Florida Department of Transportation

New Directions for Florida Post-Tensioned Bridges

Volume 10 B: Load Rating Post-Tensioned Concrete Beam 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 B – Load Rating Post-Tensioned Concrete Beam Bridges

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Chapter 1 – Introduction

Load rating post-tensioned concrete beam bridges has not normally presented difficulties for Owners or Engineers since normal beam load rating procedures have been long established.

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.

Introduction of *AASHTO Load and Resistance Factor Design* (Ref. 1.4) has led to many changes in the verification of post-tensioned beam and segmental bridges at ultimate load (strength) limits. In the case of prestressed beam bridges in general, and segmental bridges in particular, except for increases in live loads, the LRFD design requirements at service load limits are not otherwise significantly different from LFD requirements. As a result, LRFD itself has not led to any significant change in load rating of beam bridges nor resolved any issues (such as zero tension in joints) regarding 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. 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 and beam bridges that are consistent with LRFR. Documents important to this task are listed 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 and beam bridges, form the basis for completing this task. FDOT LRFR for segmental bridges was released in July 2003. This document extends the same philosophy to prestressed (i.e. pre- and post-tensioned) beam bridges including spliced girder construction.

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 Precast Concrete Girder Bridges. Concrete girders include: typical AASHTO I-beams, Bulb-T's, Florida U-beams, double T's and similar precast components.

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 with discontinuous reinforcing 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 (NCRHP 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 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 Precast Concrete Beam 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 this context, LRFR does not directly address serviceability limitations by the control of specific tensile stresses to limit or avoid cracking or to enhance durability. These limitations require that recommendations be developed that usefully extend the scope of LRFR to service level ratings. 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 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 extending LRFR 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 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 precast concrete girder 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 precast concrete components at erection, the deck formwork system, the sequence and timing of casting the deck slab, making closures at splices, information on the installation of temporary or permanent, intermediate diaphragms and the sequence and timing of the installation and stressing of post-tensioning tendons.

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 the use of a different size of beam section, changes in beam spacing, thickness of webs or slabs, repositioning or re-sizing of mild reinforcing, and any changes to longitudinal post-tensioning layouts, locations of beam splices, use of different temporary supports, their locations and construction phase in which they are removed.

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 bridge, this might have involved changing span lengths, beam type, spacing, cross section dimensions, post-tensioning layout or construction method. It is essential to obtain the approved VECP plans used for construction before load rating the bridge. For a VECP currently proposed, under review or 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 precast concrete girder 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 precast concrete girder bridge should be documented in construction project records, Shop Drawings and As-Built Drawings. For continuous spans, depending upon the complexity of the erection and post-tensioning process, additional information may be contained in a project specific “Erection” or “Post-Tensioning Manual” or similar record. This should provide, in step-by-step detail, the sequence in which each span should have been erected, the deck slab cast and tendons installed and post-tensioned. It should also document the introduction and removal of temporary supports or construction loads. This should provide much of the information necessary for re-creating a structural analysis model using a time-dependent computer program.

Information on continuous span bridges to be gleaned from construction records should include:

- Casting date for each beam.
- Erection date for each beam.
- Concrete strength for each beam or closure splice.
- Concrete density or weights of actual beams.
- Dates of casting closure splices.
- Dates of introduction and removal of temporary supports.
- Magnitude and location of erection equipment or other temporary loads.
- Dates of placing or removing erection equipment 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.
- Dates tendons are grouted.
- Dates, sequence and length of deck slab cast.

Actual Concrete Strength:

Whenever possible, the actual concrete strength of precast and cast-in-place concrete should be used when load rating a concrete girder bridge. In the longitudinal direction, ratings may be controlled by limiting tensile stresses at the Service Limit State; this would be particularly acute at splices or connections with discontinuous reinforcing where no tension could be allowed. In such cases, nothing can be gained by considering actual concrete strengths. However, when a Service Limit State rating is controlled by tension in the precast girder, by compression or principal tension in webs, the use of actual concrete strengths should improve load rating results.

Construction records should be reviewed prior to load rating in order to establish the actual mean concrete strength of the precast girders and deck slabs 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 (location)” or “leaks at splice 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 prestressed (pre and post-tensioned) concrete 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 beam bridge usually comprises several parallel beams with a cast-in-place concrete deck slab. Span lengths may range from about 30 to over 200 feet, for example, for channel crossings on major bridges. Many bridges for grade separations, river, stream or railroad crossings are made of simply-supported, pre-tensioned AASHTO I-beams; the most common being Types II, III and IV. The latter (Type IV) usually has a maximum, simple span of about 100 feet. The Type V, VI and modified Type VI facilitate simply-supported spans up to about 130 feet. Occasionally, spans over 100 feet have been achieved by adding post-tensioning to a Type IV beam to provide full continuity over an interior pier (for example, Nursery Road Bridge over I-10). The introduction of the prestressed (i.e. both pre-tensioned and post-tensioned) Florida Bulb-T beam initially enabled spans of 140 feet, later as high as 160 feet (e.g. SR 56 / I-75). Longer channel spans have been attained by adding haunches and using cantilever and “drop-in” spliced girder construction.

Simply-supported pre-tensioned and post-tensioned bridges can be readily analyzed by established techniques, including hand calculations, and computer programs for prestressed beams. Loss of prestress from elastic shortening, creep, shrinkage and steel relaxation is taken into account. The weight of the wet deck slab and formwork is easily calculated and applied to the beam section alone. Subsequent superimposed dead load is carried by the composite section. The lateral distribution of live load to the composite-section girders is usually based upon distribution factors given in the LRFD code or through use of the Department’s computer program BRUFEM. Load rating by models involving other types of finite elements may be appropriate for specific structures or for posting avoidance.

Large, spliced girder bridges are erected using a phased construction process that may involve several intermediate statical schemes with temporary supports and a specific sequence for casting various portions of the deck slab. The appropriate prediction of forces and 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, taking into account changes in the cross section from beam alone to the composite section with the appropriate installation and stressing of longitudinal post-tensioning. Loss of post-tensioning due to initial friction, wobble, and wedge set, loss due to elastic shortening of sequentially stressed tendons, and subsequent loss due to long-term concrete shrinkage, creep and steel relaxation are taken into account.

For such bridges, longitudinal construction analyses for FDOT LRFD load ratings should be performed using a two-dimensional plane frame computer program to generate permanent structural conditions that represent long-term service.

For simple span I-girder and similar minor bridges when the same load type is applied in each load lane, Live Load Distribution Factors (LLDF) may be taken according to LRFD Chapter 4. LLDF in LRFD were developed primarily on the basis of simple spans. Continuous spans are more rigorously and correctly analyzed by finite elements, where it is possible to take into account the three-dimensional geometric properties of the cross sections, beam spacings, span lengths, effects of haunches and the eccentricity of the slab relative to the girders. For mixed traffic conditions where different types of loads may be applied in different lanes (i.e. regular traffic mixed with a Permit vehicle) and for continuous bridges and long span I-girder and similar major bridges, Live Load Distribution Factors should be generated using BRUFEM.

4.2 Construction Analysis Program

The construction analysis program should be capable of modeling construction in phases where prestressed girders transform from non-composite to composite sections with the slab and to transform post-tensioning steel to an effective composite section upon grouting tendons. The program should incorporate time-dependent material properties in accordance with Chapter 5.

The selected computer program should have the ability to define temporary and permanent post-tensioning. 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. Differential shrinkage is not required for design. However, it should be included for rating if casting data is available.

The geometry of the structure should be defined as a two-dimensional plane frame with nodes having three degrees of freedom (horizontal displacement, vertical displacement and in-plane rotation), located at frequent intervals along a span; *at least 10th points are recommended.* Other typical node locations are at spliced joints, temporary and permanent supports, critical shear sections, and 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 member length and beam cross-section properties for both the non-composite and composite sections.

Account should be taken of geometric features of the structure, such as the vertical eccentricity of the deck slab relative to the basic beam, allowing for build-ups. For example, vertical rigid links may be used to connect the nodes of the slab to the nodes of the beam. Also, for internal, bonded tendons, account should be taken of the equivalent concrete cross-sectional area of the transformed post-tensioning steel once the tendons are grouted. In addition to the superstructure definition, the stiffness of bridge bearings, columns and foundations should be appropriately included in the longitudinal analysis.

For load rating purposes using step-by-step construction analysis, it is only necessary to consider one beam line with its effective deck slab width (i.e. it is not necessary to separately consider the exterior beams).

The construction analysis should include as much information as possible according to construction records of the bridge (Chapter 3). When the exact dates of key activities are not available, it is recommended that the analysis follow the sequence of construction and that missing dates be approximated. If no casting and erections dates are available, the following may be assumed:

Assume the age of the beams at erection is *56* days.
Assume each continuity diaphragm or beam splice takes 1 week.
Assume it requires a week to erect formwork for each span.
Assume it requires a week to place reinforcing steel in each span.
It may be assumed that the latter three activities are concurrent but in different spans.
Assume each portion of a sequentially poured deck slab is cast to a half week cycle.
In the absence of more complete information, assume that it takes approximately 4 weeks to completely cast a deck slab for a four-span continuous unit.

