

January 1, 1999 Revisions

to the

**STRUCTURES DESIGN GUIDELINES
FOR
LOAD AND RESISTANCE FACTOR DESIGN**

Topic No.: 625-020-150-c

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M-E-M-O-R-A-N-D-U-M

TO: All Recipients of the 1998 Version of the Structures Design Guidelines for Load and Resistance Factor Design (Topic No.: 625-020-150-c)

FROM: William N. Nickas, P.E., State Structures Design Engineer

COPIES: R. E. Nichols, Freddie Simmons

SUBJECT: January 1, 1999 Revisions

We are enclosing the subject January 1, 1999 Revisions to the 1998 Metric (SI) version of the Structures Design Guidelines for Load and Resistance Factor Design. In addition to editorial revisions and coordinated revisions to the general sections of "Table of Contents," "List of Figures," "LRFD/SDG Cross-Reference Index," and "Key Word Index," the following specific administrative and technical modifications have been made:

Chapter I - Introduction:

1. The existence and responsibilities of the previously described Technical Committees of the Structures Design Office have been terminated, and, in lieu thereof, those same responsibilities regarding the Structures Design Guidelines are assigned to SDO Staff.
2. Articles I.9 and I.9.1 describe the SSDE's capability, authority and limitations regarding the issuance of temporary "Design Bulletins."

Chapter 3 - Loads and Load Factors:

1. Article 3.2.1 clarifies and redefines the Load Factor for Temperature Gradient for continuous concrete structures.
2. Similarly, the new Article 3.8.4 restricts the effect of Temperature Gradient to be applied to continuous concrete structures only.

Chapter 5 - Substructure Elements and Appurtenances:

1. Article 5.4 contains a new, inserted second paragraph which defines concrete components for which Mass Concrete requirements apply. The definition was inadvertently omitted from the current LRFD version.

Chapter 7 - Concrete Structures:

All Registrants of the FDOT's Structures Design Guidelines

January 5, 1999

Page Two

1. Article 7.2.1 contains the revised minimum thickness of cast-in-place deck slabs for beam and girder bridges to be consistent with Specification rideability requirements.
2. In Table 7.2, the value for Modulus of Elasticity of Class VI Concrete has been corrected.
3. In Article 7.4.3.C.6 on Page No. 7-6, the use of time-dependent, inelastic creep and shrinkage is explicitly and specifically **disallowed** in the design of simple span prestressed beams. A commentary is also provided.
4. In Article 7.4.3.C.7 on Pages Nos. 7-6 & 7-7, the use of transformed section properties in the design of simple span prestressed beams is now **required**.
5. Article 7.15 for designing adhesive anchor systems has been completely rewritten subsequent to completion of the testing program conducted at the University of Florida this year. The Article contains criteria for designing anchors that are subject to tensile stress, shear stress, or a combination thereof, and three design examples have been included. To use Article 7.15, however, the anchor system must comply with the new Section 937 of the Specifications.
6. Article 7.18 regarding the design criteria for continuous concrete I-Girder superstructures has been added. Values for ultimate creep and shrinkage strain are provided as is the required beam age to use when the deck is cast. The criteria of this Article **must** be used for all continuous prestressed concrete I-Girder superstructures.

Chapter 8 - Steel Bridge Structures:

1. Article 8.13 describes the SDO's structural preference for designing and detailing bearing stiffeners which is consistent with the latest industry practice.

Please remove these same pages from your current SDG-LRFD copy and replace with the enclosed copies. Sufficient extra copies of this memorandum and enclosures have been included for each hardcopy and each CD-ROM that you originally ordered.

Please also note that the SDO's Web Page will contain an electronic version of these same revisions both separately as a stand-alone file and as page replacements in a consolidated January 1, 1999 version of the Structures Design Guidelines.

WNN:ren

STRUCTURES DESIGN GUIDELINES

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INTRODUCTION

STRUCTURES DESIGN GUIDELINES

I.1 PURPOSE:

This manual, the Structures Design Guidelines (SDG), sets forth the basic Florida Department of Transportation (FDOT) design criteria that are exceptions to those included in the AASHTO/LRFD Bridge Design Specifications published in SI Units by the American Association of State Highway and Transportation Officials (AASHTO) for Load and Resistance Factor Design (LRFD), and hereinafter simply referred to as "LRFD." These exceptions may be in the form of deletions from, additions to, or modifications of the LRFD specifications. These two documents must always be used jointly and must be utilized in concert with the FDOT's Plans Preparation Manual (Topic No.: 625-000-005) in order to prepare properly the contract plans and specifications for structural elements and/or systems that are included as part of the construction work in FDOT projects. Such elements and/or systems include, but are not necessarily limited to, bridges, overhead sign structures, earth retaining structures, and miscellaneous roadway appurtenances.

I.2 AUTHORITY:

Section 334.044(2), Florida Statutes.

I.3 SCOPE:

This manual is required to be used by anyone performing structural design or analysis for the Florida Department of Transportation.

I.4 GENERAL:

This manual is prepared and presented in 8-1/2 x 11 format and consists of text, figures, charts, graphs, and tables as necessary to provide engineering standards, criteria, and guidelines to be used in developing and designing structures for which the Structures Design Office (SDO) has overall responsibility. The manual is written in the same general format and specification-type language as LRFD and includes article-by-article commentary when appropriate. In addition to its companion documents referenced in Article I.1 above, the manual is intended to compliment and be used in conjunction with the SDO's Detailing Manual (Topic No.: 625-020-200), Standard Drawings (Topic No.: 625-020-300), and CADD User's Manual.

I.5 DISTRIBUTION:

This SDG is furnished to FDOT personnel at no charge, but only upon request. Generally, for registrants external to the Department, first-time acquisition of the manual and subsequent, total reprints must be obtained by purchase from the address furnished below. At the discretion of the SDO, minor, intermediate modifications to the manual in the form of individual pages to be inserted into a previously printed version may be sent to all manual registrants free of charge.

This manual and any modifications thereto may be obtained from the following office and address:

Structures Design Office
Mail Station 33
605 Suwannee Street
Tallahassee, Florida 32399-0450
Tel.: (850) 414-4255

I.6 REGISTRATION:

Each copy of the manual and each set of intermediate modifications thereto will include a registration page. This page must be completed by the authorized, registered user and returned to the address indicated in order for the registrant to be assured of obtaining notification of future editions of the manual and/or any intermediate modifications.

I.7 ADMINISTRATIVE MANAGEMENT:

Administrative Management of the Structures Design Guidelines (SDG) occurs through the cooperative effort of SDO staff and the nine voting members of the Technical Advisory Group (TAG).

I.7.1 The Technical Advisory Group (TAG):

The TAG provides overall guidance and direction for the SDG and has the final word on all proposed modifications. The TAG comprises the State Structures Design Engineer (SSDE) and the eight District Structures Design Engineers (DSDE). In matters of technical direction or administrative policy, when unanimity cannot be obtained, each DSDE has one vote, the SSDE has two votes, and the majority rules.

I.7.2 SDO Staff:

SDO Staff comprises the Assistant State Structures Design Engineers and Senior Structures Design Engineers selected by the SSDE.

I.8 MODIFICATIONS AND IMPROVEMENTS:

All manual users are encouraged to suggest modifications and improvements to the SDG. The majority of modifications to the manual that become necessary are the direct result of changes in FDOT specifications, FDOT organization, Federal Highway Administration (FHWA) regulations, and AASHTO requirements; or occur from recent experience gained during construction, through maintenance, and as a result of research. Many other improvements to the manual, however, have been suggested by users. This has been particularly true with suggestions to improve the clarity of the text and to include design criteria not previously addressed by the manual or by any accompanying code or specification. Suggestions to modify and improve the manual should be transmitted in writing to the SSDE at the distribution address included above.

Modifications and improvements to the SDG can occur as adopted revisions through either of the two processes described hereinafter in "Adoption of Revisions."

I.9 ADOPTION OF REVISIONS:

Revisions to the SDG occur either as Temporary Design Bulletins issued by the SSDE or as Permanent Revisions according to a formal adoption process. Temporary Design Bulletins provide the SDO with the flexibility to address in an efficient and responsive manner any mandatory modification or any modification considered by the SSDE to be essential either to production or to structural integrity issues. Temporary Design Bulletins are active for the limited time period of 360 days after the date of issuance after which time they automatically terminate. At any time during their life, Temporary Design Bulletins may be proposed by the SDO as Permanent Revisions to the SDG.

I.9.1 Temporary Design Bulletins:

A Temporary Design Bulletin is a revision to the SDG that is deemed by the SSDE to be mandatory and in need of immediate implementation. The conditions that dictate the implementation of a Temporary Design Bulletin may comprise issues in such bridge design areas as plans production, safety, structural design methodology, critical code changes, or new specification requirements.

Temporary Design Bulletins supercede the requirements of the current version of the SDG to which they apply and may be issued by the SDO at any time. Temporary Design Bulletins

must reference the particular portion or portions of the SDG that will be affected and may include commentary regarding the need for their issuance. Temporary Design Bulletins are not official or effective until signed by the State Structures Design Engineer.

Temporary Design Bulletins are effective for a period of time not exceeding 360 calendar days. They become null and void after the 360-day period, by issuance of subsequent Permanent Revisions to the SDG, or when superseded by a subsequent Temporary Design Bulletin. Temporary Design Bulletins automatically become proposed Permanent Revisions unless withdrawn from consideration by the SSDE.

Temporary Design Bulletins will indicate their effective date of issuance, will include the reference Topic Number, be numbered sequentially with reference to both the SDG version number and their year of issuance, and be issued on color-coded paper. For example, Temporary Design Bulletin No. C98-2 would be the second Bulletin issued in 1998 for SDG Topic Number 625-020-150-c.

I.9.2 Permanent Revisions:

Permanent Revisions to the manual are made on an "as-needed" basis but not less frequently than once yearly. If an individual revision, or an accumulation of several revisions, is considered by the SDO to be substantive, a complete reprinting of the manual will occur. Otherwise, color-coded pages for insertion into the most recently printed manual will be published. The following steps are required for adoption of a revision to the manual.

A. Revision Assessment:

Proposed revisions to the manual, either developed internally by the Department or in response to external suggestions or requirements, will be assessed by SDO Staff for inclusion in the manual. The SDO Staff has the responsibility of developing the initial draft of all proposed SDG modifications for the SSDE's approval.

B. Revision Research and Proposal:

The SDO Staff will conduct the necessary research, coordinate the proposed modification with all other affected offices and, if the proposed modification is deemed appropriate, prepare a complete, written modification with any needed commentary. The SSDE's approval signifies the SDO's position on the proposed modifications.

C. Revision Distribution to TAG and Other Review Offices:

The proposed, completed modification and all other similar, proposed modifications will be collected by the SDO and mailed to each TAG member

such that the revision package will be received by the TAG members not less than two (2) weeks prior to the next scheduled TAG meeting. The DSDE members of TAG are responsible for coordinating the proposed modifications with other all other appropriate offices at the district level.

Concurrent with the distribution to TAG, the assembled, proposed modifications will be transmitted by the SDO to all other appropriate FDOT/FHWA offices affected by the SDG, which offices shall be given the same two (2) weeks to reply to the proposed modifications. These offices include, but are not necessarily limited to: Office of Construction, State Maintenance Office, State Materials Office, State Roadway Design Office, Organization and Procedures, and FHWA.

D. Revision Review by TAG and SDO:

The proposed modifications will be reviewed by each TAG member prior to the meeting. Similarly, the SDO will review all modification comments received from other FDOT/FHWA offices in preparation for presentation at the meeting.

Each proposed modification will be brought forward for discussion at the TAG meeting. Also, any additional review comments received by the SDO and/or DSDE's during the review process will be presented for discussion and resolution.

E. TAG Adoption Recommendation:

Immediately after the TAG meeting, each proposed modification will be returned to the SDO with one of the following three recommendations:

1. Recommended for adoption as presented.
2. Recommended for adoption with resolution of specific changes.
3. Not recommended for adoption.

F. Revision Recommendations to SSDE:

Within two (2) weeks after the TAG meeting, each modification recommended for adoption by the TAG shall have all TAG comments resolved by the SDO Staff, and the assembled modifications shall be provided to the SSDE for final approval.

G. Revision Adoption and Implementation:

Once approved by the SSDE, the SDG revision modifications will be assigned to the Design Technology Group Leader for final editing.

H. Official Distribution of Revisions as New SDG Version:

Within four (4) weeks after receipt of the approved modifications from the SSDE, or when otherwise agreed upon, the SDO shall print and distribute the SDG version modifications. This process includes coordinating with the Organization and Procedures Office for updating of the Standard Operating System and any electronic media prior to hardcopy distribution of the SDG.

I.10 TRAINING:

No specific training is made available for the requirements of this manual.

I.11 FORMS:

No forms are required by this manual.

Chapter 1

GENERAL REQUIREMENTS

1.1 Introduction

These Structures Design Guidelines, hereinafter referred to as “SDG,” incorporate technical design criteria only and supersede the requirements of the AASHTO/LRFD Bridge Design Specifications, First Edition, and the 1996, 1997, and 1998 Interims thereto, hereinafter referred to as “LRFD,” either by additions, deletions, or modifications.

Because the SDG is designed to supplement the technical design requirements of LRFD for bridges and structures for the Florida Department of Transportation, and to help facilitate the plans production process for the Department, all general administrative, geometric, shop drawing, and plans processing requirements have been incorporated in the Plans Preparation Manual (Topic No.: 625-010-105), hereinafter referred to as “PPM.”

1.2 Format

The SDG chapters are organized more by “component,” “element,” or “process” than by “material” as for the LRFD. As a result, the chapter numbers and content of the SDG do not necessarily align themselves in the same order or with the same number as LRFD. However, LRFD references are provided whenever possible so that the designer may quickly coordinate and associate SDG criteria with that of LRFD. The LRFD references may occur within article descriptions, the body of the text, or in the commentary. Such references are shown in bold-face type within brackets; i.e., **[1.3]**, **[8.2.1]**, etc.

1.3 Coordination

The designer shall fully coordinate all plans production activities and requirements between the SDG, PPM, and LRFD. Each of these documents has criteria pertaining to bridge or structures design projects, and, normally, all three must be consulted to assure proper completion of a project for the Department.

Questions concerning the applicability or requirements of any of these or other referenced documents to a structures project for the Department shall be addressed to the appropriate District Structures Design Engineer for resolution.

1.4 Special Requirements for Box Girder Inspection and Access

Accessibility to box girders and the safety of bridge inspectors and maintenance personnel shall be appropriately evaluated during preliminary engineering and shall be adequately considered when selecting the configuration of the structure.

1.4.1 Box Girder Height [2.5.2.6.3]

The minimum desirable interior, clear height of box girders for inspection and maintenance access is 1.9 meters. Any height less than 1.9 meters will require approval from the SDO. In structures where the box depth required by structural analysis is less than 1.5 meters, the SDO and the District Structures and Facilities Engineer shall be consulted for a decision on the box height and access details.

1.4.2 Electrical Service

Box girder bridges shall include interior lighting and electrical outlets in all boxes which shall be detailed in accordance with SDO's Standard Drawings, Index 512.

1.4.3 Inspection and Maintenance Access

Exterior access to box girders shall be provided through the bottom flange near the abutments, and near the end of each continuous unit. Exterior access openings and interior openings in diaphragms, etc. preferably shall have minimum rectangular dimensions of 815 mm by 1100 mm. The openings shall **not** be used for utilities or other attachments. Exterior openings shall be covered by a hinged door that opens inward, shall have a hasp and lock, and shall be located in the bottom flange. Bottom flange access openings shall be sized and located with consideration given to the structural effects on the girder and to the safety of the inspectors and traveling public. Access locations over traffic lanes and locations that will require extensive maintenance of traffic operations shall be avoided.

