sectors of industry, particularly segmental bridges, to continue using the 1983 version. Also, because no significant casting curve or erection elevation problems had arisen, this practice continued.

The very existence of the 1990 Code brought pressure from some owners to use it on the basis of it being more recent and, therefore, presumably "better". There is no objection to its use in Florida on a project by project basis. However, the Designer or Load Rater should satisfy himself that it is appropriate for the bridge and circumstances.

There is no direct way to convert estimates of creep and shrinkage from one version of the code to another. For comparison purposes, Figures 5.1 and 5.2 and Table 5.7, show final total shortening due to elasticity plus creep and due to shrinkage for different codes relative to the CEB-FIP 1983 Code. The graphic results are summarized as the mean from one precast girder (Choctawhatchee Bay) and four segmental bridges (Broadway, Mid-Bay, Port of Miami and Long Key). For elasticity and creep, the concrete was assumed to be loaded at 28 days. Shrinkage was calculated from the day of casting. These results are for information only and should in no way be used for conversions from one code to another.

5.2.3 Creep and Shrinkage According to ACI 209

The American Concrete Institute (ACI) published a report on creep and shrinkage in 1984 (SP 76) that became ACI 209. This takes into account similar influences, such as type and age of concrete, notional thickness and humidity, for example, but the formulae are very different than CEB-FIP. Parametric studies on the same sample of bridges show that relative to CEB-FIP 1983, ACI 209 tends to underestimate creep by some 20 to 33%; but results in much greater shrinkage (26 to 59%). Again, see Figures 5.1 and 5.2 and Table 5.7 for comparisons.

The overall long-term results for creep plus shrinkage for the ACI and CEB-FIP methods are approximately the same. Significant differences between codes due to the combined effect of creep and shrinkage might be anticipated during construction or early service life.

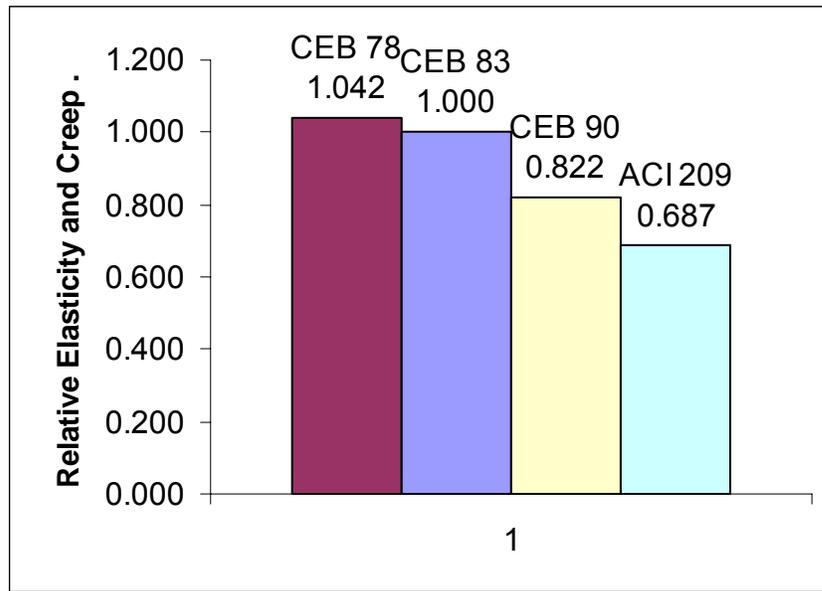


Figure 5.1 – Relative Elasticity and Creep according to Different Codes

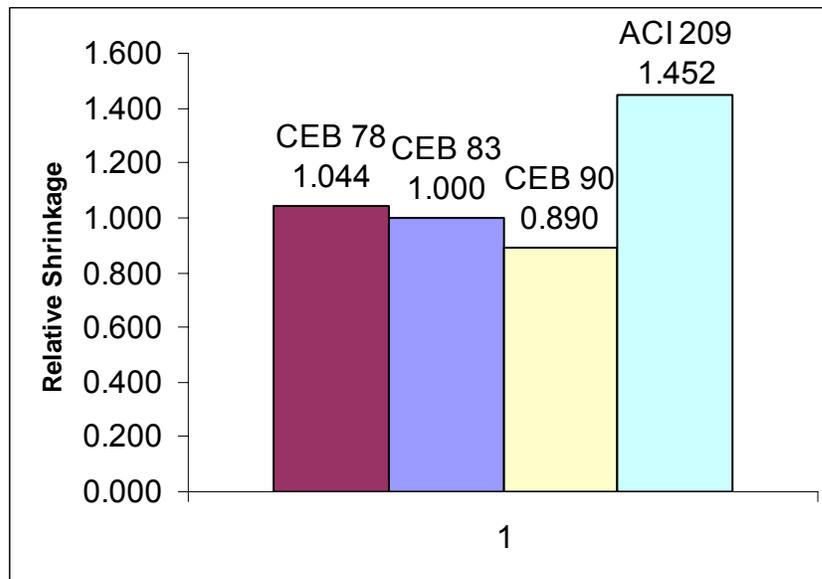


Figure 5.2 – Relative Shrinkage according to Different Codes

5.3 Properties of Prestressing Steel

For 270 ksi, 7-wire strand tendons:

- Modulus of elasticity = 28,500 ksi (LRFD 5.4.4.2).
- Relaxation characteristics with time - as per Segmental Guide Specifications (Sec. 10.4).
- Stress-strain information from Existing Plans should be used if available. If this information is not available, the stress-strain relationship shown in Design Aid 11.2.5 of the PCI Design Handbook (Ref. 5.4), with a limiting fracture strain of 0.055 in/in, should be used.

The following parameters are not material properties, but are required input for two-dimensional construction analyses. If not available from the Existing Plan information the following should be used:

- Anchor set = 3/8".
- Friction and Wobble coefficients as per LRFD Table 5.9.5.2.2b-1.
- Jacking force that would result in a maximum stress along the length of the tendon of $0.70f_{pu}$ immediately after anchor set.

For 150 ksi post-tensioning bars (permanent and temporary):

- Modulus of elasticity = 30,000 ksi (LRFD 5.4.4.2).
- Relaxation characteristics with time for temporary and permanent bars stressed in excess of $0.55f_{pu}$ - as per Segmental Guide Specifications (Sec. 10.4).

The following parameters are not material properties, but are required input for two-dimensional construction analyses. If not available from the Existing Plan information the following should be used:

- Anchor set = 3/8".
- Friction and Wobble coefficients – A value of 0.0 should be used for bars in straight ducts or external bars. Values provided in LRFD Table 5.9.5.2.2b-1 should be used for bars in contact with ducts (curved girder construction).
- Jacking stress for permanent bars = $0.70f_{pu}$. Jacking stress for temporary bars = $0.50f_{pu}$.

Age of Concrete (days)	E_c / E_{c28}	$f_c(t_0) / f_c(\infty)$
3	0.73	0.27
7	0.87	0.46
14	0.94	0.59
28	1.00	0.68
100	1.07	0.83
300	1.10	0.93
1000	1.13	0.98
10000	1.14	1.00

Table 5.1 – CEB-FIP 1978 Ratios for Modulus and Strength Development

Ambient Environment	Relative Humidity	Coefficient for Creep ϕ_{t1}	Coefficient for Shrinkage ϵ_{s1}	Coefficient λ
Water		0.95*	0.00010	30
Very Damp Atmosphere	90%	1.3*	-0.00013	5
Outside In General	70%	2.00	-0.00032	1.5
Very Dry Atmosphere	40%	3.00	-0.00052	1
FDOT LRFR Values	75%	1.83*	-0.00027	2.0

* Based on CEB-FIP 1983 formula correction to 1978 Appendix E.

Table 5.2 – CEB-FIP 1978 Coefficients ϕ_{t1} , ϵ_{s1} and λ

Notional Thickness		Coefficient for Creep ϕ_{t2}	Coefficient for Shrinkage ϵ_{s2}
(mm)	(in)		
50	2	1.85	1.20
100	4	1.70	1.05
200	8	1.55	0.90
400	16	1.40	0.86
600	24	1.30	0.80
800	32	1.25	0.75
>1600	48	1.12	0.70

Table 5.3 – CEB-FIP 1978 Coefficients ϕ_{t2} and ϵ_{s2}

Time Interval $t-t_0$ (days)	Delayed Elasticity $\beta_D(t-t_0)$
3	0.33
7	0.37
14	0.43
28	0.51
100	0.70
300	0.87
1000	0.99
10000	1.00

Table 5.4 – CEB-FIP 1978 Coefficients for Delayed Elasticity

Age of Concrete t (days)	Delayed Plasticity $\beta_r(t)$			
	$h_0 = 4"$	$h_0 = 8"$	$h_0 = 16"$	$h_0 = 32"$
3	0.13	0.13	0.13	0.13
7	0.21	0.20	0.20	0.18
14	0.30	0.28	0.26	0.24
28	0.41	0.37	0.33	0.30
100	0.61	0.56	0.48	0.42
300	0.77	0.72	0.63	0.54
1000	0.89	0.85	0.80	0.72
10000	1.00	1.00	1.00	1.00

h_0 = Notional Thickness

Table 5.5 – CEB-FIP 1978 Coefficients for Delayed Plasticity

Age of Concrete t (days)	Shrinkage $\beta_s(t)$			
	$h_0 = 4"$	$h_0 = 8"$	$h_0 = 16"$	$h_0 = 32"$
3	0.11	0.04	0.00	0.00
7	0.19	0.08	0.03	0.01
14	0.27	0.15	0.07	0.02
28	0.40	0.23	0.12	0.04
100	0.62	0.42	0.24	0.11
300	0.81	0.63	0.43	0.19
1000	0.94	0.87	0.72	0.45
10000	1.00	1.00	1.00	1.00

h_0 = Notional Thickness

Table 5.6 – CEB-FIP 1978 Coefficients for Shrinkage

	Broadway (Seg)	Mid-Bay (Seg)	P of Miami (Seg) (limerock)	Long Key (Seg) (limerock)	Choctaw'e (I-girder)	"Mean"
<p>CREEP (day 4,000) loaded at t = 28 days</p> <p>CEB 78 / CEB 83 = CEB 83 =</p> <p>CEB 90 / CEB 83 =</p> <p>ACI 209 / CEB 83 =</p>	<p>1.054</p> <p>1.000</p> <p>0.836</p> <p>0.683</p>	<p>1.043</p> <p>1.000</p> <p>0.819</p> <p>0.714</p>	<p>1.042</p> <p>1.000</p> <p>0.818</p> <p>0.660</p>	<p>1.027</p> <p>1.000</p> <p>0.817</p> <p>0.664</p>	<p>1.045</p> <p>1.000</p> <p>0.819</p> <p>0.714</p>	<p>1.042</p> <p>1.000</p> <p>0.822</p> <p>0.687</p>
<p>SHRINKAGE (day 4,000) after casting at day t = 0</p> <p>CEB 78 / CEB 83 = CEB 83 =</p> <p>CEB 90 / CEB 83 =</p> <p>ACI 209 / CEB 83 =</p>	<p>1.061</p> <p>1.000</p> <p>0.872</p> <p>1.414</p>	<p>1.055</p> <p>1.000</p> <p>0.897</p> <p>1.574</p>	<p>1.049</p> <p>1.000</p> <p>0.898</p> <p>1.261</p>	<p>1.000</p> <p>1.000</p> <p>0.884</p> <p>1.435</p>	<p>1.056</p> <p>1.000</p> <p>0.897</p> <p>1.574</p>	<p>1.044</p> <p>1.000</p> <p>0.890</p> <p>1.452</p>

Table 5.7 – Relative Elasticity plus Creep and Shrinkage according to Different Codes

References:

5.1 CEB-FIP Model Code for Concrete Structures, Comite Euro-International de Beton (CEB) et Federation International de Precontraint (FIP), 1978 (1983).

5.2 CEB-FIP Model Code for Concrete Structures, Comite Euro-International de Beton (CEB) et Federation International de Precontraint (FIP), 1990.

5.3 ACI Committee 209. *Prediction of Creep, Shrinkage and Temperature Effects in Concrete Structures*. ACI 209R-82. American Concrete Institute, Farmington Hills, MI, 1982.

5.4 *PCI Design Handbook*, 5th Edition, Precast/Prestressed Concrete Institute, Chicago, Illinois, 1999.

Chapter 6 – Loads

This Chapter recommends loading requirements for load rating concrete segmental bridges in Florida performed in accordance with LRFR. It also offers explanations for the adoption of various values for different parameters, such as "m", load factors for permanent, transient and live loads and the use of the number of striped lanes for operating service conditions.

6.1 Dead Loads

Dead loads include the self weight of the structure components (DC). Self weight (DC) should be found from available records including:

- (a) Design Plan or Shop Drawing dimensions and assumed average density for concrete, reinforcement and embedded items.
- (b) As-built dimensions, concrete thicknesses, and concrete density determined from construction records, adjusted for weight of embedded reinforcing.
- (c) Actual segment weights measured during construction.

Weights of diaphragms, deviator ribs, interior anchor blisters, and cast-in-place concrete closure pours should also be included in the self weight (DC). The weight of any other cast-in-place additions to the structure during construction or from subsequent repairs should be included in the structural dead load (DC). Traffic barriers and handrails may be included in "DC" when their weights are accurately known; otherwise include under "DW".

Structure dead loads are also comprised of superimposed dead loads (DW). Superimposed dead loads (DW) include all elements added to the structure after it has been erected. This usually includes utilities and wearing surfaces.

Since the introduction of the first edition of the AASHTO Guide Specification for Segmental Bridges (1989), cantilever bridges have been verified for a 2% differential dead load applied to one cantilever. This is intended for stability checks during construction and should not be included in load ratings.

For load rating, the dead load factor (γ_{DC}) and superimposed dead load factor (γ_{DW}) should be in accordance with Table 8.1.A1 "Load Factors for Segmental Bridges" and Table 8.1.A2 "Load Combinations for Segmental Bridges". Accurately verified segment weights or superimposed loads may be used to justify a lower component dead load factor (γ_{DC}) or superimposed dead load factor (γ_{DW}) in order to avoid posting (Chapter 9).

6.2 Other Permanent Loads

Other permanent loads to be used in FDOT load ratings of concrete segmental bridges are:

- Permanent effects of erection loads introduced by phased construction (EL).
- Secondary moments introduced when post-tensioning continuous structures (EL).

- Forces induced in superstructures monolithic with substructures as a consequence of long-term creep and shrinkage (CR, SH).
- Creep redistribution moments introduced by long-term creep and changes in statical schemes during construction (CR).

(Refer to Chapter 8 and Table 8.1.A1 for load factors for permanent loads).

6.3 Thermal Effects

Thermal effects are transient but may, under certain circumstances, induce forces and stresses that should be considered during load rating. Non-linear thermal gradients induce stresses in both simple span and continuous bridges. Concrete segmental bridges are designed for the effects of half thermal gradient ($0.5 \cdot TG$) with full design live loads at Service Limit State.

Thermal gradient effects do not apply when verifying the design of bridges at Strength Limit State. Inventory Ratings are performed at the same level of reliability as new designs, and should similarly include the effects of thermal gradients as per LRFD at the Service Limit State. In reality, the probability of the effects of the absolute maximum permissible live load occurring simultaneously with the maximum effect of thermal gradient is small. As a result, for Inventory Ratings, only $0.5 \cdot TG$ is taken with the design live load at service; for Operating Ratings, the effects of thermal gradient are not included at either the Service or Strength Limit States.

In most cases, longitudinal expansion and contraction of concrete bridges is accommodated by sliding or flexible bearings, with little resulting effect on the superstructure. Forces induced by thermal expansion and contraction (TU) should be considered where superstructures are rigidly restrained to substructures. These forces should only be included at the Service Limit State for Inventory Ratings.

6.4 Live Loads

For Inventory Ratings of concrete bridges in Florida, the Design Load is that specified in LRFD. Operating Ratings are required for the Design Load, Florida Legal Loads and Florida Permit Loads. (Refer to Chapter 8 and Table 8.1.A for load combinations and load factors.)

6.4.1 Design Load

6.4.1.1 Longitudinal Ratings

The Design Load for longitudinal Inventory and Operating Ratings is the group of loads that together in LRFD are called the HL93 Design Load. Summarizing, this comprises:

- 72 kip Truck (Previous "HS20" Truck), or,
- 50 kip Axle Tandem (25 kips per axle) with axles 4 feet apart, and,
- Uniform lane load (without impact) of 0.64 kips per foot coincident with the Truck or Tandem leaving no gaps.
- For negative moment regions over supports, 90% of two 72 kip Trucks spaced 50 feet apart along with 90% of a uniform load of 0.64 kips per foot.
- Dynamic Load Allowance, IM, (impact) of 33% applied to Truck or Tandem only.

6.4.1.2 Transverse Ratings or Local Structural Details

Transverse ratings of post-tensioned deck slabs of segmental boxes are limited by axle loads. The Design Load for transverse Inventory and Operating Ratings or for checks of local structural details should comprise the following axle configurations:

- Single Axle of the HL93 Truck.
- Double Axle of the HL93 Tandem.

For transverse load rating, the uniform lane load specified in the HL93 Design Load should NOT be applied in combination with any of the above axle loads. *This also applies to the design of new bridges.* The transverse design of top slabs is governed by local axle loads. An axle load is a specific, known value. In fact, maximum credible axle loads are less uncertain than maximum credible vehicle loads because axle loads are limited by the bending resistance of vehicle axles themselves. Therefore, inclusion of the uniform lane load in a transverse design or inventory rating is inappropriate.

6.4.2 Legal Loads

The following FDOT Legal Loads should be used for longitudinal and transverse load ratings:

- SU4, C5 and ST5 Trucks.
- Dynamic Load Allowance (IM) of 33% (Refer to Chapter 9, "Posting Avoidance").
- No uniform lane loads are to be applied with Legal Loads.
- The same Legal Load is placed in each loaded lane.
- The AASHTO Type 3-3 Legal Load is required for limiting critical cases (see 8.2.3).
- The HL93 72 kip GVW Truck component only for longitudinal conditions and either the Truck or the Tandem only for transverse conditions may be required for comparisons or to facilitate posting decisions by Maintenance Offices.

6.4.3 Permit Loads

A portion of the 160,000 lb FDOT Permit Vehicle comprises a Triple Axle unit (Figure 6.1). This should be used for Permit ratings for transverse deck slab flexure or for similar checks of local details. Parametric studies indicate that this triple axle configuration has an effect of a similar order of magnitude to the HL93 Truck or Tandem, but in some situations, it might control.

The FDOT T160 Permit Vehicle is to be used for longitudinal load ratings (Figure 6.2).

For longitudinal and transverse rating purposes, only one Permit Vehicle should be placed on a bridge at a time. For blanket (annual) permits under mixed traffic conditions, other lanes should be loaded with HL93 Design Load. For longitudinal ratings, this should include all features of the HL93 Design Load; for transverse ratings the load in the adjacent lanes should comprise the maximum of the HL93 Truck or Tandem only (i.e. with no uniform lane load).

For spans over 200 feet, a uniform lane load of 0.20 kip / LF should be applied in the same lane as, but beyond the footprint of, the permit vehicle; for convenience, this may be applied over the footprint of the vehicle.

6.4.4 Number of Live Load Lanes

For precast segmental bridges, Operating Ratings at the Service Limit State shall be performed using the number of striped load lanes with no limitation on the lateral location of loads. Operating Ratings at the Strength Limit State and Inventory Ratings at both Strength and Service Limits States shall be performed using the number of design live load lanes as specified in LRFD.

For AASHTO LRFR, the target reliability at Inventory is approximately 3.5 and at Operating, approximately 2.5, based upon strength limits. At the Strength Limit State this was achieved by means of reduced live load factors; for example, 1.35 versus 1.75.

However, for structures governed by service conditions, in order to attain similar benefits of reduced target reliability at the Service Limit State, it is necessary either to increase the allowable tensile stress or reduce the magnitude of the live load, or both. For precast segmental or similar structures with discrete joints and discontinuous mild-steel reinforcing, designed for no tension, an increase in allowable tensile stress is not possible. Consequently, it is necessary to reduce the live load.

Use of the number of striped lanes for Operating Ratings at the Service Limit State is intended to attain reduced target reliability (i.e. approximately $\beta = 2.5 < 3.5$) for bridges with discrete joints (i.e. discontinuous mild-steel reinforcing).

However, use of the number of striped lanes is not intended to limit the lateral location of loads. Loads should be placed so as to create the maximum effects, i.e. in shoulders if necessary. The same applies for a transverse rating of a slab where wheel loads should be placed for maximum load effect. Live load factors and load combinations should be according to Chapter 8.

6.4.5 Multiple Presence Factor (m)

For AASHTO LRFD, the "notional" Design Load (Article 3.6.1.2) comprising both truck and uniform lane load components was normalized for longitudinal conditions on the basis that the governing condition is for two lanes loaded. The value of the multiple presence factor for one lane loaded, i.e. $m = 1.20$, is to allow for the probability of a single heavy truck exceeding the weight of two or more fully correlated, heavy side-by-side trucks. Values for three or more lanes reduce because the probability of heavier simultaneous presence reduces. Consequently, for longitudinal load rating using the HL93 notional load, the multiple presence factors in LRFD are appropriate.

However, the possibility remains that an individual truck or axle load itself may exceed the specified, design load value. Therefore, it is appropriate to retain and apply the maximum multiple presence factor of $m = 1.20$ for a single lane of load at Inventory. (To illustrate this point, consider the Strength Limit for a single lane of live load. Using LRFD, the factored effect would be $1.75 \times 1.20 = 2.10$ times the truck load effect. This is essentially the same as previous LFD design; i.e. $1.30 \times 1.67 = 2.17$. If allowance is made for the difference in dynamic load allowance (impact) between LRFD and LFD, the results are even closer).

When a load rating in the longitudinal direction is required for a specific vehicle, i.e. for Legal or Permit Loads, the load rating should be evaluated for that particular vehicle's specified weight. Consequently, a maximum multiple presence factor, m , of 1.00 should be applied to Legal and

Permit Loads.