4.3 Cumulative Permanent Effects in Continuous Spliced-beam Bridges

For a continuous spliced-beam bridge, built and post-tensioned in phases, the cumulative permanent effects should be determined as follows:

1. Basic beam with pre-tensioned strands; apply beam self weight (DC beam) to gross concrete beam-only section.

2. If supported on temporary, in-span towers, determine static reactions or out-of-balance cantilever reactions on each temporary and permanent support.

3. Cast beam closure splices (deck slab not yet cast).

4. Install first-stage longitudinal post-tensioning (PT-1), but do not grout – still using gross concrete beam-only section.

5. Determine changes in permanent support and temporary tower reactions due to secondary PT effects from first stage post-tensioning (EL PT-1).

6. If removable formwork is used, apply weight of formwork (EL forms) to continuous, gross concrete beam-only section. Otherwise apply “Stay-in-Place” formwork load and skip Step 7.

7. Determine changes in permanent support and temporary tower reactions from weight of formwork (EL forms).

8. Apply weight of wet, deck slab concrete (DC slab) to the continuous, gross concrete beam section only for the full length of the continuous unit. For a rigorously correct solution, the slab load should be applied in the exact sequence in which the deck was cast, and the section properties of the portions already hardened would be taken for the composite section using transformed (E_{slab} / E_{beam}) widths. In theory, this would stiffen the already-built portions and attract more, subsequent dead load to those portions. However, this is not a practical approach for load rating. Not only may the deck slab pouring sequence not be known, but the rigor of this method is judged not likely to significantly affect overall effects and load rating. Consequently, if it is judged necessary to account for intermediate construction effects, it is suggested that the assumed pouring sequence be: (a) all slab load applied to the mid-span of each span between quarter points, followed by (b) all portions of slab over intermediate permanent

piers, where, in both cases, the slab load is applied only to the gross concrete section of the basic beam.

9. Determine changes in permanent support and temporary tower reactions from weight of deck slab concrete (DC slab) using continuous, gross concrete non-composite section.

10. After deck slab concrete attains sufficient strength, install second stage longitudinal post-tensioning (PT-2). Apply this to composite section based upon the effective slab width per ratio of E_{slab} / E_{beam}.

11. Determine changes in permanent support and temporary tower reactions due to secondary PT effects from second stage post-tensioning (EL PT2) applied to the composite section.

12. Grout longitudinal tendons.

13. Remove erection loads and temporary towers, namely, weight of any removable formwork (EL forms). Remove accumulated temporary tower reactions by applying equal and opposite loads on the completed, permanent structure. Base this upon the composite section properties, taking into account the transformed areas of post-tensioning steel after grouting. Consider this step as the "End of Construction" (EOC). [Note: some computer programs (e.g. BD 2) automatically account for the removal of reactions or loads upon command so that it is not necessary to apply an equal and opposite load. Likewise, when accounting for transformed steel areas].

14. Allow time to pass and determine long-term creep and shrinkage effects, such as long-term loss of prestress and redistribution of forces, using suitable method (e.g. BD 2 program). In this regard, it is NOT necessary to account for the effects induced by differential shrinkage of younger deck slab concrete atop more mature precast concrete beams, but it is necessary to determine the overall shrinkage and creep losses and redistribution.

15. Use final results at infinity (day 4,000) of dead load forces and stresses.

4.4 Application and Distribution of Live Loads

The determination of the overall lateral distribution of live load effects should be based upon:

- Using stiffness of gross concrete composite section of girder and slab.
- Using slab width modified only by ratio of E_{slab} / E_{beam}.
- Do not transform any steel (PT or rebar) to equivalent concrete.
- Do not deduct section for ducts whether grouted or not.

For a "Major bridge" – i.e. continuously post-tensioned beam bridge where construction-phase analysis is necessary – use BRUFEM program to determine a maximum Live Load Distribution Factor (LLDF) per cross section, per beam, for each live load moment and shear force for the particular type of live load and number of live load lanes of interest (i.e. from 1 lane to the maximum number as appropriate). It is not necessary to account for edge stiffening provided by sidewalks and barriers, unless necessary for Posting Avoidance (see Chapter 9).

For a “Minor bridge” – i.e. simply-supported beam bridge where construction-phase analysis is not necessary – LRFD Live Load Distribution Factors may be used.

BRUFEM should always be used for any bridge subjected to mixed traffic – i.e. normal traffic with a Permit vehicle.

When using BRUFEM, account should be taken of the vertical eccentricity of the deck slab relative to the basic beam - for example, using vertical rigid links to connect the nodes of the slab to the nodes of the beam. This provides a more realistic assessment of load distribution than by using a co-planar slab-beam model.

For a major bridge, determine the longitudinal live load moment and shear force for one lane of longitudinal live load using the live load generator feature of the program (e.g. BD 2). Modify the live load results according to the maximum LLDF per section from BRUFEM to determine the maximum moment and shear per beam.

4.5 Determination of Stresses in Composite Bridge Decks

4.5.1 Dead Load Forces

Distribution of dead load and prestress axial force, bending moment and shear are found from the construction-phase analysis (e.g. BD 2).

4.5.2 Dead Load Stresses

Dead load flexure, shear and principal tension stresses are found from the accumulation of stress according to the step-by-step erection process – this requires accumulating the incremental change in stress at each location and fiber of interest according to changes in section properties from this process.

If the location for checking principal tensile stress lies within a vertical distance of 1 duct diameter from the top or bottom of an internal duct, deduct half the duct size from the web-width (whether duct is grouted or not) to determine the *dead load shear stress* component of the principal tension. However, the axial prestress force and flexural dead load (if any) components of the principal stress may be based on using full web width.

4.5.3 Live Load Forces

Lateral LLDF’s are obtained by the finite element (e.g. BRUFEM) global analysis based on gross (concrete only) composite section properties. The live load flexure and shear effects are produced by one lane of specified truck traveling along the length of the plane frame structural model. These force effects are then modified by the appropriate LLDF to give the distributed live load effect to the beam line under investigation.

4.5.4 Live Load Stresses

Positive moment flexure:

Determine the positive live load flexural tensile stress based upon the gross (concrete only) composite section properties including the transformed post-tensioning steel. For bonded tendons, tension is limited to stresses given in Chapter 8.

Negative moment flexure:

When the slab is already in compression (for example, from the second stage of post-tensioning of a continuous, draped tendon at the section over an interior pier), determine the live load longitudinal flexural tension stress in the top of the slab based on the effective composite section properties including the transformed post-tensioning steel. Tension should be limited to the stresses given in Chapter 8.

When the slab is not already in compression (for example, a full-height reinforced concrete splice over an interior pier with NO top longitudinal post-tensioning), assume that the top slab is cracked and that top tensile effects are carried by the slab reinforcement only. Determine the stress in the rebar according to established LRFD formulae for reinforced concrete. The section must have ability to develop full bottom flange compression through intimate contact of the cast-in-place splice. (This situation could be considered “partially continuous” as opposed to a fully continuous section where the PT passes through the splice).

Due to shear force:

Determine live load shear stress based on the gross (concrete only) composite section.

If the location for checking principal tensile stress lies within a vertical distance of 1 duct diameter from the top or bottom of an internal duct, deduct half the duct size from the web-width (whether duct is grouted or not) to determine the *live load shear stress* component of the principal tension. The live load flexural stress component of the principal tension (if any) can be taken according to the gross (concrete only) composite section using the full web width.

4.6 Transverse Analysis

Transverse analysis for FDOT LRFD load ratings of the top slabs of concrete beam bridges is not normally considered if the effective length of the slab does not exceed 13.5 feet. The effective length of the slab is the distance between flange tips, plus the flange overhang, taken as the distance from the extreme flange tip to the face of the web, disregarding any fillets (Reference: LRFD 9.7.2.3 and 9.7.2.4).

Should inspection indicate distress and the possible need to check the deck slab, then an appropriate check for local concentrated wheel load effects may be made using influence surfaces (by Pucher or Homberg) or by a suitable grillage or finite element model of the slab. Slab boundary conditions should closely approximate the degree of support provided by the beams. Wheel loads should be represented as distributed surface forces in accordance with the tire contact area taken according to LRFD 3.6.1.2.5, dispersed at 45 degrees to the mid-depth of the slab.

4.7 Analysis of Local Details

Concrete bridges sometimes contain local details that must be appropriately designed to carry local loads or distribute forces to the main bridge components (beams). Although forces in these details can vary as a function of the applied live loads (with the exception of in-span beam splices), it is recommended that they not be included in the load rating. Rather, the capacities of such details should be checked only for critical loads or ratings and then only if there is evidence of distress (e.g. cracks).

Important local details in concrete bridges include beam splices within a span, dapped hinges, diaphragms and details, such as corbels, that support expansion joint devices and anchorages for post-tensioning tendons. The behavior of these details and the forces to which they are subjected may be determined by appropriate models or hand calculations. Analysis methods and design procedures are available in LRFD; for example, strut and tie analysis.

4.7.1 Beam Splices within a Span

Beam splices within a span are frequently used to connect end-span beams and main, drop-in, span beams to the ends of cantilever girders erected over the main piers of a three span bridge or similar applications. Such splices usually require reinforcing bars projecting from the ends of the precast beams and into a reinforced, cast-in-place transverse diaphragm. Longitudinal post-tensioning ducts are connected and tendons pass through the splice.