1.4.4 Exterior Holes

Ventilation or drain holes of 50 mm minimum diameter shall be provided in each box girder at a spacing of approximately 15 meters or as needed to provide proper drainage. All exterior holes in box girders not covered by a door shall be covered by a screen of not more than 6 mm mesh opening to prevent wildlife from entering the box. This includes holes in webs through which drain pipes pass, ventilation holes, drain holes, and openings in diaphragms at abutments and at expansion joints.

1.5 Special Requirements for Inspection and Access of Other Box Sections

Accessibility to other box sections such as precast hollow pier segments shall be provided in a manner similar to that for box girders, particularly concerning the safety of bridge inspectors and maintenance personnel. As with box girders, the need for proper inspection and access shall be appropriately evaluated during preliminary engineering and shall be adequately considered when selecting the configuration of the structure.

Because of the possible variety of shapes and sizes of hollow sections such as precast concrete pier segments as well as the numerous possible different site constraints and environmental conditions that may exist, each such application shall be considered on an individual, project-by-project basis. In all such cases, however, the SDO shall be contacted for guidance in providing adequate inspection access and safety measures.

1.6 Special Requirements for Movable Bridges

Because the criteria for the mechanical and electrical design aspects of movable bridges are not addressed in LRFD, a comprehensive description of the engineering services required by the Department for preparing plans for movable bridge projects is included in Chapter 10.

Chapter 2

SITE AND MATERIAL DURABILITY CRITERIA

2.1 General

This Chapter clarifies, supplements, and contains deviations from the information in LRFD Sections [2], [5], and [6]. These combined requirements establish material selection criteria for durability to meet the 75-year design life requirement established by the Department.

2.2 Establishment of Environmental Classification

All new bridge sites shall be environmentally classified by the District Materials Engineer or the Department's Environmental/Geotechnical Consultant. Additionally, the need for environmental classification shall be considered as a criterion for minor widenings and is required for major widenings which are defined in Chapter 9. The work shall be accomplished prior to or during the development of the BDR/30% Plans Stage, and the results shall be included in the documents. The bridge site shall be tested, and separate classifications for superstructures and substructures shall be provided.

The bridge plan General Notes shall include the environmental classification for both the superstructure and substructure according to the following classifications:

- Slightly Aggressive
- Moderately Aggressive
- Extremely Aggressive

For the substructure, additional descriptive data is required to supplement the environmental classification. The source and magnitude of the environmental classification parameter(s) resulting in the classification shall be contained in parentheses after the classification.

Commentary: As an example, for a proposed bridge located in a freshwater swampy area where the substructure is determined to be in an Extremely Aggressive environment due to low soil pH of 4.5 and the superstructure to be in a Slightly Aggressive environment, the format on the bridge plans will be:

ENVIRONMENTAL CLASSIFICATION:

*Substructure: Extremely Aggressive (Soil - pH = 4.5)
Superstructure: Slightly Aggressive*

2.2.1 Environmental Classification Parameters

Bridge substructures and superstructures shall be classified either as being situated in Slightly Aggressive, Moderately Aggressive, or Extremely Aggressive environments according to the following criteria.

- A. For substructure components and systems because of the conditions of the soil and/or water:
 - 1. Slightly Aggressive when **all** of the following conditions exist:
 - a. pH greater than 6.6
 - b. Resistivity greater than 3,000 ohm-cm
 - c. Sulfates less than 150 ppm
 - d. Chlorides less than 500 ppm
 - 2. Moderately Aggressive:

This classification shall be used at all sites not meeting requirements for either Slightly Aggressive or Extremely Aggressive Environments.
 - 3. Extremely Aggressive when **any one** of the following conditions exists:
 - a. For concrete structures: pH less than 5.0
 - b. For steel structures: pH less than 6.0
 - c. Resistivity less than 500 ohm-cm
 - d. Sulfates greater than 1,500 ppm
 - e. Chlorides greater than 2,000 ppm
- B. For bridge superstructures, the following environmental classifications apply:
 - 1. Slightly Aggressive:

Any superstructure situated in an environment that is not classified either as Moderately or Extremely Aggressive (see Figure 2-1).
 - 2. Moderately Aggressive:

Any superstructure located within 800 meters of any coal burning

industrial facility, pulpwood plant, fertilizer plant or any other similar industry where, in the opinion of the District Materials Engineer, a potential environmental corrosion condition exists, or any other specific conditions and/or locations described in Figure 2-1.

3. Extremely Aggressive:

Any superstructure situated in an area such that a combination of environmental factors indicates that significant corrosion potential exists, or with specific conditions and/or locations described in Figure 2-1.

2.2.2 Substructure and Superstructure Definitions for Environmental Consideration

The division between the substructure and the superstructure shall be as stated in the FDOT Standard Specifications for Road and Bridge Construction, Section 1, except as noted below.

- A. Box culverts, retaining wall systems (including MSE walls), and bulkheads shall be considered as substructures.
- B. Abutments/end bents shall always be governed by the most stringent condition.
- C. If, at a given site, the superstructure is classified Moderately or Extremely Aggressive and the substructure is classified Slightly Aggressive, all portions of each component of the superstructure exposed to the atmosphere down to the substructure element shall be considered as superstructure.
- D. Approach slabs shall be considered as superstructure; however, Class II Concrete (Bridge Deck) will be used for all environmental classifications.

2.2.3 Chloride Content Determinations

- A. The chloride content of the water and soil at the proposed bridge site will be determined and/or approved by the District Materials Engineer using established procedures and test methods.
- B. The chloride content of major bodies of water within 800 meters of the proposed bridge site will, in most instances, be available in the Department's Bridge Corrosion Analysis database. In the unlikely event chloride values are unavailable, the major body of water must be tested unless it is known to be a freshwater body.
- C. Generally, all superstructures that are situated within line-of-sight and within 800 meters of the Atlantic Ocean or the Gulf of Mexico are subject to increased chloride intrusion rates on the order of $0.019 \text{ kg/m}^3/\text{year}$ at a 50-mm concrete

depth. The intrusion rate decreases rapidly with distance from open waters and/or when obstacles such as rising terrain, foliage or buildings alter wind patterns. In order to optimize the materials selection process, the Designer and/or District Materials Engineer shall have the option of obtaining representative cores for determination of chloride intrusion rates for any superstructure situated within 800 meters of any major body of water containing more than 12,000 ppm chlorides. Cores shall be taken from bridge superstructures in the immediate area of the proposed superstructure and in sufficient number as to represent the various deck elevations. The sampling plan for such structures shall be coordinated with the State Corrosion Engineer. The District Materials Engineer shall be responsible for obtaining cores. The chloride content and intrusion rates of core samples shall be determined by the State Materials Office, Corrosion Laboratory, where a data base of such data is maintained. In the absence of core samples and testing all such superstructures shall be classified as Extremely Aggressive.

After representative samples are taken and tested, Table 2.1 shall be used to correlate the core results (the chloride intrusion rate in $\text{kg}/\text{m}^3/\text{year}$ at a depth of 50 mm) with the classification.

Table 2.1 Chloride Intrusion Rate/Environmental Classification

CHLORIDE INTRUSION RATE	CLASSIFICATION
$\geq 0.019 \text{ kg}/\text{m}^3/\text{year}$	Extremely Aggressive
$< 0.019 \text{ kg}/\text{m}^3/\text{year}$	Moderately Aggressive

2.3 Concrete Structures [5.12.1]

2.3.1 Concrete Cover

The requirements for concrete cover over reinforcing steel are listed in Table 2.2. Examples of concrete cover are shown in Figures 2.2 through 2.5.

When deformed reinforcing bars are in contact with other embedded items such as post-tensioning ducts, the actual bar diameter, including deformations, must be taken into account in determining the design dimensions of concrete members and in applying the design covers of Table 2.2.

2.3.2 Concrete Class and Admixtures for Corrosion Protection

The “General Notes” for both bridge plans and wall plans require the clear identification of, and delineation of use for, concrete class and admixtures used for strength and durability considerations.

A. Concrete Class Requirements:

When the environmental classifications for a proposed structure have been determined, then those portions of the structure located in each classification shall use the class of concrete described in Table 2.3 for the intended use and location unless otherwise directed or approved by the Department.

Unless otherwise specifically designated or required by the FDOT, the concrete strength utilized in the design shall be consistent with the 28-day compressive strength given in the specification.

Commentary: The following example is given:

*Component - submerged piling
Environment - Extremely Aggressive over saltwater
Concrete Class - Table 2.3 Class V (Special)
Quality Control and Design Strength at 28 days - 41 MPa*

B. Admixtures for Corrosion Protection:

Primary components of structures located in Moderately or Extremely Aggressive environments utilize either Class IV, V, V (Special), or VI Concrete. These concrete classes use either fly ash, slag, microsilica, and/or cement type to reduce permeability.

Structures located in Extremely Aggressive marine environments may require additional measures as defined below. These additional measures and their location must be clearly identified in the General Notes. Special Provisions to the specifications may be required for their implementation.

The use of concrete admixtures to enhance durability shall be consistent with these guidelines; however, if deemed necessary the EOR may request that additional measures be used. These additional measures must be approved by the State Corrosion Engineer and the State Structures Design Engineer.

1. Corrosion Inhibitor - Calcium Nitrite:

When the environmental classification is Extremely Aggressive due to the presence of chloride in the water, calcium nitrite shall be employed in accordance with the following criteria:

a. Superstructure:

In all superstructure components situated less than 4.0 meters above Mean High Water.

b. Substructure:

In all pile-bent or pier caps situated less than 4.0 meters above Mean High Water and having concrete cover less than 100 mm.

c. Retaining Walls:

In all retaining walls, including MSE walls, situated less than 4.0 meters above Mean High Water and within 100 meters of the shoreline.

2. Microsilica (Silica Fume):

When the environmental classification is Extremely Aggressive due to the presence of chloride in the water, microsilica shall be employed in the "splash zone" of a structure. The splash zone is the 6.0-meter vertical distance from 2.0 meters below to 4.0 meters above Mean High Water (MHW). Microsilica shall be employed in accordance with the following criteria:

a. Piles of Pile Bents:

In all piles exposed to the splash zone.

b. Columns or Walls of Cofferdam-type Piers:

In all columns or walls exposed to the splash zone.

Commentary: Microsilica concrete must be used in the splash zone and may be used for the entire column height.

2.3.3 Penetrant Sealers

Penetrant sealers shall **not** be used on new bridge structures.

On bridge widening projects where the existing bridge deck does not conform with the current reinforcing steel cover requirement and the superstructure environment is classified as Extremely Aggressive due to the presence of chlorides, the existing bridge deck shall be sealed after grooving. If the existing deck is not to be grooved, then a penetrant sealer shall **not** be used. A note and a sketch describing the concrete surfaces to be coated with

penetrant sealer shall be included in the "General Notes" of the bridge plans. Refer to Chapter 9 for requirements for grooving widened bridges.

2.3.4 Stay-in-Place Forms

Stay-in-place forms shall **not** be used for superstructures located over water containing more than 2,000 ppm chloride; however, for the internal portions of box girders (closed portions between webs), metal stay-in-place forms may be used without exception.

The "General Notes" for each bridge project shall include a note clearly stating whether or not stay-in-place forms are permitted for the project and how the design was modified for their use; e.g., dead load allowance.

2.3.5 Prestressed Concrete Piles

Fender piles generally have a short life expectancy. Fender piles are considered to be sacrificial; therefore, no additional corrosion protection requirements, beyond the use of concrete class as shown in Table 2.3, are required.

Piling used in Extremely Aggressive salt water are specified with a minimum dimension of 610 mm. Using a pile less than 610 mm square can be acceptable subject to the concurrence of the District Maintenance Engineer or his designated representative. This decision is dependent upon site specific conditions and the history of piles in the vicinity. However, if pile bents are exposed to wet/dry cycles such that jacketing will be required in the future, then a 610 mm minimum pile shall be used.

2.3.6 Epoxy Coated Reinforcing Steel

Epoxy coated reinforcing steel shall **not** be used in any bridge element or other highway related structure.

2.4 Steel Structures

2.4.1 Uncoated Weathering Steel

The design and use of weathering steel for bridges shall conform with the guidelines in FHWA Technical Advisory T5140.22 and the criteria stated herein.

- A. Site Locations for Usage:

Suitable locations for the use of weathering steel without long-term, on-site panel testing shall meet or exceed the following criteria:

1. Locations in excess of 800 meters from a salt water body (salt water contains 2,000 ppm chloride or more).
2. Locations in excess of 800 meters from any coal burning plant, pulpwood plant, fertilizer plant, or similar industry.
3. Located in excess of 3.0 meters above any freshwater body.

B. Superstructure Joint and Drainage Considerations:

Generally, weathering steel should be combined with jointless deck construction and, preferably, integral abutments. The use of scuppers for deck drainage shall be minimized. Scuppers or deck drains shall pipe the drainage to the ground or use extended downspouts. Weathering steel requires the use of ASTM A325M Type 3 bolts. Crevice creating details shall be eliminated when possible; therefore, the use of stiffeners and bracing should be minimized.

C. Handling and Cleaning:

Weathering steel shall be cleaned and handled in accordance with the Specifications, Article 460-2. Substructures shall be protected from staining as described in this specification article. Painting of the exterior girder/fascia may be required for aesthetic appearance.

2.4.2 Three-Coat Inorganic Zinc Paint System

All structural steel bridge members that require coatings will receive a three-coat paint system in accordance with FDOT specifications.

2.4.3 Steel Box Girder Interior Painting

When structural steel is to be painted, the interior of box girders will be painted white or gray. The color of zinc primer is acceptable for use.

2.4.4 Existing Structures with Hazardous Material

In order to comply with the regulations of the Occupational Safety and Health Administration (OSHA) and the Environmental Protection Agency (EPA) for the handling and disposal of hazardous materials, it is critical on all projects to determine if any existing structure contains hazardous materials; e.g., lead-based paint, asbestos-graphite bearing pads, etc. The

determination must be made from information provided by the Department about each structure, or by site testing, as appropriate. If an existing structure is determined to contain hazardous materials and any work is to be performed that will disturb the material such as new painting or the repair, demolition, welding, or removal of existing structural members, the project documents will include the necessary requirements for protecting the workers and for proper handling and disposal of the material.

Assistance in preparing the construction documents for those projects that include demolition and removal of members containing hazardous material as well as those that involve overcoating of existing hazardous material such as lead-based paint shall be obtained from the FDOT design project manager. The assistance will include furnishing the technical special provision required for the project and for the particular hazardous material.

NOTE: No project on which any existing structure contains hazardous material shall be developed without adequate provisions for worker safety, handling, storage, shipping, and disposal of the hazardous material and/or its waste.

2.4.5 Galvanizing

A. Galvanizing of Structural Steel Members:

Galvanizing of structural steel and accessories and the application of zinc paint over welded areas of galvanized structural members and over areas of previously galvanized members on which galvanizing has become significantly damaged shall be performed in accordance with the Department's specifications.

B. Galvanizing of Bolts for Bridges:

Normally, all anchor bolts, tie-down hardware, and miscellaneous steel (ladders, platforms, grating, etc.) are to be hot-dip galvanized. While ASTM A307 (coarse thread) bolts shall be hot-dip galvanized, A325M (fine thread) bolts must be mechanically galvanized when utilized either with galvanized steel components or with single-coat inorganic paint systems when slip critical connections are utilized. Other applications not requiring full torquing of the bolts may use hot-dip galvanized A325M bolts.