For transverse ratings of the design load, a maximum multiple presence factor of 1.20 is appropriate to allow for the possibility of rogue vehicles. However, transverse load ratings for Operating conditions are evaluated for specific axle loads; so it is not appropriate to apply a multiple presence factor, m , greater than 1.00.

For both longitudinal and transverse ratings where *more than one lane* causes the maximum effect, multiple presence factors specified in LRFD are appropriate to account for lower probability of heavier simultaneous presence and should be applied to Design, Legal and Permit Loads. For example, for three lanes (one Permit and two Design), $m = 0.85$ is applied to all three).

6.4.6 Live Load Factor at Service Limit State

For concrete tensile stress conditions, a service level live load factor of 0.80 was introduced in AASHTO LRFD and LRFR as a "load calibration" to recognize the fact that prestressed concrete bridges have generally performed well. They have shown little sign of distress due to flexural tension, despite being subjected to actual traffic loads of a similar magnitude to those of the new LRFD design load and despite having been designed to the previous LFD standard.

Since Inventory Rating is a check of current design conditions, it follows that it should have the same benefit for concrete flexural tension at the SERVICE III Limit State. This applies to longitudinal conditions for prestressed concrete structures. So, for Inventory SERVICE III conditions, a live load factor of $\gamma_L = 0.80$ is appropriate, along with a maximum multi-presence factor of 1.20, for ratings of the HL93 notional load when checking concrete tension at the Service Limit State (SERVICE III) and is used for the design of both prestressed girders and segmental bridges with discrete joints. The SERVICE I Limit State should be used for transverse conditions for both transversely post-tensioned deck slabs and for deck slabs in beam-type bridges.

In order to attain reduced target reliability at operating conditions, it is necessary to use the number of striped lanes for segmental bridges while retaining "no tension" in the joints using a service level live load factor of $\gamma_L = 1.00$ for specific (defined) truck loads. By comparison, in order to attain similar conditions for beams, it is necessary to increase the allowable tensile stress (e.g. from 3.0 to $7.5\sqrt{f'_c}$ psi) and to apply a reduced live load factor of $\gamma_L = 0.80$ on Legal and Permit loads.

A live load factor of $\gamma_L = 1.00$ should be used for transverse load ratings of Legal and Permit loads at the Service Limit State because they are carried out for specific axle loads.

In order to attain a reduced target reliability ($\beta = 2.5$ approx. < 3.5) parametric studies were performed on eight post-tensioned beam and three precast segmental bridges. Along with studies of Legal and Permit Loads, comparisons were also made for Operating conditions under full notional HL93 Design Load. For beam bridges, these studies considered various reduced live load factors for Design, Legal and Permit Loads and combinations of various amounts of HL93 Design load with a Permit load in mixed traffic. For segmental bridges, the studies also considered the use of the number of striped lanes. For beam bridges, relative good correlation was found between LRFR and previous LFD practice for a combination of the T160 Permit vehicle in one lane and approximately 0.65 times the HL93 load in the adjacent lanes. However,

since the order of magnitude of the T160 effects were found to be similar to those of a lane of HL93, it was decided to simplify the combination and apply a load factor value of $\gamma_L = 0.80$ to the sum of both for beam bridges. For segmental bridges, rating for mixed traffic of the Permit plus HL93 Design load is made using a load factor of $\gamma_L = 1.00$ but for the number of striped lanes.

Although it was not possible to ascertain definite target reliabilities from a relatively small sample, the studies generally verified the acceptability of using the number of striped lanes for segmental bridges and helped identify acceptable load factors and allowable stresses at the Service Limit for beam bridges.

6.4.7 Dynamic Load Allowance (IM)

For all load ratings, a Dynamic Load Allowance (IM) of 1.33 should be applied to all truck, tandem, axle or wheel load components, but not to uniform lane loads. This is necessary for the HL93 truck loads in order to increase the vehicle weight to a value consistent with trucks in service nationwide. The same value (1.33) should be applied to Legal and Permit Load conditions with mixed traffic mainly in order to allow for rogue vehicles. Under certain conditions, a reduced Dynamic Load Allowance may be considered in order to avoid posting (Chapter 9).

Chapter 7 – Capacity Factors

This Chapter presents recommendations for capacity factors to be used for load rating Florida concrete segmental bridges in accordance with LRFR Strength Limit States. Capacity factors are not used for Service Limit States. These capacity factors were developed by extending the concepts of “structural condition” and “structural redundancy” given in LRFR to the particulars of concrete segmental bridges. In addition, the concept of internal redundancy present in LRFR has been extended to the Multiple Tendon Path strategy introduced in conjunction with “New Directions for Florida Post-Tensioned Bridges”.

The Strength Limit State capacity factors to be used in LRFR load ratings are:

- ϕ = LRFD Strength Reduction Factor as appropriate to type of load effect (flexure, shear, torsion) and structural configuration or detail.
- ϕ_C = Condition Factor - takes a value from 0.85 to 1.10.
- ϕ_S = System Factor - takes a value from 0.85 to 1.30.

Condition and System Factors apply to the Strength Limit State. In accordance with LRFR, the product of the Condition and System Factors ($\phi_C \times \phi_S$) need not be taken less than 0.85 and in no case shall be greater than 1.30. One lower-bound value may be used or different values of these factors may be applied appropriately at different sections along the length of the bridge, if necessary.

7.1 Condition Factor, ϕ_C

7.1.1 General

The Condition Factor (ϕ_C) for post-tensioned concrete segmental bridges represents the degree of damage or loss of concrete or post-tensioning section due to some circumstance, such as corrosion or accidental damage.

For an existing bridge, the condition factor may be estimated from Table 7.1. Section 7.1.2 offers illustrative examples of Condition Factors for typical conditions of Florida concrete segmental bridges. When actual conditions have been determined by thorough inspection and measurement, the estimated condition factor may be increased by 0.05 but may not exceed 1.00. Measurement should include member thickness, loss of concrete section and an accurate estimate of loss of post-tensioning tendons or loss of rebar to corrosion or other damage.

Borescope investigation may verify surface corrosion or it may reveal broken wires. If the tendon is internal and is well bonded with grout, force transfer between the strands may develop greater effective force remote from the damaged section.

In general, if a structure is cracked and if there are any signs of significant rust or efflorescence emanating from cracks or joints that intersect internal tendon ducts or anchorages, then a close (in-depth) examination is warranted. In this case, either the actual section or post-tensioning force loss should be determined, or an appropriate Condition Factor (e.g., 0.85) should be assumed, until verified by in-depth inspection.

After repair or rehabilitation, a revised Condition Factor based upon the repaired condition (but $\phi_C \leq 1.00$) may be adopted. (Refer to FDOT Manuals, Volume 9, “Condition Inspection and Maintenance of Florida Post-Tensioned Bridges”).

7.1.2 Illustrative examples for ϕ_C

- Segmental bridge designed and built strictly in accordance with the FDOT “New Directions for Florida Post-Tensioned Bridges”: $\phi_C = 1.10$
- Precast segmental balanced cantilever bridge with internal tendons, relatively new, completed using latest grouting criteria enforced via specifications and training etc. not leaking, well-sealed epoxy joints, but without special duct connectors, for longitudinal evaluation: $\phi_C = 1.00$
- Precast segmental balanced cantilever bridge with internal tendons, relatively old, with no evidence of leaks at epoxy joints or any rust stains or efflorescence, generally in good or satisfactory condition, for longitudinal evaluation: $\phi_C = 1.00$
- Precast segmental balanced cantilever with internal tendons, relatively old, with leaks at epoxy joints, with rust staining or efflorescence and where inspections indicate evidence of corrosion of longitudinal tendons, and generally in poor condition, for a preliminary longitudinal evaluation: $\phi_C = 0.85$
- Same condition as previous example but after an in-depth inspection reveals no pitting corrosion and less than 5% wire breaks: $\phi_C = 0.90$
- Precast span-by-span with external tendons – inspected to reveal no corrosion to tendons and no evidence or indications of tendon loss or corrosion damage and generally in good or satisfactory condition, for longitudinal evaluation: $\phi_C = 1.00$
- Precast span-by-span with external tendons – inspected to reveal corroded tendons or indications of section loss or damage: $\phi_C = 0.85$
- Same condition as previous example but after an in-depth inspection reveals no pitting corrosion and less than 5% wire breaks: $\phi_C = 0.90$
- For transverse internal tendons in top slabs, diaphragms or similar, where there is no visible evidence of corrosion damage such as rust stains or efflorescence coming from the tendons or other indications of distress such as longitudinal flexural cracks in the top surface of cantilever wings or in the underside middle portion of top slabs: $\phi_C = 1.00$
- For similar transverse internal tendons where there is evidence of rust, efflorescence, possible corrosion damage or cracks that has developed: $\phi_C = 0.85$

For structures and intermediate conditions between any of the above, engineering judgment

may be used to select an appropriate value for ϕ_C between 0.85 and 1.00. A value greater than 1.00 may only be used for bridges designed and built strictly in accordance with the FDOT “New Directions for Florida Post-Tensioned Bridges.”

If corrosion damage to bonded tendons is localized to one region or to one or more particular cross sections and the rest of the structure is otherwise satisfactory, then the low value (0.85) may be applied to those areas and an appropriately higher value to others. However, damage to an internal tendon at one section may mean that it may be only partially effective at other sections and caution is advised.

7.2 System Factor, ϕ_S

The System Factor (ϕ_S) is related to the degree of redundancy in the total structural system. In LRFR, bridge redundancy is defined as the capability of a structural system to carry loads after damage or failure of one or more of its members. LRFR recognizes that structural members of a bridge do not behave independently, but interact with one another to form one structural system.

Current LRFR System Factors do not adequately address the characteristic behavior of post-tensioned segmental box girder construction in three areas:

- Longitudinal Continuity – The research upon which LRFR is based (NCHRP 406), considered longitudinal bridge continuity. However, longitudinal continuity that makes a structure statically indeterminate and increases the overall redundancy is not acknowledged in LRFR.
- Continuum of the Closed Box Girder – LRFR primarily considers bridge superstructures comprised of multiple girders with composite deck slabs. LRFR does not give consideration to enhanced behavior of box girders with regard to torsion, and therefore the participation of the entire cross section in resisting loads.
- Multiple Tendon Paths and Internal Redundancy – LRFR introduces the concept of Internal Redundancy provided by multiple, independent component load paths (single welds versus bolted connections). This concept of internal redundancy is not extended in similar fashion to multiple tendon paths typical of segmental construction.

The System Factors presented in this Chapter incorporate the benefits of these three features for concrete segmental box girder bridges. Appendix A provides background information regarding the development of System Factors for Segmental Bridges.

7.2.1 Longitudinal Flexure

System Factors for longitudinal flexure at the Strength Limit State should be taken from Table 7.2. System Factors in this table are given for different types of segmental construction, different degrees of longitudinal continuity expressed in terms of the number of plastic hinges required to create a mechanism, the number of webs (two and three or more) and number of tendons per web. For longitudinal flexure, System Factors range, for example from 0.85 for a simple span with only two tendons per web to 1.30 for a box with three or more webs each with four or more tendons per web.

7.2.2 Shear and Torsion

System Factors for longitudinal shear or shear combined with torsion should be taken as a single value of $\phi_S = 1.00$.

7.2.3 Transverse Flexure

Where there is a closed continuum to the cross section of the box structure, the System Factor for transverse flexure should be $\phi_S = 1.00$.

7.3 Local Details

Local Details including dapped hinges within a span, diaphragms at interior and expansion joint piers, and deviators, are not part of the Load Rating procedure. These details should be reviewed, however, to ensure that the details can support the load ratings predicted for the major bridge elements.

In general, a System Factor (ϕ_S) depends upon the degree of redundancy provided by the local post-tensioning and reinforcing. System Factors for local details should be taken as 0.90 when only one post-tensioning tendon (or bar) contributes to or provides the main resistance of the detail. The System Factor of 1.00 may be used when two or more post-tensioning tendons or bars provide the resistance.

Structural Condition of Member	NBI Rating	Condition Factor (ϕ_c)
Good or Satisfactory	> 6	1.00
Fair	5	0.95
Poor	< 4	0.85
Bridges built to recommendations of "New Directions for Florida Post-Tensioned Bridges," FDOT, 2002	>> 6	1.10

(See LRFR Table 6.4.2.3-1 and Commentary)

Table 7.1 – Relationship between NBI Rating and ϕ_c

Bridge Type	Span Type	# of Hinges to Failure	System Factors (ϕ_s)			
			No. of Tendons per Web			
			1/web	2/web	3/web	4/web
Precast Balanced Cantilever Type A Joints	Interior Span	3	0.90	1.05	1.15	1.20
	End or Hinge Span	2	0.85	1.00	1.10	1.15
	Statically Determinate	1	n/a	0.90	1.00	1.10
Precast Span-by-Span Type A Joints	Interior Span	3	n/a	1.00	1.10	1.20
	End or Hinge Span	2	n/a	0.95	1.05	1.15
	Statically Determinate	1	n/a	n/a	1.00	1.10
Precast Span-by-Span Type B Joints	Interior Span	3	n/a	1.00	1.10	1.20
	End or Hinge Span	2	n/a	0.95	1.05	1.15
	Statically Determinate	1	n/a	n/a	1.00	1.10
Cast-In-Place Balanced Cantilever	Interior Span	3	0.90	1.05	1.15	1.20
	End or Hinge Span	2	0.85	1.00	1.10	1.15
	Statically Determinate	1	n/a	0.90	1.00	1.10

(For box girder bridges with 3 or more webs, table values may be increased by 0.10).

Table 7.2 – System Factors for Longitudinal Flexure

Chapter 8 – Rating Equation and Load Combinations

Six (6) features of concrete segmental bridges are to be load rated at both Inventory and Operating Levels. Three of these criteria are at the Service Limit State and three at the Strength Limit State, as follows:

At the Service Limit State:

- Longitudinal Box Girder Flexure
- Transverse Top Slab Flexure
- Principle Web Tension

At the Strength Limit State:

- Longitudinal Box Girder Flexure
- Transverse Top Slab Flexure
- Web Shear

This Chapter presents the General Load Rating Equation and recommended Load Factors and Load Combinations for use in rating the above six features of concrete segmental bridges.

8.1 General Load Rating Equation

In accordance with AASHTO LRFR Equation 6-1, the general Load Rating Factor, RF, should be determined according to the formula:

$$RF = \frac{C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{EL}(P + EL) - \gamma_{FR}FR - \gamma_{CR}(TU + CR + SH) - \gamma_{TG}TG}{\gamma_L(LL + IM)}$$

Where:

For Strength Limit States:

C = Capacity = $(\phi_c \times \phi_s \times \phi) R_n$.

ϕ_c = Condition Factor per Chapter 7.

ϕ_s = System Factor per Chapter 7.

ϕ = Strength Reduction Factor per LRFD.

R_n = Nominal member resistance as inspected, measured and calculated according to formulae in LRFD - with the exception of shear, for which, capacity is calculated according to the AASHTO Guide Specification for Segmental Bridges.

For Service Limit States:

C = f_R = Allowable stress at the Service Limit State (*Table 8.2.A*).

Allowable stress levels have been established in order to limit cracking and protect the integrity of corrosion protection afforded post-tensioning tendons. This is particularly important for posting in order to limit the effects of excessive loads and rogue vehicles.

Load Effects and Nomenclature per LRFD / LRFR:

- DC = Dead load of structural components (includes barriers if accurately known).
- DW = Dead load of permanent superimposed loads such as wearing surface and utilities (applies to barriers when weight is not accurately known).
- P = Permanent effects other than dead load (LRFR), including prestress.
- EL = Permanent effects of erection forces (e.g. from erection equipment, changes in statical scheme) and includes secondary effects of post-tensioning.
- FR = Forces from fixed bearings, bearing friction or frame action, otherwise zero.
- TU = Uniform temperature effects from fixed bearings or frame action, otherwise zero.
- CR = Creep.
- SH = Shrinkage.
- TG = Thermal gradient.
- LL = Live load.
- IM = Dynamic Load Allowance (Impact).
- γ_{DC} = Load factor for structural components.
- γ_{DW} = Load factor for permanent superimposed dead loads.
- γ_{EL} = Load factor for secondary PT effects and locked-in erection loads.
- γ_{FR} = Load factor for bearing friction or frame action.
- γ_{CR} = Load factor for uniform temperature, creep and shrinkage.
- γ_{TG} = Load factor for thermal gradient.
- γ_L = Live load factor.

8.2 Load Factors and Load Combinations

Load factors and load combinations for the Strength and Service Limit States should be made in accordance with *Table 8.1.A1, "Load Factors for Segmental Bridges"* and *Table 8.1.A2, "Load Combinations for Segmental Bridges"*. Table 8.1.A1 is separated horizontally into longitudinal and transverse requirements and vertically into Inventory or Operating conditions. Load factors for permanent (e.g. dead) loads and transient (e.g. temperature) loads are provided. Note: one-half thermal gradient (0.5TG) is used only for longitudinal Service Inventory conditions.

Altogether, load combinations (Table 8.1.A2) are given for eleven basic cases, labeled "#1" through "#11", which are necessary to satisfy FDOT and AASHTO LRFR. The first two (#1 and #2) are for Inventory (design) conditions. #3 and #4 are for Operating conditions using Design loads. #5 addresses FDOT Legal Loads. #6 and #7 address AASHTO limiting critical (legal) loads. For Permit vehicles in mixed traffic, two combinations must be added together: the permit is applied in one lane (#8) with HL93 in the remaining live load lanes (either #9 or #10) as appropriate. The last (#11) is for a lone permit vehicle crossing.

STRENGTH I and II and SERVICE I and III conditions are used in the context of their definitions as given in Table 8.1.A1, summarizing:

STRENGTH I - applies to Inventory and Operating conditions for Design and Legal loads.

STRENGTH II - applies only to Permit Loads.

SERVICE I - applies primarily for concrete in compression but is also to prevent yield of tension face reinforcement or prestress under overloads (permits). This condition is extended to

concrete tension in transversely prestressed deck slabs, typical of most segmental bridges.

SERVICE III - applies to concrete in longitudinal tension and principal tension. Load factors for SERVICE III for Operating conditions have been selected to attain the benefits of reduced reliability when used in conjunction with either higher allowable tensile stress or, in the case of joints that cannot carry tension, use of the number of striped lanes. For consistency with the tension side, where the allowable stress cannot be augmented, use of the number of striped lanes is retained to achieve appropriate reductions in reliability for SERVICE I compression (See 6.4.4).

The following is a detailed checklist of the load applications, combinations and circumstances necessary to satisfy FDOT and AASHTO LRFR ratings.

8.2.1 Inventory Rating – Design Loads

Transverse:

- Apply HL93 Truck or Tandem (*Table 8.1.A2, load combination #1*).
- Do not apply uniform lane load.
- Apply same axle loads in each lane.
- Apply Dynamic Load Allowance, IM = 1.33 on Truck or Tandem.
- For both Strength and Service Limit States, use number of load lanes per LRFD.
- Apply multi-presence factor: one lane, $m = 1.20$; two lanes, $m = 1.00$; three, $m = 0.85$; four or more, $m = 0.65$. (Maximum value of $m = 1.20$ is the appropriate AASHTO LRFD / LRFR current criteria to allow for rogue vehicles).
- Place loads in full available width as necessary to create maximum effects.
- Apply pedestrian live load as necessary (counts as one lane for “m”).
- Apply no Thermal Gradient transversely.
- Use SERVICE I Limit State with live load factor, $\gamma_L = 1.00$ and limit concrete transverse flexural stresses to values in Table 8.2.A. (Note: $\gamma_L = 1.00$ as AASHTO LRFR).
- For STRENGTH I Limit State use live load factor, $\gamma_L = 1.75$.

Longitudinal:

- Apply HL93 Truck or Tandem, including 0.64 kip/ft uniform lane load (*Table 8.1.A2, load combination #1*).
- Apply same load in each lane.
- Apply Dynamic Load Allowance, IM = 1.33 on Truck or Tandem only.
- For both Strength and Service Limit States, use number of load lanes per LRFD.
- Apply multi-presence factor: one lane, $m = 1.2$; two lanes, $m = 1.00$; three, $m = 0.85$; four or more, $m = 0.65$. (Maximum value of $m = 1.20$ is the appropriate AASHTO LRFD / LRFR current criteria for notional loads and rogue vehicles).
- For negative moment regions: apply 90% of the effect of two Design Trucks of 72 kip GVW spaced a minimum of 50 feet apart between the leading axle of one and the trailing axle of the other, plus 90% of uniform lane load (*Table 8.1.A2, load combination #2*).
- Place loads in full available width as necessary to create maximum effects.
- Apply pedestrian live load as necessary (counts as one lane for “m”).
- For Thermal Gradient, apply 0.50TG with live load for Service but zero TG for Strength.
- Use SERVICE III Limit State with live load factor, $\gamma_L = 0.80$. (Note: use of $\gamma_L = 0.80$ is for

- load calibration as adopted by AASHTO LRFR).
- For SERVICE III Limit State, limit concrete Longitudinal Flexure Tensile Stress to values in Table 8.2.A as appropriate.
 - For SERVICE III Limit State, limit Principal Tensile Stress at the neutral axis to $3\sqrt{f_c}$ (psi) at Inventory. (During construction, a temporary overstress to $4.5\sqrt{f_c}$ (psi) may be allowed).
 - Use SERVICE I Limit State with live load factor, $\gamma_L = 1.00$ and limit concrete longitudinal flexural compressive stress to values in Table 8.2.A. (Note: $\gamma_L = 1.00$ as AASHTO LRFR).
 - For STRENGTH I Limit State use live load factor, $\gamma_L = 1.75$.