Beam splices are typically near inflection points; consequently, live load effects may induce longitudinal tensile stress in the top or bottom. Therefore, the longitudinal tendons are approximately concentric, i.e. at mid-depth of the composite section. It is necessary to check longitudinal flexure and shear effects at in-span beam splices.

4.7.2 Dapped Hinges within a Span

Dapped hinges are rarely used in beam bridges in Florida. Forces acting through dapped hinges within a span should be calculated for statically determinate structures or be determined as a part of the time-dependent construction analysis for indeterminate structures. Maximum live load reactions should also be calculated. Once all reaction forces are known, local analyses should be performed to develop the hinge forces into the main beam components using suitable strut-and-tie techniques. An alternate approach would be to develop three-dimensional finite element models to analyze the flow of forces.

4.7.3 Diaphragms

The main purpose of transverse diaphragms is to provide lateral stability to girders during construction and improve the overall lateral load distribution under traffic. In the latter case, transverse reinforcing should be continuous or transverse tendons should be used to provide sufficient capacity for this purpose.

Transverse diaphragms themselves need not be analyzed as part of a routine load rating. Only if there is evidence of distress (such as cracks, efflorescence or rust stains) should it be necessary to more closely consider the forces and stresses in a diaphragm.

The stiffness of any transverse diaphragms should be included, if significant and appropriate, in any finite element analysis program (e.g. BRUFEM) used to establish Live Load Distribution Factors.

4.7.4 Support for Expansion Joint Devices

Expansion joint devices are usually contained in a recess formed in the top of the end of the top slab and transverse diaphragm. Occasionally, depending upon the need to accommodate other details, such as drainage systems, this may involve a corbel - usually as a contiguous part of the expansion joint diaphragm. It is not necessary to analyze such a detail for routine load rating. Only if there is evidence of distress (such as cracks, efflorescence or rust stains) should it be necessary to more closely consider the forces and stresses in such a detail.

4.7.5 Anchorages for Post-Tensioning Tendons

Anchorages are normally contained in a widened portion of the web at the ends of a beam. It is not necessary to analyze anchorage details for routine load rating. Only if there is evidence of distress (such as cracks, efflorescence or rust stains) should it be necessary to more closely consider the forces and stresses in such a detail itself.

Changes in the gross section properties at anchor block zones should be properly accounted for in any finite element analysis program (e.g. BRUFEM) used to establish Live Load Distribution Factors.

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: $h_o = \lambda(\frac{2A_c}{\mu})$

Where:

λ = 2.0 (See Table 5.2 for 75% relative humidity).
A_c = area of concrete section.
μ = 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,

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

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.

$\phi_d \beta_d (t\text{-}t_0) / E_{C28}$ 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 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:

ϕ_d = 0.4 = delayed modulus of elasticity.

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

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

β_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 $(t - t_o)$ is given by:

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

Where:

ε_{s0} = $\varepsilon_{s1} \times \varepsilon_{s2}$
ε_{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).

β_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.

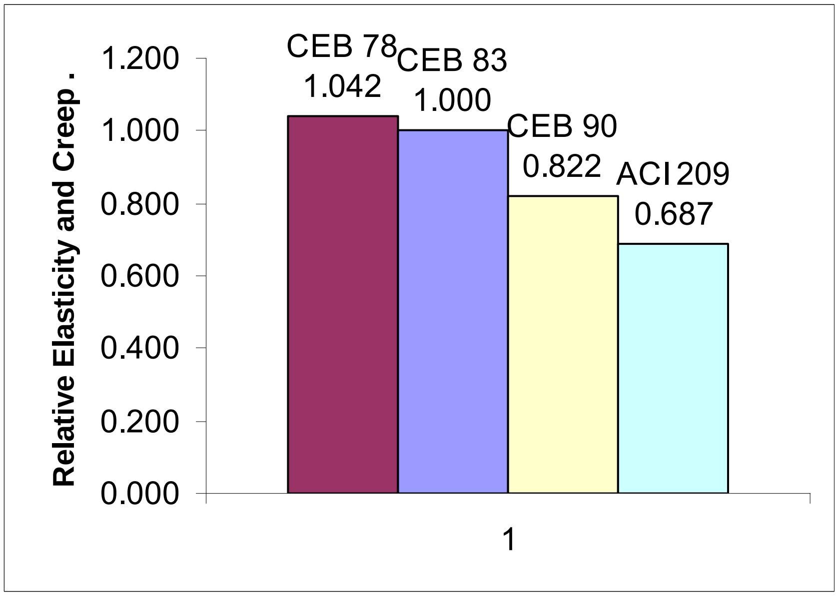


Figure 5.1 – Relative Elasticity and Creep according to Different Codes

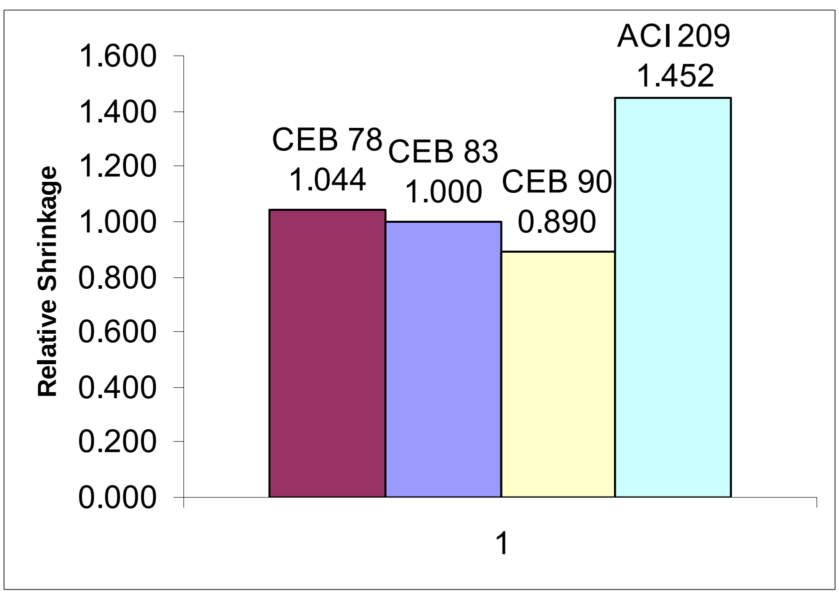


Figure 5.2 – Relative Shrinkage according to Different Codes

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_{f1}	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_{f1}, ε_{s1} and λ

Notional Thickness		Coefficient for Creep ϕ_{f2}	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_{f2} 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 $\beta_f(t)$			
	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"
CREEP (day 4,000) loaded at t = 28 days						
CEB 78 / CEB 83 =	1.054	1.043	1.042	1.027	1.045	1.042
CEB 83 =	1.000	1.000	1.000	1.000	1.000	1.000
CEB 90 / CEB 83 =	0.836	0.819	0.818	0.817	0.819	0.822
ACI 209 / CEB 83 =	0.683	0.714	0.660	0.664	0.714	0.687
SHRINKAGE (day 4,000) after casting at day t = 0						
CEB 78 / CEB 83 =	1.061	1.055	1.049	1.000	1.056	1.044
CEB 83 =	1.000	1.000	1.000	1.000	1.000	1.000
CEB 90 / CEB 83 =	0.872	0.897	0.898	0.884	0.897	0.890
ACI 209 / CEB 83 =	1.414	1.574	1.261	1.435	1.574	1.452

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 prestressed concrete beam bridges in Florida performed in accordance with LRFR. It also offers explanations for the adoption of various values for different parameters, such as "m", and load factors for permanent, transient and live loads.

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 beam weights if measured during construction.

Weights of cast-in-place deck slabs, build-ups, diaphragms, ribs, additional 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”.

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

For load rating, the dead load factor (γ_{DC}) and superimposed dead load factor (γ_{DW}) should be in accordance with Table 8.1.B “Load Combinations for Beam 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 beam 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.B 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 to include the effects of half thermal gradient (0.5*TG) with full design live loads at Service Limit State. This standard should also be used for precast concrete beam bridges because they experience very similar thermal gradient effects as segmental bridges.

Thermal gradient effects do not apply when verifying the design of new 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*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.B 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

If and when it is necessary to check transverse post-tensioned deck slabs, the Design Load for Inventory and Operating Ratings 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.