C. Galvanizing of Bolts for Miscellaneous Structures:

Bolts for connections of structural steel members of Miscellaneous Structures other than bridges, including overhead sign structures, traffic mast arms, ground-mounted signs, etc., shall be ASTM A 325M, Type 1, and shall be hot-dip galvanized.

**Table 2.2 Minimum Concrete Cover Requirements
for Design and Detailing**

ITEM/DESCRIPTION	CONCRETE COVER (mm)	
	S ⁽¹⁾ or M ⁽²⁾	E ⁽³⁾
Superstructure (Precast) Internal and external surfaces (except riding surfaces) of segmental concrete boxes, and external surfaces of prestressed beams (except the top surface).	50	50
Top surface of girder top flange.	25	25
Top deck surfaces.	60 ⁽⁴⁾	60 ⁽⁴⁾
All components and surfaces not included above (including barriers).	50	50
Superstructure (Cast-in-Place) All external and internal surfaces (except top surfaces).	50	50
Top deck surfaces.	60 ⁽⁴⁾	60 ⁽⁴⁾
Substructure (Cast-in-Place) External surfaces cast against earth and surfaces in water.	100	115
External formed surfaces, columns, and tops of footings.	75	100
Internal surfaces.	75	75
Substructure (Precast)	75	100
Substructure (Girder Pedestals)	75	75
Prestressed Piling	75	75
Drilled Shafts	150	150
Retaining Walls (Cast-in-Place or Precast)	50	75
Culverts (Cast-in-Place or Precast)	50	75
Bulkheads (Cast-in-Place)	100	100

- NOTES: (1) S = Slightly Aggressive environment
(2) M = Moderately Aggressive environment
(3) E = Extremely Aggressive environment
(4) Cover dimension includes a 10-mm allowance for milling

Table 2.3 Structural Concrete Class Requirements

CONCRETE LOCATION AND USAGE		ENVIRONMENTAL CLASSIFICATION		
		SLIGHTLY AGGRESSIVE	MODERATELY AGGRESSIVE	EXTREMELY AGGRESSIVE ⁽¹⁾
SUPERSTRUCTURE	CAST-IN-PLACE (OTHER THAN BRIDGE DECKS)	CLASS II	CLASS IV	CLASS IV
	CAST-IN-PLACE BRIDGE DECKS (INCL. DIAPHRAGMS)	CLASS II (BRIDGE DECK)	CLASS IV	CLASS IV
	APPROACH SLABS	CLASS II (BRIDGE DECK)	CLASS II (BRIDGE DECK)	CLASS II (BRIDGE DECK)
	PRECAST OR PRESTRESSED	CLASSES III, IV, V OR VI	CLASSES IV, V, OR VI	CLASSES IV, V, OR VI
SUBSTRUCTURE	CAST-IN-PLACE (OTHER THAN SEALS)	CLASS II	CLASS IV	CLASS IV OR CLASS V (SPEC.)
	RETAINING WALLS	CLASS II OR III	CLASS IV	CLASS IV
	CAST-IN-PLACE SEALS	CLASS III (SEAL)	CLASS III (SEAL)	CLASS III (SEAL)
	PRECAST OR PRESTRESSED (OTHER THAN PILING)	CLASSES III, IV, V OR VI	CLASSES IV, V, OR VI	CLASSES IV, V, OR VI
	COLUMNS LOCATED DIRECTLY IN SPLASH ZONE	CLASS II	CLASS IV	CLASS V (SPEC.)
	PILING	CLASS V (SPEC.)	CLASS V (SPEC.)	CLASS V (SPEC.)
	DRILLED SHAFTS	CLASS IV (DRILLED SHAFTS)	CLASS IV (DRILLED SHAFTS)	CLASS IV (DRILLED SHAFTS)

NOTES: (1) Corrosion Protection Measures:

The use of calcium nitrite and/or microsilica may be required to be added to the concrete as an admixture. The use of the admixtures shall conform to the requirements of the article of this Chapter entitled "Concrete Class and Admixtures for Corrosion Protection."

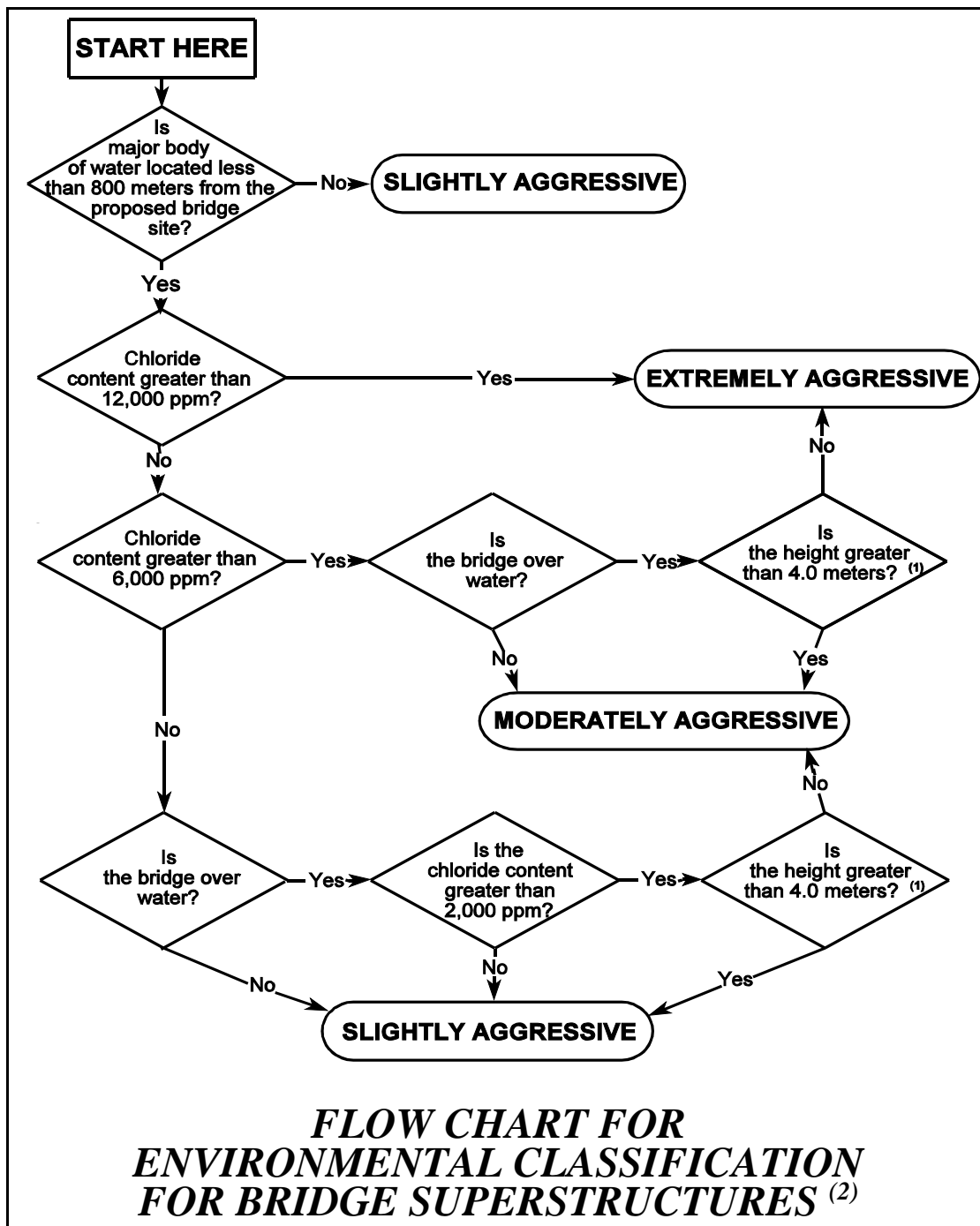


Figure 2-1

- NOTES: (1) The height of 4.0 meters may be increased for extreme exposure conditions. Height refers to the distance between the lowest superstructure elevation and the water surface at Mean High Water (MHW).
(2) The chart applies to water crossings but is not inclusive of other bridge types.