8.2.2 Operating Rating – Design Load (HL93)

Transverse:

- Apply one HL93 Truck or Tandem per lane (*Table 8.1.A2, load combination #3*).
- Do not apply uniform lane load.
- Apply same axle loads in each lane.
- Apply Dynamic Load Allowance, IM = 1.33 on Truck or Tandem.
- For both Strength and Service Limit States, use number of load lanes per LRFD.
- Apply multi-presence factor: one and two lanes, $m = 1.0$; three, $m = 0.85$; four or more, $m = 0.65$. (Maximum limit of 1.0 applies because this is a rating for specific (defined) axle loads, not notional loads or rogue vehicles).
- Place loads in full available width as necessary to create maximum effects.
- Apply pedestrian live load as necessary (counts as one lane for “m”).
- Apply no Thermal Gradient transversely.
- Use SERVICE I Limit State with live load factor, $\gamma_L = 1.00$ and limit concrete transverse flexural stresses to values in Table 8.2.A. (Note: use of $\gamma_L = 1.00$ is necessary because loads are actual (defined) axle loads. Reduced reliability is obtained in this case by an increased allowable tensile stress - i.e. $6\sqrt{f_c}$ (psi) at Operating compared to $3\sqrt{f_c}$ (psi) at Inventory, Table 8.2.A).
- For STRENGTH I Limit State use live load factor, $\gamma_L = 1.35$.

Longitudinal:

- Apply HL93 Truck or Tandem, including 0.64 kip/ft uniform lane load (*Table 8.1.A2, load combination #3*).
- Apply same load in each lane.
- Apply Dynamic Load Allowance, IM = 1.33 on Truck or Tandem only.
- For the Strength Limit State, use number of load lanes per LRFD.
- For the Service Limit State use the number of striped lanes (this is to attain the benefits of reduced reliability).
- Place loads in full available width as necessary to create maximum effects (for example, in shoulders).
- Multi-presence factor: HL93 Design Load (including uniform lane load) one lane, $m = 1.20$; two lanes, $m = 1.00$; three, $m = 0.85$; four or more, $m = 0.65$. (The maximum value of 1.20 for one lane is necessary because the load is a notional load with a uniform lane load component).
- For negative moment regions, apply 90% of the effect of two Design Trucks of 72 kip GVW each spaced a minimum of 50 feet apart between the leading axle of one and the

- trailing axle of the other, plus 90% of 0.64 kip/LF uniform lane load (*Table 8.1.A2, load combination #4*).
- Apply pedestrian live load as necessary (counts as one lane for “m”).
 - Apply no Thermal Gradient.
 - Use SERVICE III Limit State with live load factor, $\gamma_L = 1.00$ and limit concrete longitudinal flexural tensile and principal tensile stresses to values in Table 8.2.A as appropriate. (Note: use of $\gamma_L = 1.00$ is appropriate because reduced reliability for large boxes is attained through the use of the number of striped lanes. At Operating, although no increase in allowable tensile stress (i.e. zero) can be allowed in precast joints, an increase is allowed from 3 to $7.5\sqrt{f_c}$ (psi) in reinforced joints and the Principal Tensile Stress at the neutral axis is raised to $4\sqrt{f_c}$ (psi) to attain the benefit of reduced reliability per Table 8.2.A)
 - Use SERVICE I Limit State with live load factor, $\gamma_L = 1.00$ and limit concrete longitudinal flexural compressive stress to values in Table 8.2.A. (Note: $\gamma_L = 1.00$ AASHTO LRFR).
 - For STRENGTH I Limit State use live load factor, $\gamma_L = 1.35$.

8.2.3 Operating Rating – Florida Legal Loads

Transverse:

- Apply FDOT Legal Load Trucks *SU4, C5 and ST5* (*Table 8.1.A2, load combination #5*).
- Also, apply *HL93 Truck or Tandem only* (*load combination #5*). This is to facilitate comparison and posting decisions.
- Do not apply any uniform lane load.
- Apply same axle loads in each lane using only one truck per lane (i.e. do not mix Trucks).
- Apply Dynamic Load Allowance, $IM = 1.33$ on Legal, HL93 Truck or Tandem (see Chapter 9, “Posting Avoidance”).
- For both Strength and Service Limit States, use number of load lanes per LRFD.
- Apply multi-presence factor: one and two lanes, $m = 1.0$; three, $m = 0.85$; four or more, $m = 0.65$. (Maximum limit of 1.0 applies because this is a rating for specific (defined) axle loads, not notional loads or rogue vehicles).
- Place loads in full available width as necessary to create maximum effects.
- Apply no pedestrian live load (unless very specifically necessary for the site - in which case it counts as one lane for establishing “m”).
- Apply no Thermal Gradient transversely.
- Use SERVICE I Limit State with live load factor, $\gamma_L = 1.00$ and limit concrete transverse flexural stresses to values in Table 8.2.A. (Note: use of $\gamma_L = 1.00$ is necessary because this is a rating for specific (defined) axle loads. Reduced reliability is obtained in this case by an increased allowable tensile stress - i.e. $6\sqrt{f_c}$ (psi) at Operating compared to $3\sqrt{f_c}$ (psi) at Inventory, Table 8.2.A).
- For STRENGTH I Limit State use live load factor, $\gamma_L = 1.35$.

Longitudinal:

- Apply FDOT Legal Load Trucks *SU4, C5 and ST5* (*Table 8.1.A2, load combination #5*).
- Also, apply *HL93 Truck only* - i.e. *72 kip GVW* (*load combination #5*). This is to facilitate comparison and posting decisions.
- Apply same Truck load in each lane using only one truck per lane (i.e. do not mix Trucks).

- Apply no uniform lane load.
- Apply Dynamic Load Allowance, $IM = 1.33$ on Legal, HL93 Truck or Tandem (see Chapter 9, "Posting Avoidance").
- For the Strength Limit State, use number of load lanes per LRFD.
- For Service Limit States, use number of striped lanes (this is to attain the benefits of reduced reliability).
- Place loads in full available width as necessary to create maximum effects (for example, in shoulders).
- Use multi-presence factor: one and two lanes, $m = 1.00$; three, $m = 0.85$; four or more, $m = 0.65$. (Maximum limit of 1.0 applies because loads are specific (defined) truck loads, not notional loads or rogue vehicles).
- Apply no pedestrian live load (unless very specifically necessary for the site - in which case it counts as one lane for establishing "m").
- Apply no Thermal Gradient.
- Use SERVICE III Limit State with live load factor, $\gamma_L = 1.00$ and limit concrete longitudinal flexural tensile and principal tensile stresses to values in Table 8.2.A as appropriate. (Note: use of $\gamma_L = 1.00$ is appropriate because reduced reliability for large boxes is attained through the use of the number of striped lanes. At Operating, although no increase in allowable tensile stress (i.e. zero) can be allowed in precast joints, an increase is allowed from 3 to $7.5\sqrt{f'_c}$ (psi) in reinforced joints and the Principal Tensile Stress at the neutral axis is raised to $4\sqrt{f'_c}$ (psi) to attain the benefit of reduced reliability per Table 8.2.A)
- Use SERVICE I Limit State with live load factor, $\gamma_L = 1.00$ and limit concrete longitudinal flexural compressive stress to values in Table 8.2.A. (Note: $\gamma_L = 1.00$ AASHTO LRFR).
- For STRENGTH I Limit State, use live load factor, $\gamma_L = 1.35$.
- Negative moments load ratings may be limited by AASHTO LRFR 6.4.4.2.1, as follows. Determine the AASHTO Limiting Critical Load effects from a lane load of 0.20 K/LF combined with 0.75 times the effect of two AASHTO Type 3-3 Trucks in the same lane, heading in the same direction and separated by 30ft. (Table 8.1.A2, load combination #6). If the value of the Rating Factor for the AASHTO Limiting Critical Load is less than 1.00, then the basic rating factor for all FDOT Legal Loads shall be reduced by multiplying by this value.
- In addition, load rating may be limited by AASHTO LRFR 6.4.4.2.1. For spans less than 200 feet, determine AASHTO Limiting Critical Load effects for one AASHTO Type 3-3. For spans over 200 feet, determine effects for one AASHTO Type 3-3 multiplied by 0.75 combined with a lane load of 0.20 K/LF (Table 8.1.A2, load combination #7). If the value of the Rating Factor for the AASHTO Limiting Critical Load is less than 1.00, then the basic rating factor for all FDOT Legal Loads shall be reduced by multiplying by this value.

8.2.4 Operating Rating – Florida Permit Loads

Transverse, annual blanket permits, mixed traffic:

- Apply ONE Permit Vehicle in one load lane. Use T160 vehicle with its triple axle units of 3 axles of 22 kips each (Table 8.1.A2, load combination #8 plus #9).
- Apply HL93 Truck or Tandem axles only in each of the other load lanes as necessary to create maximum effects (Table 8.1.A2, load combination #8 plus #9).
- Do not apply any uniform lane load.

- Apply Dynamic Load Allowance, IM = 1.33 on Permit, HL93 Truck or Tandem (See Chapter 9 “Posting Avoidance”).
- Do not mix Permit Load with Legal Load.
- For both Strength and Service Limit States, use number of load lanes per LRFD.
- Apply multi-presence factor: one and two lanes, $m = 1.0$; three, $m = 0.85$; four or more, $m = 0.65$. (Maximum limit of 1.0 applies because this is a rating for specific (defined) axle loads, not notional loads or rogue vehicles).
- Place loads in full available width as necessary to create maximum effects.
- Apply no pedestrian live load (unless very specifically necessary for the site - in which case it counts as one lane for establishing “m”).
- Apply no Thermal Gradient transversely.
- Use SERVICE I Limit State with live load factor, $\gamma_L = 1.00$ and limit concrete transverse flexural stresses to values in Table 8.2.A. (Note: use of $\gamma_L = 1.00$ is necessary because this is a rating for specific (defined) axle loads. Reduced reliability is obtained in this case by an increased allowable tensile stress - i.e. $6\sqrt{f_c}$ (psi) at Operating compared to $3\sqrt{f_c}$ (psi) at Inventory, Table 8.2.A).
- Use SERVICE I Limit State with live load factor, $\gamma_L = 1.00$ if it is necessary to evaluate the rating according to the maximum allowable tensile stress (i.e. 90% yield) in reinforcing or prestressing steel closest to the tension fiber (AASHTO LRFR).
- For STRENGTH II Limit State, use live load factor, $\gamma_L = 1.35$.
- Reduced Dynamic Load Allowance (IM) or live load factor (γ_L) may be considered only to avoid restrictions (Chapter 9).

Transverse, special or limited, escorted trip, any Permit vehicle, no other traffic on bridge:

- As above, but without HL93 Loads (i.e. *Table 8.1.A2, load combination #11*).
- Place load to produce maximum effect or, if necessary, in a designated location (for example, straddling a web) providing that location is strictly enforced.
- Apply no pedestrian live load, unless it is determined that pedestrians will be present, otherwise ensure that pedestrians are restricted.
- Dynamic Load Allowance, IM = 1.33.
- Multi-presence factor, $m = 1.00$.
- Use SERVICE I Limit State with live load factor, $\gamma_L = 1.00$ and limit concrete transverse flexural stresses to values in Table 8.2.A. (Note: use of $\gamma_L = 1.00$ is necessary because this is a rating for specific (defined) axle loads. Reduced reliability is obtained in this case by an increased allowable tensile stress - i.e. $6\sqrt{f_c}$ (psi) at Operating compared to $3\sqrt{f_c}$ (psi) at Inventory, Table 8.2.A).
- Use SERVICE I Limit State with live load factor, $\gamma_L = 1.00$ if it is necessary to evaluate the rating according to the maximum allowable tensile stress (i.e. 90% yield) in reinforcing or prestressing steel closest to the tension fiber (AASHTO LRFR).
- For STRENGTH II Limit State, use live load factor, $\gamma_L = 1.15$.
- Reduced Dynamic Load Allowance (IM) or live load factor (γ_L) may be considered only to avoid restrictions (Chapter 9).

Longitudinal, annual “blanket” permits, mixed traffic:

- Apply ONE T160 Permit Vehicle in one load lane (*Table 8.1.A2, load #8*).
- Apply HL93 Truck of 72 kips GVW in each of the other load lanes as necessary to create

- maximum effects, including 0.64 kip / LF uniform lane load (*Table 8.1.A2, load #9*).
Combine #8 with #9.
- Alternatively, for negative moment regions: in conjunction with the Permit vehicle in its lane, apply to the other lanes 90% of the effect of two Design Trucks of 72 kip GVW each spaced a minimum of 50 feet apart between the leading axle of one and the trailing axle of the other, plus 90% of 0.64 kip/LF uniform lane load (*Table 8.1.A2, load #10*). Combine #8 with #10.
 - For spans over 200 feet, apply a uniform lane load of 0.20 kip / LF in the lane with the permit vehicle. This uniform lane load should be applied beyond the footprint of the vehicle to create the maximum effects. However, for convenience, it may be applied coincident with the vehicle.
 - For the Strength Limit State, use number of load lanes per LRFD.
 - For Service Limit States, use number of striped lanes (this is to attain the benefits of reduced reliability).
 - Place loads in full available width as necessary to create maximum effects (for example, in shoulders).
 - Use multi-presence factor: one and two lanes, $m = 1.00$; three, $m = 0.85$; four or more, $m = 0.65$. (Maximum limit of 1.0 applies because loads are specific (defined) Permit loads, not notional loads or rogue vehicles).
 - Do not mix Permit Load with Legal Loads.
 - Dynamic Load Allowance, $IM = 1.33$ on Permit and HL93 Trucks (see Chapter 9, "Posting Avoidance").
 - Apply no pedestrian live load (unless very specifically necessary for the site - in which case it counts as one lane for establishing "m").
 - Apply no Thermal Gradient.
 - Use SERVICE III Limit State with live load factor, $\gamma_L = 1.00$ and limit concrete longitudinal flexural tensile and principal tensile stresses to values in Table 8.2.A as appropriate. (Note: use of $\gamma_L = 1.00$ is appropriate because reduced reliability for large boxes is attained through the use of the number of striped lanes. At Operating, although no increase in allowable tensile stress (i.e. zero) can be allowed in precast joints, an increase is allowed from 3 to $7.5\sqrt{f_c}$ (psi) in reinforced joints and the Principal Tensile Stress at the neutral axis is raised to $4\sqrt{f_c}$ (psi) to attain the benefit of reduced reliability per Table 8.2.A)
 - Use SERVICE I Limit State with live load factor, $\gamma_L = 1.00$ and limit concrete longitudinal flexural compressive stress to values in Table 8.2.A. (Note: $\gamma_L = 1.00$ AASHTO LRFR).
 - For STRENGTH II Limit State, use live load factor, $\gamma_L = 1.35$.
 - Reduced Dynamic Load Allowance (IM) or live load factor (γ_L) may be considered only to avoid restrictions (Chapter 9).

Longitudinal, special or limited, escorted trip, any Permit Vehicle, no other traffic on bridge:

- As above except, apply ONE T160 Permit Vehicle with no other live load (*i.e. Table 8.1.A2, load combination #11*).
- Place load to produce maximum effect or, if necessary, place in a designated location (for example, straddling web) providing that the location is strictly enforced.
- Apply no pedestrian live load unless it is determined that pedestrians will be present. Otherwise ensure pedestrians are restricted.
- Dynamic Load Allowance, $IM = 1.33$ (but also refer to Chapter 9, "Posting Avoidance").

- Multi-presence factor, $m = 1.00$.
- Use SERVICE III Limit State with live load factor, $\gamma_L = 1.00$ and limit concrete longitudinal flexural tensile and principal tensile stresses to values in Table 8.2.A as appropriate. (Note: use of $\gamma_L = 1.00$ is appropriate because reduced reliability for large boxes is attained through the use of the number of striped lanes. At Operating, although no increase in allowable tensile stress (i.e. zero) can be allowed in precast joints, an increase is allowed from 3 to $7.5\sqrt{f'_c}$ (psi) in reinforced joints and the Principal Tensile Stress at the neutral axis is raised to $4\sqrt{f'_c}$ (psi) to attain the benefit of reduced reliability per Table 8.2.A)
- Use SERVICE I Limit State with live load factor, $\gamma_L = 1.00$ and limit concrete longitudinal flexural compressive stress to values in Table 8.2.A. (Note: $\gamma_L = 1.00$ AASHTO LRFR).
- For STRENGTH II Limit State, use live load factor, $\gamma_L = 1.15$.
- Reduced Dynamic Load Allowance (IM) or live load factor (γ_L) may be considered only to avoid restrictions (Chapter 9).

8.3 Capacity – Strength Limit State

The capacity of a section in transverse and longitudinal flexure may be determined using any of the relevant formulae or methods in the LRFD Code, or AASHTO Guide Specification for Segmental Bridges, including more rigorous analysis techniques involving strain compatibility. The latter should be used in particular where the capacity depends upon a combination of both internal (bonded) and external (unbonded) tendons.

For Load Rating, the capacity should be determined based upon actual rather than specified or assumed material strengths and characteristics. Concrete strength should be found from records or verified by suitable tests. If no data is available, the specified design strength may be assumed, appropriately increased for maturity (Chapter 3).

In particular, for shear or combined shear with torsion, the capacity at the Strength Limit State for segmental bridges should be calculated according to the AASHTO Guide Specification for Segmental Bridges. The “Modified Compression Field Theory” (MCFT) of LRFD may be used as an alternative, but only for structures with continuously bonded reinforcement (e.g. large boxes cast-in-place in cantilever or on falsework).

8.4 Allowable Stress Limits – Service Limit State

Allowable stresses for the Service Limit State are given in *Table 8.2.A*. The intent is to ensure a minimum level of durability for FDOT bridges that avoids the development or propagation of cracks or the potential breach of corrosion protection afforded to post-tensioning tendons. Also, these are recommended for the purpose of designing new bridges.

8.4.1 Longitudinal Tension in Joints

Type “A” Joints with Minimum Bonded Reinforcement

The Service level tensile stress is limited to $3\sqrt{f'_c}$ (psi) for cast-in-place joints with continuous longitudinal mild steel reinforcing for both Inventory and Operating Ratings. (Reference: AASHTO Guide Specification for Segmental Bridges and LRFD Table 5.9.4.2.2-1). Reduced

reliability at Operating conditions is attained by using the number of striped lanes and by allowing an increase in tensile stress to $7.5\sqrt{f_c}$ (psi) (Table 8.2.A).

Type “A” Epoxy Joints with Discontinuous Reinforcement

The Service level tensile stress is limited to zero tension for epoxy joints for both inventory and operating ratings. (Reference: AASHTO Guide Specification for Segmental Bridges and LRFD Table 5.9.4.2.2-1). Reduced reliability at operating conditions is attained by using the number of striped lanes.

Type “B” Dry Joints

Early precast segmental bridges with external tendons and non-epoxy filled, Type-B (dry) joints were designed to zero longitudinal tensile stress. In 1989, a requirement for 200 psi residual compression was introduced with the first edition of the AASHTO Guide Specification for Segmental Bridges. This was subsequently revised in 1998 to 100 psi compression. Service level Inventory Ratings shall be based on a residual compression of 100 psi for dry joints. For Operating Ratings, the limit is zero tension. (Reference: AASHTO Guide Specification for Segmental Bridges and LRFD Table 5.9.4.2.2-1). Reduced reliability at operating conditions is attained by using the number of striped lanes.

8.4.2 Transverse Tensile Stress

For a transversely prestressed deck slab, the allowable flexural stresses for concrete tension are provided in Table 8.2.A: namely, for Inventory $3\sqrt{f_c}$ (psi) and for Operating $6\sqrt{f_c}$ (psi). For Florida, no distinction is made for different environmental conditions. This is deliberate. It is intended to provide a degree of confidence in the durability of the deck.

8.4.3 Principal Tensile Stress – Service Limit State

A check of the principal tensile stress has been introduced to verify the adequacy of webs for longitudinal shear at service. This is to be applied to both for the design of new bridges and Load Rating. The verification, made at the neutral axis, is the recommended minimum prescribed procedure, as follows:

Sections should be considered only at locations greater than “H/2” from the edge of the bearing surface or face of diaphragm, where classical beam theory applies: i.e. away from discontinuity regions. In general, verification at the elevation of the neutral axis may be made without regard to any local transverse flexural stress in the web itself given that in most large, well proportioned boxes the maximum web shear force and local web flexure are mutually exclusive load cases. This is a convenient simplification. However, should the neutral axis lie in a part of the web locally thickened by fillets, then the check should be made at the most critical elevation, taking into account any coexistent longitudinal flexural stress. Also, if the neutral axis (or critical elevation) lies within 1 duct diameter of the top or bottom of an internal, grouted duct, the web width for calculating stresses should be reduced by half the duct diameter.

All stresses at the elevation of the neutral axis due to thermal gradient at Inventory conditions may be disregarded for principal tension checks.

Classical beam theory and Mohr's circle for stress should be used to determine shear and principal tensile stresses. At the Service Limit State, the shear stress and Principal Tensile Stress should be determined at the neutral axis (or critical elevation) under the long-term residual axial force, maximum shear and/or maximum shear force combined with shear from torsion in the highest loaded web, using a live load factor, $\gamma_L = 1.00$. The live load should then be increased in magnitude so that the shear stress in the highest loaded web increases until the Principal Tensile Stress reaches its allowable maximum value (Table 8.2.A).

The Rating Factor at the Service Limit State is the ratio between the live load shear stress required to induce the maximum Principal Tensile Stress to that induced by a live load factor of 1.00.

8.5 Local Details

Important Local Details in concrete segmental bridges are discussed in Chapter 4. Load rating should not be based upon the capacity of local details. However, if a detail shows signs of distress (cracks), a structural evaluation should be performed for the Strength Limit State. Capacity, condition and system factors for local details should be taken according to Chapter 7.