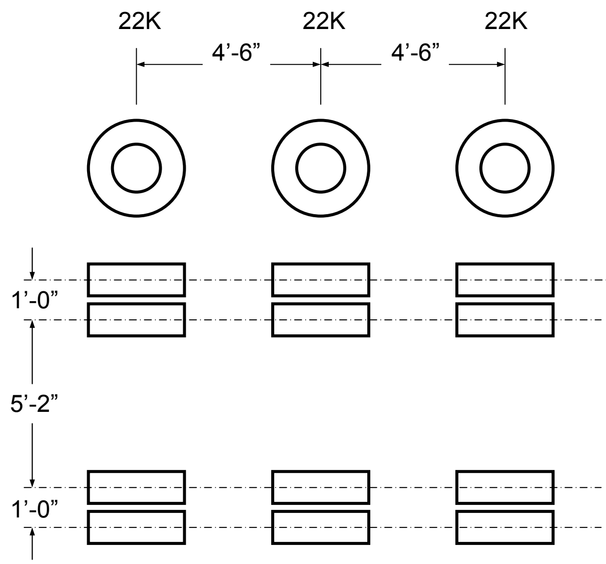


Figure 6.1 – FDOT 22Kip Triple Axle Load

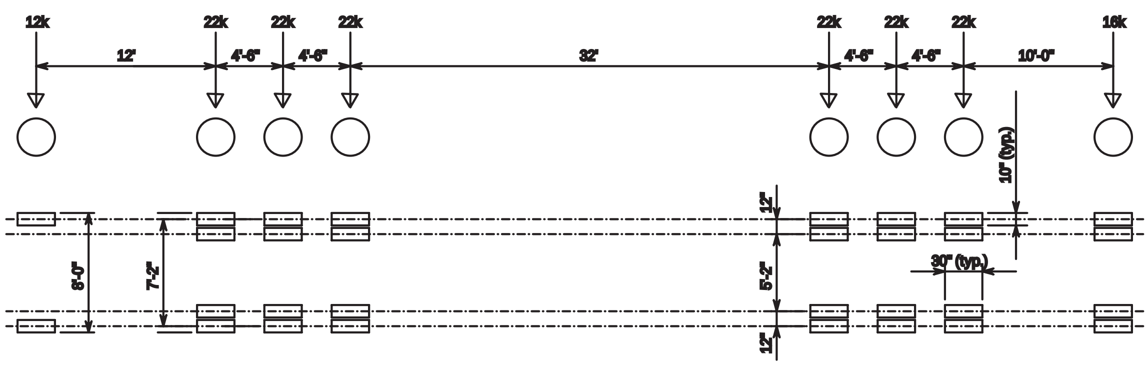


Figure 6.2 – FDOT Permit Vehicle T160
160,000 lb Eight Axle Truck

6.4.4 Number of Live Load Lanes

For beam type bridges, Inventory and Operating Ratings at both Service and Strength Limit States shall be performed using the number of design live load lanes as specified in LRFD.

Laterally, there is no limitation on the location of the live load. Loads should be placed so as to create the maximum longitudinal effects, as determined according to Chapter 4. Live load factors and load combinations should be according to Chapter 8.

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 beam bridges, it is necessary to do both in order to attain reduced target reliability (i.e. approximately $\beta = 2.5 < 3.5$).

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 load lanes for segmental bridges and helped identify acceptable load factors and allowable stresses at the Service Limit for beam bridges.

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*1.20 = 2.10 times the truck load effect. This is essentially the same as previous LFD design; i.e. 1.30*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 γ_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 γ_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 γ_L = 0.80 on Legal and Permit loads.

A live load factor of γ_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.

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 precast concrete beam 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 with consideration to similar factors developed for segmental bridges. In particular, this incorporates the concept of internal redundancy, introduced in AASHTO LRFR and extended to the Multiple Tendon Path strategy of “New Directions for Florida Post-Tensioned Bridges”.

The Strength Limit State capacity factors for 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 1.25.

Condition and System Factors apply to the Strength Limit State. In accordance with LRFR, the product of the Condition and System Factors (ϕ_C x ϕ_S) need not be taken less than 0.85 and in no case shall be greater than 1.25. 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 prestressed (pre- and post-tensioned) concrete beam bridges represents the degree of damage or loss of concrete or prestress at a 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 concrete 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 prestress or loss of rebar to corrosion or other damage.

Borescope investigation may verify surface corrosion or it may reveal broken wires in post-tensioning tendons. 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 loss or loss of prestressing force 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 Post-Tensioned Bridges").

7.1.2 Illustrative examples for ϕ_c

- Post-tensioned beam bridge designed and built strictly in accordance with the FDOT "New Directions for Florida Post-Tensioned Bridges": $\phi_C = 1.10$
- Precast concrete beam bridge with internal tendons, relatively old, but with no evidence of leaks, efflorescence or rust stains, generally in good or satisfactory condition: $\phi_C = 1.00$
- Precast concrete beam bridge with internal tendons, relatively old, that contains galvanized ducts and tendons rise into slab at piers, otherwise in good or satisfactory condition: $\phi_C = 0.95$
- Precast concrete beam bridge with internal tendons, relatively old, with evidence of leaks, with efflorescence or rust stains 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: $\phi_C = 0.90$

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 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 concrete beam bridges with regard to the following:

- Longitudinal Continuity – The research upon which LRFR is based (NCHRP 406), examined simply-supported and continuous steel and concrete girder bridges. However, the type of longitudinal continuity that makes a structure statically indeterminate and increases the overall redundancy, by virtue of its ability to fully mobilize compression flanges without buckling at the Strength Limit State, is not acknowledged in LRFR.

- 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 post-tensioned construction.

The System Factors presented in this Chapter endeavor to incorporate these features for precast concrete beam bridges.

7.2.1 Longitudinal Flexure

System Factors for longitudinal flexure at the Strength Limit State should be taken from Table 7.3. System Factors in this table are given for the number of girders in the cross section, different degrees of longitudinal continuity expressed in terms of the number of plastic hinges required to create a mechanism, and the number of post-tensioning tendons per web. Use of the number of girders in the cross section is a subjective approach and is not intended to transcend the need for an appropriate live load distribution to the critical beam.

For longitudinal flexure, System Factors in Table 7.3 range from 0.85 to 1.20. These values may be increased by 0.05 for spans with 3 or more diaphragms within each span in addition to the diaphragms at the end of each span. This table is a guide. Higher values may be considered on a case-by-case basis with the approval of the Department. However, the maximum system factor value shall not exceed 1.25 and the minimum value need not be less than 0.85.

Pre-tensioned beam bridges with a deck slab poured and reinforced to be continuous for traffic over the top of the ends of beams, but where those ends have a gap and are not in direct contact to transmit longitudinal compression flange forces (i.e. “poor-boy” joints), shall be considered as simply-supported.

System factors for spans made fully continuous by virtue of longitudinal post-tensioning passing continuously through beam splices or cast-in-place diaphragms generally may be increased by 0.10 over that of a simply-supported span. This is reflected in Table 7.3.

(The format of Table 7.3 has been deliberately arranged to match that of Table 7.2 for different types of Segmental Bridges to facilitate possible merging).

7.2.2 Shear and Torsion

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

7.2.3 Transverse Flexure

Transverse rating of a deck slab is not required. However, if checks of load effects are necessary, the System Factor should be taken as ϕ_S = 1.00.

7.3 Local Details

Local Details including splice joints, dapped hinges, diaphragms, expansion joint support details and post-tensioning anchorages are not part of the Load Rating procedure. Such details should be reviewed, however, to ensure that they 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 to 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

NOTE: Table 7.2 applies to segmental bridges and is not shown here

Number of Girders in Cross Section	Span Type	# of Hinges required for mechanism	System Factors (ϕ_s) No. of Tendons per Web 1	2	3	4
2	Interior span	3	0.85	0.90	0.95	1.00
	End span	2	0.85	0.85	0.90	0.95
	Simple span	1	0.85	0.85	0.85	0.90
3 or 4	Interior span	3	1.00	1.05	1.10	1.15
	End span	2	0.95	1.00	1.05	1.10
	Simple span	1	0.90	0.95	1.00	1.05
5 or more	Interior span	3	1.05	1.10	1.15	1.20
	End span	2	1.00	1.05	1.10	1.15
	Simple span	1	0.95	1.00	1.05	1.10

- Above values may be increased by 0.05 for spans containing more than 3 intermediate, evenly spaced diaphragms in addition to the diaphragms at the end of each span.
- Higher values may be considered on a case-by-case basis with the approval of the Department.
- In no case shall the System Factor exceed 1.25.
- System Factor need not be less than 0.85.

Table 7.3 – System Factor Values for Post-Tensioned Beams

(NOTE: format of above table has been chosen to match that of Table 7.2 for Segmental in order to facilitate possible merging later, if preferred).

(For information only: A previous draft format of the above table addressed values in terms of the percentage of live load effect carried by the beam in question. Concern was expressed that this might encroach upon "Live Load Distribution Factors" inherently built into the LRFD load distribution tables. For this reason, values are now expressed in terms of the "number of girders in the cross section", which is very much more clear and explicit. However, previous higher system factor values (upper diagonal) have been reduced a little to provide a "mean" value of about 1.00 for the most commonly occurring bridge decks. This then allows for the recognition of certain beneficial redundancies without pushing the envelope too far at this time.

An engineer may consider higher values on a case-by-case basis with the approval of FDOT).

Chapter 8 – Rating Equation and Load Combinations

Post-tensioned concrete beam bridges are to be load rated at both Inventory and Operating Levels using two longitudinal checks at the Service Limit State and two at the Strength Limit State, as follows:

At the Service Limit State:
- Longitudinal Box Girder Flexure
- Principal Web Tension

At the Strength Limit State:
- Longitudinal Box Girder Flexure
- Web Shear

This Chapter presents the General Load Rating Equation and recommended Load Combinations for use in load rating with the above four features of concrete beam bridges.

Load rating of the top deck slabs of concrete beam bridges is not required when the effective length of the slab is less than 13.5 feet. The effective length of the slab is the distance between flange tips, plus the flange overhang, taken as the distance from the extreme flange tip to the face of the web, disregarding any fillets (Reference: LRFD 9.7.2.3 and 9.7.2.4). If the effective length of the slab exceeds 13.5 feet, or if it is otherwise considered necessary (e.g. if a slab is transversely prestressed), load rating of the slab should be performed in accordance with the procedures for the transverse load rating of the top slab of a segmental bridge *(Volume 10 A).*

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. (For beams, shear capacity is by Modified Compression Field Theory (MCFT).