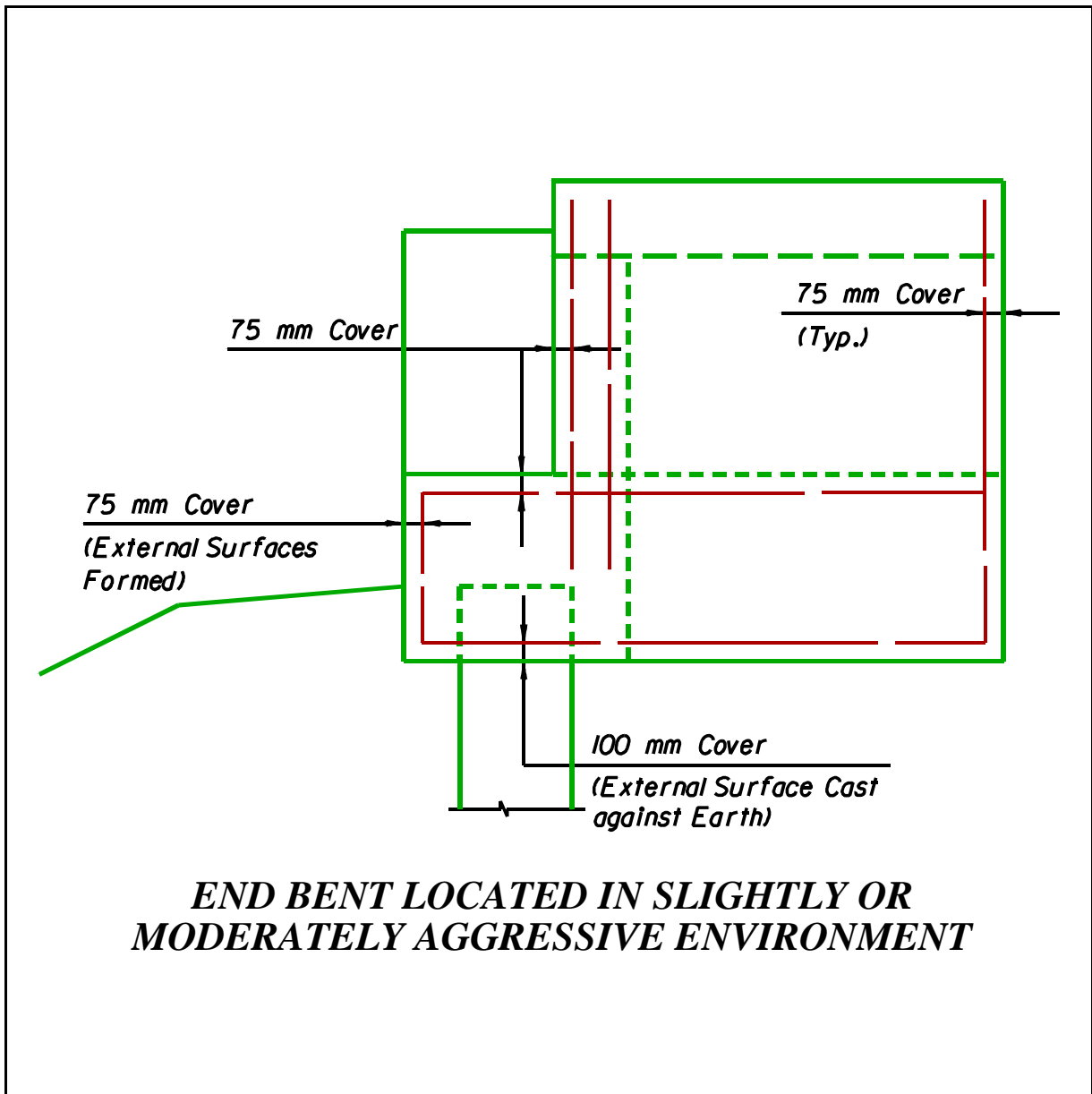


Figure 2-2

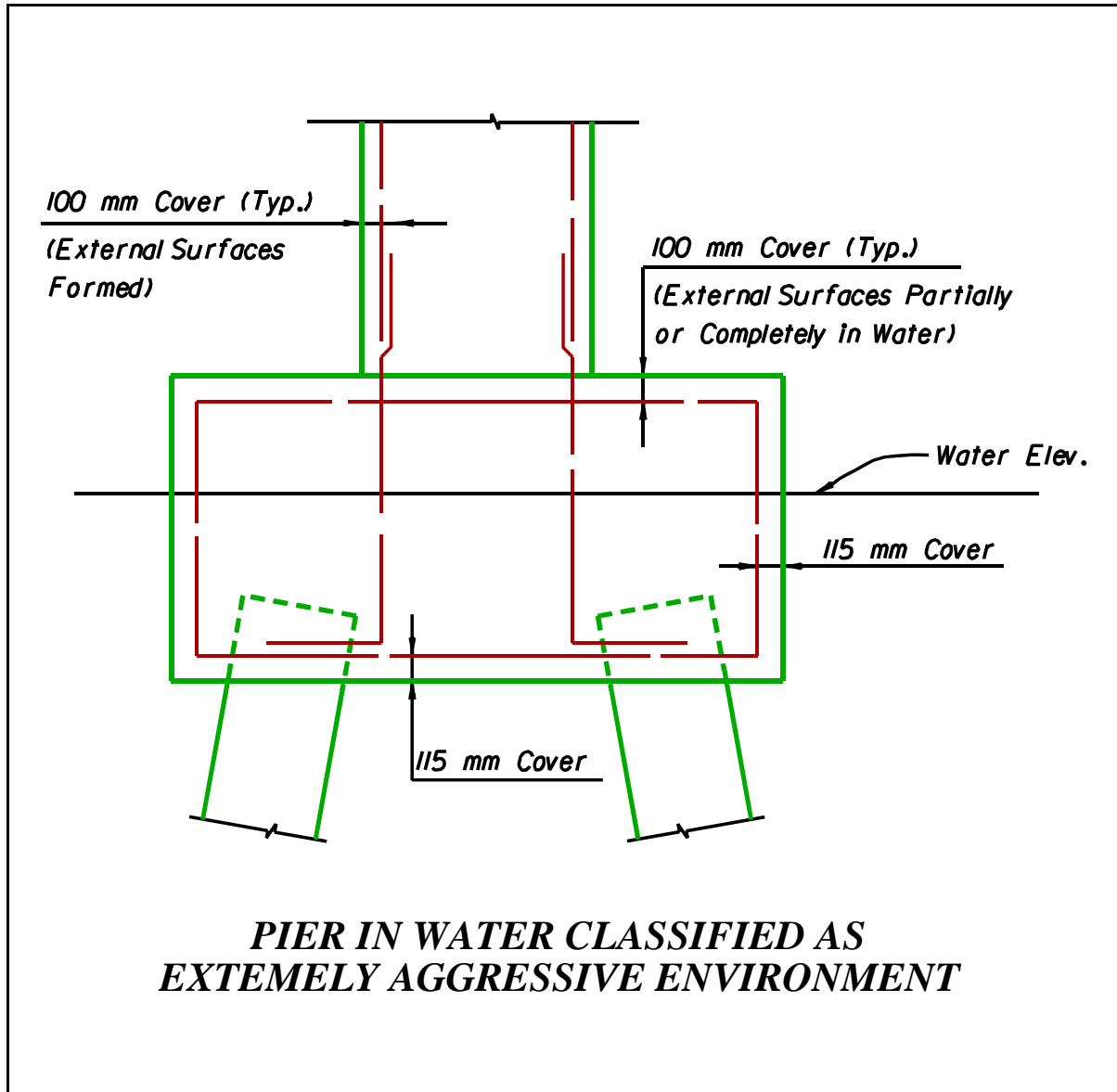


Figure 2-3

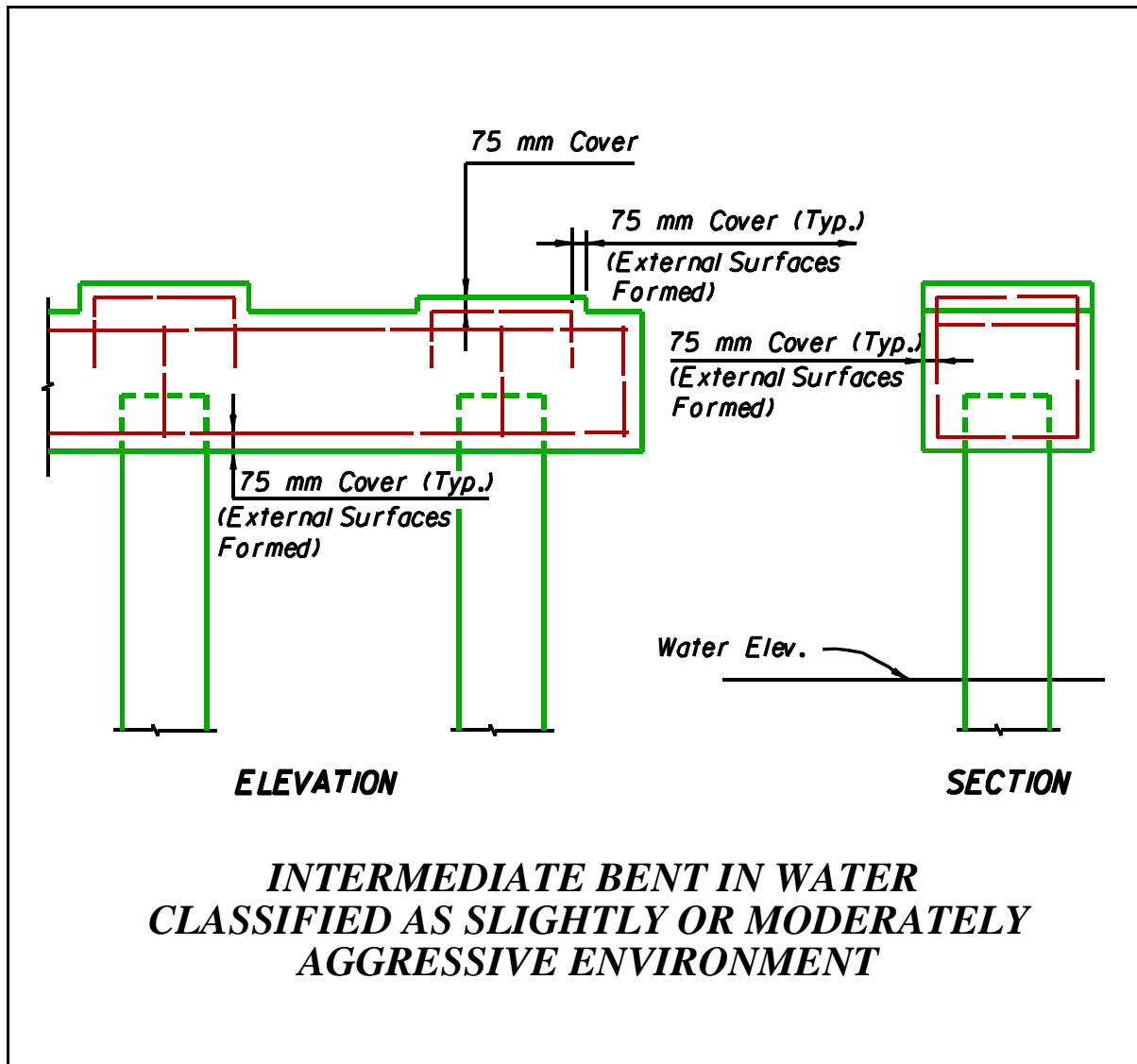


Figure 2-4

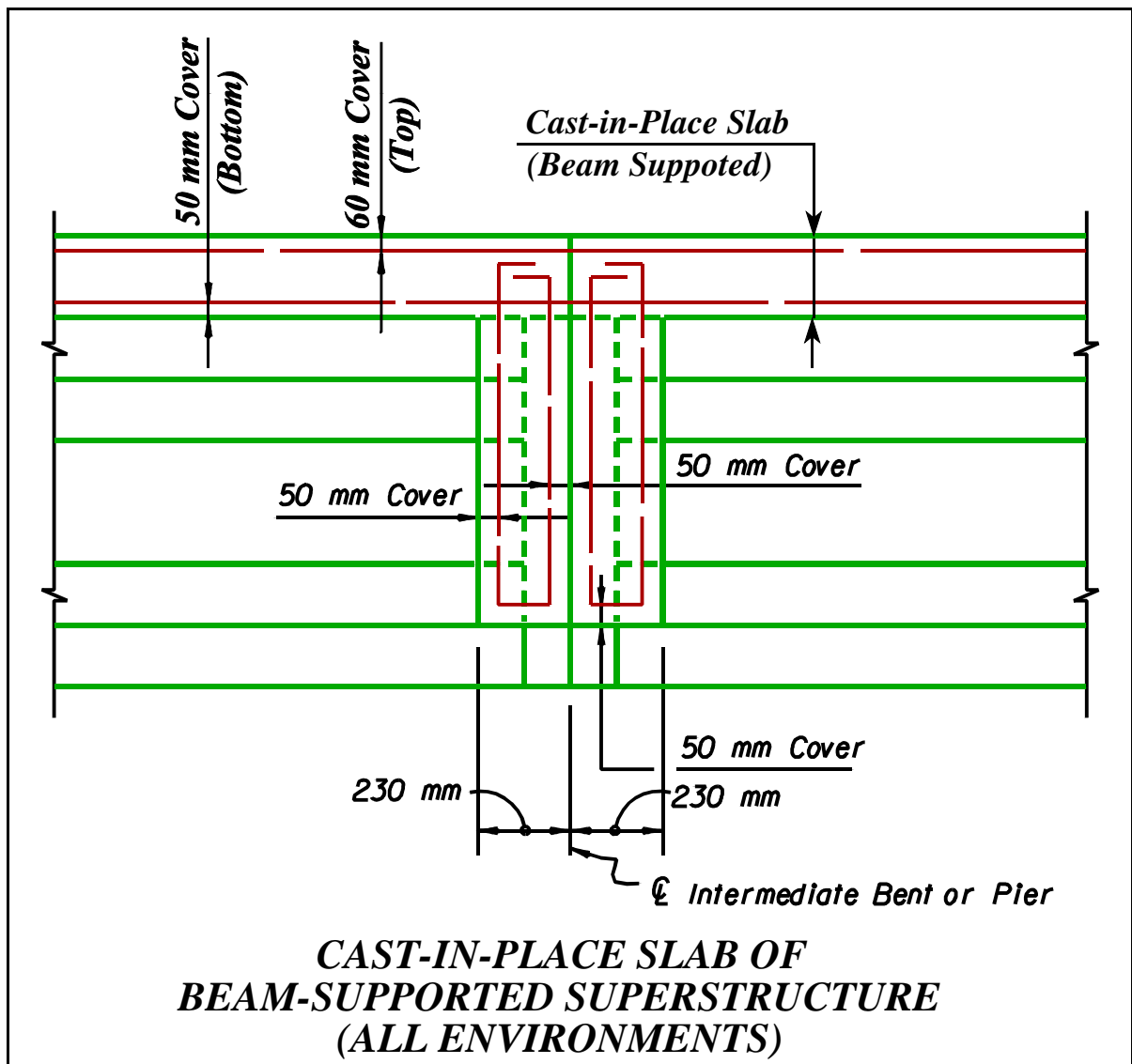


Figure 2-5

Chapter 3

LOADS AND LOAD FACTORS

3.1 General

This Chapter contains information related to loads, loadings, load factors, and load combinations. It also contains deviations from LRFD regarding Loads and Load Factors as well as characteristics of a structure that affect each.

3.2 Load Factors and Load Combinations [3.4.1]

LRFD Table 3.4.1-1 does not show values for the load factors " γ_{TG} ," " γ_{EQ} ," and " γ_{SE} ." The following sub-articles specify the values that will be used for these load factors.

3.2.1 Load Factor for Temperature Gradient (γ_{TG})

γ_{TG} = Zero (0.0) for all strength limit states.

γ_{TG} = 0.50 for service limit states of continuous concrete superstructures .

3.2.2 Load Factor (γ_{EQ}) for Extreme Event-I Load Combination

γ_{EQ} = 0.50.

3.2.3 Load Factor for Foundation Settlement (γ_{SE})

γ_{SE} = 1.0.

3.3 Vehicular Collision Force [3.6.5]

The requirements of LRFD [3.6.5.1] and [3.6.5.2] are deleted.

3.4 Horizontal Wind Pressure [3.8.1]

To account for the higher wind velocities which occur in South Florida, the calculated wind

pressure shall be increased by 20% for bridges located in Palm Beach, Broward, Dade, and Monroe counties.

3.5 Criteria for Deflection and Span-to-Depth Ratios [2.5.2] [3.6.1]

The criteria for Span-to-Depth Ratios in LRFD [2.5.2.6.3] apply. The criteria for deflection in LRFD [2.5.2.6.2] and [3.6.1.3.2] only apply for the design of bridges with pedestrian traffic.

3.6 Dead Loads [3.5.1]

The Dead Loads of Table 3.1 shall be used unless more accurate values are known.

Table 3.1 Dead Loads

ITEM	UNIT	LOAD
Barrier, Traffic Railing	kN/m	6.1
Barrier, Median	kN/m	7.1
Barrier, Sidewalk/Bicycle	kN/m	6.2
Barrier, Pedestrian/Bicycle	kN/m	3.1
Concrete, Counterweight (Plain)	kN/m ³	22.9
Concrete, Structural	kN/m ³	23.6
Future Wearing Surface	kN/m ²	0.72*
Soil, Compacted	kN/m ³	18.1
Stay-in-Place Metal Forms	kN/m ²	0.96**

* The Future Wearing Surface allowance applies to all new bridges. The allowance also applies to widenings unless otherwise authorized by the Department.

** Unit load of metal forms and concrete required to fill the form flutes to be applied over the projected plan area of the metal forms.

3.7 Earthquake Effects [3.10.9] [3.10.9.2]

The majority of Florida bridges will not require seismic design or the design of restrainers. For such bridges, only the minimum bearing support dimensions need to be satisfied as required

by LRFD [4.7.4.4]. Such bridges exempt from seismic design include those with superstructures comprising simple-span or continuous flat slabs, simple-span prestressed slabs or double-tees, simple-span AASHTO or Florida Bulb-Tee girders, and simple-span steel girders.

Structures that **do** require seismic design include those comprising unusual design, continuous steel or concrete girders, steel or concrete box girders, concrete segmental bridges, curved bridges with a radius less than 300 meters, those that rely on a system of fixed or sliding bearings for transmitting elongation changes, spans in excess of 50 meters, cable supported bridges, and bridges that do not use conventional elastomeric bearing pads.

The “Uniform Load Elastic Method” specified in LRFD [4.7.4.3.1] and [4.7.4.3.2], or any higher level analysis, shall be used for calculating the seismic forces to be resisted by the bearings or other anchorage devices. Only the connection forces between the superstructure and substructure are calculated.

Commentary: Neoprene bearing pads, as used in the majority of Florida bridges, have sufficient shearing strength and slip resistance to transfer the calculated seismic forces to the substructure.

For most Florida bridges, seismic forces will not govern over other load combinations; however, for some designs, even where the acceleration coefficient is as low as 1%, seismic loads can govern. For example, a long continuous superstructure, supported on a system of sliding bearings at all piers except one short, stiff pier where fixed bearings are used. The fixed bearings, in such cases, must be designed to transmit the calculated seismic forces. These seismic forces will likely govern over other load cases.

3.8 Uniform Temperature [3.12.2]

3.8.1 Segmental Box Girders

See Chapter 7 for temperature considerations when designing precast segmental box girders.

3.8.2 Joints and Bearings

For the design of bearings and expansion joints use the temperature ranges in Chapter 6.

3.8.3 All Other Bridge Components

For the design of all other bridge components, use the temperature ranges for Moderate Climate shown in LRFD Table 3.12.2.1-1.

3.8.4 Temperature Gradient [3.12.3]

Delete the second paragraph of LRFD [3.12.3] and, in lieu thereof, substitute the following:

“The effects of Temperature Gradient shall be included in the design of continuous concrete superstructures **only**. The vertical Temperature Gradient may be taken as shown in Figure 3.12.3-2.”

3.9 Barrier/Railing Distribution for Beam-Slab Bridges [4.6.2.2]

In lieu of the requirements of LRFD [4.6.2.2.1] for permanent loads, and in lieu of a more refined analysis, the dead load of barriers and railings attributed to exterior beams and stringers (W_{ext}), in kN/m, may be determined by the following equation when superstructure spans are not less than 12 meters nor more than 46 meters in length:

$$W_{ext} = \frac{(W)(C_1)(C_2)}{100} \quad [Eq. 3-1]$$

$$C_1 = 1.53\sqrt{S^3(3K-8)} + \frac{(10-K)^2 + 39}{1.4} \quad [Eq. 3-2]$$

$$C_2 = 2.2 - 1.1(L/10) + 0.3(L/10)^2 - 0.028(L/10)^3 \quad [Eq. 3-3]$$

Where:

- K = Number of beams in span, 10 maximum
- L = Span length (m)
- S = Beam spacing, center-to-center (m)
- W = Total uniform dead load weight of one barrier or railing (kN/m)

The balance of the total barrier/railing weight may be distributed equally among the interior beams or stringers.

When a barrier (or railing) is located on one side of a span only, 75% of the value of W_{ext} computed above may be used for the dead load of the barrier attributed to the exterior beam adjacent to the barrier, and the balance of the barrier weight distributed equally among the remaining beams.

The distribution methods described above apply to concrete or steel, longitudinal beam or stringer, superstructures on which a concrete slab is cast, compositely, prior to installation of the barrier or railing. For superstructures not conforming to the limitations stated above, the barrier/railing distribution may be determined in accordance with LRFD [4.6.2.2.1].

A. Example Problem 1:

(New Construction, 2-lane bridge)

L = 30.00 m

K = 6

S = 2.44 m

$C_1 = 57.73$

$C_2 = 0.846$

W = 6.1 kN/m

$$W_{\text{ext}} = \frac{(6.100)(57.73)(0.846)}{100} = 2.979 \text{ kN/m}$$

$$W_{\text{int}} = \frac{(2)(6.100 - 2.979)}{(6 - 2)} = 1.561 \text{ kN/m}$$

B. Example Problem 2:

(Widening from 2 to 4 lanes. Same as Example 1 except for a barrier on one side only and a longitudinal expansion joint at the junction with the existing slab.)

$$W_{\text{ext}} = (2.979)(0.75) = 2.234 \text{ kN/m (at exterior beam on barrier side)}$$

$$W_{\text{int}} = \frac{(6.100 - 2.234)}{(6 - 1)} = 0.773 \text{ kN/m (all other beams)}$$

3.10 Operational Importance [1.3.5]

Unless otherwise approved in writing by the State Structures Design Engineer, the value of the Operational Importance Factor (η_1) in LRFD [1.3.5] for the strength limit state shall be taken as:

$$\eta_1 = 1.0 \text{ for all bridges.}$$

Bridges that are considered critical to the survival of major communities, or to the security and defense of the United States, should utilize a higher value for η_1 ; however, as above, the higher value must be approved in writing by the State Structures Design Engineer.