Table 8.2.A - Allowable Stresses for Concrete Bridges

At the Service Limit State after losses	Stress Limit INVENTORY Rating	Stress Limit OPERATING Rating	Source of Criteria
<p>Compression (Longitudinal or Transverse):</p> <ul style="list-style-type: none"> Compressive stress under effective prestress, permanent loads, and transient loads Allowable compressive stress shall be reduced according to AASHTO Guide Specification for Segmental Bridges when slenderness of flange or web is greater than 15 (For both New Design and Load Rating purposes) 	0.60f _c	0.60f _c	LRFD Table 5.9.4.2.1-1 Seg Guide Spec 9.2.2.1 Seg Guide Spec 9.2.2.1
<p>Longitudinal Tensile Stress in Precompressed Tensile Zone: (Intended for Pre and Post-Tensioned Beams and similar construction) For components with bonded prestressing tendons or reinforcement that are subject to not worse than: For (a) an aggressive corrosion environment and (b) moderately aggressive corrosion environment For components with unbonded prestressing tendons</p>	3√f _c psi tension No Tension	7.5√f _c psi tension No Tension	LRFD Table 5.9.4.2.2-1 and FDOT FDOT no distinction for Environ't LRFD Table 5.9.4.2.2-1
<p>Longitudinal Tensile Stress through Joints in Precompressed Tensile Zone: (Intended for Segmental and similar construction)</p> <ul style="list-style-type: none"> Type A joints with minimum bonded auxiliary longitudinal reinforcement sufficient to carry the calculated longitudinal tensile force at a stress of 0.5f_y; for internal and/or external PT (e.g. cast-in-place construction) For (a) an aggressive corrosion environment and (b) moderately aggressive corrosion environment Type A joints without the minimum bonded auxiliary longitudinal reinforcement through the joints; internal and/or external PT (e.g. match-cast epoxy joints or unreinforced cast-in-place closures between precast segments or between spliced girders or similar components.) Type B joints (dry joints - no epoxy); external tendons: 	3√f _c psi tension No Tension 100 psi min comp	7.5√f _c psi tension No Tension No Tension	LRFD Table 5.9.4.2.2-1 Seg Guide Spec 9.2.2.2 FDOT no distinction for Environ't Ditto and FDOT Seg. Rating Criteria Seg Guide Spec 9.2.2.2 FDOT Seg. Rating Criteria
<p>Transverse Tension, Bonded PT:</p> <ul style="list-style-type: none"> Tension in the transverse direction in precompressed tensile zone calculated on basis of uncracked section (i.e. top prestressed slab) For (a) an aggressive corrosion environment and (b) moderately aggressive corrosion environment 	3√f _c psi tension	6√f _c psi tension	Seg Guide Spec 9.2.2.3 LRFD Table 5.9.4.2.2-1 FDOT no distinction for Environ't FDOT Seg. Rating Criteria
<p>Tensile Stress in Other Areas:</p> <ul style="list-style-type: none"> Areas without bonded reinforcement Areas with bonded reinforcement sufficient to carry the tensile force in the concrete calculated on the assumption of an uncracked section is provided at a stress of 0.5f_y (< 30 ksi) 	No tension 6√f _c psi tension	No tension 6√f _c psi tension	Seg Guide Spec 9.2.2.4 LRFD Table 5.9.4.2.2-1 Seg Guide Spec 9.2.2.4 LRFD Table 5.9.4.2.2-1
<p>Principal Tensile Stress at Neutral Axis in Webs (Service III):</p> <ul style="list-style-type: none"> All types of segmental or beam construction with internal and/or external tendons.* 	3√f _c psi tension	4√f _c psi tension	FDOT LRFR Rating Criteria
<p>* Principal tensile stress is calculated for longitudinal stress and maximum shear stress due to shear or combination of shear and torsion, whichever is the greater. For segmental box, check neutral axis. For composite beam, check at neutral axis of beam only and at neutral axis of composite section and take the maximum value. Web width is measured perpendicular to plane of web. For segmental box, it is not necessary to consider coexistent web flexure. Account should be taken of vertical compressive stress from vertical PT bars provided in the web, if any, but not including vertical component of longitudinal draped post-tensioning - the latter should be deducted from shear force due to applied loads. Check section at H/2 from edge of bearing or face of diaphragm, or at end of anchor block transition, whichever is more critical. For the design of a new bridge, a temporary principal tensile stress of 4.5√f_c may be allowed during construction - per AASHTO Seg. Guide Spec. Initial load ratings for new design should be based upon specified concrete strength. Load rating of an existing bridge should be based upon actual concrete strength from construction or subsequent test data.</p>			

Table 8.2.B - Allowable Stresses for Concrete Bridges

At the Service Limit State after losses	Stress Limit INVENTORY Rating	Stress Limit OPERATING Rating	Source of Criteria
<p>Compression (Longitudinal or Transverse):</p> <ul style="list-style-type: none"> Compressive stress under effective prestress, permanent loads, and transient loads Allowable compressive stress shall be reduced according to AASHTO Guide Specification for Segmental Bridges when slenderness of flange or web is greater than 15 (For both New Design and Load Rating purposes) 	0.60f _c	0.60f _c	LRFD Table 5.9.4.2.1-1 Seg Guide Spec 9.2.2.1 Seg Guide Spec 9.2.2.1
<p>Longitudinal Tensile Stress in Precompressed Tensile Zone: (Intended for Pre and Post-Tensioned Beams and similar construction) For components with bonded prestressing tendons or reinforcement that are subject to not worse than: For (a) an aggressive corrosion environment and (b) moderately aggressive corrosion environment For components with unbonded prestressing tendons</p>	3√f _c psi tension No Tension	7.5√f _c psi tension No Tension	LRFD Table 5.9.4.2.2-1 and FDOT FDOT no distinction for Environ't LRFD Table 5.9.4.2.2-1
<p>Longitudinal Tensile Stress through Joints in Precompressed Tensile Zone: (Intended for Segmental and similar construction)</p> <ul style="list-style-type: none"> Type A joints with minimum bonded auxiliary longitudinal reinforcement sufficient to carry the calculated longitudinal tensile force at a stress of 0.5f_y; for internal and/or external PT (e.g. cast-in-place construction) For (a) an aggressive corrosion environment and (b) moderately aggressive corrosion environment Type A joints without the minimum bonded auxiliary longitudinal reinforcement through the joints; internal and/or external PT (e.g. match-cast epoxy joints or unreinforced cast-in-place closures between precast segments or between spliced girders or similar components.) Type B joints (dry joints - no epoxy); external tendons: 	3√f _c psi tension No Tension 100 psi min comp	7.5√f _c psi tension No Tension No Tension	LRFD Table 5.9.4.2.2-1 Seg Guide Spec 9.2.2.2 FDOT no distinction for Environ't Ditto and FDOT Seg. Rating Criteria Seg Guide Spec 9.2.2.2 FDOT Seg. Rating Criteria
<p>Transverse Tension, Bonded PT:</p> <ul style="list-style-type: none"> Tension in the transverse direction in precompressed tensile zone calculated on basis of uncracked section (i.e. top prestressed slab) For (a) an aggressive corrosion environment and (b) moderately aggressive corrosion environment 	3√f _c psi tension	6√f _c psi tension	Seg Guide Spec 9.2.2.3 LRFD Table 5.9.4.2.2-1 FDOT no distinction for Environ't FDOT Seg. Rating Criteria
<p>Tensile Stress in Other Areas:</p> <ul style="list-style-type: none"> Areas without bonded reinforcement Areas with bonded reinforcement sufficient to carry the tensile force in the concrete calculated on the assumption of an uncracked section is provided at a stress of 0.5f_y (< 30 ksi) 	No tension 6√f _c psi tension	No tension 6√f _c psi tension	Seg Guide Spec 9.2.2.4 LRFD Table 5.9.4.2.2-1 Seg Guide Spec 9.2.2.4 LRFD Table 5.9.4.2.2-1
<p>Principal Tensile Stress at Neutral Axis in Webs (Service III):</p> <ul style="list-style-type: none"> All types of segmental or beam construction with internal and/or external tendons.* 	3√f _c psi tension	4√f _c psi tension	FDOT LRFR Rating Criteria
<p>* Principal tensile stress is calculated for longitudinal stress and maximum shear stress due to shear or combination of shear and torsion, whichever is the greater. For segmental box, check neutral axis. For composite beam, check at neutral axis of beam only and at neutral axis of composite section and take the maximum value. Web width is measured perpendicular to plane of web. For segmental box, it is not necessary to consider coexistent web flexure. Account should be taken of vertical compressive stress from vertical PT bars provided in the web, if any, but not including vertical component of longitudinal draped post-tensioning - the latter should be deducted from shear force due to applied loads. Check section at H/2 from edge of bearing or face of diaphragm, or at end of anchor block transition, whichever is more critical. For the design of a new bridge, a temporary principal tensile stress of 4.5√f_c may be allowed during construction - per AASHTO Seg. Guide Spec. Initial load ratings for new design should be based upon specified concrete strength. Load rating of an existing bridge should be based upon actual concrete strength from construction or subsequent test data.</p>			

Table 8.1.A1 - Load Factors for Segmental Bridges

Nomenclature per LRFD:		LRFD Dead and Permanent Loads			LRFD Transient Loads			Inventory	No. of Ld. Lanes = No. of Design Lanes per LRFD	Operating							
		DC	DW	EL including PT sec	FR	TU ^(B) CR SH	TG ^(B)			Design Loads LC #1, #2	No. of Ld. Lanes = No. of Design Lanes per LRFD	No. of Ld. Lanes = No. of Striped Lanes placed for max. effects (A)	Design Loads LC #3, #4	Legal Loads LC #5, #6, #7	FDOT Permit Loads		
							Inv.								Oper.	LC #8, #9, #10	LC #11
LONGITUDINAL	STRENGTH I	$\gamma = 1.25$	1.50	1.00	1.00	0.50	0.00	0.00	X		X	1.35	1.35	-	-		
	STRENGTH II	$\gamma = 1.25$	1.50	1.00	1.00	0.50	0.00	0.00	X		X	-	-	1.35	1.15		
	SERVICE I	$\gamma = 1.00$	1.00	1.00	1.00	1.00	0.50	0.00	X		X	1.00	1.00	1.00	1.00		
	SERVICE III	$\gamma = 1.00$	1.00	1.00	1.00	1.00	0.50	0.00	X		X	1.00	1.00	1.00	1.00		
TRANSVERSE OR LOCAL DETAILS	STRENGTH I	$\gamma = 1.25$	1.50	1.00	n/a	n/a	n/a		X		X	1.35	1.35	-	-		
	STRENGTH II	$\gamma = 1.25$	1.50	1.00	n/a	n/a	n/a		X		X	-	-	1.35	1.15		
	SERVICE I	$\gamma = 1.00$	1.00	1.00	n/a	n/a	n/a		X		X	1.00	1.00	1.00	1.00		

SERVICE I:

Load combination relating to the normal operational use of the bridge with a 55 MPH wind and all loads taken at their nominal values. In the context of concrete segmental bridges, SERVICE I is extended to apply to transverse analysis relating to flexural tension and compression in transversely prestressed (pre- and post-tensioned) deck slabs. In accordance with AASHTO LRFR 6.5.4.2.2.2, the following SERVICE I check of Permit load combinations for reinforced and prestressed concrete components is considered optional. During Permit Load Rating, the stress in reinforcing bars or prestressing steel nearest the extreme tension fiber should not exceed 0.90 of the yield point stress for unfactored loads (i.e. cracked). Absent well defined yield stress for prestressing steels, the following may be assumed:
 Low Relaxation Strand 0.90 fpu
 Stress Relieved Strand and Type 1 High-Strength Bar 0.85 fpu
 Type 2 High Strength Bar 0.80 fpu

SERVICE III:

Load combination for longitudinal analysis relating to tension in prestressed concrete superstructures with the objective of crack control and to principal tension in prestressed concrete webs under normal, unlimited number of, repeat loads (i.e. durability at inventory level). This is attained by limits on tensile stress in Table 8.2.A.

STRENGTH I:

Basic load combination relating to the normal vehicular use of the bridge without wind.

STRENGTH II:

Load combination relating to the use of the bridge by Owner-specified special design vehicles, evaluation permit vehicles, or both without wind.

- (A) Reduced reliability is attained by using only the no. of striped lanes, with a live load factor of 1.00, for Operating SERVICE conditions.
- (B) Temperature (TU & TG) is considered for SERVICE I & III, Inventory Rating.

No. of Live Load Lanes, n		Multiple Presence Factor, m			
		Inventory		Operating	
		Design	Design	Legal	Permit
1	Long. (L)	1.20	1.20	1.00	1.00
	Trans. (T)	1.20	1.00	1.00	1.00
2	L or T	1.00			
3	L or T	0.85			
≥ 4	L or T	0.65			

Table 8.1.A2 - Load Combinations for Segmental Bridges

LOAD COMBINATION (LC) NO.:				Inventory		Operating									
				Design Loads ^(A)		Design Loads ^(A)		Legal Loads ^(A)			FDOT Permit Loads ^(A)				
				#1	#2	#3	#4	#5	#6 ^(B)	#7 ^(B)	Annual Permits, Mixed Traffic ^(C)		Permit Only		
				#1	#2	#3	#4	#5	#6 ^(B)	#7 ^(B)	#8	#9	#10	#11 ^(D)	
LONGITUDINAL	All regions, All spans	HL93 Truck or Tandem plus 0.64 k/lf uniform lane load in All Load Lanes (except Permit Lane for LC #9)	LRFD 3.6.1.2 LRFR 6.4.3	X		X						X			
		FDOT Legal Loads - one SU4, C5 or ST5 in each load lane. For comparison and posting decisions, one HL93 Truck Only per load lane.	FDOT					X							
		One FDOT T160 Permit Vehicle in One Load Lane (For Spans >200', apply 0.2 k/lf uniform lane load in same lane, beyond footprint of permit vehicle for max. effects, or coincident with vehicle for convenience).	FDOT LRFR 6.4.5.4								X				X
	Negative moment regions, All spans	90% of Two HL93 Trucks in same lane spaced at 50 ft minimum plus 90% of 0.64 k/lf uniform lane load	LRFD 3.6.1.3.1		X		X							X	
		Two Type 3-3 Vehicles in same lane times 0.75 separated by 30 ft plus 100% of 0.2 k/lf uniform load	LRFR 6.4.4.2.1							X					
	Pos. mom. & shear, For spans ≤ 200' For spans > 200'	One Type 3-3 Vehicle	LRFR 6.4.4.2.1								X				
One Type 3-3 Vehicle times 0.75 plus 100% of 0.2 k/lf uniform load															
TRANSVERSE OR LOCAL DETAILS	All regions, All spans	HL93 Truck or Tandem Only (one per lane) (no uniform lane load) in All Load Lanes (except Permit Lane for LC #9)	LRFD 3.6.1.2 LRFR 6.4.3	X		X						X			
		FDOT Legal Loads - one SU4, C5 or ST5 in each load lane. For comparison and posting decisions, one HL93 Truck or Tandem Only per load lane.	FDOT					X							
		One FDOT T160 Permit Vehicle in One Load Lane (Triple axle, 3 at 22 kips)	FDOT LRFR 6.4.5.4								X				X

- (A) Apply added Dynamic Load Allowance, IM, of 33% to Vehicle or Axle Loads only. Pedestrian Load per LRFD may be included as necessary. Pedestrian load counts as ONE LANE for determining "m".
- (B) In negative moment regions for all span lengths and in positive moment regions in spans over 200 ft, if the value of the Rating Factor for the AASHTO Limiting Critical Load is less than 1.0, then the basic Rating Factors for all FDOT Legal Loads shall be reduced by multiplying by this value.
- (C) FDOT Permit Load Rating for annual permits and mixed traffic is LC (#8 + #9) or (#8 + #10).
- (D) Single trip, escorted Permit vehicle only on bridge. No other traffic allowed. Permit vehicle may be placed at the most favorable transverse location (e.g. straddling web if necessary), providing that it is strictly enforced.

Chapter 9 – Posting Avoidance

The following methods of posting avoidance are presented in an approximate hierarchy judged to return the greatest benefit for the least cost or effort. This hierarchy is not absolute and may change depending on the particular bridge being load rated.

Under no circumstance shall a posting avoidance technique or restrictions on Permit loads be allowed when load rating a newly designed bridge. Rating Factors for all Design, Legal and Permit Loads at both the Inventory and Operating Levels shall not be less than 1.0.

Posting avoidance techniques require either a Variance or an Exception. A Variance must be approved by the FDOT District Structures Engineer with a copy sent to the State Structures Design Engineer. An Exception requires the approval of the State Structures Design Engineer and may require notification of the Federal Highway Administration.

9.1 Dynamic Load Allowance (IM) for Specific Vehicle Loads (Variance)

For Legal Loads, the Dynamic Load Allowance may be reduced from 1.33 to 1.25. For slow moving (< 10 mph) Permit vehicles, the Dynamic Load Allowance may be eliminated (Reference: LRFR 6.4.5.5).

9.2 Dynamic Load Allowance (IM) for Improved Surface Conditions (Variance)

On the basis of field observations and judgment of the Engineer, and with the concurrence of the District Maintenance Engineer, the Dynamic Load Allowance may be reduced for the following conditions:

- Where there are minor surface imperfections or depressions, the Dynamic Load Allowance (IM) may be reduced to 20%.
- Where there is a smooth riding surface on the bridge and where the transitions from the bridge approaches to the bridge deck across the expansion joints are smooth, the Dynamic Load Allowance (IM) may be reduced to 10%. (An example of this would be a box girder top slab finished by grinding and grooving to remove irregularities with no bumps or steps at expansion joints).

9.3 More Sophisticated Analyses (Variance)

More sophisticated structural analyses (e.g. using finite elements) may be performed in order to establish a more refined load rating. The more sophisticated analyses do not supersede the need for a time-dependent construction analysis for overall longitudinal effects.

9.4 Stiffness of Traffic Barrier (Exception)

Traffic barriers act to longitudinally stiffen box girder cantilever wing slabs. This stiffening increases the longitudinal distribution of wheel loads in the cantilever wings, which reduces the peak transverse moments at the root of the cantilever and increases the load rating of that

section. The presence of median barriers may provide the same benefits at other locations in the top slab.

Barrier stiffness should be considered and appropriately included if necessary. Inclusion of the barriers acting compositely with the box girder section should improve longitudinal load ratings. When barriers are considered in this manner, the difference in the modulus of elasticity of the lower strength barrier concrete relative to that of the segments should be taken into account. The presence of joints in a barrier reduces the overall effective section at the joint to that of the box girder cross section. This may result in a local concentration of longitudinal stress that should be appropriately considered.

Nevertheless, both longitudinal and local transverse load ratings should benefit from reasonable consideration of barrier stiffness.

9.5 Longitudinal Tension in Epoxy Joints (Exception)

The AASHTO Guide Specification for Segmental Bridges and LRFD limit longitudinal tensile stresses to zero at epoxied match-cast joints under Service level conditions. The ability of the epoxy joint to accept tension is not considered. However, in properly prepared epoxy joints the bond usually exceeds the tensile strength of the concrete. Consequently, for posting avoidance, tensile stresses may be accepted as a function of the location and quality of the epoxy joint:

- For top fiber stresses on the roadway surface – no tension is permitted for all load rating calculations.
- For bottom fiber stresses –
 - (a) Allow 200 psi tension at good quality epoxy joints (i.e. no leaks and fully sealed).
 - (b) No tension allowed for poor quality epoxy joints (i.e. leaky or not filled, gaps).

9.6 Transverse Tensile Stress Limit in Top Slab (Exception)

For Legal and Permit loads, the permissible tensile stress in a transversely post-tensioned slab is set at $6.0\sqrt{f_c}$, regardless of the environment (Table 8.2.A). For posting avoidance, up to $7.5\sqrt{f_c}$ may be allowed providing that:

- a) There is sufficient bonded reinforcement to carry the calculated tensile force in the concrete computed on the assumption of an uncracked section at a stress of $0.5f_y$, and,
- b) It is verified by field inspection that there are no cracks in the bridge deck as a consequence of routine or historically heavy vehicular traffic.

9.7 Principal Tensile Stress (Variance)

If the load rating based upon the limiting principal tensile stress at the neutral axis is not satisfactory, the rating factor with regard to principal tension may be taken as 1.00 providing that:

- (a) There is no visible evidence of any representative cracking in the webs.
- (b) The capacity is satisfactory under the required Strength Limit State.

However, if during field inspection, cracks are discovered at or near a critical section where, by calculation, the principal tensile stress is found to be less than the allowable, then further study is recommended to determine the origin of the cracks and their significance to normal use of the structure. A check should be made for any significant out of plane flexural web stress that might lead to a high local tension when combined with other effects. A check should be made for local stress dispersal from applied concentrated forces, such as at certain types or arrangements of anchorage blisters, or eccentric effects. If possible, a check should be made of construction records to determine if there was any change of construction, temporary loads or support reactions that may have induced a significant but temporary local effect.

9.8 Reduced Structural (DC) Dead Load (Exception)

A lower dead load factor may be considered in accordance with the following criteria. Under no circumstance should this load factor be less than 1.10. For the self weight determined by:

- (a) Design Plan or Shop Drawing dimensions and assumed average density for concrete, reinforcement and embedded items: $\gamma_{DC} = 1.25$.
- (b) As-built dimensions, concrete thicknesses, and concrete density determined from construction records, adjusted for weight of embedded reinforcing: $\gamma_{DC} = 1.15$.
- (c) Actual segment weights measured during construction: $\gamma_{DC} = 1.10$.

Cases (b) and (c) may only be used providing that neither additional structural component (DC) nor superimposed dead loads (DW) has been added whose weight cannot be accurately ascertained.

In using either (a) or (b) above, and when it is known that the original design was based on an assumed density for normal concrete and that a check or investigation can verify that a bridge has been constructed with Florida Limerock, then the unit weight may be reduced to 138 lbs per cubic foot for the concrete plus an allowance for the weight of steel.

9.9 Reduced Superimposed (DW) Dead Load (Exception)

The load factor for superimposed dead loads including wearing surface and utilities is normally $\gamma_{DW} = 1.50$. A lower factor may be considered if weights are determined from an accurate survey. Under no circumstance should this be taken less than $\gamma_{DW} = 1.25$ (See LRFR – notes to Table 6-1, October 2003).

9.10 Shear Capacity by AASHTO Guide Specification for Segmental Bridges (Variance)

If the shear capacity at the Strength Limit State, when calculated in accordance with the AASHTO Guide Specification for Segmental Bridges based upon an assumed crack angle of 45° , leads to an unsatisfactory load rating, then the assumed angle of crack may be reconsidered and the capacity recalculated according to the procedure in Appendix B.