For Service Limit States:

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

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).
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.B1, "Load Factors for Post-Tensioned Beam Bridges"* and *Table 8.1.B2, "Load Combinations for Post-Tensioned Beam Bridges".* Table 8.1.B1 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.B2) 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.B1, 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). Applies to the rating analysis of prestressed concrete bridge deck slabs.

SERVICE III – applies to concrete in tension. Load factors for SERVICE III for Operating conditions have been selected to attain the benefits of reduced reliability when used in conjunction with higher allowable tensile stresses and to provide similar operational ratings to historical practice. However, no similar change (reduction) has been made in the load factor for Service I compression stress conditions. This is mainly for simplicity; on the basis that it is very unlikely that compression would control a rating and, if it did, then the use of an increased allowable compression should be considered to avoid posting or restrictions.

The following is a detailed checklist of the load applications, combinations and circumstances necessary to satisfy FDOT and AASHTO LRFR ratings which are summarized in Table 8.1.B1. Brief explanations are also included as to why a parameter (e.g. "m" or "γL") is assigned a particular value.

8.2.1 Inventory Rating – Design Loads

Longitudinal:

- Apply HL93 Truck or Tandem, including 0.64 kip/ft uniform lane load *(Table 8.1.B2, 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 multiple 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.B2, 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 at Strength.
- Use SERVICE III Limit State with live load factor, γ_L = 0.80. (Note: use of γ_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.B as appropriate.
- For SERVICE III Limit State, limit concrete Principal Tensile Stress 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, γ_L = 1.00 and limit concrete longitudinal

flexural compressive stress to values in Table 8.2.B. (Note: γ_L = 1.00 as AASHTO LRFR).

- For Strength I Limit State use live load factor, γ_L = 1.75.

8.2.2 Operating Rating – Design Load (HL93)

Longitudinal:

- Apply HL93 Truck or Tandem, including 0.64 kip/ft uniform lane load *(Table 8.1.B2, load combination #3).*
- 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.
- Place loads in full available width as necessary to create maximum effects (for example, in shoulders).
- Multiple presence factor: for 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.B2, 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, γ_L = 0.80 and limit concrete longitudinal flexure and principal tensile stresses to values in Table 8.2.B as appropriate; namely, longitudinal flexure 7.5√f’c (psi) and principal tension 4√f’c (psi). (Note: use of γ_L = 0.80 in conjunction with increased allowable tensile stress are, in these cases, to attain the benefits of reduced reliability).
- Use Service I Limit State with live load factor, γ_L = 1.00 and limit concrete longitudinal flexural compressive stress to values in Table 8.2.B.
- For Strength I Limit State use live load factor, γ_L = 1.35.

8.2.3 Operating Rating – Florida Legal Loads

Longitudinal:

- Apply FDOT Legal Load Trucks SU4, C5 and ST5 *(Table 8.1.B2, load combination #5).*
- Also, apply HL93 Truck only – i.e. 72 kip GVW (load combination column #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 both Strength and Service Limit States, use number load lanes per LRFD.
- Place loads in full available width as necessary to create maximum effects (for example, in shoulders).
- Use multiple 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.
- 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.B2, 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.B2, 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.
- Use SERVICE III Limit State with live load factor, γ_L = 0.80 and limit concrete longitudinal flexure and principal tensile stresses to values in Table 8.2.B as appropriate; namely, longitudinal flexure 7.5√f'c (psi) and principal tension 4√f'c (psi). (Note: use of γ_L = 0.80 in conjunction with increased allowable tensile stress are, in these cases, to attain the benefits of reduced reliability).
- Use SERVICE I Limit State with live load factor, γ_L = 1.00 and limit concrete longitudinal flexural compressive stress to values in Table 8.2.B. (If concrete compressive stress, as opposed to tension stress, should become a controlling rating concern, then consider increasing the allowable compression to avoid posting).
- For STRENGTH I Limit State, use live load factor, γ_L = 1.35.

8.2.4 Operating Rating – Florida Permit Loads

Longitudinal, annual "blanket" permits, mixed traffic:

- Apply ONE T160 Permit Vehicle in one load lane. *(Table 8.1.B2, load #8).*
- Apply HL 93 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.B2, 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.B2, 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 both Strength and Service Limit States, use number of load lanes per LRFD.
- Place loads in full available width as necessary to create maximum effects (for example,

in shoulders).

- Use multiple presence factor: for two lanes, m = 1.00; three, m = 0.85; four or more, m = 0.65. (Maximum limit of 1.0 applies because two lanes are minimum number that can be loaded as opposed to some of loads being specific (defined) Permit loads).
- 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, γ_L = 0.80 and limit concrete longitudinal flexure and principal tensile stresses to values in Table 8.2.B as appropriate; namely, longitudinal flexure 7.5√f’c (psi) and principal tension 4√f’c (psi). (Note: use of γ_L = 0.80 in conjunction with increased allowable tensile stresses are, in these cases, to attain the benefits of reduced reliability).
- Use SERVICE I Limit State with live load factor, γ_L = 1.00 and limit concrete longitudinal flexural compressive stress to values in Table 8.2.B. (If concrete compression stress, as opposed to tension stress, should become a controlling rating concern, then consider increasing the allowable compression to avoid restrictions).
- For STRENGTH II Limit State, use live load factor, γ_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:

- Apply ONE Permit Vehicle with no other live load *(Table 8.1.B2, 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”.
- Multiple presence factor, m = 1.00.
- Use SERVICE III Limit State with live load factor, γ_L = 1.00 and limit concrete longitudinal flexure and principal tensile stresses to values in Table 8.2.B as appropriate; namely, longitudinal flexure 7.5√f’c (psi) and principal tension 4√f’c (psi). (Note: use of γ_L = 1.00 is necessary because this is a rating for a specific (defined) load. Limited benefits of reduced reliability are possible only through the higher allowable tensile stresses).
- Use SERVICE I Limit State with live load factor, γ_L = 1.00 and limit concrete longitudinal flexural compressive stress to values in Table 8.2.B. (If concrete compression stress, as opposed to tension stress, should become a controlling rating concern, then consider increasing the allowable compression to avoid restrictions).
- For STRENGTH II Limit State, use live load factor, γ_L = 1.15.
- Reduced Dynamic Load Allowance (IM) or live load factor (γ_L) may be used to avoid restrictions (Chapter 9).

8.3 Capacity – Strength Limit State

The capacity of a section in longitudinal flexure may be determined using any of the relevant formulae or methods in the LRFD Code, or by more rigorous analysis techniques involving strain compatibility. The latter should be used in particular if 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 capacity at the Strength Limit State should be calculated according to the "Modified Compression Field Theory" (MCFT) of LRFD.

8.4 Allowable Stress Limits – Service Limit State

Allowable stresses for the Service Limit State are given in *Table 8.2.B.* 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 with Discontinuous Reinforcement

For Load Rating purposes, longitudinal tension in joints with discontinuous longitudinal mild steel reinforcing and unbonded (external tendons) is limited to zero (LRFD Table 5.9.4.2.2-1). (In the case of post-tensioned beams, this might apply at some spliced girder connections).

8.4.2 Transverse Tensile Stress

For a reinforced concrete slab, when the effective length of a deck slab exceeds 13.5 feet, or if it is otherwise considered necessary, load rating of the slab at the Service Limit State shall be based upon the limitations set by LRFD 5.7.3.4 "Control of Cracking by Distribution of Reinforcement" so that the tensile stress in the reinforcement does not exceed 0.6fy or the tension in the gross concrete section does not exceed 80% of the modulus of rupture given by LRFD 5.4.2.6.

For a slab that is transversely prestressed, the load rating at the Service Limit State shall be determined based upon the allowable stresses given in Table 8.2.B: namely, for Inventory $3\sqrt{f'c}$ (psi) and for Operating $6\sqrt{f'c}$ (psi); as for the deck slab of a segmental type box. 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 in service. This shall be applied to both for the design of new bridges and Load Rating. The following check is the recommended minimum prescribed procedure.

Critical sections should be considered only at the following locations:

Either: where the normal beam section with the minimum web width extends to the support - at a distance of one half the overall depth of the composite section from the edge of the bearing or face of a supporting diaphragm.
Or: at the end of the anchor block taper, whichever is more critical.

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 axial, shear and principal tensile stresses at the critical section in the highest loaded web. The shear stress and principal tensile stress should be determined at the elevation of the neutral axis of the original beam section and at the elevation of the neutral axis of the composite section. This should be done under the conditions of long-term residual axial, shear and flexural stress. [This requires a proper incremental accounting of stresses at these locations from initial casting to long-term service, including changes as a consequence of construction phases].

The live load shear stress and corresponding principal tensile stress should be found first using a live load factor γ_L = 1.00 for the Service Limit State. Then, while holding the longitudinal stress constant, the live load shear component of stress should be increased until it induces the maximum allowable principal tensile stress.

The Rating Factor at the Service Limit State is the ratio of the live load shear force required to induce the allowable maximum Principal Tensile Stress to the live load shear force under a live load factor of 1.00.