3.11 Vessel Collision [3.14]

3.11.1 General [3.14.1]

The design of all bridges over navigable waters must include consideration for possible Vessel Collision. Such collisions generally occur from barges or oceangoing ships. EOR's shall conduct a vessel risk analysis to determine the most economical method for protecting the bridge. This shall include either designing and constructing the bridge to withstand Vessel Collision, or protecting it with dolphins and/or fenders. The Engineer of Record shall also comply with the procedure described hereinafter.

3.11.2 Research and Information Assembly

A. Data Sources:

1. U.S. Army Corps of Engineers, Waterborne Commerce Statistics Center, P.O. Box 61280, New Orleans, LA 70161. Telephone: (504) 862-1472.
2. U.S. Army Corps of Engineers, "Waterborne Commerce of the United States (WCUS), Parts 1 & 2," Water Resources Support Center (WRSC), Fort Belvoir, VA.
3. U.S. Army Corps of Engineers, "Waterborne Transportation Lines of the United States," WRSC, Fort Belvoir, VA.
4. U.S. Army Corps of Engineers (COE), District Offices.
5. U.S. Coast Guard, Marine Safety Office (MSO).
6. Port Authorities and Water Dependent Industries.
7. Pilot Associations and Merchant Marine Organizations.
8. National Oceanic and Atmospheric Administration (NOAA), "Tidal Current Tables; Tidal Current Charts and Nautical Charts," National Ocean Service, Rockville, Maryland.
9. Bridge tender record for bascule bridge at the District Maintenance Office.
10. Local tug and barge companies.

B. Assembly of Information unless provided by the Department:

1. Characteristics of the waterway including the following:
 - a. Nautical chart of the waterway.
 - b. Type and geometry of bridge.
 - c. Preliminary plan and elevation drawings depicting the number, size and location of the proposed piers.
 - d. Navigation channel, width, depth and geometry.
 - e. Average current velocity across the waterway.
2. Characteristics of the vessels and traffic including the following:

- a. Ship, tug and barge sizes (length, width and height)
 - b. Number of passages for ships, tugs and barges per year (last five years and prediction to end of 25 years in the future).
 - c. Vessel displacements.
 - d. Cargo displacements (deadweight tonnage).
 - e. Draft (depth below the waterline) of ships, tugs and barges.
 - f. The overall length and speed of tow.
- C. Accident reports.
- D. Bridge Importance Classification.

3.11.3 Design Vessel [3.14.4] [3.14.5.3]

When utilizing the FDOT's Mathcad software for conducting the Vessel Collision risk analysis, a "Design Vessel," which represents a group of vessels, is not required. The software can easily calculate the risk of collision for every vessel type with every pier. When calculating the geometric probability, the overall length of each vessel (LOA) shall be used instead of the LOA of a "Design Vessel."

3.11.4 Design Methodology - Damage Permitted [3.14.13]

In addition to utilizing the general design recommendations presented in LRFD, the EOR shall also use the following design methodology:

- A. At least one iteration of secondary effects in columns shall be included; i.e., axial load times the initial lateral deflection.
- B. The analysis must include the effect on adjacent piers from the transfer of lateral forces up to the superstructure. Bearings, including neoprene pads, may transfer lateral forces to the superstructure. Analysis of force transfer through the mechanisms at the superstructure/substructure interface shall be evaluated by use of generally accepted theory and practice.
- C. The ultimate capacity of axially loaded piles shall be limited to the compressive and/or tensile loads determined in accordance with the requirements of Chapter 4. For battered pile foundations, load redistribution shall not be permitted when the axial pile capacity is reached; rather, axial capacity shall be limited to the

ultimate limit as established by elastic analysis.

- D. Lateral soil-pile response shall be determined by concepts utilizing a coefficient of subgrade modulus provided or approved by the Geotechnical Engineer. Group effects must be considered.
- E. For the designer's Vessel Collision risk analysis, the FDOT will determine whether a bridge is critical or non-critical.
- F. Load Combination "Extreme Event II" shown in LRFD Table 3.4.1-1 shall be used for Vessel Collision. Nonlinear structural effects shall be included and can be significant, particularly for the pier components.

3.11.5 Pile Bents

Bridges with pile bents that are subject to minor vessel impact shall be designed to remain open for traffic after a vessel collision even if any one pile is lost as a result of the collision, thereby permitting traffic to be rerouted into designated, safe, striped traffic lanes. In this event, the load combination for Extreme Event II in LRFD [3.4.1] shall apply; however, the Load Factor for Live Load shall be increased to 1.0.

3.11.6 Widening

Bridge structures that span over navigable waterways and that are classified as major widenings in accordance with Chapter 9 shall be designed for Vessel Collision. Bridge structures that span over navigable waterways and that are classified as minor widenings in accordance with Chapter 9 shall be considered for Vessel Collision design requirements on an individual basis.

3.11.7 Movable Bridges

Movable bridges shall comply with the requirements of this chapter without exception.

3.11.8 Main Span Length for Barges

The length of the main span between centerlines of piers at the navigable channel shall be based upon the Coast Guard requirements, the Vessel Collision risk analysis (in conjunction with a least-cost analysis), and aesthetic considerations.

3.11.9 Scour with Vessel Collision [3.14.1]

The probability of the simultaneous occurrence of an extreme Vessel Collision load by a ship or barge and some amount of scour being present is a valid concern. For this reason, the substructure shall be designed to withstand the following two Load/Scour (LS) combinations:

A: Load/Scour Combination 1:

$$LS_{(1)} = \text{Vessel Collision} + \frac{1}{2} \text{Long-Term Scour} \quad [\text{Eq. 3-4}]$$

Where:

Vessel Collision: Assumed to occur at normal operating speed.
Long-Term Scour: Defined in Chapter 4 of the FDOT Drainage Manual (Topic No. 625-040-001).

B: Load/Scour Combination 2:

$$LS_{(2)} = \text{Minimum Impact Vessel} + \frac{1}{2} \text{100-Year Scour} \quad [\text{Eq. 3-5}]$$

Where:

Min. Impact Vessel: Defined in LRFD [3.14.1] with related collision speed.
100-Year Scour: Defined in Chapter 4 of the FDOT Drainage Manual (Topic No. 625-040-001).

When preparing the soil models for computing the substructure strengths, and when otherwise modeling stiffness, the EOR must exercise judgement in assigning soil strength parameters to the soil depth that is subject to Local and Contraction Scour that may have filled back in. The soil model shall utilize strength characteristics over this depth that are compatible with the type soil that would be present after having been hydraulically redeposited. In many cases, there may be little difference between the soil strength of the natural stream bed and that of the soil that is redeposited subsequent to a scour event.

3.11.10 Application of Impact Forces [3.14.14]

For long narrow footings in the waterway when the length to width ratio, L/W, is 2.0 or greater, the longitudinal force shall be applied within the limits of the distance that is equal to the length minus twice the width, (L-2W), in accordance with Figure 3-1.

3.11.11 Impact Forces on Superstructure [3.14]

Vessel Impact Forces on the superstructure shall be applied in accordance with LRFD [3.14].

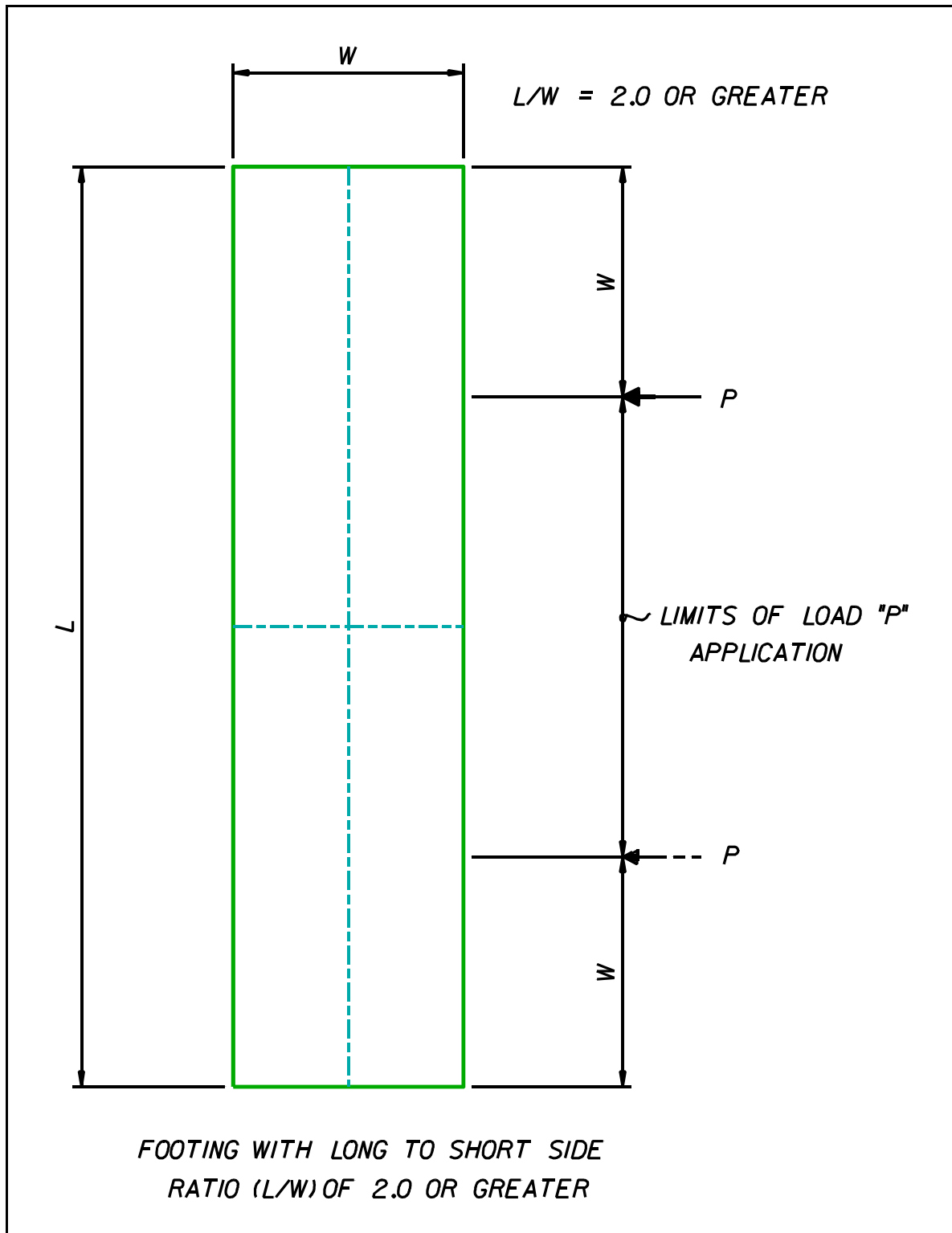


Figure 3-1

Chapter 4

FOUNDATIONS

4.1 General

This Chapter supplements LRFD Sections [2] and [10] and contains deviations from those sections. This Chapter also contains information and requirements related to soil properties, foundation types and design criteria, fender pile considerations, and cofferdam design criteria to be used in the design of bridge structures.

4.2 Scope

The Structural Engineer, utilizing input from the Geotechnical and Hydraulic Engineers, shall determine the structure loads and the pile/shaft section or spread footing configuration.

4.3 Scour Considerations for Foundation Design [2.6]

These three engineering disciplines have specific responsibilities in considering scour as a step in the foundation design process.

4.3.1 The Structural Engineer

- A. Provides the preliminary design configuration of a bridge structure utilizing all available geotechnical and hydraulic data.
- B. Provides axial (compression and tension) and lateral loads.

4.3.2 The Hydraulics Engineer

- A. Provides the following three scour elevations:
 - 1. EL_{100} , corresponding to the design flood up to the 100-year scour event.
 - 2. EL_{500} , corresponding to the superflood up to the 500-year extreme scour event.
 - 3. Long-term scour if vessel collision design is required.

4.3.3 The Geotechnical Engineer

- A. Provides the factored axial (compression and tension) curves for the various scour conditions.
- B. Provides lateral stability soil properties for evaluation.

Commentary: This is a multi-discipline effort involving Geotechnical, Structural, and Hydraulics Engineers. The leader of this multi-discipline team is the Structural Engineer who is responsible for the coordination and resolution of all issues raised. The process listed below often requires several iterations. The foundation design must satisfactorily address the various scour conditions and also furnish sufficient information for the Contractor to provide adequate equipment and construction procedures.

4.4 Determination of Soil Properties [10.4]

The District Geotechnical Engineer or the contracted Geotechnical firm shall issue a Geotechnical Report for most projects. This report shall describe the soil conditions in detail and recommend suitable foundation types with consideration for constructability issues. The report shall include appropriate design parameters. Input data for COM 624, Florida Pier, and other design programs shall be included. The report shall be prepared in accordance with the Department's Soils and Foundations Manual which is available through Maps and Publications Sales in Tallahassee.

The core boring drawings shall be included in the Geotechnical Report. These drawings reflect the foundation data acquired from field investigations.

Geotechnical Reports shall conform to the FHWA Report "Checklist and Guidelines for Review of Geotechnical Reports and Preliminary Plans and Specifications" prepared by the Geotechnical and Materials Branch, FHWA, Washington, D.C., October 1985. Contact the District Geotechnical Engineer to receive a copy of this document. The appropriate checklists provided in the subject document are required to be properly executed and submitted as part of the Geotechnical Report.

In the event that the Geotechnical Report is prepared by a contracted Geotechnical firm, it generally will be reviewed by both the State and District Geotechnical Engineers for Category 2 Structures and the District Geotechnical Engineer for Category 1 Structures as defined in the Plans Preparation Manual (Topic No.: 625-000-005), Chapter 26. Final acceptance of the report is contingent upon the District Geotechnical Engineer's approval. Concurrence by the State Geotechnical Engineer is required for all Category 2 Structures. The contracted Geotechnical firm is to interact in a timely manner with the District Geotechnical Engineer throughout the course of the geotechnical activities. The scope of services as well as the proposed field and laboratory investigations shall be discussed with the Structural Engineer and the District Geotechnical Engineer prior to commencing any operations.

4.5 Resistance Factors [10.5.5]

The calibrated resistance factors of Tables 4.1 and 4.2 should be used for piles and drilled shafts, respectively:

Table 4.1 Resistance Factors for Piles

Foundation Type	Application	Design Consideration	Design Methodology	Resistance Factor, ϕ
Piles	All Structures	Compression	SPT97	0.65
			PDA (EOD)	0.65
			Wave Equation Analysis	0.35
			Static Load Testing	0.75
		Uplift	SPT97	0.55
			Static Load Testing	0.65
		Lateral	FLPier	1.00

Table 4.2 Resistance Factors for Drilled Shafts

Foundation Type	Application	Design Consideration	Design Methodology	Resistance Factor, ϕ
Drilled Shafts	Bridge Foundation	Compression	FHWA-HI-88-042 on soils with $N < 15$ correction suggested by O'Neil. For rock socket, use McVay's method to determine unit skin friction by:	
			1. Neglecting end bearing	0.60
			2. Including 1/3 end bearing	0.55
		3. Static Load Testing	0.75	
	Uplift	Same as above for side friction	0.50	
	Lateral	FLPier	1.00	
	Misc. Structures	Compression	Same as above	0.55
		Uplift	Same as above	0.50
Lateral Load		Brom's Method (FDOT D647/F205)	0.90	
Torsion		FDOT Structures Office	0.95	

4.6 Spread Footing General [10.5.5] [10.6]

In the event that the Geotechnical Report recommends a spread footing foundation, the maximum soil pressures, the minimum footing widths, the minimum footing embedment, and the LRFD Table 10.5.5-1 Resistance Factors (ϕ) used shall be indicated in the Geotechnical Report. The Structural Engineer shall determine the factored design load and proportion the footings to provide the most cost effective design without exceeding the recommended maximum soil pressures. The Structural Engineer shall communicate with the Geotechnical Engineer as necessary to insure that the corresponding settlements do not exceed the tolerable limits. Sliding, overturning, and rotational stability of the footings shall also be verified. Dewatering, when recommended in the Foundation Report, shall be so noted in the plans. If the ground water elevation is within 600 mm of the bottom of the footing, or higher, dewatering must be specified.