9.11 Resal Effect (Variance)

For a continuous cantilever of variable depth, account may be taken of the Resal effect from the curved longitudinal profile of the bottom flange to enhance can the shear capacity, if necessary.

Chapter 10 – Strengthening

Methods for strengthening concrete segmental bridges are presented in this Chapter. Each method calls for adding post-tensioning to the structure. The effects of the additional post-tensioning on the existing bridge should be evaluated prior to strengthening.

10.1 Re-Use of Temporary Post-Tensioning Blisters

Some segmental bridges contain internal blisters (Figure 10.1) on the underside of the top slab and the top of the bottom slab. These would have been used during construction to anchor temporary post-tensioning bars for erection and/or closure of epoxy joints. These blisters may be re-used to add additional permanent post-tensioning.

The temporary post-tensioning bars would typically have been 150 ksi bars with diameters ranging from 1" to 1-3/8" in diameter. The bars would have most likely been stressed to no more than 0.5fpu. The reinforcing details in the Design Plans and Shop Drawing details should be reviewed to determine the amount of new permanent post-tensioning that can be anchored in the blisters.

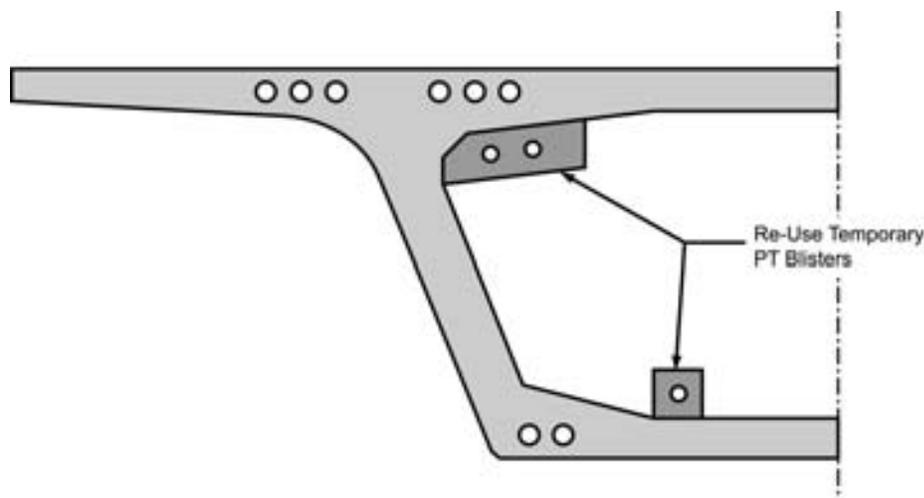


Figure 10.1 – Re-Use of Temporary Post-Tensioning Blisters

10.2 Use of Future Post-Tensioning Provisions

Most segmental structures built since the mid-1980's should have included details for the placement of additional post-tensioning tendons. Typical provisions would have been to leave holes through diaphragms, deviator saddles, or blisters to allow the placement of additional tendons, with anchor blocks to be cast later if required. In some cases, empty ducts for internal tendons may have been installed to allow for additional post-tensioning needs.

In more recent segmental construction (1990 to the present), provisions for future post-tensioning were formalized. It is possible that not only ducts but also anchorages may have been cast into diaphragms or blisters. The Design Plans may also include conceptual details for

additional (future) post-tensioning. In this case, the plans should be reviewed and post-tensioning added in accordance with the details and based on appropriate analysis.

10.3 Adding Longitudinal Post-Tensioning

Longitudinal post-tensioning may be added even in concrete segmental bridges built without provisions for future tendons. Taking appropriate care, ducts may be cored through diaphragms and anchor blisters cast in-situ.

Draped, external post-tensioning tendons provide an efficient layout for continuous structures. This efficiency may be offset, however, by the cost of new ribs needed to provide the desired tendon profile. Adding straight tendons, on the other hand, will be less effective in compressing deficient areas, but may require only the addition of end anchor blocks (Figure 10.2).

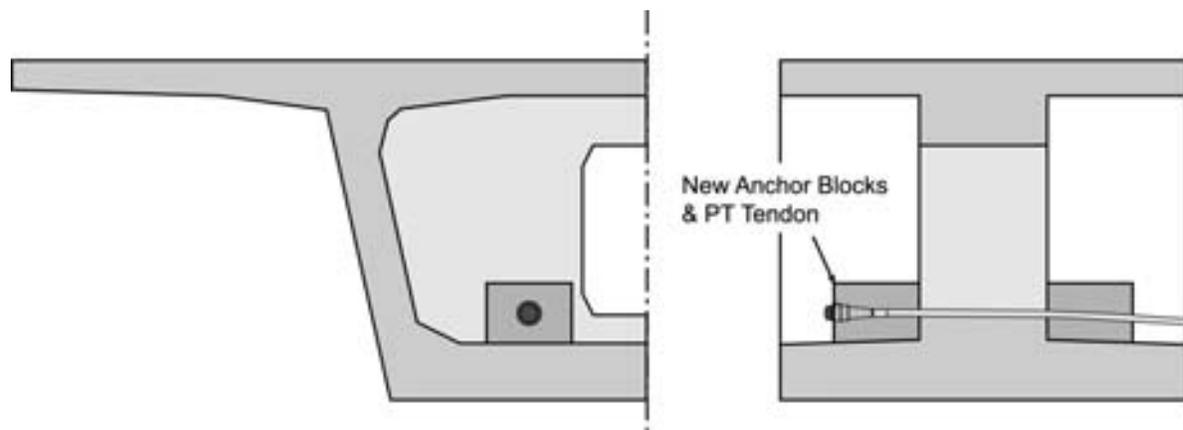


Figure 10.2 – Adding Longitudinal Post-Tensioning

10.4 Transverse Strengthening

Transverse strengthening of the top slab of a concrete segmental bridge may be accomplished by adding transverse tendons. The additional tendons are placed in troughs cut across the full width of the bridge deck. Troughs can be effectively cut by hydro-blasting. Additional concrete at the ends of the cantilever wings is removed to provide space to place the anchorages for the new post-tensioning tendons. After placing and stressing the additional post-tensioning, the trough is filled with an appropriate, high strength, non-shrink material.

Repairs of this nature have been successfully completed in the United States. The scope of these repairs, however, has been small, providing only local strengthening at the location of damaged tendons.

10.5 Strengthening of Local Details

Strengthening of local details, such as dapped hinges and diaphragms, typically includes adding external post-tensioning to offset unexpected cracking. Anchor blocks must be properly added to the bridge to allow the additional post-tensioning to effectively strengthen the local detail.

Appendix A – Development of System Factors

The System Factor (ϕ_s), given in Table 7.2 is related to degree of redundancy in the total structural system. In LRFR, bridge redundancy is defined as the capability of a structural system to carry loads after damage or failure of one or more of its members. LRFR recognizes that structural members of a bridge do not behave independently, but interact with one another to form one structural system.

Current LRFR System Factors do not adequately address the characteristic behavior of post-tensioned segmental box girder construction in three areas:

- Longitudinal Continuity – The research upon which LRFR is based (NCHRP 406), considered longitudinal bridge continuity. However, longitudinal continuity that makes a structure statically indeterminate and increases the overall redundancy is not acknowledged in LRFR.
- Continuum of the Closed Box Girder – LRFR primarily considers bridge superstructures comprised of multiple girders with composite deck slabs. LRFR does not give consideration to enhanced behavior of box girders with regard to torsion, and therefore the participation of the entire cross section in resisting loads.
- Multiple Tendon Paths and Internal Redundancy – LRFR introduces the concept of Internal Redundancy provided by multiple, independent component load paths (an example of this in LRFR is single welds versus bolted connections). This concept of internal redundancy is not extended in similar fashion to multiple tendon paths typical of segmental construction.

The effect of these features of segmental box girder bridges is discussed in the following sections.

A.1 Longitudinal Continuity

System Factors in NCHRP 406 were developed considering both simple span and continuous span bridges. NCHRP 406 concluded that there was no beneficial enhancement to basic system factors unless full section capacity of the superstructure over continuous supports could be developed and maintained. In practice, this can only be achieved (in steel structures) by using compact sections at these locations. LRFR recognizes that, in reality, bridges of this nature are seldom if ever built, as the steel webs in the negative moment regions would be too thick to be cost-effective. As a result, the variation in NCHRP 406 System Factors in relation to continuity was not included in the development of LRFR.

Continuous, post-tensioned box girder bridges, on the other hand, can effectively develop full negative moment capacity over interior supports without significant changes to the basic box girder cross section. Several research projects, including those at the University of Texas (Ref. 2.1) demonstrated the post-elastic response of post-tensioned segmental construction. Review of these research projects indicate that provided that sufficient post-tensioning is provided, the webs and bottom slab will effectively develop plastic hinging and redistribution of loads.

The work in NCHRP recognized a 7% benefit resulting from the continuity of beams with compact sections. Given other aspects of segmental bridges, such as transverse continuum of the box and considerations from research (Ref 7.1), a value of 10% is considered suitable for the influence of continuity for current purposes (Ref 7.2).

A.2 Continuum of the Box Girder

A large box section is a closed-continuum, and as a result the number of webs cannot be considered the same as the number of girders in a beam bridge. Consider, for example, a wide three web box. In torsion, the section behaves almost the same as of a box comprised only of the perimeter webs - the center web carries no torsion. However, in shear, a higher proportion of load is usually carried by the center web. In flexure, the entire section contributes. Hence, in terms of "system-redundancy" the whole of a box section participates but in different ways depending upon the load effect. This is very different to I-girder bridges.

For a variety of reasons, the majority of segmental bridges comprise two-web (single cell) boxes. Although some three-web boxes have been built, few, if any, four-web boxes have been built as a complete precast section. In most cases, wide bridges are made of two separate two-web boxes connected by a longitudinal deck closure strip. Complex difficulties with casting tolerances, geometry and camber control usually rule-out connecting both top and bottom slabs of a pair of boxes and usually, there is little structural need to do so. Also, casting machines are simpler and more economical for two-web boxes. Usually, it is more efficient and economical to precast a wide two-web (single-cell) box for a wide section than introduce a third web. Much depends upon the overall proportions, segment weights, delivery sizes, transport, and erection systems. Hence most segmental bridges are made of a two-web, single-cell box.

In the context of the number of webs and integrity of the structure, the most serious externally inflicted damage to a segmental bridge would likely be that from a dump truck, crane boom or vessel superstructure punching a hole in a web or bottom slab. Even so, given the usual structural form of the box comprising a wide bottom slab contiguous with the webs, the same accidental impact energy will not likely destroy the outside web and flange compared to the relative ease with which some edge I-girders have otherwise been removed from overpasses.

Nevertheless, segmental box structures with three or more webs already exist (e.g. Acosta, Port of Miami) and more could be built. So there is some need to address redundancy in terms of multiple webs. However, in order to have meaningful application, this must be expressed in terms of the "System Factor". From studies, experience and review of research, it is estimated that if three or more webs are used, then the overall system factor could be increased by 0.10 (i.e. about 10%). In addition, the benefit of an extra web lies more in providing a location for additional tendons – as considered in "Multiple Tendon Paths" – below.

A.3 Multiple Tendon Paths

A significant feature of segmental bridges is that they are post-tensioned with multiple tendons.

Post-tensioning may comprise internal or external tendons or both. There are a multiple number of tendons passing through any given section. In this respect, multiple tendons can be envisioned as a multiple number of load paths – somewhat analogous to multiple rivets or bolts in a steel connection. This fundamental aspect of post-tensioned structures is not addressed at

all in LRFR. However, it is a key feature not only of segmental bridges but also of many other types of prestressed post-tensioned construction.

Moreover, the durability and structural behavior of individual tendons affects the overall integrity and capacity of the structure. Durability is a condition (such as corrosion) to be addressed through the "Condition Factor".

The structural behavior of an individual tendon comprises not only the longitudinal tendon force but also flexural and shear effects induced by tendon eccentricity, radial and lateral deviation forces. The overall effect of multiple tendons is the summation of all individual effects. Indeed, segmental bridges are analyzed in this manner. So the contribution of "Multiple Tendon Paths" (MTP) ought to be properly accounted for in the overall redundancy of the system.

The concept of "Multiple Tendon Paths" (MTP) has already been introduced in Florida in the context of Design, Construction and Maintenance not only for segmental but for all types of post-tensioned structures under policies implemented through FDOT "New Directions in Florida Post-Tensioned Bridges". In this Rating Manual, MTP is addressed under the "System Factor".

A.4 Justification for System Factor Values

The following justification is offered with respect to particular values selected for System Factors in Table 7.2. Certain values in this table are pivotal in that they were determined by analyses, from experience or performance of existing bridges. Other values were then interpolated. The pivotal values were established based upon engineering studies, knowledge of many existing bridges and experience including key considerations from background to codes and research.

A. *Flexure – precast and cast-in-place balanced cantilever, end spans or interior span with hinge, two webs, one and two tendons per web*

For flexural conditions, the pivotal values of 0.85 (one tendon per web) and 1.00 (two tendons per web) for internal tendons in precast segmental cantilever bridges stem from examination and knowledge of existing cantilever bridges – some in Florida, but also others elsewhere – in which only one tendon per web passes through the bottom (tension face) of some end-span segments. This borders upon "condition" but is not strictly a function of it.

After studies of existing bridges and adoption of "Multiple Tendon Paths" as a policy by FDOT in "New Directions for Florida Post-Tensioned Bridges" (2001), it was realized that providing at least two tendons per web and applying a "System Factor" of 1.00 to the section capacity calculation led to a comfortably conservative result. Although a larger system factor might be justified, 1.00 would certainly be a minimum for such situations. However, the same could not be argued for only one (intact) tendon per web on the tension face – so a value of 0.85 was chosen. This decision involves engineering judgment but it is based upon sound observations and experience.

(Note that the above refers to the few sections in the end span of a continuous cantilever – not to the entire end span. It does not refer to a span-by-span bridge with full span length tendons. In fact, there is no known case of a span-by-span bridge with only one tendon per web.)

B. *Flexure – precast span-by-span, interior span, two webs, one and two tendons per web*

In addition to the above (A), there are some first generation span-by-span bridges in the Florida Keys with only two external tendons per web in some spans. The fact that these bridges have been performing satisfactorily provides some confidence to adopt 1.00 as the lowest possible "System factor" relating to MTP when applied to continuous, interior spans only, using external tendons. There is far less confidence and comfort in providing only one external tendon per web, even if, theoretically, this were sufficient to satisfy structural design requirements. In fact, there is no known case of only one external tendon per web. This consideration led to the insertion of "n/a" in Table 7.2 (meaning "not applicable" or "not allowed") and the choice of 0.85 as the "bottom line" if ever such a case were found to exist.

C. Flexure – all segmental types, two-webs, 4 or more tendons per web, influence of longitudinal continuity

Based upon the approach in NCHRP 406 and studies of its application to segmental bridges, System Factors for the design of simple and continuous segmental bridges with two-webs could be 1.10 and 1.20, respectively; with potentially greater values pending the results of further studies and research. Considering the need to address "Multiple Tendon Paths" and that, under the "New Directions for Florida Post-Tensioned Bridges", a minimum of four external tendons per web is recommended for span-by-span construction. Therefore, it is considered appropriate to apply the 1.10 and 1.20 values to the case of simple and continuous spans of span-by-span bridges. Because precast segmental cantilever bridges usually contain more than 4 cantilever tendons per web, these same values can be safely applied to cantilever construction.

D. Longitudinal continuity – simple and continuous spans and plastic hinges

Longitudinal continuity is recognized through the simple concept of the number of plastic hinges needed to form a collapse mechanism: that is to say;

- 1 hinge for a simple span or statically determinate structure;
- 2 hinges for the end span of a continuous unit, and
- 3 for an interior span or monolithic portal frame

The same logic applies whether a bridge is built using span-by-span or balanced cantilever construction. The significance of the distinction between simple and continuous spans really refers to the difference between statically determinate or indeterminate structures. Hence a "statically determinate" cantilever bridge (i.e. two cantilevers with a suspended "drop-in" span) is treated like a simple span bridge. Hence, for an interior span or statically indeterminate structure, the System Factor is set at 1.20. For an end span or statically determinate bridge, the System Factor is 1.10 for two-web boxes (providing that there are at least four tendons per web).

E. Flexure with Three or more webs – upper limit 1.30

For longitudinal flexure, an enhancement of 0.10 may be added to the System Factors given for boxes with two webs to account for redundancy afforded by three or more webs. Most segmental bridges have two webs. Recognition of the enhancement afforded by a third web is appropriate as it ties into the concepts of multi-tendon paths and a closed continuum. However, this increase 0.10 applies to the third web only; it does not increase any further for more webs. Consequently,

For two webs:

Base value for concrete I-Girder:	1.00
Closed continuum two webs (> than I-girders with diaphragms)	0.10
Difference in reliability β_f for concrete versus steel	<u>0.10</u>
Base for two-web box:	1.20
Multiple Tendon Paths (4 or more tendons per web)	0.10
Longitudinal continuity (structural redundancy)	<u>0.10</u>
Possible upper base limit for 2-web box, sub-total:	1.40
Add a third Web or more,	<u>0.10</u>
Possible upper base limit for 3 or more webs, total:	1.50

Segmental concrete bridges are beyond the scope of the AASHTO LRFR provisions which were not intended for bridges as redundant as closed cell box girders with multiple load and tendon paths. So a system factor in excess of 1.20 is justified for appropriate situations. However, at this time, pending further experience or research, it is proposed to limit this to 1.20 for 2 webs and a limit of 1.30 should be used for 3 or more webs. Both of these limits apply to conditions with at least 4 tendons per web and reduce for fewer tendons.

F. Flexure – Intermediate numbers of tendons per web

Intermediate conditions (e.g. to account for 3 tendons per web) were selected by interpolation.

G. Shear and Shear Torsion

For longitudinal shear and shear-torsion, the system factor is taken as 1.00 for the Strength Limit State for all circumstances at this time. (This might be reconsidered after further studies or research).

H. Transverse Flexure

With transverse post-tensioning to the deck slab, a segmental box is simply a prestressed concrete structure - so for transverse flexure a system factor of 1.00 is appropriate, regardless of the spacing of tendons - likewise for the local detail of a transverse beam support to an expansion joint device, although the possibility of having only one tendon in the effective section is recognized by reducing the system factor to 0.90.

For punching shear of a slab, the system factor is taken as 1.00.

I. Local Details

For local details involving local shear and/or strut and tie action or analysis, a system factor of 1.00 is considered appropriate where two or more tendons provide capacity to the detail - but a reduced factor of 0.90 where only one tendon provides that capacity.

Note that as regards rating for any particular load, the capacity of a tendon deviator, or an anchor block or diaphragm designed to restrain or anchor one or more tendons, is unaffected by

live load. Live load only affects dapped hinges, expansion joint support beams and diaphragms designed that transfer live loads to bearings.

References:

7.1 McGregor, Kreger and Breen, Research Report 365-3F, "Strength and Ductility of Externally Post-tensioned 3-Span Box Girder Bridge Model", University of Texas at Austin, January 1989.

7.2 Dr. Dennis Mertz: "Discussion of System Factors for Segmentally Constructed Concrete Bridges" and related correspondence, University of Delaware, October 2002.

Appendix B

B.1 General Load-Rating Equation

The Rating Factor, RF, is determined according to the general formula:

$$RF = \frac{C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_{EL}(P + EL) - \gamma_{FR}FR - \gamma_{CR}(TU + CR + SH) - \gamma_{TG}TG}{\gamma_L(LL + IM)} \quad (\text{Eqn B.1})$$

For definition of terms, refer to Chapter 8.

B.2 Load Rating Based Upon Allowable Principal Tensile Stress at Service Limit State

The following process may be used to derive a load rating based upon the allowable limit for the principal tensile stress taken at the neutral axis only. Appropriate section properties should be used. In general, it is sufficient to use gross section properties. However, where a section is near the end of a span or close to a support (at H/2, where H = overall depth), and the effects of discontinuities or shear lag may be significant, section properties may be determined based upon the reduced section in accordance with LRFD.

The neutral axis is adopted to simplify the check. This is not an exhaustive analysis. No attempt is made to determine the likely onset of shear cracks initiated by flexure in a manner that would occur in the span, further from the support. Also, if the neutral axis lies in a portion of the web thickened by a local fillet, then the more critical elevation should be checked. Account should then be taken of the corresponding flexural stress; or it may be discounted if its effect is small. Appropriate allowance should be made for reduced web width if internal ducts are present.

At the service level, the shear stress due to permanent loads is a combination of vertical shear and torsional shear in the critical web and is given by:

$$v_D = (V_{DC} + V_{DW} + V_{EL} + V_{CR} + V_{SH} - V_{PT}) \left(\frac{Q}{I \cdot b_{WT}} \right) \quad (\text{Eqn B.2})$$

- Q = first moment of area of section above neutral axis.
- I = second moment of area of full or effective section (inertia).
- b_{WT} = total web width perpendicular to median slope of web, at neutral axis.
(Refer to Table 8.1.A1 for load combination multipliers)
- V_{PT} = vertical component of final post-tensioning after losses from draped tendons.
It is assumed to act to assist shear capacity in above equation B.2.

And, the torsional shear stress from permanent loads:

$$t_D = (T_{DC} + T_{DW} + T_{EL} + T_{CR} + T_{SH} - T_{PT}) \left(\frac{1}{2A_0 \cdot b_{W1}} \right) \quad (\text{Eqn B.3})$$

Where:

- T = Torsion: the subscript "PT" denotes prestress.
(T_{PT} is assumed to assist capacity in above equation B.3).
- A_0 = Torsional Area (i.e. total area enclosed by the median line of the exterior webs and slabs of the box section).
- b_{w1} = width of one outer web perpendicular to median slope of web, at neutral axis.

Note: In most straight or curved structures with internal tendons, T_{PT} is likely to be small. However, it may be significant in curved structures with external, deviated, tendons. It may add to or subtract from other load effects. Correct interpretation of direction of action is essential.