Unless the tendon path geometry makes another elevation more critical, when the elevation of neutral axis of the basic beam or composite section lies within 1 duct diameter of the top or bottom of an internal, grouted duct, the web width for calculating stresses shall be reduced by half the duct diameter.

8.5 Local Details

Important Local Details in concrete beam 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
Compression (Longitudinal or Transverse): ● 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
Longitudinal Tensile Stress in Precompressed Tensile Zone: (Intended for Pre and Post-Tensioned Beams and similar construction) For components with bonded pretressing tendons or reinforcement that are subject to not worse than: For (a) an aggressive corrosion environment and (b) moderately aggressive corrosion environment	$3\sqrt{f'c}$ psi tension	$7.5\sqrt{f'c}$ psi tension	LRFD Table 5.9.4.2.2-1 and FDOT FDOT no distinction for Environ't
For components with unbonded prestressing tendons	No Tension	No Tension	LRFD Table 5.9.4.2.2-1
Longitudinal Tensile Stress through Joints in Precompressed Tensile Zone: (Intended for Segmental and similar construction) ● Type A joints with minumum bonded auxiliary longitudinal reinforcement sufficient to carry the calculated longitudinal tensile force at a stress of 0.5fy; for internal and/or external PT (e.g.cast-in-place construction) For (a) an aggressive corrosion environment and (b) moderately aggressive corrosion environment	$3\sqrt{f'c}$ psi tension	$7.5\sqrt{f'c}$ psi tension	LRFD Table 5.9.4.2.2-1 Seg Guide Spec 9.2.2.2 FDOT no distinction for Environ't
● 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.)	No Tension	No Tension	Ditto and FDOT Seg. Rating Criteria
● Type B joints (dry joints - no epoxy); external tendons:	100 psi min comp	No Tension	Seg Guide Spec 9.2.2.2 FDOT Seg. Rating Criteria
Transverse Tension, Bonded PT: ● 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	$3\sqrt{f'c}$ psi tension	$6\sqrt{f'c}$ 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
Tensile Stress in Other Areas: ● Areas without bonded reinforcement	No tension	No tension	Seg Guide Spec 9.2.2.4 LRFD Table 5.9.4.2.2-1
● 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 0.5fy (< 30 ksi)	$6\sqrt{f'c}$ psi tension	$6\sqrt{f'c}$ psi tension	Seg Guide Spec 9.2.2.4 LRFD Table 5.9.4.2.2-1
Principal Tensile Stress at Neutral Axis in Webs (Service III): ● All types of segmental or beam construction with internal and/or external tendons.*	$3\sqrt{f'c}$ psi tension	$4\sqrt{f'c}$ psi tension	FDOT LRFR Rating Criteria
* 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 $4.5\sqrt{f'c}$ 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.			

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
Compression (Longitudinal or Transverse): ● Compressive stress under effective prestress, permanent loads, and transient loads	$0.60f'c$	$0.60f'c$	LRFD Table 5.9.4.2.1-1 Seg Guide Spec 9.2.2.1
● 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)			Seg Guide Spec 9.2.2.1
Longitudinal Tensile Stress in Precompressed Tensile Zone: (Intended for Pre and Post-Tensioned Beams and similar construction) For components with bonded pretressing tendons or reinforcement that are subject to not worse than: For (a) an aggressive corrosion environment and (b) moderately aggressive corrosion environment	$3\sqrt{f'c}$ psi tension	$7.5\sqrt{f'c}$ psi tension	LRFD Table 5.9.4.2.2-1 and FDOT FDOT no distinction for Environ't
For components with unbonded prestressing tendons	No Tension	No Tension	LRFD Table 5.9.4.2.2-1
Longitudinal Tensile Stress through Joints in Precompressed Tensile Zone: (Intended for Segmental and similar construction) ● Type A joints with minumum bonded auxiliary longitudinal reinforcement sufficient to carry the calculated longitudinal tensile force at a stress of 0.5fy; for internal and/or external PT (e.g.cast-in-place construction) For (a) an aggressive corrosion environment and (b) moderately aggressive corrosion environment	$3\sqrt{f'c}$ psi tension	$7.5\sqrt{f'c}$ psi tension	LRFD Table 5.9.4.2.2-1 Seg Guide Spec 9.2.2.2 FDOT no distinction for Environ't
● 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.)	No Tension	No Tension	Ditto and FDOT Seg. Rating Criteria
● Type B joints (dry joints - no epoxy); external tendons:	100 psi min comp	No Tension	Seg Guide Spec 9.2.2.2 FDOT Seg. Rating Criteria
Transverse Tension, Bonded PT: ● 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	$3\sqrt{f'c}$ psi tension	$6\sqrt{f'c}$ 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
Tensile Stress in Other Areas: ● Areas without bonded reinforcement	No tension	No tension	Seg Guide Spec 9.2.2.4 LRFD Table 5.9.4.2.2-1
● 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 0.5fy (< 30 ksi)	$6\sqrt{f'c}$ psi tension	$6\sqrt{f'c}$ psi tension	Seg Guide Spec 9.2.2.4 LRFD Table 5.9.4.2.2-1
Principal Tensile Stress at Neutral Axis in Webs (Service III): ● All types of segmental or beam construction with internal and/or external tendons.*	$3\sqrt{f'c}$ psi tension	$4\sqrt{f'c}$ psi tension	FDOT LRFR Rating Criteria

* 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 $4.5\sqrt{f'c}$ 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.

Table 8.1.B1 - Load Factors for Beam Bridges

			LRFD Dead and Permanent Loads			LRFD Transient Loads					Inventory (A)	Operating (A)			
											Design Loads	Design Loads	Legal Loads	FDOT Permit Loads	
	Nomenclature per LRFD:		DC	DW	EL including PT sec	FR	TU (B) CR SH	TG (B) Inv.	TG (B) Oper.		LC #1, #2	LC #3, #4	LC #5, #6, #7	LC #8, #9, #10	LC #11
LONGITUDINAL	**STRENGTH I**	γ =	1.25	1.50	1.00	1.00	0.50	0.00	0.00	γ_L =	1.75	1.35	1.35	-	-
LONGITUDINAL	**STRENGTH II**	γ =	1.25	1.50	1.00	1.00	0.50	0.00	0.00	γ_L =	-	-	-	1.35	1.15
LONGITUDINAL	**SERVICE I**	γ =	1.00	1.00	1.00	1.00	1.00	0.50	0.00	γ_L =	1.00	1.00	1.00	1.00	1.00
LONGITUDINAL	**SERVICE III**	γ =	1.00	1.00	1.00	1.00	1.00	0.50	0.00	γ_L =	0.80	0.80	0.80	0.80	1.00
TRANSVERSE OR LOCAL DETAILS	**STRENGTH I**	γ =	1.25	1.50	1.00	n/a	n/a	n/a		γ_L =	1.75	1.35	1.35	-	-
TRANSVERSE OR LOCAL DETAILS	**STRENGTH II**	γ =	1.25	1.50	1.00	n/a	n/a	n/a		γ_L =	-	-	-	1.35	1.15
TRANSVERSE OR LOCAL DETAILS	**SERVICE I**	γ =	1.00	1.00	1.00	n/a	n/a	n/a		γ_L =	1.00	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 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.B.

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) No. of Ld. Lanes = No. of Design Lanes per LRFD
(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
1	Trans. (T)	1.20	1.00	1.00	1.00
2	L or T	1.00			
3	L or T	0.85			
$\geq$ 4	L or T	0.65			

Table 8.1.B2 - Load Combinations for Beam Bridges

				Inventory		Operating								
				Design Loads (A)		Design Loads (A)		Legal Loads (A)			FDOT Permit Loads (A)			
											Annual Permits, Mixed Traffic (C)			Permit Only
			LOAD COMBINATION (LC) NO.:	**#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 fooprint 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 One Type 3-3 Vehicle times 0.75 *plus* 100% of 0.2 k/lf uniform load	LRFR 6.4.4.2.1							X				
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 deck 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 an enhanced live load distribution and improved load rating. For fully continuous structures, a more sophisticated analysis of this type does not eliminate the need for a time-dependent construction analysis to determine overall longitudinal effects from permanent conditions (e.g. BD 2 analysis).

For simply-supported structures and for structures with deck slabs continuous only for live load (i.e. “poor-boy” joints), a time-dependent construction analysis may be used to establish enhanced permanent compression stress levels as an alternative procedure to forces and stresses found by routine application of formulae in the AASHTO LRFD Code.

9.4 Stiffness of Traffic Barrier (Exception)

Barrier stiffness should be considered and appropriately included if necessary. Inclusion of the barriers acting compositely with the deck slab and beams 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 deck slab and to that of the beams should be taken into account. The presence of joints in a barrier reduces the overall effective section at the joint to that of the deck slab plus beam. This may result in a local concentration of longitudinal stress that should be appropriately considered.

Nevertheless, load ratings should benefit from reasonable consideration of barrier stiffness.

9.5 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.B). 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 0.5fy, 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.

9.6 Principal Tensile Stress (Variance)

If the load rating based upon the limiting principal tensile stress at the neutral axis of the basic beam or composite section 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. 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 affect.

9.7 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: γ_{DC} = 1.25.

(b) As-built dimensions, deck slab thickness and build-up using concrete density determined from construction records, adjusted for weight of embedded reinforcing: γ_{DC} = 1.15.

(c) Actual beam weights measured during construction: γ_{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.