4.7 Lateral Load [10.7.3.8] [10.8.3.8]

The soil resistance factors for foundation systems as specified in LRFD shall be used for all load cases, except that a resistance factor of 1.0 shall be used for lateral analysis.

4.8 Spacing, Clearances Embedment and Size [10.7.1.5] [10.8.1.6]

Minimum pile or shaft spacing center-to-center shall be at least three (3) times the least width of the deep foundation element measured at the ground line.

The minimum size of square prestressed concrete piles is 455 mm. The minimum diameter for drilled shafts shall be 915 mm; however, larger-diameter drilled shafts are preferred. An exception to the minimum size requirement for prestressed piles is the size of piles for fender systems.

Commentary: The Structural and Geotechnical Engineers shall consider constructability in the selection of the foundation system. Such issues as existing utilities (both underground and overhead), pile-type availability, use of existing structures for construction equipment, phase construction, conflicts with existing piles and structures, effects on adjacent structures, etc. shall be considered in evaluating foundation design alternatives.

4.9 Battered Piles [10.7.1.6]

Plumb piles are preferred; however, if the design dictates the use of battered piles, a single batter, either parallel or perpendicular to the centerline of the cap or footing, is preferred. If the design dictates that a compound batter is required, the pile shall be oriented so that the direction of batter will be perpendicular to the face of the pile. The Structural Engineer shall evaluate the effects of length and batter on the selected pile size. The following maximum

batters, measured as the vertical-to-horizontal ratio, "v:h," shall not be exceeded:

End bents and abutments	6:1
Piers without ship impact	12:1
Intermediate bents	6:1
Piers with ship Impact	4:1

Commentary: Longer piles are more susceptible to bending, especially if an obstruction is hit during driving. Also, when driving long slender piles on batters, the tendency is for the tip to bend downward due to gravity which creates additional bending stresses. These deflections can be severe enough to cause significant, additional stresses due to bending when hit with a pile driving hammer. The addition of these stresses can lead to pile failure in bending. Hard subsoil layers can also deflect piles outward in the direction battered causing pile breakage in bending. The ability to install battered piles must be determined during the design phase.

4.10 Minimum Pile Tip [10.7.1.11] [10.8.2.4]

The minimum pile or drilled shaft tip elevation must be the deeper of the minimum elevations that satisfy lateral stability requirements for the load cases and satisfy the minimum penetration requirements of FDOT Specification A455-3.9. The minimum tip elevation may be set lower to satisfy any unique soil conditions.

4.11 Anticipated Pile Lengths [10.7.1.11]

The anticipated pile lengths are used only to estimate quantities and set test pile lengths. These lengths are determined by using the lower of the minimum tip elevations specified on the plans or the axial capacity elevations predicted by the pile capacity curves.

4.12 Test Piles [10.7.1.13]

Test piles include static load test piles, dynamic load test piles, and confirmation piles ("unloaded test piles"). As a minimum, one test pile shall be located approximately every 60 meters of bridge length with a minimum of two test piles per bridge structure. These requirements shall apply for each size and pile type in the bridge except at end bents. Test piles in end bents are not normally required and should not be specified unless specific justification is provided in the Geotechnical Report. For bascule piers and high-level crossings that require large footings or cofferdam-type foundations, a minimum of one test pile shall be driven at each pier.

Test piles shall be required for all projects unless, in the opinion of the District Geotechnical Engineer, pile driving records for the existing structure include enough information (i.e., stroke

length, hammer type, cushion type, etc.) to adequately determine the authorized pile lengths and driving criteria.

Test piles should be at least 5.0 meters longer than the estimated length of production piles. Additional length may be required by the load frame geometry when static load tests are used. The Structural Engineer shall coordinate his recommended test pile lengths and locations with the District Geotechnical Engineer, or Geotechnical Consultant, prior to finalization of the plans.

Commentary: Test piles are installed and tested in order to verify the soil capacity, the pile driving system, the pile driving ability, to determine production pile lengths, and to establish driving criteria. When determining the location of test piles, maintenance of traffic requirements, required sequence of construction, geological conditions, and pile spacing must be considered. For phase construction, it is preferable to locate all test piles in the first phase to be constructed. The Geotechnical Engineer shall verify the adequacy of the test pile locations. Because test piles are exploratory in nature, they may be driven harder, deeper, and to a greater bearing value than required for permanent piling or may be used to establish soil freeze parameters. (See FDOT Specifications Section 455). Therefore, the Structural Engineer shall take these facts into consideration in establishing estimated lengths.

4.13 Load Tests [10.7.3.6] [10.8.3.6]

Load tests include static load tests, dynamic load tests, Osterberg load tests, and Statnamic load tests. Both design phase and construction phase load testing should be investigated. Factors to be considered when evaluating the benefits and costs of providing loaded test piles, include: soil stratigraphy, design loads, pile type and number, type of loading, testing equipment, and mobilization.

Commentary: In general, the more variable the subsurface profile, the less cost-effective are static load tests. When soil variability is an issue, other options include additional field exploration, more laboratory samples, in-situ testing, and pullout tests.

4.13.1 Static Load Test [10.7.3.6]

If static load test piles or drilled shafts are called for on the design plans, the number of required tests, pile or shaft type, pile or shaft size, and test loads shall be shown in the plans. All piles which will be statically load-tested shall first be subjected to dynamic load testing. Static load tests shall be designed to test the pile to failure as stipulated in Section 455 of the Specifications.

Commentary: Test piles or drilled shafts can be subjected to static compression, tension, or

lateral test loads. Static load tests may be desirable when foundation investigations reveal sites where the soils cause concern regarding the development of the required pile capacity at the desired depths, and/or the possibility that considerable cost savings will result if higher soil capacities can be obtained. Furthermore, static load tests will reduce the driving effort since a higher Performance Factor is applied to the Ultimate Bearing Capacity formula.

4.13.2 Dynamic Load Test [10.7.1.15]

When dynamic testing is required, all test piles shall have dynamic load tests. The need for dynamic load tests shall be addressed in the geotechnical report. This requirement shall be shown by means of a note on the pile layout sheet.

Commentary: Dynamic load testing consists of instrumenting piles with strain transducers and accelerometers which measure pile force and acceleration, respectively, during driving operations. A Pile Driving Analyzer (PDA) unit is used for this purpose.

4.13.3 Special Considerations

Load testing of foundations which will be subjected to subsequent scour activity requires special attention. The necessity of isolating the resistance of the scourable material from the load test results must be considered.

4.14 Fender Piles

There are no specific design requirements for fender piles; however, the length established shall meet or exceed the minimum pile penetration set forth in Section A455-3.9 of the Specifications.

Commentary: The pile installation constructability must be looked at by the Geotechnical Engineer to insure that the pile tips shown in the plans can be reasonably obtained by the Contractor.

4.15 Pile Driving Resistance [10.7.1.3]

The Geotechnical Engineer calculates an Ultimate Bearing Capacity (UBC) as:

$$UBC = \frac{\text{Factored Design Load} + \text{Net Scour} + \text{Downdrag}}{\phi} \quad [Eq. 4-1]$$

The Ultimate Bearing Capacity shall not exceed the following values unless specific justification is provided and accepted by the Department:

455 mm pile	2,700 kN
510 mm pile	3,200 kN
610 mm pile	4,000 kN
760 mm pile	5,400 kN

4.16 Pile Jetting and Preforming

When jetting or preforming is to be allowed, then the depth of jetting or preforming must be limited as necessary to comply with all the design criteria. For projects with scour, jetting or preforming will not normally be permitted below the 100-year scour elevation (EL_{100}).

If jetting or preforming is utilized to EL_{100} , the Net Scour Resistance to that depth is assumed to be equal to 0.0 kN (provided the hole remains open or continuous jetting is being done).

If jetting is specified in the plans, the Structural Engineer must verify that it will not violate any of the environmental permits.

4.17 Foundation Installation Table

All plans for projects utilizing driven piles or drilled shafts shall contain a "Pile Data Table" and notes or a "Drilled Shaft Data Table" and notes included hereinafter, as applicable. For items that do not apply, place "N/A" in the column; however, the table itself shall not be revised or modified. Loads should be rounded to the nearest 10 kN. Elevations and pile lengths should be rounded to the nearest 0.5 meters.

4.17.1 Piles

In the Pile Data Table, Table 4.3, the values for driven piles for the column entitled "Ultimate Bearing Capacity (UBC)" shall be determined from Equation 4-1 above.

Additional plan notes for bridges with pile supported substructures are:

- Minimum Tip Elevation is required _____(reason must be completed by Structural Engineer; e.g., "for lateral stability")
- When a required jetting elevation is shown, the jet shall be lowered to the elevation and continue to operate at this elevation until the pile driving is completed. **If jetting or preforming elevations differ from those shown on the table, the Engineer shall be responsible for determination of the**

required driving resistance.

- In order to achieve the required minimum tip elevation, it is estimated that a driving resistance value higher than that required for load capacity may be encountered in some locations.
- No jetting, except that shown in the Pile Installation Table (and only to that depth) or to the 100-year scour elevation, will be allowed without the approval of the Engineer. The Contractor should not anticipate being allowed to jet piles below this elevation.

Table 4.3 Pile Data Table

PILE DATA TABLE														
INSTALLATION CRITERIA							DESIGN CRITERIA							
Pier or Bent	Pile Size (mm)	Ultimate Bearing Capacity (kN)	Tension Capacity (kN)	Min. Tip Elev. (m)	Test Pile Length (m)	Req'd Jet Elev. (m)	Req'd Preform Elev. (m)	Factored Design Load (kN)	Down Drag (kN)	Total Scour Resist. (kN)	Net Scour Resist. (kN)	100 Yr. Scour Elev. (m)	Long Term Scour Elev. (m)	ϕ

Total Scour Resistance - an estimate of the ultimate static side friction resistance provided by the scourable soil.

Net Scour Resistance - an estimate of the ultimate static side friction resistance provided by the soil from the required preformed or jetting elevation to the scour elevation.

Tension Capacity - the ultimate static side friction capacity that must be obtained below the 100 year scour elevation to resist pullout of the pile. (Specify only when design requires tension capacity)

4.17.2 Drilled Shafts

In a manner similar to that for piles, a Drilled Shaft Data Table, identical in format and information to Table 4.4 below, shall be included in the plans for all projects on which drilled shafts will be utilized.

Table 4.4 Drilled Shaft Data Table

DRILLED SHAFT DATA TABLE									
INSTALLATION CRITERIA					DESIGN CRITERIA				
Pier or Bent No.	Shaft Size (mm)	Tip Elev. (m)	Min. Tip Elev. (m)	Min. Rock Socket Length (m)	Factored Design Load (kN)	Downdrag (kN)	100-yr Scour Elev. (m)	Long-term Scour Elev. (m)	ϕ

Tip Elevation - the elevation to which the shaft shall be constructed unless test load data, rock cores, or other geotechnical data obtained during construction allows the Engineer to authorize a different tip elevation.

Min. Tip Elevation - the highest elevation that the shaft tip may be constructed if adjustments are made to the tip elevation specified.

4.18 Cofferdams

When showing seal dimensions in the bridge plans, the EOR shall show the water depth assumed for the seal design. This depth is derived from the maximum safe elevation of the water outside the cofferdam when it has been completely dewatered.

For design of the cofferdam seal, assume the maximum allowable stresses from Table 4.5 which apply at the time of complete dewatering of the cofferdam.

In the event greater stress values are required, the contact surfaces of the foundations with the seal shall contain mechanical connectors such as weldments or shear connectors. When such artificial means of increasing shear capacity are to be used, the EOR shall detail their locations and connections on the drawings and provide substantiating calculations for their use.

Table 4.5 Cofferdam Design Values

MAXIMUM ALLOWABLE STRESSES AT TIME OF COMPLETE DEWATERING OF THE COFFERDAM	
Maximum tension in seal concrete as a result of hydrostatic pressure	1.400 MPa
Cohesive shear stress between seal concrete and concrete piles or shafts	0.270 MPa
Cohesive shear stress between seal concrete and steel piles or casings	0.035 MPa

Commentary: Generally, cofferdams are designed and detailed by the Contractor and reviewed by the EOR as a shop drawing. In many instances, however, the EOR must design the seal because it constitutes a significant load for the foundation design, and a seal quantity is often required for bidding purposes.

Chapter 5

SUBSTRUCTURE ELEMENTS AND APPURTENANCES

5.1 General

This Chapter contains information and criteria related to the design, reinforcing, detailing, and construction of substructure elements and appurtenances and includes deviations from and supplementary requirements to LRFD criteria for substructure designs. In particular, this Chapter supplements LRFD Sections [5] and [11].

5.2 Splash Zone Definition

The Splash Zone is defined as the vertical distance from 2.0 meters below to 4.0 meters above Mean High Water (MHW).

5.3 Bents with Drilled Shafts

When drilled shafts are used in bents in Moderately or Extremely Aggressive Environments and extend through the water, the shafts shall be detailed to eliminate construction joints within the Splash Zone. Additionally, it is preferred that such shafts extend to the bottom of the bent cap without a construction joint.

5.4 Mass Concrete [5.10.8.3]

Mass Concrete is defined as any large volume of cast-in-place or precast concrete with dimensions large enough to require that measures be taken to cope with the generation of heat and attendant volume change so as to minimize cracking.

When the minimum dimension of a concrete component exceeds 1.0 meters and the ratio of its volume to surface area is greater than 300 mm, then Mass Concrete requirements apply. The surface area includes all the cumulative area of all surfaces of the concrete component being considered including the full underside (bottom) surface of footings, caps, etc. The volume shall be measured in units of cubic meters, and the area in units of square meters.

The EOR shall consider the consequences of Mass Concrete requirements in selecting member sizes and shall avoid Mass Concrete whenever practicable. However, when Mass Concrete is unavoidable, the EOR shall indicate on the plans those portions of the concrete elements in the bridge that are Mass Concrete. In this event, the estimated pay quantities shall be arranged so that Mass Concrete quantities are shown separately, and are identified either as Mass Concrete (Substructure) or Mass Concrete (Superstructure).

Seal concrete shall **not** be considered as Mass Concrete.

5.5 Crack Control Requirements for Piers, Footings, and Walls

5.5.1 Secondary Reinforcing Requirements for Pier Footings

In addition to the main reinforcing required in the bottoms and/or tops of pier footings, a two-way cage of top and side reinforcement must also be provided. The need for such reinforcement is twofold: (1) to assist the green concrete in resisting tensile stresses that occur during the hydration process, and (2) to resist secondary loads that often are not included in the design process. The reinforcement used to form the cage in each direction shall be sized and spaced according to the Mass Concrete requirements of **[5.10.8.3]** but shall not be less than No. 19 Bars and shall be spaced at not more than 420 mm.