Therefore, the shear stress due to permanent loads at service is given by:

$$V_{PERM} = V_D + t_D \quad (\text{Eqn B.4})$$

The axial compression stress (taken as positive) at the neutral axis:

$$\sigma_x = \frac{P_{XEFF}}{A_C} \quad (\text{Eqn B.5})$$

Where P_{XEFF} = effective prestress after losses, and A_C = area of concrete section (gross or effective, as determined for the section and location) and the subscript "X" denotes the longitudinal direction.

The effective vertical compression stress in the web under consideration:

$$\sigma_y = \frac{P_{YEFF}}{b_w} \quad (\text{Eqn B.6})$$

Where P_{YEFF} = effective vertical prestress force per linear foot of web, after losses, and the subscript "Y" denotes the vertical direction.

Any vertical prestress effect from longitudinally draped tendons is accounted for by reducing the applied shear force (i.e. V_{PT}). Although there might be some vertical prestress stress in the web from this effect, it is discounted in determining σ_y . Only the actual vertical prestress in the web itself, due to vertical PT bars for example, is to be counted in determining P_{YEFF} and σ_y . In most bridges with no vertical PT bars in the webs, $\sigma_y = 0$.

Under the condition of permanent loads only, the center of Mohr's circle for stress is given by:

$$c = \frac{(\sigma_x + \sigma_y)}{2} \quad (\text{for } \sigma_x > \sigma_y) \quad (\text{Eqn B.7})$$

In most cases the vertical compressive stress, $\sigma_y = 0$, as illustrated in Figure B.1.

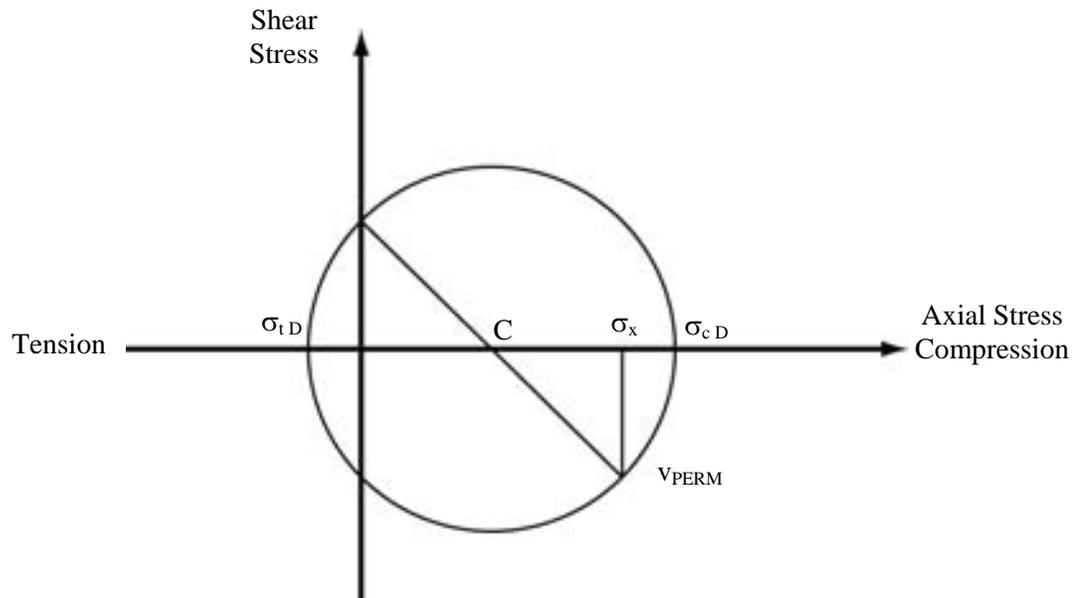


Figure B.1 – Mohr's Circle for All Permanent Loads (shown for case where $\sigma_y = 0$)

And the radius of the circle under permanent loads is given by:

$$r_p = \sqrt{\left[(c - \sigma_y)^2 + (v_{PERM})^2 \right]} \quad (\text{Eqn B.8})$$

The principal tensile stress (tension negative) under permanent loads is given by:

$$\sigma_{tD} = (c - r_p) \quad (\text{Eqn B.9})$$

Where the subscript “t” signifies tension and “D” permanent load effects.

The shear stress due to unfactored vertical live load and torsion is given by:

$$v_{LL} = \left(\frac{V_{LL} \cdot Q}{I \cdot b_{wt}} \right) \quad (\text{Eqn B.10})$$

Torsional shear stress due to live load:

$$t_{LL} = \left(\frac{T_{LL}}{2A_o b_{w1}} \right) \quad (\text{Eqn B.11})$$

So the increase in shear stress due to unfactored live load:

$$v_{1LL} = v_{LL} + t_{LL} \quad (\text{Eqn B.12})$$

When live load is applied, there is no net change to any of the permanent stresses due to dead load and post-tensioning at the neutral axis. The only increase is that of the shear and torsion-shear stress due to the live load (Figure B.2).

The principal tensile stress under this condition is given by:

$$\sigma_{tD+L} = c - \sqrt{c^2 + (v_{PERM} + v_{1LL})^2} \quad (\text{Eqn B.13})$$

The subscript, "D+L", signifies the effect due to permanent load and the unfactored live load.

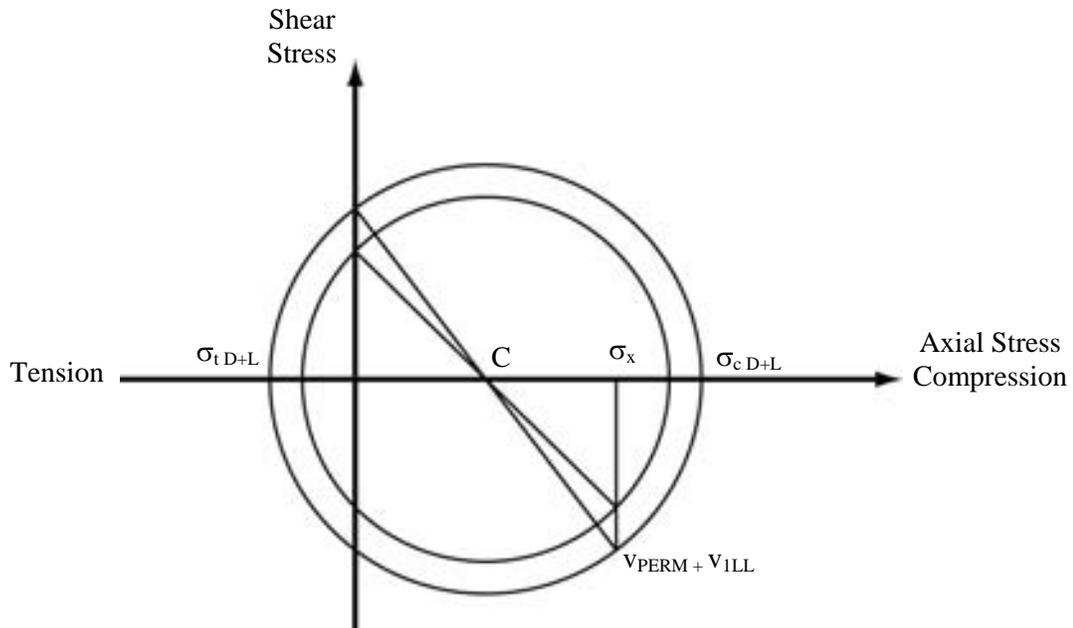


Figure B.2 – Mohr's Circle for All Permanent Loads + 1 Lane of Live Load

Now, the maximum principal tensile stress is limited to:

$$\sigma_{TMAX} = \phi_{SERVICE} \cdot n \sqrt{f'_c} \quad (\text{Eqn B.14})$$

Where; $\phi_{SERVICE} = 1.0$ and the multiple “n” is taken from Table 8.2.

Therefore, the maximum radius of the Mohr’s circle corresponding to the maximum principal tensile stress limit is:

$$R_{MAX} = (c + \sigma_{TMAX}) = \left(\left(\frac{\sigma_x + \sigma_y}{2} \right) \right) + (\phi_{SERVICE} \cdot n \sqrt{f'_c}) \quad (\text{Eqn B.15})$$

Therefore the maximum shear stress capacity available for resisting all loads is given by:

$$v_{CAP} = v_{LLMAX} + v_{PERM} = \sqrt{(R_{MAX}^2 - (\sigma_x - c)^2)} \quad (\text{Eqn B.16})$$

And the Rating Factor for the applied live load corresponding to the maximum principal tensile stress is given by:

$$RF = \frac{(v_{CAP} - v_{PERM})}{v_{ILL}} \quad (\text{Eqn B.17})$$

It is entirely appropriate to express this rating factor in terms of shear stresses because it relates directly to the vector sum in Mohr’s circle of axial stress and shear stress that induces the limiting principal tensile stress, based on conditions at the neutral axis (Figure B.3)

It should be pointed out that our primary interest here is tension, in order to limit the potential for cracking at service or to determine an appropriate angle of crack at which tensile cracking could occur as a part of the exercise to determine the ultimate section strength. Although there is a complementary principal compression stress, it is of no interest to our immediate purpose. Furthermore, in most practical situations, it is considered unlikely that a principal compression would approach an allowable compression stress limit.

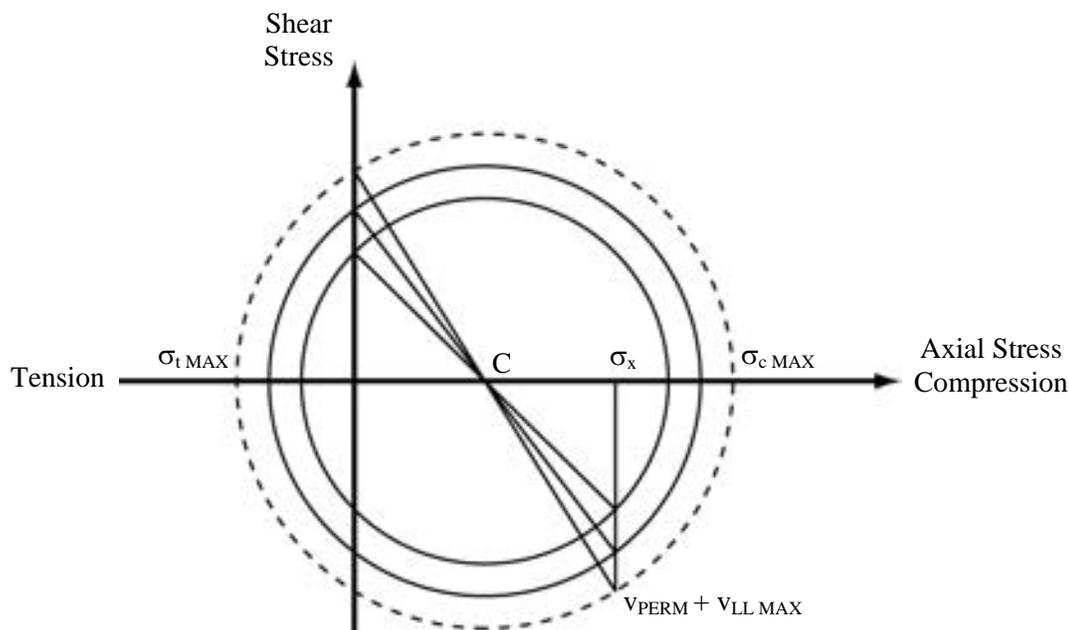


Figure B.3 – Mohr's Circle for All Permanent Loads + Max Live Load (for $\sigma_y = 0$)

B.3 Load Rating for Shear and Torsion at Strength Limit State

In the transition from Load Factor Design to Load and Resistance Factor Design, the underlying formulae and procedures for calculating the ultimate capacity of a prestressed section in flexure remain essentially the same. There is really only a shift in emphasis on the corresponding load and resistance factors. However, for shear this is not so.

In LRFD, the procedure for calculating shear capacity has changed completely from the previous approach in the Standard Specification and AASHTO Guide Specification for Segmental Bridges. However, in some segmental boxes, the associated longitudinal strain may be relatively small and only nominal shear reinforcement is required. Consequently, it might lie outside the range of LRFD Table 5.8.3.4.2-1.

For Load Rating, the shear capacity may be determined in accordance with the method in LRFD or in accordance with the established formulae in the AASHTO Guide Specification for Segmental Bridges (Second Edition, 1999), which has proven to be satisfactory based upon its use on many bridges, as adapted in the following method.

The analysis of a segmental box must combine shear and torsion for the same load conditions. Torsion may arise from eccentric loads and from the effects of plan curvature. Consequently, it is usually necessary to consider many possible load combinations with various spans with different numbers and eccentricities of live load lanes all with corresponding multi-presence

factors. It is a significant bookkeeping exercise particularly if the worst case is not intuitively obvious. Simplification is necessary and appropriate, particularly for load rating.

The simplified procedure is intended for the Strength Limit State, although the process utilizes Mohr's circle calculations similar to those for the Principal Tensile Stress.

The process uses beam theory section analysis to combine the (factored) shear force from vertical loads with that force induced by the corresponding (factored) torsional load. In one web of a two-web box the torsional shear stress acts in the same direction as the shear stress due to vertical loads and in the opposite direction in the other web. It is possible to determine an equivalent shear force in the critical web by adding torsion to the corresponding vertical shear force in that web. The web may then be designed or load rated for the combined shear.

The procedure is suitable for most well-proportioned segmental boxes found in Florida where the torsional shear stresses are relatively small compared to vertical shear stresses, where the critical shear occurs in a region at or near a support (approximately within twice the box depth) and where shear is not initiated by a flexural tension crack within the span. (The latter is unlikely for most situations). The procedure may not be applicable to a very narrow box with long cantilever wings, where torsion is high relative to shear and transverse flexure might locally affect the web. Likewise, the procedure may not apply to a box with a significantly unsymmetrical cross section. However, it does apply to both straight and curved bridges meeting the above qualifications.

At the Strength Limit State, (ULS*), the total shear force on the section is given by:

$$V = \gamma_{DW} V_{DW} + \gamma_{DC} V_{DC} + \gamma_{EL} V_{EL} + \gamma_{CR} (V_{CR} + V_{SH}) + \gamma_L V_{(LL+IM)} - \gamma_{EL} (V_{PT} + V_{RESAL}) \quad (\text{Eqn B.18})$$

(*Hereinafter, the abbreviation and subscript "ULS" is used to signify when a calculation or parameter is based on or refers to factored (ultimate) loads at the "Strength Limit State" to distinguish it from similar situations for the "Service Limit State").

And the corresponding total torsion is given by:

$$T = \gamma_{DW} T_{DW} + \gamma_{DC} T_{DC} + \gamma_{EL} T_{EL} + \gamma_{CR} (T_{CR} + T_{SH}) + \gamma_L T_{(LL+IM)} - \gamma_{EL} T_{PT} \quad (\text{Eqn B.19})$$

V_{PT} = the vertical component of the longitudinal post-tensioning force from draped or deviated tendons (not from vertical PT bars in the web) after losses.

V_{RESAL} = vertical component of longitudinal force in the bottom compression flange for the flexural conditions corresponding to the shear or shear plus torsion load case under consideration. This only applies to sections with a significant longitudinal sloping soffit – i.e. deep, variable depth boxes. For boxes of constant depth (i.e. horizontal soffit) this effect is zero. In most cases, it is conservative to disregard the Resal effect – but it may offer some relief in marginal situations. For simplicity, the Resal effect is not considered in the following.

It is necessary to address both cases - boxes with two webs and those with more than two and consider the worst effect where the torsion adds to the shear.

For a typical segmental box the torsional shear stress in the exterior webs is given by:

$$t = \frac{T}{2A_0 b_{w1}} \quad (\text{Eqn B.20})$$

Where:

- T = Total torsion on whole section from all applied dead, live and post-tensioning forces at factored (ultimate) load conditions.
- A₀ = Torsional Area (i.e., total area enclosed by the median line of the exterior webs and slabs of the box section).
- b_{w1} = web width (perpendicular to median slope of web). Some judgment must be exercised for a web of varying thickness – perhaps to use a mean value or check at the thinnest portion if it coincides with the maximum shear stress due to vertical loads. Usually b_{w1} is taken at the neutral axis.

The magnitude of the torsional shear force in the critical web (V_{TW1}) is given by multiplying by the thickness and (slope) length, S₁, of that web, thus:

$$V_{TW1} = \frac{T \cdot b_{w1} \cdot S_1}{2A_0 \cdot b_{w1}} = \frac{T \cdot S_1}{2A_0} \quad (\text{Eqn B.21})$$

Where S₁ = length of median of web measured on slope from mid-depth of top slab to mid-depth of bottom slab. Again, for a web of varying thickness, some judgment is necessary to account for maximum effects. However, usually effects are taken at the neutral axis.

So, for a box section, the combined effective shear force due to vertical shear force and torsion on the (one) critical web is given by:

$$V_1 = (G \cdot V) + V_{TW1} \quad (\text{Eqn B.22})$$

Where “G” is a constant representing the proportion of total shear carried by one web. G = ½ for a symmetrical box with two-webs.

For torsion, the same approach can be applied to a box with three or more webs. The torsional shear force in the outer web is determined from Equation B.20, where A₀ remains the total area enclosed by the median perimeter line taken through the top and bottom slabs and outer webs.

However, the vertical shear force on each individual web depends upon the relative stiffness characteristics of the box section – it may not necessarily be carried equally by each web. The shear may be distributed to the webs according to the flexural stiffness (inertia) of each individual web relative to that of the whole or effective section. For four or more webs, a more rigorous analysis may be necessary (e.g., finite element, space frame or grillage) in order to establish the proportion of total shear to distribute to each web.

The shear stress at any point within the depth of the section is determined using the classical

beam formula:

$$v = \frac{VQ}{Ib_w} \quad (\text{Eqn B.23})$$

Where Q = first moment of area of the effective portion of the cross section above the elevation location of interest in the web taken, about the neutral axis. This formula can be applied to any point within the depth of the section to determine the shear stress distribution. However, it is normal practice to consider shear conditions at the neutral axis. This is usually the location of the maximum shear stress, and the longitudinal stress is simply that due to the axial prestress involving no flexural component.

If the web is thinner at a particular elevation away from the neutral axis, then the shear stress should be checked at that elevation and compared to that at the neutral axis. If it is significantly greater than that at the neutral axis, a check of shear stress distribution should be made. If necessary, for a first estimate and for simplicity, the peak value of shear stress may be applied in the following formulae (which apply to conditions at the neutral axis), even if that peak is not actually at the neutral axis.

The shear stress due to the combination of shear force and torsional-shear force at the neutral axis of the (one) critical web is given by:

$$v_1 = \frac{V_1 Q_1}{I_1 b_{w1}} \quad (\text{Eqn B.24})$$

Where V_1 = value from Equation B.22 above and Q_1 , I_1 and b_{w1} are taken for the cross section properties for the one web under consideration (at the level of the neutral axis). Alternatively, for webs of equal width, this may be taken as:

$$v_1 = \frac{2V_1 Q}{I b_w} \text{ for 2 webs} \quad (\text{Eqn B.25})$$

Where the terms Q , I and b_w are taken for the whole (or effective) section.

Thus far we know the shear force on the worst affected web, V_1 , (equation B.22) due to the combined action of shear and torsion on the whole section and the corresponding shear stress, v_1 , (equation B.24) combined with torsional stress t_1 (equation B.20) all taken at factored conditions.

The effective longitudinal prestress (compression positive), after all losses, at the neutral axis is simply;

$$\sigma_x = \frac{P_{x\text{EFF}}}{A_C} \quad (\text{Eqn B.26})$$

Where $P_{x\text{EFF}}$ = total effective prestress force after losses, and A_C = area of whole (or effective) concrete section and the subscript "X" denotes the longitudinal direction.

The effective vertical compression stress in the web under consideration;

$$\sigma_y = \frac{P_{yEFF}}{b_{w1}} \quad (\text{Eqn B.27})$$

Where P_{yEFF} = effective vertical prestress force in each web per unit length of bridge, after losses, due only to vertical PT (normally bars) in the web itself; the subscript “Y” denotes the vertical direction.

Note that although there is a vertical prestress effect from longitudinally draped tendons, it is accounted for in Equation B.18 by reducing the applied load effect. Although there may be some vertical prestress stress in the web from this effect, it is discounted in determining σ_y . Only actual vertical prestress in the web itself, due to vertical PT bars for example, is to be counted in determining P_{yEFF} and σ_y .

The principal tensile stress is determined using Mohr’s circle, thus:

The center of circle, c, is given by:

$$c = \frac{(\sigma_x + \sigma_y)}{2} \quad (\text{for } \sigma_x > \sigma_y) \quad (\text{Eqn B.28})$$

And the radius of the circle is given by:

$$r_p = \sqrt{[(c - \sigma_y)^2 + (v_1)^2]} \quad (\text{Eqn B.29})$$

Note that in this equation B.29, “ v_1 ” is the effective shear stress due to both shear force and torsional shear force in the worst web.**

The principal tensile stress (tension negative) is given by:

$$\sigma_{TULS} = (c - r) \quad (\text{Eqn B.30})$$

And the angle of the potential crack due to the principal tension is given by:

$$\theta_{CULS} = 0.5 \cdot \sin^{-1} \left(\frac{v_1}{r} \right) \quad (\text{Eqn B.31})$$

Note: It may be preferable to keep separate account of the shear stress due to shear force and the shear stress due to torsion-shear – in which case equation B.29 would become:

$$r_p = \sqrt{[(c - \sigma_v)^2 + (v + t)^2]} \quad (\text{Eqn B.29a})$$

Where v = shear stress due to vertical shear force only and t = shear stress due to torsional shear stress only from equation B.20.

Then, the principal tensile stress (tension negative) is given by:

$$\sigma_{TULS} = (c - r) \quad (\text{Eqn B.30a})$$

And the angle of the potential crack due to the principal tension is given by:

$$\theta_{CULS} = 0.5 \cdot \sin^{-1} \left(\frac{v + t}{r} \right) \quad (\text{Eqn B.31a})$$

At the Strength Limit State, where the appropriate Load Factors from Table 8.1 are applied, the resulting principal tension (Equation B.30) represents that stress which would exist at the neutral axis if the loads were increased to ultimate conditions.