9.8 Reduced Superimposed (DW) Dead Load (Exception)

The load factor for superimposed dead loads including wearing surface and utilities is normally γ_{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 γ_{DW} = 1.25 (See LRFR – notes to Table 6-1, October 2003).

Chapter 10 – Strengthening

Post-tensioned beam bridges are not designed to incorporate provisions for future strengthening or for adding tendons in the future as is normal practice for segmental bridges. The effects of the additional post-tensioning on the existing bridge should be evaluated prior to strengthening.

Longitudinal post-tensioning bars have been added to some beam bridges to enhance capacity, usually as a consequence of a mistake or omission during construction. In such cases, it has been relatively simple, technically, to add external concrete blisters to accommodate bar anchorages. Blisters are secured to the existing beam by drilling for reinforcing anchors that are usually a specialty item from an approved commercial manufacturer. In one case, PT bars were installed longitudinally in the deck slab prior to casting, over a pier, to compensate for a draped tendon that could not be installed in a web due to a duct blockage. Such remedies are relatively straightforward.

If it were necessary to increase longitudinal capacity for additional loads then systems of draped external tendons could be installed on some types of continuous spliced-girder bridges.

Strengthening of local details, such as splices, hinges, diaphragms or pier caps, typically includes adding external post-tensioning to offset unexpected cracking. Anchor blocks must be properly installed to properly disperse the locally added post-tensioning and effectively strengthen the local detail.

Appendix – Examples

Examples are given for a continuous three-span, main channel unit of a bulb-T girder bridge, (Figure A1). The spans are 160 – 200 – 160 feet. The bulb-T girders are 6'-0" deep with an extra 2'-0" depth of bottom flange haunch at the two interior piers. There are five girders in the cross section spaced at 9'-8" centers. A reinforced concrete deck slab of 7-1/2" thickness provides an overall width of 46'-10" including barriers. The build-up over the top of the bulb-T beam varies slightly with camber and cross-slope, but has an average thickness of 1-7/8".

Each haunched portion is approximately 95 feet long. It was precast with top pretensioning strands, and erected in asymmetric cantilever with approximately 50 feet extending into the side spans. The out-of-balance load was supported by a temporary tower at the location of the side-span splice joint. After erecting and splicing the side-span portions of constant depth girder, the main span, "drop-in" portion was erected and spliced.

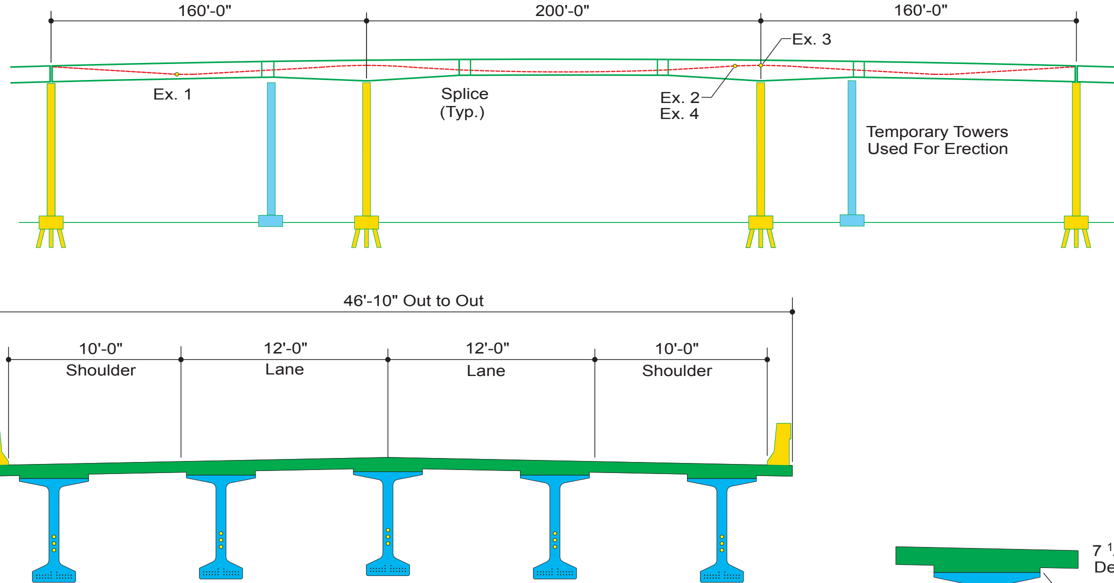
Each beam contains three tendons of draped longitudinal profile. Two longitudinal post-tensioning tendons were installed and tensioned from end to end of the unit, acting only upon the girder section itself. The deck slab was then cast in a series of longitudinal phases that minimized the tensile effects in the deck slab itself over the interior piers. Finally, an additional longitudinal tendon was installed and tensioned on the composite section.

The four key ratings required for beam bridges are (Chapter 8):

At the Service Limit State:
- Longitudinal Flexure
- Principle Web Tension

At the Strength Limit State:
- Longitudinal Flexure
- Web Shear

To determine the permanent structural forces and effects, a complete time-dependent, construction-phase computer model was assembled and analyzed. This accounted for:
- Self-weight of the precast beam elements
- Internal forces and stresses from pre-tensioning strands in precast elements only
- Support reactions at the temporary towers
- Weight of forms and concrete in girder splices
- First stage of longitudinal internal draped post-tensioning on the non-composite girder section from end to end of the three span structure
- All losses in pre- and post-tensioning forces due to elastic shortening, anchor set, friction, wobble, steel relaxation, creep and shrinkage from the time of casting prior to casting the deck slab
- Release of the temporary support tower reactions
- Weight of formwork and wet concrete applied in the actual longitudinal sequence that minimized longitudinal tensile stresses in the deck slab itself over the interior piers
- Longitudinally, the change in section properties from non-composite to composite as portions of the slab hardened and cured



SECTION PROPERTIES				
Property	At Midspan		At Pier (Haunch)	
	Beam	Composite	Beam	Composite
A (ft^2)	6.46	12.01	11.20	16.75
I (ft^4)	31.25	69.20	72.64	172.17
$c_{top\ slab}$	——	2.26	——	3.81
$c_{top\ beam}$	3.12	1.48	4.74	3.03
$c_{bot\ beam}$	2.88	4.52	3.15	4.87
$Q_{NA\ comp}$	——	12.84	——	25.41

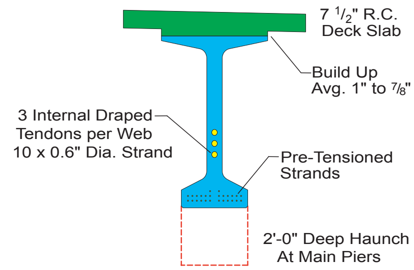


Figure 1: Continuous Three Span Main Channel Unit - Haunched Bulb-Tee Girder Bridge

- The second (final) stage of longitudinal post-tensioning was introduced on the composite section from end to end after the deck slab was completely cast
- The change from un-bonded to bonded properties for the internal longitudinal tendons upon grouting
- Additional prestress losses in the pre- and first and second stages of post-tensioning due to further elastic shortening, friction, wobble, relaxation, creep and shrinkage, etc.
- The effect of the eccentricity of the deck slab relative to the centroidal axis of the non-composite section and the difference in concrete strength, maturity and shrinkage

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

Live load effects were determined using the FDOT “BRUFEM” finite element computer program. For this particular structure, it was determined that the maximum effects in one girder (the first interior girder) occur under the application of two lanes of live load, rather than one or three lanes, after allowing for the appropriate multiple presence factors.

The examples have been chosen deliberately to illustrate various nuances that affect the choice of key parameters, such as, the number of live load lanes which determines the multiple presence factor and the associated live load factors (Table 8.1.B1), load combinations (Table 8.1.B2) and allowable stresses (Table 8.2.B). The examples are:

1. Longitudinal Service Flexure at Inventory: The example is that for the controlling section for Inventory Rating at service flexure. It illustrates the use of the live load factor of 0.80 for bottom fiber tension and when thermal gradient is taken into account.

2. Longitudinal Flexural Strength: This example shows the limiting AAHSTO Type 3-3 Vehicle for negative moment over an interior pier. This load case is not a controlling one but the example illustrates the selection of condition and system redundancy factors.

3. Principal Web Tension at Service: This example is for the T160 Permit Vehicle alone and in mixed (HL93) traffic. The ratings are calculated on the basis of the shear stresses that induce the maximum allowable principal tensile stress determined from a Mohr's circle analysis. (The latter is according to Appendix B of Volume 10 A).

4. Web Shear Strength: For the SU4 Legal Load, this example illustrates the use of the condition and system factors in the context of post-tensioned beams.

In the examples, flexural strength capacities have all been calculated according to formulae in AASHTO LRFD. Shear strength capacities have been determined according to the Modified Compression Field Theory (MCFT) of LRFD.

Service longitudinal flexural stress conditions are given for the top of the deck slab, top of the girder and the bottom of the girder. Principal tensile stress conditions were checked at the location of the neutral axis of the beam and the composite section; the results are for the latter which was found to be the more critical.

Example 1: Longitudinal Service Flexure at Inventory

Bulb-T Girder Bridge - Channel Unit of Three Continuous, Haunched, Spans
Consider Inventory Rating for section of Maximum Positive Moment in First End Span.