5.5.2 Control of Shrinkage Cracking **[5.10.3]** **[5.10.8]**

A. Long Walls:

For long walls and other similar construction where one lift of concrete is placed on top of a previously placed lift, the EOR is advised to consider the following:

1. The minimum percentage of reinforcement to control shrinkage/thermal cracking shall meet the requirements of **[5.10.8]**.
2. Limiting the length of a lift to a maximum of 9.0 meters between vertical construction joints. See also the limits of concrete pours in tall piers elsewhere in this Chapter.
3. Required construction joints shall be clearly detailed on the plans.
4. Construction or expansion joints shall be fitted with a water barrier wherever water leakage has to be prevented.

B. Footings:

Footings shall be cast monolithically. Struts and other large attachments shall be attached as secondary castings.

C. Pier Caps:

Pier caps shall be sequenced to minimize shrinkage cracking. Unless otherwise specifically approved by the Department, construction joints shall be spaced at not less than 6.0 meters nor more than 25.0 meters.

D. Keyways:

Keyways shall be provided at formed surfaces of construction joints and elsewhere as necessary to transfer applied loads from one cast section to an adjacent, second pour. Keyways shall be trapezoidal in shape for ease of forming and stripping, and, as an example, a typical joint shall have a keyway about 50 mm deep and about 150 mm wide (or one third the thickness of the member for members less than 455 mm in thickness) running the full length.

Keyways shall not be placed in horizontal construction joints between large-sized components such as pier columns to footings and caps.

5.5.3 Construction Joints in Tall Piers and Columns

Construction joints shall be detailed on the plans for tall piers or columns to limit the concrete lifts to 8.0 meters. When requested by the EOR and approved by the Department, a maximum lift of 9.0 meters may be allowed to avoid successive small lifts (less than approximately 5.0 meters) which could result in vertical bar splice conflicts or unnecessary splice length penalties.

Splices shall be detailed for vertical reinforcing at every horizontal construction joint; except that the splice requirement may be disregarded for any lift of 3.0 meters or less.

The EOR shall verify that the lift heights (and construction joint locations) shown on the plans are in accordance with the concrete placement requirements of the specifications.

5.6 Retaining Walls

Retaining wall plans preparation and administrative requirements are included in Chapter 30 of the Plans Preparation Manual (Topic No. 625-000-005) which must be used in conjunction with the design requirements of this Section. Refer to Chapter 2 for the retaining wall concrete class and reinforcing steel cover requirements. The retaining wall concrete class is required to be shown on the plans.

5.6.1 Wall Types (Site Applications)

There are a number of types of retaining walls presently being used by the Department. Wall site considerations, economics, aesthetics, maintenance and constructability should all be considered when determining wall type to be used. The following wall types are being used or considered for use by the Department.

A. Conventional Cast-in-Place (CIP) Walls:

CIP walls are normally used in either a cut or fill situation. These walls are sensitive to foundation problems. The foundation soil must be capable of withstanding the design toe pressure and must exhibit very little differential

settlement. Judgement should be exercised to assure that during the life of the structure the bearing capacity of the soil will not be diminished (i.e., french drains in close proximity to the walls) thus requiring pile supported walls. This type wall has an advantage over MSE walls because they can be built with conventional construction methods even in Extremely Aggressive Environments. Another advantage over MSE walls is on cut/widening projects where the area behind the wall is not sufficient for the soil reinforcing (See Figures 5-2 and 5-3).

The relative cost of CIP walls is greater than MSE walls when the site and environment are appropriate for each wall type. This is assuming the area of wall is greater than 100.0 m² and greater than 3.0 m in height.

B. Pile Supported Walls:

Pile supported walls are utilized when the foundation soil is not capable of supporting the retaining wall and associated dead and live loads on a spread footing.

Pile supported retaining walls are extremely expensive compared to CIP cantilevered and MSE walls and are only appropriate when foundation soil conditions are not conducive for CIP or MSE walls. Pile supported walls are appropriate for cut or fill sites. Temporary sheeting may be required in cut sites (See Figures 5-4 and 5-5).

C. Mechanically Stabilized Earth (MSE) Walls:

MSE walls are not a cure-all for poor foundation soil and are not appropriate for all sites. MSE walls are very adaptable to both cut and fill conditions and will tolerate a greater degree of differential settlement than CIP walls. Because of their adaptability MSE walls are being used almost exclusively. The design of MSE walls with metallic reinforcement, however, is sensitive to the electrochemical properties of the backfill material and to the possibility of a change in the properties of the backfill materials due to submergence in water classified as Extremely Aggressive or from heavy fertilization. The use of geosynthetic reinforcement shall be considered in areas where the water is classified as an Extremely Aggressive Environment when the 100-year flood can infiltrate the backfill, and when the wall is within 4.0 meters of the Mean High Water (MHW). MSE walls are generally the most economical of all wall types when the area of retaining wall is greater than 100.0 m², and the wall is greater than 3.0 m in height (See Figures 5-6 and 5-7).

D. Precast Counterfort Walls:

Precast counterfort walls are applicable in cut or fill locations. Their advantage is in cut locations such as removing front slopes under existing bridges and in certain widening applications where sheet piling would be required to stabilize

excavation for earth reinforcements for MSE walls. Also, their speed of construction is advantageous in congested areas where maintenance of traffic is a problem. This type of wall is also applicable in areas where the backfill is or can become classified as extremely corrosive.

This type of wall is generally not as economical as MSE walls but is competitive with CIP walls (See Figure 5-8) and may offer aesthetic and constructability advantages.

E. Steel Sheet Pile Walls (Permanent and Temporary):

Steel sheet pile walls are applicable for use in permanent locations (i.e., bulkheads) but their more common use is for temporary use (i.e., phase construction). Generally steel sheet pile walls can be designed as cantilevered walls up to approximately 4.5 m in height. Steel sheet pile walls over 4.5 m are tied back with either prestressed soil anchors or tiebacks (deadmen).

Steel sheet piles walls are relatively expensive initially and require periodic maintenance (i.e. painting, cathodic protection). This type of wall should only be used if there are no other more economical alternates (See Figure 5-9).

F. Concrete Sheet Piles:

Concrete sheet piles are primarily used as bulkheads in either fresh or saltwater. Rock in close proximity to the ground surface is a concern with this type of wall as they are normally installed by jetting. Concrete sheet piles when used as bulkheads are normally tied back with deadmen.

This type of wall is relatively costly and should only be used when less costly alternates are not appropriate or when the environment is appropriate (See Figure 5-10).

G. Wire Faced Walls (Temporary):

Wire faced walls are applicable in temporary situations, phase construction where large amounts of settlement are anticipated or where surcharges are required to accelerate settlement. This type of wall is a form of MSE wall. The soil reinforcement may be either steel or geogrid. The designer shall verify the systems he intends to use are on the Department's Qualified Products List (See Figure 5-11).

H. Soil Nails (Temporary Walls):

The Department is allowing the use of soil nails in temporary and experimental walls only; however, the relative cost of this type wall has not been determined because Contractors have yet to bid this alternate (See Figure 5-12).

I. Soldier Pile/Panel Walls:

This type of wall is applicable in bulkheads and retaining walls where the environment is Extremely Aggressive and/or rock is relatively close to the ground surface. The cost of this type wall is very competitive with concrete sheet pile walls (See Figure 5-13).

J. Modular Block Walls:

Modular blocks consist of drycast unreinforced blocks which are sometimes used as a gravity wall and sometimes used as a wall facing for a MSE variation normally utilizing a geogrid for soil reinforcement. The Department is considering systems of this type for low-height, non-structural applications only.

K. Geogrid Soil Reinforcement for Retaining Walls:

Geogrids are presently approved for use in temporary and select (Extremely Aggressive Environments) permanent walls only. Their use in permanent walls is being reviewed. Some of the concerns about geogrids are the long term stress-strain characteristics of polymers, hydrolysis of polyesters, environmental stress cracking, and brittle rupture of polyolefins (high density polyethylene, polyethelene and polypropylene) and appropriate factors to ensure an allowable stress condition at the end of the structure's service life.

L. Permanent - Temporary Wall Combination:

As more highways are widened, we have encountered a problem at existing grade separation structures. The existing front slope at the existing bridge is required to be removed to accommodate a new lane and a retaining wall must be built under the bridge. The trick is how do you remove the existing front slope and maintain the stability of the remaining soil? Several methods have been used. One method is to excavate slots or pits in the existing fill to accommodate soldier beams. The soil is then excavated and timber lagging is placed horizontally between the vertical soldier beams. The soldier beams are tied-back by the use of prestressed soil anchors. This procedure will maintain the soil while the permanent wall is built. The permanent wall should be designed to accept all appropriate soil, dead and live loads. The temporary wall shall not contribute to the strength of the permanent wall (See Figure 5-14).

5.6.2 Design of MSE Walls

A. Corrosion Rates:

The corrosion rates for metallic reinforcement shall follow LRFD **[11.9.8.1]**; however, they apply to non-corrosive (Slightly or Moderately Aggressive) Environments only. Metallic reinforcement shall not be used in Extremely

Aggressive Environments without approval from the State Structures Design Engineer.

B. Soil Reinforcement:

1. Soil Reinforcement and connections for permanent walls shall be designed for a design life of 75 years except for walls supporting abutments on spread footings which shall be designed for a design life of 100 years.
2. Soil Reinforcement for temporary walls shall be designed for a design life of not less than the contract time of the project or three years whichever is greater.
3. The steel soil reinforcement shall not exceed $0.55F_y$ for steel straps, and $0.48F_y$ for welded wire mats or grids. The steel shall be ASTM A82, Grade 450.
4. Vertical stresses at each reinforcement level shall include consideration for local equilibrium of all forces to that level only and shall include the effects of any eccentricity.
5. Steel soil reinforcement for permanent walls shall be galvanized. Steel soil reinforcement for temporary walls may be galvanized or uncoated (black) steel.
6. Epoxy coated reinforcement of LRFD [11.9.5.1.6] is not permitted. Passive metal (i.e., stainless steel, aluminum alloys, etc.) soil reinforcement are permitted for use only with written approval of the SDO.
7. Geosynthetic geogrids included on the Department's QPL should be used.
8. Soil reinforcement shall not be skewed more than 15 degrees from a position normal to the wall panel unless necessary and clearly detailed for acute corners.
9. Soil reinforcement lengths, measured from the back of the facing element, shall not be less than the greater of the following:
 - a. MSE Walls and Walls in Front of Abutments on Piling [11.9.5.1.4]
 - (1) $L = 2.5$ m minimum length
 - (2) $L = 0.7 H_1$, $H_1 =$ mechanical height of wall, in m (See Figures 5-15 and 5-16)

- (3) $L =$ length in m, required for external stability design
- b. MSE Walls In Front of Abutments on Spread Footings **[11.9.7]**
 - (1) $L = 6.7$ m
 - (2) $L = 0.6 H_1 + 1.8$ m
 - (3) $L = 0.7 H_1$

Commentary: As a rule of thumb, for an MSE structure with reinforcement lengths equal to 70% of mechanical height, the anticipated maximum calculated bearing pressure can be anticipated to be about 135% of the overburden weight of soil and surcharge. It may be necessary to increase the reinforcement length for external stability to assure that the allowable bearing pressure specified equals or exceeds this value.

10. External Stability:

H_1 , the mechanical height, of an MSE structure shall be taken as the height measured to the point where the potential failure plane (line of maximum tension) intersects the ground surface, as shown in Figure 5-15 and 5-16 and can be calculated using the equation provided in Figure 5-15. The methodology for this analysis is included in FHWA publication RD-89-043 "Reinforced Soil Structures, Volume I." In addition to the two conditions shown in AASHTO, horizontal backslope and infinite sloping backfill, we are including Figures 5-15 and 5-16 to illustrate broken back slope conditions with and without traffic surcharge. If the break in the slope behind the wall facing is located within two times the height of the wall ($2H$), then the broken back slope design shall apply. If a traffic surcharge is present and is located within $0.5H$ of the back of the reinforced soil mass, then it shall be included in the analysis.

The Geotechnical Engineer of Record for the project is responsible for designing the strap lengths for the external conditions shown in Figure 5-1 and any other conditions that are appropriate for the site.

11. Acute corners less than 75 degrees (MSE Walls):

When two walls intersect forming an internal angle of less than 75 degrees, the nose section shall be designed as a bin wall. The calculations for this special design shall be submitted with the plans for review and approval. Structural connections between the wall elements within the nose section shall be designed to create an at-rest bin effect without eliminating the flexibility of the facing to tolerate differential settlements. The design for connections to facing elements and allowable stresses in the connecting members shall conform to the

requirements for the wall design in general. The nose section shall be designed to settle differentially from the remainder of the structure with a slip joint. Facing panel overlap or interlock or rigid connection across that joint is not permitted.

Soil reinforcements shall be designed to restrain the nose section by connecting directly to each of the facing elements in the nose section and running back into the backfill of the main reinforced soil mass at least to a plane 3 m beyond a Rankine failure surface. Design for facing connections, obstructions, pullout and allowable stresses in the reinforcing elements shall conform to the requirements of the wall design in general (See Figure 5-17).

12. Minimum Wall embedment in the soil shall be determined by consideration of both scour and bearing capacity. The District Drainage and Geotechnical Engineers shall be consulted in determining the elevation of the top of leveling pad which must also conform to a minimum embedment to the top of leveling pad of 500 mm as determined in Figure 5-18.

13. Apparent Coefficient of Friction (F^*):

The coefficient of friction (F^*) need not be reduced for backfills that are saturated and are properly placed and compacted.

14. Connections (MSE Walls):

The soil reinforcement to facing panel connection shall be designed to assure full contact. Normally mesh and bar mats are connected to the facing panel by a pin passing through loops at the end of the reinforcement and loops inserted into the panels. If these loops are not aligned, then some reinforcement will not be in contact with the pins causing the remaining reinforcement to be unevenly stressed and/or over stressed. If the quality of this connection cannot be assured then the allowable stress in the soil reinforcement and its connections should be reduced accordingly.

15. End Bents Behind MSE Walls:

- a. End Bents on Piling:

- (1) All end bent piles must be plumb.
- (2) The front face of the end bent cap or footing shall be a minimum distance of 610 mm behind the front face of the wall. (610 mm minimum clear distance between face of piling and leveling pad). This dimension is based upon the

use of 455 mm piles. For larger piles, the clear distance between the wall and pile shall be increased such that no strap is skewed more than 15° .

- (3) Soil reinforcement to resist the overturning produced by the earth load, friction, and temperature shall be attached to end bents. If the long-term settlement exceeds 100 mm, the reinforcement shall not be attached to the end bent. In this event, a special wall behind the backwall shall be designed to accommodate the earth load.

b. End Bents on Spread Footings:

- (1) The spread footing shall be sized so that the bearing pressure shall not exceed 192 kPa.
- (2) The footing shall be a minimum distance of 305 mm behind the facing panel.
- (3) The minimum distance between the centerline of bearing on the end bent and the back of the facing panel shall be 1.2 meters.

16. Facing Panels (MSE Walls) **[11.9.5.1.6]:**

The basic (typical) panel size shall not exceed 3.75 m^2 in area, and special panels (top out, etc.) shall not exceed 4.65 m^2 in area. Full-height facing panels shall not exceed 2.4 meters in height. Use of larger panels will be considered by the SDO on a case-by-case basis. The reinforcing steel concrete cover shall comply with Table 2.2.

17. Soil reinforcement shall not be attached to piling, and abutment piles shall not be attached to any retaining wall system.

18. The soil reinforcement connection at the face of the wall shall be equal to a minimum of $100\% T_{\max}$ **[11.9.5.1.2]**; where:

$$T_{\max} = T_a \text{ (maximum stress in soil reinforcement).}$$

19. Creep reduction factors in geosynthetic soil reinforcement shall be included in seismic designs.

20. Overturning **[11.9.4.3]:**

Use a soil resistance factor of 0.67 for overturning.