The resulting angle of the tension crack, θ_{CULS} , given by Equation B.31, is important because it determines the horizontal length over which sufficient vertical prestress and reinforcement must be provided in order to resist the ultimate loads. (In the absence of any longitudinal or vertical prestress, this angle would be 45° – as for a simple reinforced section.)

The longitudinal length to be protected by vertical prestress and reinforcement is given by:

$$L_C = \frac{d_{CT}}{\tan \theta_{CULS}} \quad (\text{Eqn B.32})$$

Where d_{CT} = distance between the center of compression force and center of tensile force, but need not be taken as less than $0.80H$, where H = overall depth of the section.

The strength provided by the (vertical) reinforcement (in one web) is given by:

$$V_{S1} = (A_{ps} f_{py} + A_s f_y)(L_C) \quad (\text{Eqn B.33})$$

Where,

- A_{ps} = area of vertical prestress steel in web per linear foot
- f_{py} = yield strength of vertical prestress steel in web ($0.90 F_{pu}$)
- A_s = area of mild reinforcing steel per linear foot

f_y = yield strength of reinforcing steel (60 ksi)

However, the capacity attributed to the reinforcement and any vertical web prestress (PT bars) together should not be taken greater than $8\sqrt{f'_c}b_{w1}d$

For the one critical web, the strength attributed to the concrete itself is given by:

$$V_{C1} = 2k\sqrt{f'_c} (b_{w1}d) \quad (\text{Eqn B.34})$$

Where,

$$k = \sqrt{\left(1 + \frac{f_{pc}}{2\sqrt{f'_c}}\right)} \quad k \leq 2.0 \quad (\text{Eqn B.35})$$

And,

f_{pc} = axial stress due to post-tensioning at the neutral axis of the section after all losses, creep, shrinkage and any other permanent restraint forces (= P_{EFF} / A_C)

The total ultimate capacity of the (one) critical web is given by:

$$C_1 = \phi_C \phi_S \phi (V_{S1} + V_{C1}) \quad \phi_C \phi_S \geq 0.85 \quad (\text{Eqn B.36})$$

Where:

ϕ = strength reduction factor for shear
 ϕ_C = condition factor
 ϕ_S = system factor

For design purposes, C_1 (from equation B.36) $\geq V_1$ (from equation B.22)

However, in order to perform a load rating, it is necessary to know the magnitude of all shear and torsion shear effects from the permanent loads. Eliminating live load effects from equations B.18 and B.19, we find the total factored permanent shear force on the full section is:

$$V_{PERM} = \gamma_{DW} V_{DW} + \gamma_{DC} V_{DC} - \gamma_{EL} (V_P + V_{RESAL}) \quad (\text{Eqn B.37})$$

And the corresponding total factored permanent torsion is:

$$T_{PERM} = \gamma_{DW} T_{DW} + \gamma_{DC} T_{DC} - \gamma_{EL} T_P \quad (\text{Eqn B.38})$$

(Note, in above; $V_P = V_{PT} - V_{SECONDARY}$ and $T_P = T_{PT} - T_{SECONDARY}$ as necessary).

Then, applying the same process using equations B.20, B.21 and B.22 to give the combined

effective shear force due to both vertical shear and torsion on the (one) critical web for the permanent force effects only, gives:

$$V_{1PERM} = (G \cdot V_{PERM}) + V_{TW1PERM} \quad (\text{Eqn B.39})$$

Again, applying the same process using equations B.20, B.21 and B.22 but to the factored live load effect only for the rating vehicle concerned gives the combined effective shear force due to live load in the (one) critical web:

$$V_{1LL+IM} = (G \cdot V_{LL+IM}) + V_{TW1LL+IM} \quad (\text{Eqn B.40})$$

The Rating Factor based upon factored load conditions is given by:

$$RF = \frac{C_1 - V_{1PERM}}{\gamma_L (V_{LL+IM})} \quad (\text{Eqn B.41})$$

Because the magnitude of the principal tension and the angle of inclination of the potential crack at ultimate depend upon the magnitude of the applied live load, the above calculation must clearly be performed for each type of load for which a Load Rating is required.

This is a major reason for limiting this exercise to sections near supports and simplifying the check to that at the elevation of the neutral axis.

Appendix C – Examples

A recently completed precast segmental balanced cantilever bridge (Figure C.1) illustrates the key features for calculation of Rating Factors according to the procedures in this Manual.

The bridge is a single cell (two-web) trapezoidal box girder with 13 continuous spans ranging from 96 to 264 feet, resting on 14 piers. The depth of the box varies from 7'-9" in the end-spans and at midspan to 13'-0" at the main span piers. Typical precast cantilever segments are 15'-9" long and the pier segments containing diaphragms are 9'-10" long. The overall width of the deck is 48'-4" and provides for a sidewalk of 5'-0", a shoulder of 10'-0", two travel lanes of 12'-0", a median shoulder of 6'-0", two traffic barriers and a pedestrian barrier.

The top deck slab cantilevers approximately 10'-7" beyond the outside face of each web and varies in thickness from 8-5/8" at the tip and in the center portion to 15-3/4" at the webs. The width of the bottom slab is maintained at a constant 18'-0", so the slope of the web varies with the varying depth of the section. The web thickness is constant at 13-3/4". A fillet radius of approximately 2'-3" transitions the outside web face to the soffit of the wing cantilever. For most of each span, the bottom slab has a constant thickness of 9" but increases to 16" over the two segments on each side of the pier in each of the longest spans.

Longitudinal, internal cantilever tendons of 15*0.6" strands are provided over each web. These tendons generally anchor in the end faces of the web-top slab joint at each segment. Temporary longitudinal post-tensioning bars were used for segment erection and closing of epoxy joints. Internal tendons provide continuity through the bottom slab in the mid-span region and anchor in blisters or ribs on the inside of the box. In the completed structure, the cantilever and continuity tendons are supplemented by up to three external tendons per web. These tendons typically anchor in pier segment diaphragms and pass through a pair of deviator ribs within each span.

The bridge was erected in balanced cantilever commencing with relative short, constant depth segments over the first interior pier and connected to the end-span segments at a cast-in-place joint. Cantilever erection continued over each successive pier in turn and each cantilever was connected to the previously completed spans by a closure approximately at mid-span. During cantilever erection, out-of-balance effects were carried by temporary supports adjacent to each pier. Segments were erected using a barge mounted crane.

Initially, the spans were restrained from longitudinal movement using temporary connections to the very first end-span pier. Longitudinal fixity was later transferred to bearings permanently fixed against longitudinal movement over the 7th, 8th and 9th piers. Longitudinal jacking at some of the span closures was used to compensate for movements and forces induced by the temporary restraint and sequence of erection.

Recent inspection shows that the match-cast joints are well sealed with epoxy. The internal tendons do not contain any special duct connectors in the joint faces. There are multiple shear keys in the match-cast web faces and an additional four individual shear keys in the top slab and one in the bottom slab.

The bridge was designed according to the AASHTO Standard (LFD) Specification (16th Edition) and the AASHTO Guide Specification for Segmental Bridges (1st Edition).

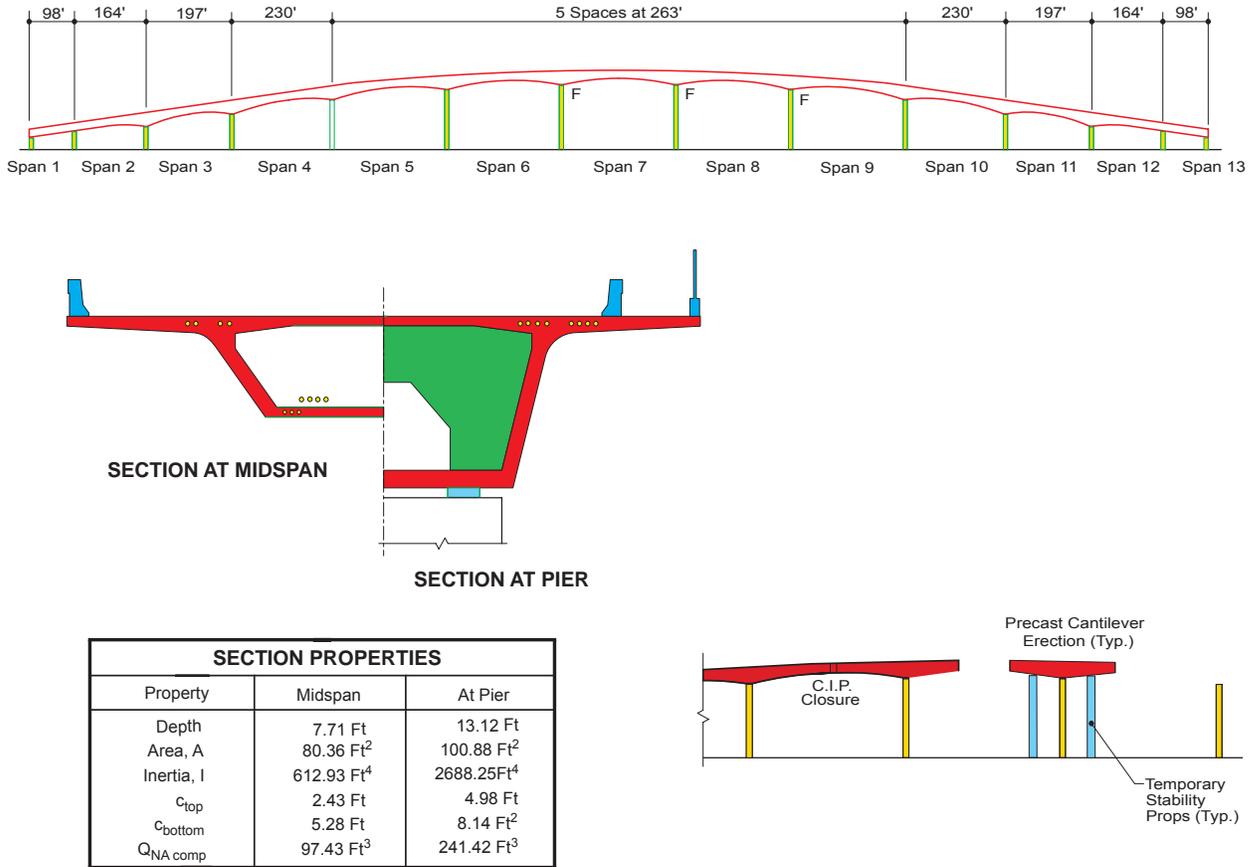


Figure C1: Precast Segmental Balanced Cantilever

The six key ratings required are (Chapter 8):

At the Service Limit State:

- Longitudinal Box Girder Flexure
- Transverse Top Slab Flexure
- Principle Web Tension

At the Strength Limit State:

- Longitudinal Box Girder Flexure
- Transverse Top Slab Flexure
- Web Shear

To determine the permanent structural forces and effects in the superstructure, a complete time-dependent, construction-phase, structural analysis, computer model was assembled in the same sequence and time-related activities as the actual structure. The two-dimensional, plane frame structural model was defined with a node at each joint in the superstructure and at each bearing location. The pier characteristics were modeled for piers 7, 8 and 9 which have longitudinally fixed, but rotationally free, bearings in the final structure. The model accounted for:

- Self weight of the precast segments
- Introduction and release of temporary stability towers during erection
- Weight of forms, concrete in closures, closure devices and their removal
- Sequential erection and post-tensioning of each segment in cantilever
- Longitudinal post-tensioning forces and all losses due to elastic shortening, anchor set, friction, wobble, steel relaxation, creep and shrinkage from the time of casting the segments, throughout construction to long term service
- The change from unbonded to bonded properties for the internal longitudinal tendons upon grouting
- Installation of external longitudinal post-tensioning and all associated loss in the external tendons
- Additional prestress loss in all previous stages of post-tensioning due to further elastic shortening, friction, wobble, relaxation, creep and shrinkage, etc. from subsequent tendons
- Longitudinal restraint at the first pier followed by release of that restraint and longitudinal jacking between cantilevers 6 and 7 then between cantilevers 8 and 9
- Application of superimposed dead loads, such as traffic barriers
- Concrete material properties according to Chapter 5, for strength, maturity, creep and shrinkage characteristics

All permanent internal forces and stress conditions were accumulated from the time of casting, throughout construction to long term service (i.e. after approximately 10 years) according to the actual sequence of casting and construction operations.

Longitudinal live loads were analyzed using the same structural model in its completed configuration. Maximum effects were determined by moving each load longitudinally.

For transverse flexure of the top slab, moments were determined using influence surfaces according to Homberg. The results were combined with the analysis of two transverse plane-frame structural models to determine the effect of the release of slab fixity moments and distribute flexural effects around the box section. The two models were each of a 1-foot unit length, for two cross sections: a deep section approximately at a distance of the depth of the structure away from the pier diaphragm and a shallow section, representing that at mid-span. The deep section model has greater transverse flexibility than the shallow section and was used to determine the maximum positive transverse moments at the mid-point of the top slab. The shallow section, being stiffer, provided the maximum negative moments in the top slab at the interior face of the web. Moments on the cantilever wing, were obtained directly from an appropriate influence surface chart.

At the Service Limit State, longitudinal and transverse flexural stresses were calculated according to classical beam theory. Principal tensile stresses were calculated at the elevation of the neutral axis using Mohr's circle for stress.

Flexural strength capacities were calculated according to formulae in AASHTO LRFD. Strain compatibility analysis was used to account for the combined effect of both internal (bonded) and external (unbonded) tendons. Shear capacities were determined by the AASHTO Guide Specification for Segmental Bridges (2nd Edition, 1999).

The following examples have been chosen deliberately to illustrate various nuances that affect the choice of key parameters, such as, the use of the number of striped lanes, associated load factors (Table 8.1.A1), load combinations (Table 8.1.A2) and allowable stresses (Table 8.2.A). Summarizing the examples:

C.1 Longitudinal Service Flexure at Inventory: The controlling section and condition is that of no-tension in the bottom fiber. This is the design load case with one half of the non-linear thermal gradient combined with the full design live load.

C.2. Transverse Top Slab Service Flexure: The controlling section is at the tangent point of the outer fillet radius at the root of the cantilever wing. This is the maximum design load case for a single lane of HL93 Tandem on the wing. For this Inventory condition, the multi-presence factor, $m = 1.20$; $IM = 1.33$; there is no uniform lane load and for Service I, $\gamma_{LL} = 1.00$.

C.3 Principal Web Tension at Service: The Operating Rating is determined for the T160 Permit Vehicle alone and in combination with mixed (HL93) traffic. As this is a service level check at Operating conditions, the number of live load lanes is the number of striped lanes, i.e. 2. (The use of the number of striped lanes is to attain the benefits of reduce target reliability at service conditions). The full HL93 live load is applied in the lane adjacent to the Permit Load. Longitudinally, an additional 0.20 kip/ ft is applied in the Permit lane in spans over 200 feet. For the lone T160 vehicle, the multi-presence factor, (m) is capped at 1.00 because it is a specific, defined, live load. This check is made using Service III with $\gamma_{LL} = 1.00$ for an allowable principal tensile stress of $4\sqrt{f_c}$ (psi). The controlling section is that in span 7 near pier 8.

C.4 Longitudinal Flexural Strength: The Operating Rating is determined for the limiting AASHTO Type 3-3 Legal Load. As this is a strength condition, the full number of live load lanes is used, i.e. 3, with the corresponding multi-presence factor $m = 0.85$. The example illustrates the selection of condition and system factors. For Legal Loads, $\gamma_{LL} = 1.35$. No thermal gradient effects need be considered at either Strength or Operating conditions.

C.5 Transverse Flexural Strength: The Operating Rating is determined for the T160 Permit vehicle alone and in combination with mixed (HL93) traffic. As a required strength check, there is no limitation on the number of live load lanes or the location of the loads. However, studies of this and other live load cases revealed that the critical section is that for positive moment flexure at the mid-span section between the webs, for which two lanes are applied. The HL93 Tandem with no uniform lane load component is applied in the lane adjacent to the T160 triple-axle unit. The multi-presence factor $m = 1.00$ and for mixed traffic, $\gamma_{LL} = 1.35$.

C.6 Web Shear Strength: The Operating Rating is determined for the FDOT SU4 Legal Load. As a strength check, all design load lanes must be loaded (i.e. 3), so $m = 0.85$ and $\gamma_{LL} = 1.35$. Condition and system factors are selected according to Chapter 7.

Example C.1: Longitudinal Service Flexure at Inventory

Inventory Rating of variable depth precast segmental balanced cantilever at Service Limit State. The controlling section is at mid-span of the fifth span of the continuous 13-spans.

Capacity for Segmental with Type-A (epoxy) joints:

$$\begin{aligned} \text{Tension} &= 0 \text{ ksf} \\ \text{Compression} &= 0.6 f'c = 475.2 \text{ ksf} \end{aligned}$$

Sum of all permanent loads (i.e. DC, DW, EL, CR, SH, and PT) from time-dependent longitudinal structural analysis model:

$$\begin{aligned} \text{Top} &= 103.5 \text{ ksf compression} \\ \text{Bottom} &= 132.0 \text{ ksf compression} \end{aligned}$$

Stress from full positive non-linear thermal gradient:

$$\begin{aligned} \text{Top} &= 142.8 \text{ ksf compression} \\ \text{Bottom} &= 43.6 \text{ ksf tension} \end{aligned}$$

Critical Stress from one lane HL93 Truck only without impact:

$$\begin{aligned} \text{Top} &= 9.75 \text{ ksf compression} \\ \text{Bottom} &= 21.15 \text{ ksf tension} \end{aligned}$$

Critical Stress from one lane HL93 Uniform Lane load only without impact:

$$\begin{aligned} \text{Top} &= 12.18 \text{ ksf compression} \\ \text{Bottom} &= 26.42 \text{ ksf tension} \end{aligned}$$

For Inventory Condition number of lanes loaded = number of design lanes = 3.

As a result, $m = 0.85$ per Table 8.1.A1 (i.e. LRFD Table 3.6.1.1.2-1)

Rating Factor Calculation:

Top Fiber: Compression (Service I)
(Load Combination #1 Table 8.1.A2)

$$RF = \frac{C - \sigma_{Perm.} - \gamma_{TG} \times \sigma_{TG}}{\#_{Lanes} \times m \times \gamma_{LL} \times (\sigma_{Truck} \times IM + \sigma_{Lane})}$$

$$\therefore RF = \frac{475.2 \text{ ksf} - 103.5 \text{ ksf} - 0.5 \times 142.8 \text{ ksf}}{3 \times 0.85 \times 1.00 \times (9.75 \text{ ksf} \times 1.33 + 12.18 \text{ ksf})} = 4.68$$

Compression in the top is likely never to be of concern in this type of bridge. It is shown only for thoroughness of the exercise.

Bottom Fiber: Tension (Service III)
(Load Combination #1 Table 8.1.A2)

$$RF = \frac{C - \sigma_{Perm.} - \gamma_{TG} \times \sigma_{TG}}{\#_{Lanes} \times m \times \gamma_{LL} \times (\sigma_{Truck} \times IM + \sigma_{Lane})}$$

$$\therefore RF = \frac{0 \text{ ksf} - 132.0 \text{ ksf} - 0.5 \times -43.6 \text{ ksf}}{3 \times 0.85 \times 0.80 \times (-21.15 \text{ ksf} \times 1.33 + -26.42 \text{ ksf})} = 0.99^*$$

* As the bridge was designed according to LFD 16th edition and AASHTO Segmental Guide Specifications, not LRFD, this rating is considered acceptable.

Example C.2: Transverse Top Slab Service Flexure

Transversely, the deck is post-tensioned with profiled tendons of 4*0.6" strands (area of one strand = 0.217 square inches) spaced at intervals that provide an effective area of 0.275 square inches per longitudinal foot of deck.

In this case, transversely, four sections are considered of potential interest with regard to tensile stress:

- a) The bottom fiber at mid-span of the interior slab between the webs, 8.66" deep
- b) The top fiber at the section at the interior web face, 15.74" deep
- c) The top fiber at a section within the outer web face fillet radius, 17.50" deep
- d) The top fiber at the section at the tangent point of the outer fillet radius, 15.04" deep

For transverse flexure at Inventory Rating, the last (d) was the most critical for SERVICE I tension under a load of a single HL93 Tandem with no uniform land load applied to the cantilever wing.

For (d) given deck slab is 15.04" deep; $A_c = 1.2533 \text{ ft}^2$ per ft; $S_{xx} = 0.2618 \text{ ft}^3$ per ft.

At this section (d), the permanent dead load moments are:

Dead load of slab	6.343 ft kip
Dead load of barrier	<u>3.780 ft kip</u>
Sum	= 10.123 ft kip

Therefore, dead load flexural stress, $f_{DC} = 10.123 / 0.2618 = \pm 38.67 \text{ ksf}$ (top in tension)

The transverse post-tensioning at this section has an eccentricity of 0.305 ft.

The final post-tensioning force after all losses is estimated at 38.6 kip / strand. This translates to an effective prestressing force,

$$P = 38.6 * (0.275 / 0.217) = 48.87 \text{ kips / ft.}$$

The final stresses due to post-tensioning only are:

$$\begin{aligned} \text{Top fiber stress, } \sigma_{P \text{ top}} &= \frac{48.87}{1.253} + \frac{48.87 * 0.305}{0.2618} = + 95.93 \text{ ksf (comp)} \\ \text{Bottom fiber stress, } \sigma_{P \text{ btm}} &= 39.00 - 56.93 = - 17.93 \text{ (tension)} \end{aligned}$$

For Inventory conditions, the allowable top tensile stress = $3\sqrt{f_c} = - 32.04 \text{ ksf}$ for concrete where $f_c = 5,500 \text{ psi}$.