Capacity for continuously reinforced PT beam: Tension = $3\times\sqrt{f'c}$ = 34.83 ksf
Compression = 0.6 f'c = 561.60 ksf

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

Top Slab	=	29.7 ksf compression
Top Beam	=	317.1 ksf compression
Bottom	=	165.8 ksf compression

Stress from full positive non-linear thermal gradient (100% TG):

Top Slab	=	92.94 ksf compression
Top Beam	=	10.05 ksf tension
Bottom	=	13.48 ksf tension

Critical Stress from two lanes HL93 with impact (from BRUFEM):

Top Slab	=	88.57 ksf compression
Top Beam	=	59.45 ksf compression
Bottom	=	164.57 ksf tension

For Inventory Condition number of lanes loaded = 2 (3 lanes with live load transverse distribution and smaller multiple presence factor is less critical), so m = 1.00 (Tab.8.1.B1)

Rating Factor Calculation:

Top Slab: Compression (Service I)
(LC #1, Table 8.1.B2)
(γ_{LL} = 1.00, Tab. 8.1.B1)

$$RF = \frac{C - \sigma_{Perm} - \gamma_{TG} \times \sigma_{TG}}{m \times \gamma_{LL} \times \sigma_{LL}}$$

$$\therefore RF = \frac{561.6\text{ ksf} - 29.7\text{ ksf} - 0.5 \times 92.94\text{ ksf}}{1.00 \times 1.00 \times 88.57\text{ ksf}} = 5.48$$

Top Girder: Compression (Service I)
(LC #1, Table 8.1.B2)
(γ_{LL} = 1.00, Tab. 8.1.B1)

$$RF = \frac{C - \sigma_{Perm} - \gamma_{TG} \times \sigma_{TG}}{m \times \gamma_{LL} \times \sigma_{LL}}$$

$$\therefore RF = \frac{561.6\text{ ksf} - 317.1\text{ ksf} - 0.5 \times -0.3 \times -10.05\text{ ksf}}{1.00 \times 1.00 \times 59.45\text{ ksf}} = 4.09$$

(The "-0.3" is for negative thermal gradient effect that induces compression not tension.)

Bottom: Tension (Service III)
(LC #1, Table 8.1.B2)
(γ_{LL} = 1.00, Tab. 8.1.B1)

$$\therefore RF = \frac{-34.83\text{ ksf} - 165.8\text{ ksf} - 0.5 \times -13.48\text{ ksf}}{1.00 \times 0.80 \times -164.57\text{ ksf}} = 1.47$$

Top compression is of minor concern compared to bottom tension in this particular case.

Example 2: Longitudinal Flexural Strength

Bulb-T Girder Bridge - Channel Unit of Three Continuous, Haunched, Spans
Consider Operating Rating for AASHTO 3-3 Vehicle for Negative Moment over Interior Pier.

Negative Flexural Capacity (per LRFD 5.7.3.2.1) at critical section, from a combination of post-tensioning tendons in the top of the girders and deck slab longitudinal rebar over pier, is:

$$C = M_n = -20{,}616 \text{ k-ft}$$

Factored permanent loads from time dependent computer analysis:

		From Table 8.1.B1	
DC	-7117 k-ft	$\gamma = 1.25$	= -8,896 k-ft
DW	0 k-ft	$\gamma = 1.50$	= 0 k-ft
EL	548 k-ft	$\gamma = 1.00$	= 548 k-ft
CR	1340 k-ft	$\gamma = 0.50$	= 670 k-ft
SH	95 k-ft	$\gamma = 0.50$	= 47 k-ft
			-7,631 k-ft

Live Load Combination #7, Table 8.1.B2:

AASHTO Critical Limiting Load Moment from AASHTO 3-3 loading, from BRUFEM, = -1,160 k-ft
This is a negative moment region. The moment is for two Type 3-3 Trucks at 75% plus uniform lane load of 0.64 kip per LF in each live load lane - See Chapter 8.2.3.

For Strength I: Number of live load lanes = 2 (3 lanes has reduced effect from transverse live load distribution and a smaller multiple presence factor), so m = 1.00 (Table 8.1.B1)

γ_{LL} = 1.35 per Table 8.1.B1

φ = 1.00 per LRFD article 5.5.4.2.1

φ_c = 1.00 per Section 7.1.2. (Internal tendons, older, no leaks or stains)

φ_s = 1.10 per Table 7.3 (5 beams, 3 tendons per web, End Span over pier)

Rating Factor Calculation:
(Strength I)

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

$$\therefore RF = \frac{1.0 * 1.1 * 1.0 * -20{,}616 \text{ k-ft} - -7{,}631 \text{ k-ft}}{1.00 * 1.35 * -1{,}160 \text{ k-ft}} = 9.60$$

In this case, the result is very much greater than 1.00 and is of no concern. Had the result been a value less than 1.00, it would have been necessary to reduce similar rating factors for all FDOT Legal Loads by multiplying by this less than 1.00 value (see Chapter 8.2.3).

Example 3: Principal Tension in Web at Service

Bulb-T Girder Bridge - Channel Unit of Three Continuous, Haunched, Spans
Consider Permit Vehicle T160 Rating at End of the Interior Span.

Using Mohr's circle, an allowable principal tension stress of $4 \times \sqrt{f'c}$ leads to an effective shear stress capacity of, $v_{Perm} + v_{LL\ Max}$ = 85.27 ksf (By Appendix B.2 of Volume 10 A).

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} = 52.50 ksf.

Shear stress from one lane T160 Permit Vehicle only with 33% impact, v_{T160} = 13.81 ksf.

Shear stress from one lane HL93 positioned one lane over from the permit vehicle with 33% impact, v_{HL93} = 13.79 ksf.

For permit vehicle alone, m = 1.00. For combined loading at Operating, controlling number of lanes loaded = 2, therefore, m = 1.00 per Table 8.1.B1. (If 3 lanes are loaded, the reduced multiple presence factor of m = 0.85 and the lateral live load distribution, using BRUFEM, leads to a less conservative result.)

Rating Factor Calculation:

Permit Alone: Principal Tension (Service III)
(LC #11, Table 8.1.B2 and γ_{LL} = 1.00, Table 8.1.B1)

$$RF = \frac{(\nu_{Perm} + \nu_{LL\_Max}) - \nu_{Perm}}{m \times \gamma_{LL} \times \nu_{T160}}$$

$$\therefore RF = \frac{85.27\text{ ksf} - 52.50\text{ ksf}}{1.00 \times 1.00 \times (13.81\text{ ksf})} = 2.37$$

Permit & Design Load: Principal Tension (Service III)
(LC #8 plus #9, Table 8.1.B2 and γ_{LL} = 0.80, Table 8.1.B1)

$$RF = \frac{(\nu_{Perm} + \nu_{LL\_Max}) - \nu_{Perm}}{m \times \gamma_{LL} \times (\nu_{T160} + \nu_{HL93})}$$

$$\therefore RF = \frac{85.27\text{ ksf} - 52.50\text{ ksf}}{1.00 \times 0.80 \times (13.81\text{ ksf} + 13.79\text{ ksf})} = 1.48$$

Refer to Chapter 9, "Posting Avoidance", for possible options to avoid restrictions if, by the above method, a Permit Rating is less than 1.00.

Example 4: Shear Strength Check

Bulb-T Girder Bridge – Channel Unit of Three Continuous, Haunched, Spans
FDOT SU4 Legal Load Rating – Shear at the End of Interior Span

Shear Capacity using LRFD Modified Compression Field Theory (MCFT):

$$C = V_n = 506 \text{ kips}$$

Factored permanent loads from time dependent computer analysis:

From Table 8.1.B1

DC	188.3 kips	γ = 1.25	= 235.4 kips
DW	0.0 kips	γ = 1.50	= 0.0 kips
EL	-14.3 kips	γ = 1.00	= -14.3 kips
PT	-10.3 kips	γ = 1.00	= -10.3 kips
CR	0.0 kips	γ = 0.50	= 0.0 kips
SH	0.2 kips	γ = 0.50	= 0.1 kips
			210.9 kips

Live Load Combination #5, Table 8.1.B2:

Critical Shear from FDOT SU4 vehicle = 86.46 kips

γ_{LL} = 1.35 as per Table 8.1.B1

For Strength 1 number of lanes = 2 (3 lanes could have been used but due to a reduced effect from transverse distribution and a smaller multiple presence factor 2 lanes controls). As a result, m = 1.00 per Table 8.1.B1.

φ = 0.85 per LRFD Table 5.5.4.2.2-1

φ_c = 1.00 per Section 7.1.2 (Internal tendons, older, no leaks or stains)

φ_s = 1.15 per Table 7.3 (5 beams, 3 tendons per web, Interior Span)

Rating Factor Calculation:
(Strength I)

$$RF = \frac{\phi \times \phi_S \times \phi_C \times V_n - V_{Perm}}{m \times \gamma_{LL} \times \sigma_{LL}}$$

$$\therefore RF = \frac{0.85 * 1.15 * 1.0 * 506 \text{ kips} - 210.9 \text{ kips}}{1.00 * 1.35 * 86.46 \text{ kips}} = 2.43$$

Parametric studies indicate that Shear Capacity determined by Modified Compression Field Theory, is less conservative than that calculated according to section analysis using the traditional Standard Specification (16th Edition).