For the top fiber, the available stress "capacity" = $C = 95.93 - (-32.04) = 127.97 \text{ ksf (comp)}$

A Homberg chart for a variable depth cantilever wing is used to determine the live load moments (Figure C.2). The HL93 Tandem wheel loads are placed as patch loads on an area given by the tire print according to LRFD 3.6.1.2.5. The load is dispersed at 45 degrees to a

depth of 4". This is a little less than half the depth of the thinnest part of the slab and is considered sufficient for this case. Integration of the volume under the influence surface, for each of the four patch wheel loads, provides the flexural moment per unit length at the root of the cantilever. The live load is moved until the maximum moment is determined for the tandem in the location shown in Figure C.2. The controlling live load moment for the HL 93 Tandem at Inventory, without dynamic allowance and without the multiple-presence factor is:

$$M_{Tandem} = 14.49 \text{ ft kip per ft.}$$

This induces a live load stress = $14.49 / 0.2618 = \pm 55.35 \text{ ksf}$ (top in tension)

For a single lane of live load at Inventory, $m = 1.20$ (Table 8.1.A1); and $IM = 1.33$

For SERVICE I, live load tensile stress check, Load Factor $\gamma_{LL} = 1.00$

Rating Factor:

$$RF = \frac{C - \sigma_{Perm.}}{\#_{Lanes} \times m \times \gamma_{LL} \times (\sigma_{Tandem} \times IM)}$$

$$\therefore RF = \frac{127.97 - 38.67}{1 \times 1.20 \times 1.00 \times (55.35 \times 1.33)} = 1.01$$

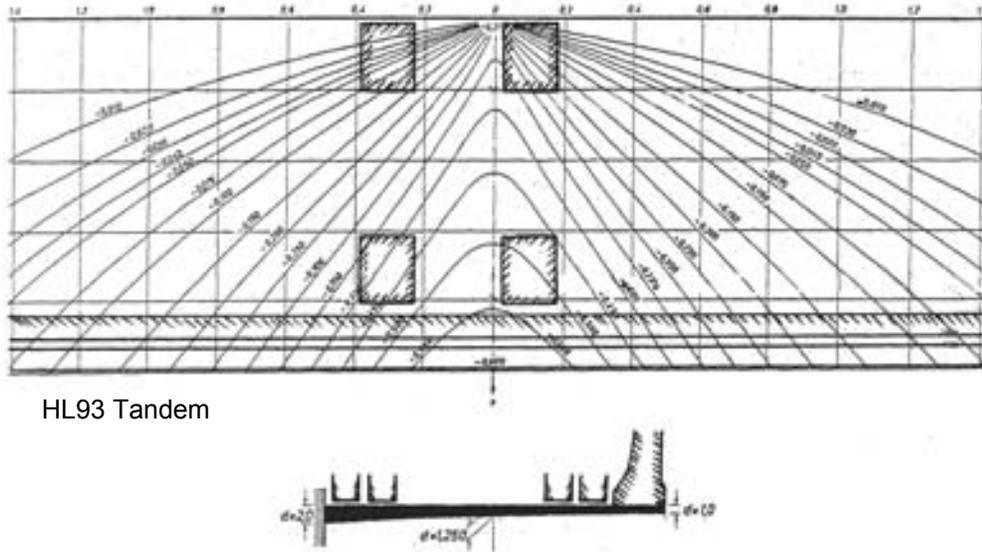


Figure C.2 - Homberg Chart for Variable Depth Cantilever Slab

Example C.3: Principal Web Tension at Service

Consider the Operating condition for a T160 Permit Truck for which the critical section is at the end of the 7th interior span, at half the depth of the section from the diaphragm at pier 8:

A principal tensile stress limit of $4\sqrt{f'c}$ leads to an allowable shear stress at the neutral axis of
 $(V_{Perm} + V_{LL_Max}) = 89.72 \text{ ksf}$ (for detailed explanation refer to Appendix B.2).

Sum of all permanent loads (i.e. DC, DW, EL, CR, SH and PT) from time-dependent computer analysis leads to a permanent shear stress: $V_{Perm} = 61.73 \text{ ksf}$.

Shear stress from one lane T160 Permit Vehicle only without impact is 5.48 ksf.

Shear stress from one lane of 200 lb/ft Uniform Lane load only without impact is 1.26 ksf.

Shear stress from one lane HL93 Truck only without impact is 2.65 ksf.

Shear stress from one lane HL93 Uniform Lane load only without impact is 4.05 ksf.

For permit vehicle alone $m = 1.00$.

For Permit and HL93 combined at Operating, number of lanes loaded = number of striped = 2, for which, $m = 1.00$ per Table 8.1.A1.

Rating Factor Calculation:

Permit Alone: Principal Tension (Service III, but using $\gamma_{LL} = 1.00$ Table 8.1.A1)
(Load Combination #11 Table 8.1.A2)

$$RF = \frac{(V_{Perm} + V_{LL_Max}) - V_{Perm}}{\#_{Lanes} \times m \times \gamma_{LL} \times (V_{T160} \times IM + V_{Lane_200})}$$

$$\therefore RF = \frac{89.72 \text{ ksf} - 61.73 \text{ ksf}}{1 \times 1.00 \times 1.00 \times (5.48 \text{ ksf} \times 1.33 + 1.26 \text{ ksf})} = 3.27$$

Permit & Design Load: Principal Tension (Service III, but using $\gamma_{LL} = 1.00$ Table 8.1.A1)
(Load Combination #8 & #9 Table 8.1.A2)

$$RF = \frac{(V_{Perm} + V_{LL_Max}) - V_{Perm}}{m \times \gamma_{LL} \times ((V_{T160} + V_{Truck} \times (\#_{Lanes} - 1)) \times IM + V_{Lane_200} + V_{Lane_640} \times (\#_{Lanes} - 1))}$$

$$\therefore RF = \frac{89.72 \text{ ksf} - 61.73 \text{ ksf}}{1.00 \times 1.00 \times (5.48 \text{ ksf} + 2.65 \text{ ksf} \times (2 - 1)) \times 1.33 + 1.26 \text{ ksf} + 4.05 \text{ ksf} \times (2 - 1)} = 1.74$$

Example C.4: Longitudinal Flexural Strength

For the precast balanced cantilever bridge, the critical section for flexural capacity for many of the live load cases is that for positive flexure in the first end span. The following is the Operating Rating for AASHTO 3-3 Vehicle at this location.

The positive flexural strength capacity calculated, using strain compatibility to combine the behavior of both bonded and unbonded tendons at the critical section, is:

$$C = M_n = 23,385 \text{ k-ft}$$

Factored permanent loads from time-dependent computer analysis:

From Table 8.1.A1

DC	5,409 k-ft	$\gamma = 1.25$	= 6,761 k-ft
DW	526 k-ft	$\gamma = 1.50$	= 789 k-ft
EL	2,042 k-ft	$\gamma = 1.00$	= 2,042 k-ft
CR	100 k-ft	$\gamma = 0.50$	= 50 k-ft
SH	-3 k-ft	$\gamma = 0.50$	= -1 k-ft
			<u>9,641 k-ft</u>

Live Load Combination #7 Table 8.1.A2:

Critical Moment from AASHTO 3-3 Vehicle = 1,092 k-ft

Since in positive moment region with span length less than 200' no uniform lane load is applied.

$$\gamma_{LL} = 1.35 \text{ per Table 8.1.A1}$$

For Strength 1 number of lanes = number of design lanes = 3 and $m = 0.85$, per Table 8.1.A1 (i.e. LRFD Table 3.6.1.1.2-1)

$$\phi = 0.90 \text{ per LRFD Table 5.5.4.2.2-1}$$

$$\phi_C = 1.0 \text{ per Section 7.1.2 illustrative examples}$$

$$\phi_S = 1.0 \text{ per Table 7.2 for 2 tendons per web}$$

Rating Factor Calculation:
(Strength I)

$$RF = \frac{\phi \times \phi_S \times \phi_C \times M_n - M_{Perm.}}{\#_{Lanes} \times m \times \gamma_{LL} \times \sigma_{Truck} \times IM}$$

$$\therefore RF = \frac{0.9 * 1.0 * 1.0 * 23,385 \text{ k-ft} - 9,641 \text{ k-ft}}{3 * 0.85 * 1.35 * 1,092 \text{ k-ft} * 1.33} = 2.28$$

Example C.5: Transverse Flexural Strength

In this case, the transverse Operating Rating is required for the T160 Permit vehicle in Mixed Traffic.

Live loads from the wheels of the 3*22 kip triple axle unit of the T160 are placed in one live load lane and the 2*25 kip HL93 Tandem is placed in the adjacent live load lane. Although this is an Operating Rating, the transverse location of the live loads is unrestricted; likewise for the longitudinal location of each axle unit. The axle units are moved both transversely and longitudinally until the mid-span positive moment is maximized. It is usual to make several trial applications to determine the optimum locations for maximum effect. In this case, it is with the wheels placed on the Homberg Influence Surface for positive flexure as illustrated in Figure C.3. This chart is for a fixed slab with a depth varying by a ratio of 2:1.

Wheel loads are dispersed at 45 degrees to the neutral axis of the slab. For convenience and simplicity, this is taken as a depth of about 4 inches in this case. The wheel loads are increased by 1.33 for Dynamic Load Allowance (IM) and applied as patch loads. The positive moment is the summation of the volume under the influence surface by the applied load for each patch. Since two lanes are loaded, the multiple presence factor (m) = 1.00 (Table 8.1.A1). The resulting, unfactored, positive moment in this case is:

$$M_{LL+} = 6.40 \text{ ft-kip per ft.}$$

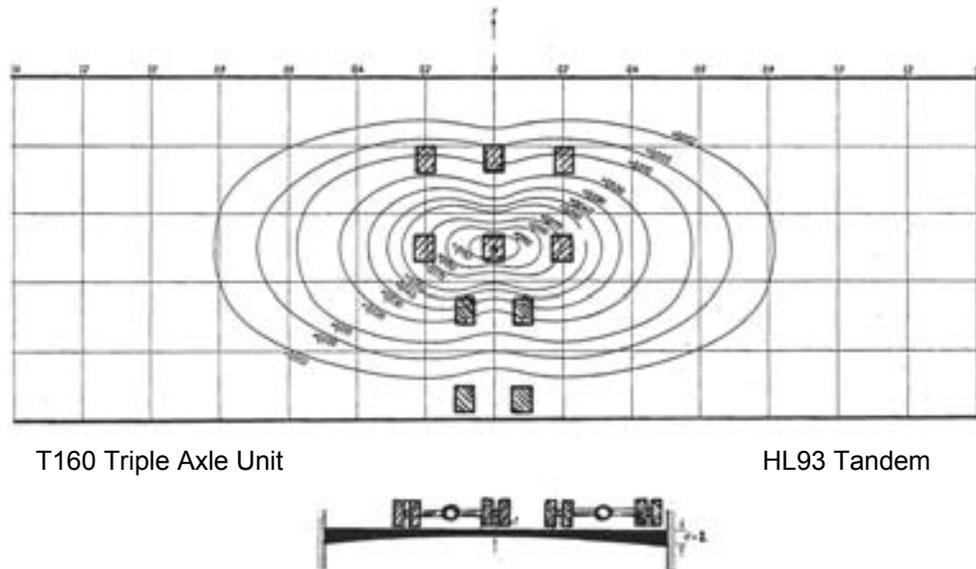


Figure C.3 - Homberg Chart for Positive Moment at Mid-Span of Slab

It is also necessary to determine the corresponding negative moment at each fixed edge of the slab. This is done using the Homberg Influence Surface for negative (fixed edge) moment illustrated in Figure C.4.

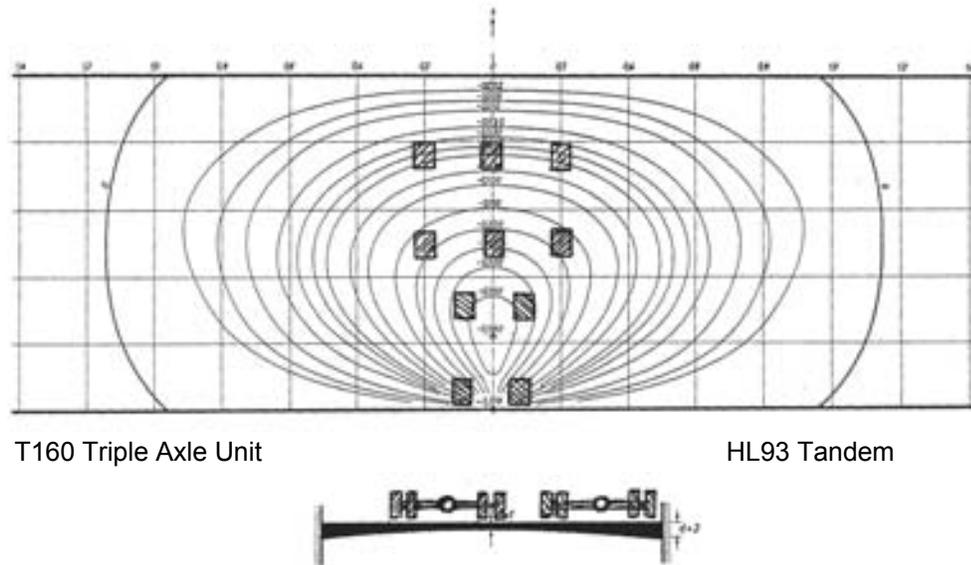


Figure C.4 - Homberg Chart for Negative Fixity Moment at Edge of Slab

In this particular case, it turns out that the left and right fixed edge moments are almost equal (this would not necessarily be so in every case):

$$M_{LL- \text{ left}} = -20.14 \text{ ft-kip per ft and } M_{LL- \text{ right}} = -20.13 \text{ ft-kip per ft}$$

Now, the real situation in the bridge is not one of fixity of the top slab at the webs. The entire box cross section can deform. The amount of deformation depends upon the relative flexural stiffness of the top slab, webs, bottom slab and the stiffness imparted from the box up- and down-station from the particular cross section of interest. Because in this case, we are only interested in the flexural performance of the top slab and not of the entire box structure, we can isolate a one-foot long box cross section and consider its transverse flexure as a frame. Because we are interested in the maximum positive flexure at the mid-span of the top slab, the frame chosen should be that which provides the greatest flexibility – in this case, that of a deep section, close to but not necessarily at the pier. (At the pier itself, the section and top slab is significantly stiffened by the local presence of the pier diaphragm. A little engineering judgment is necessary in choosing an appropriate section).

The effect of releasing the fixity of the top slab is to reduce the negative fixity moments at the webs at the expense of a corresponding increase in live load moment at the mid-span. For the proportions of the cross section near the pier, in this case, it results in an increase of mid-span live load moment by 2.42 ft-kip per ft. So the unfactored live load moments including impact are:

$$\begin{aligned} \text{At mid-span, } M_{LL+} &= + 8.82 \text{ ft-kip per ft and} \\ \text{At each side, } M_{LL-} &= -17.72 \text{ ft-kip per ft.} \end{aligned}$$

The above negative live load moment (i.e. -17.72 ft-kip per ft) is not the maximum that can occur at each side of the top slab. For the Permit Vehicle in mixed traffic, that maximum actually occurs with two lanes of load between the webs and one on the cantilever wing and has a value of -24.41 ft kip per ft. However, it is not a controlling load case for rating of the top slab. In fact it leads to a Rating Factor of 2.46 for the section at the face of the web compared to the controlling Rating Factor for mid-span positive flexure of 2.12 (below).

A transverse structural frame model is used to determine the effects from self weight (DC), superimposed dead load (DW) and transverse post-tensioning (M primary and M secondary). The transverse post-tensioning is at a slightly draped profile that rises as high as possible over the webs and drapes as low as possible in the mid-span region, given the constraints of cover and the presence of mild steel reinforcement. From this model, the permanent moments are:

		Unfactored	Factored
M_{DC}	=	1.700 ft-kip/ft	2.125 ft-kip/ft
M_{DW}	=	-0.270 ft-kip/ft	-0.405 ft-kip/ft
M_{SEC}	=	-0.687 ft-kip/ft	-0.687 ft-kip/ft
M_{PERM}	=		<u>1.033 ft-kip/ft</u>

The flexural capacity for the section is calculated according to LRFD formulae, using the following for the section at mid-span of the top slab:

Concrete strength (f'_c)	= 5,500 psi
Concrete parameter (β_1)	= 0.775
Overall depth of concrete	= 8.659 inch
Depth to centroid of strands (allowing for contact on top of duct)	= 4.801 inch
Eccentricity of strands (e_{PS})	= 0.472 inch
Area of strands (A_{PS})	= 0.275 in ² /ft
Ultimate strength of strands (f_{PU})	= 270.0 ksi
Yield stress of strands (f_{PY})	= 243.0 ksi
Spacing of 4*0.6" transverse tendons	= 3.16 ft
Effective prestressing force after losses (from model)	= 38.64 kip/strand
Effective prestressing force after losses	= 48.92 kip/ft
Area of mild steel reinforcement on tension face (A_s)	= 0.186 in ² /ft
Depth to mild steel tension rebar (d_s)	= 6.297 inch
Area of mild steel reinforcement on compression face (A'_s)	= 0.186 in ² /ft
Depth of mild compression rebar from comp. face	= 3.150 inch
Yield strength of tension and compression rebar	= 60.0 ksi

The contribution from mild steel reinforcement is relatively minor, but is included for the sake of completeness. The flexural strength capacity is calculated using Equations 5.7.3.1.1-2, 5.7.3.1.1-4 and 5.7.3.2.2-1 of the LRFD code and by applying the parameters:

Strength Reduction Factor for Flexure (LRFD), ϕ	= 1.00
Condition Factor (Chapter 7), new, good condition, ϕ_c	= 1.00
System Factor (Chapter 7), transverse conditions, ϕ_s	= 1.00

From which the Flexural Strength Capacity is:

$$C = \phi_c \cdot \phi_s \cdot M_n = 315.6 \text{ Inch-kip / ft} = 26.300 \text{ ft-kip / ft}$$

At the Strength Limit State, the available capacity for resisting live load effects is, therefore:

$$C - M_{\text{PERM}} = 26.30 - 1.033 = 25.263 \text{ ft-kip/ft}$$

The Rating Factor for the T160 Permit Vehicle in Mixed (HL93) Traffic is:

$$\begin{aligned} \therefore \text{RF} &= \frac{C - M_{\text{PERM}}}{\gamma_{\text{LL}} (M_{\text{LL}} + \text{IM})} \\ &= \frac{25.263}{1.35 * 8.82} = 2.12 \end{aligned}$$

If we wish to investigate the effect of a T160 Permit Vehicle alone, then the 3*22 kip Triple Axle unit is applied at a different transverse location on the Homberg Influence Surfaces than in the above example, but nevertheless at a location that gives a maximum (positive) value. The corresponding fixed edge moments are evaluated for the vehicle in the same location. The fixed edge moments are then released on the frame model, resulting in an increase in the slab moment.

Although the T160 Permit Vehicle is alone, the maximum multiple presence factor (m) is taken as 1.00 (not 1.20) because it is for a specific, defined, set of axle loads. The unfactored positive live load moment, after release of slab fixity, is found to be

$$M_{\text{LL}+} \text{ for lone T160,} = 6.07 \text{ ft-kip/ft.}$$

For a lone Permit Vehicle, the live load factor may be taken as 1.15 (Table 8.1.A1).

Consequently, the Rating Factor is given by:

$$\begin{aligned} \therefore \text{RF} &= \frac{C - M_{\text{PERM}}}{\gamma_{\text{LL}} (M_{\text{LL}} + \text{IM})} \\ &= \frac{25.263}{1.15 * 6.07} = 3.62 \end{aligned}$$

This is clearly a trivial case and not a controlling condition. It has been included to illustrate the need to consider different live load cases and the use of the lone Permit live load factor.

Example C.6: Web Shear Strength

For many load cases, the critical section for the shear strength is that section at the beginning of the fifth, interior span. The following is for the FDOT SU4 Legal Load at this section.

Shear Capacity as per AASHTO Segmental Guide Specifications for one web:

$$C = V_n = 1,208 \text{ kips}$$

Factored permanent loads from time-dependent computer analysis:

		From Table 8.1.A1		
DC	1,652 kips	$\gamma = 1.25$	=	2,065 kips
DW	132 kips	$\gamma = 1.50$	=	198 kips
PT	-214 kips	$\gamma = 1.00$	=	-214 kips
CR	-5 kips	$\gamma = 0.50$	=	-3 kips
SH	-1 kips	$\gamma = 0.50$	=	-0 kips
				2,046 kips

$$\text{For one web } V_{\text{Perm}} = 2,046 \text{ kips} / 2 = 1,023 \text{ kips}$$

Live Load Combination #5 Table 8.1.A2:

$$\begin{aligned} \text{Critical Shear from FDOT SU4 vehicle} &= 67.24 \text{ kips} \\ \text{For one web } V_{LL} &= 67.24 \text{ kips} / 2 = 33.62 \text{ kips} \end{aligned}$$

In this case, the maximum shear force in one web occurs under the condition of all three lanes loaded as opposed to either only one or two lanes loaded, even after allowing for the eccentric torsional effect of the eccentric live load. In the latter two cases, the eccentric torsional live load effect is discounted by Article 12.2.10 of the AASHTO Guide Specification for Segmental Bridges.

$$\gamma_{LL} = 1.35 \text{ per Table 8.1.A1}$$

$$\text{For Strength I, no. of lanes} = \text{number of design lanes} = 3, m = 0.85 \text{ per Table 8.1.A1}$$

$$\phi = 0.85 \text{ as per LRFD Table 5.5.4.2.2-1}$$

$$\phi_C = 1.0 \text{ as per Section 7.1.2.}$$

$$\phi_S = 1.2 \text{ as per Table 7.2}$$

Rating Factor Calculation:
(Strength I) RF =

$$\begin{aligned} \text{RF} &= \frac{\phi \times \phi_C \times \phi_S \times V_n - V_{\text{Perm}}}{\#_{\text{Lanes}} \times m \times \gamma_{LL} \times \sigma_{\text{Truck}} \times IM} \\ &= \frac{0.85 \times 1.0 \times 1.2 \times 1,208 \text{ kips} - 1,023 \text{ kips}}{3 \times 0.85 \times 1.35 \times 33.62 \text{ kips} \times 1.33} = 1.36 \end{aligned}$$