21. Polymeric Reinforcements in Soils **[11.9.5.1.3]:**

For geosynthetics, T_a shall not exceed the following values:

- a. $T_a = 17\% T_{ult}$ for permanent walls and critical temporary walls.
 - b. $T_a = 29\% T_{ult}$ for temporary walls.
- C. Minimum Front Face Embedment:
The minimum front face embedment shall comply with Figure 5-18.

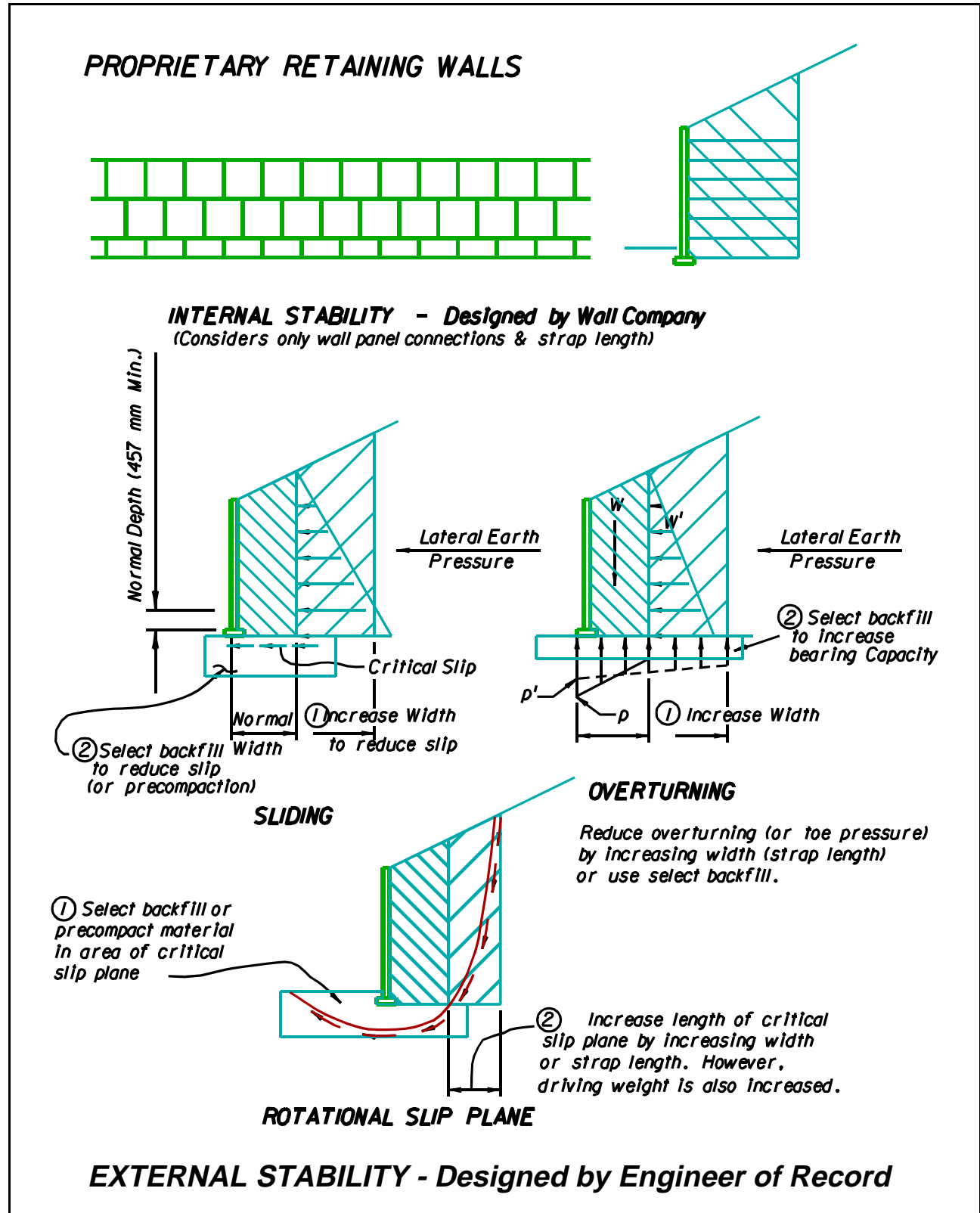


Figure 5-1

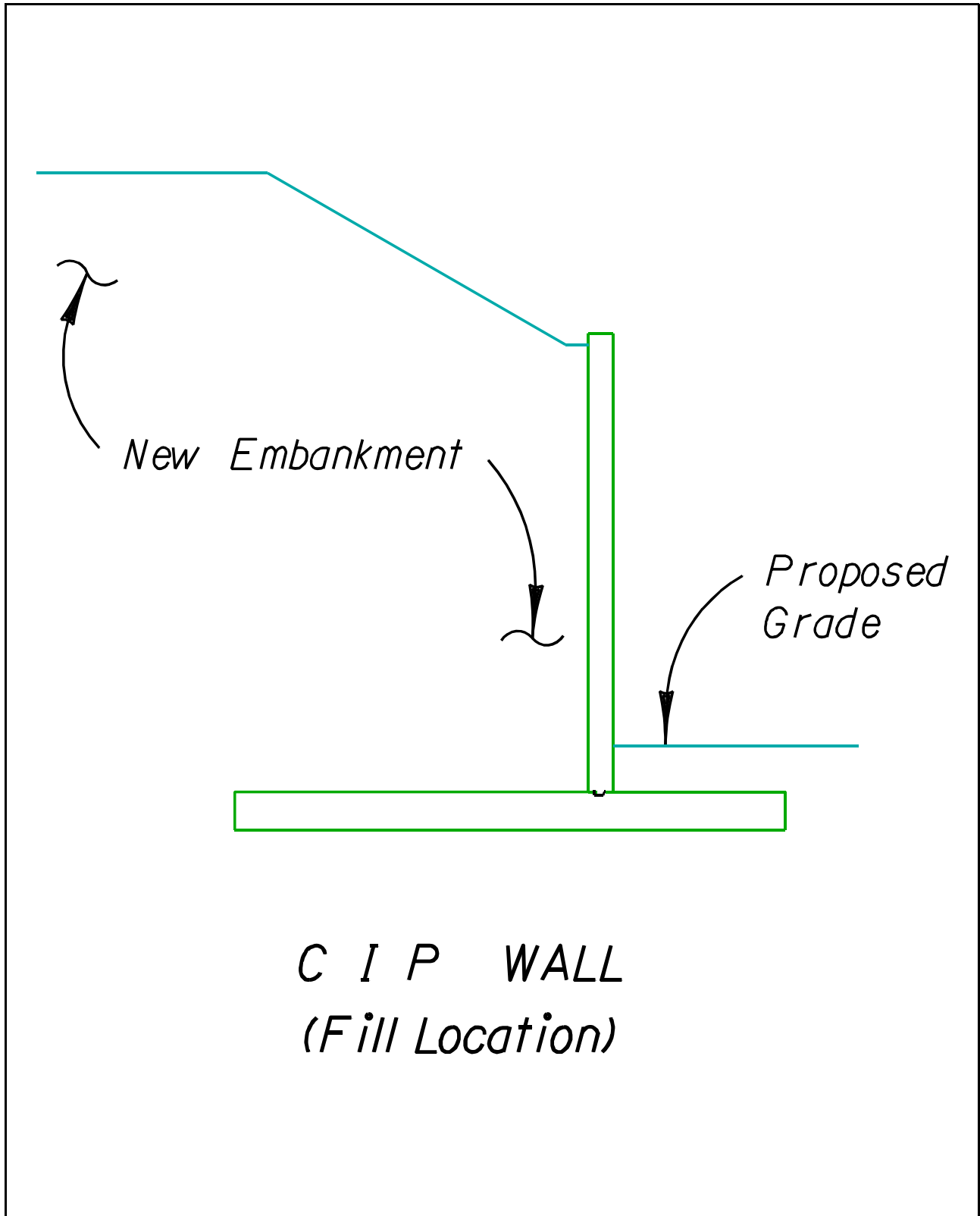


Figure 5-2

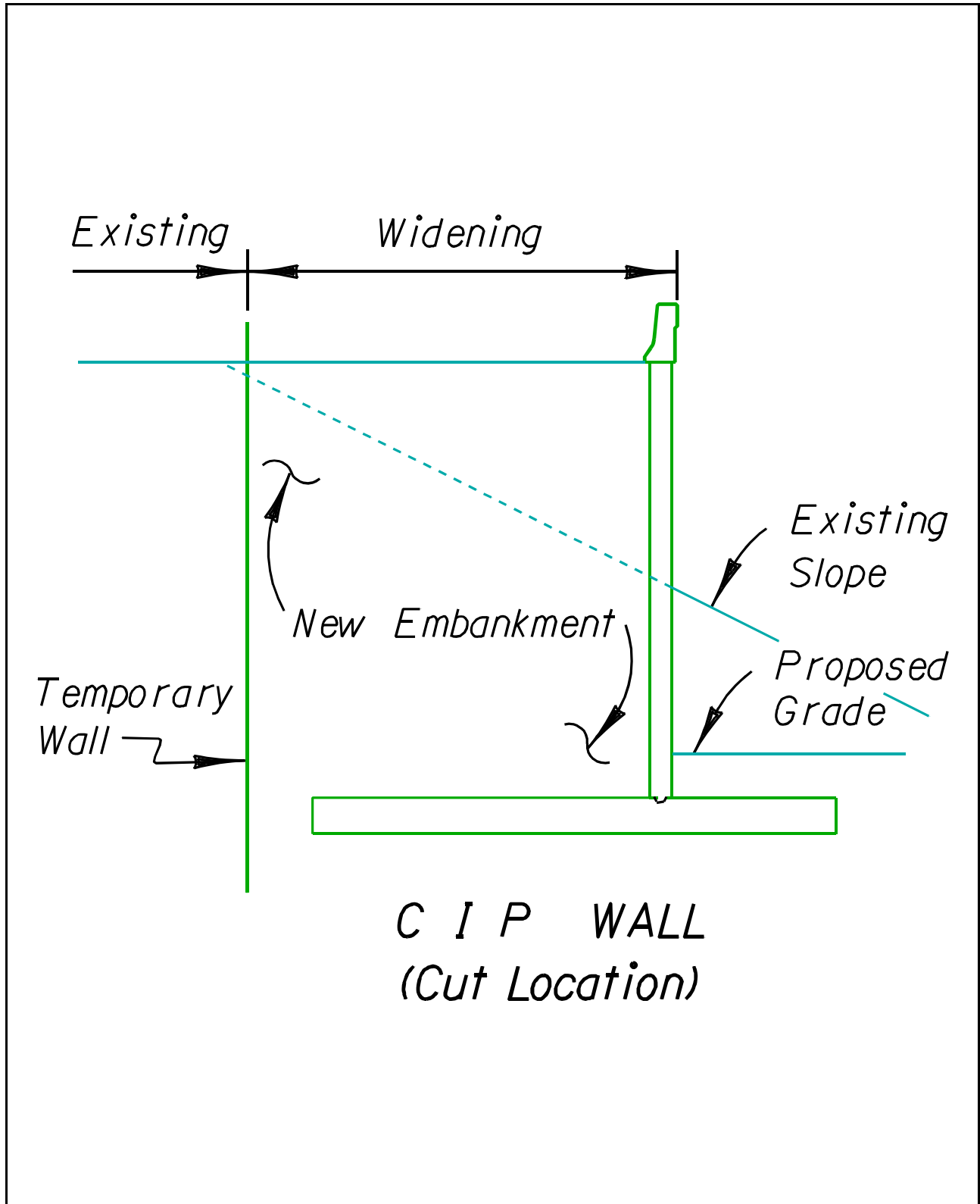


Figure 5-3

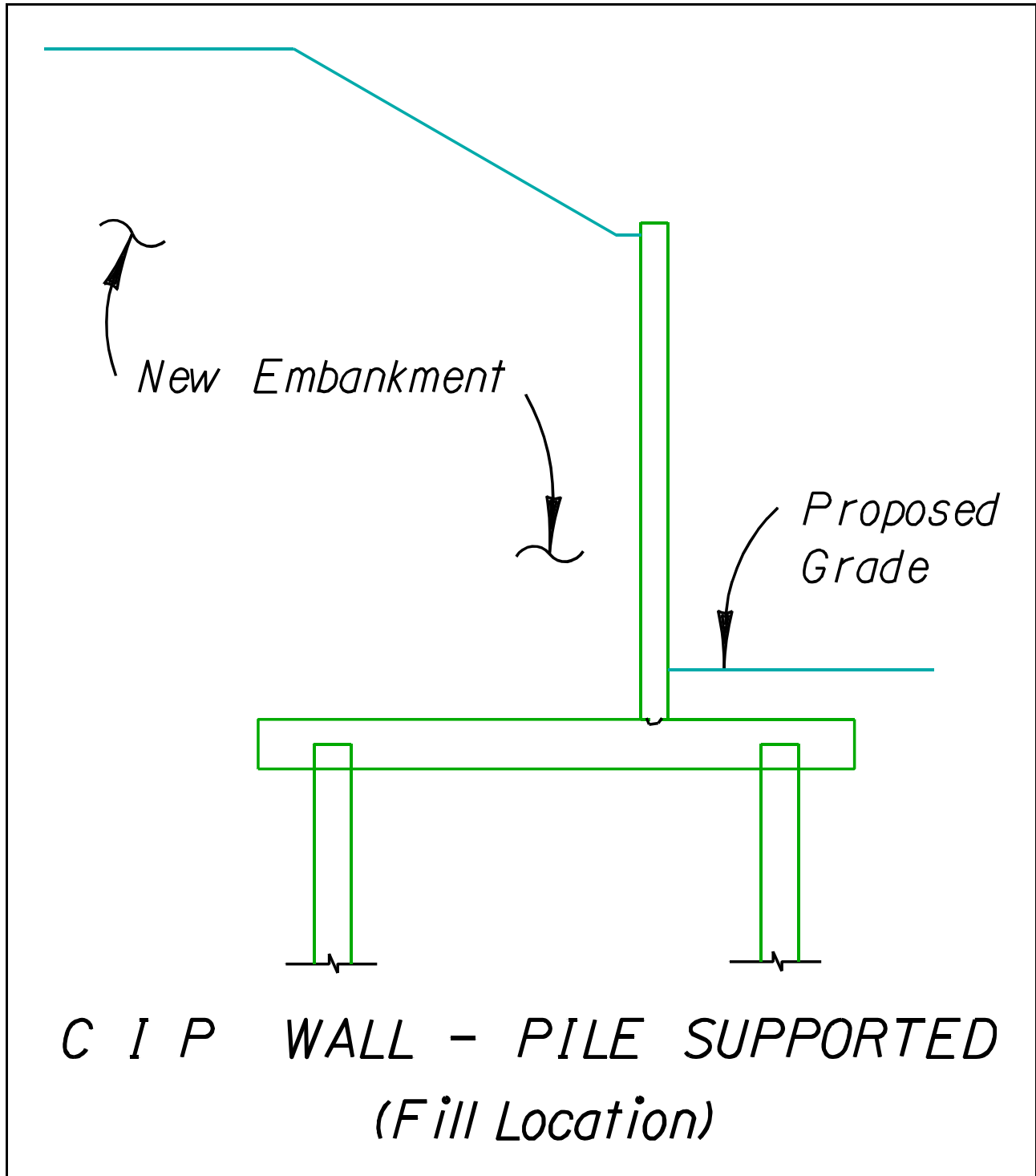


Figure 5-4

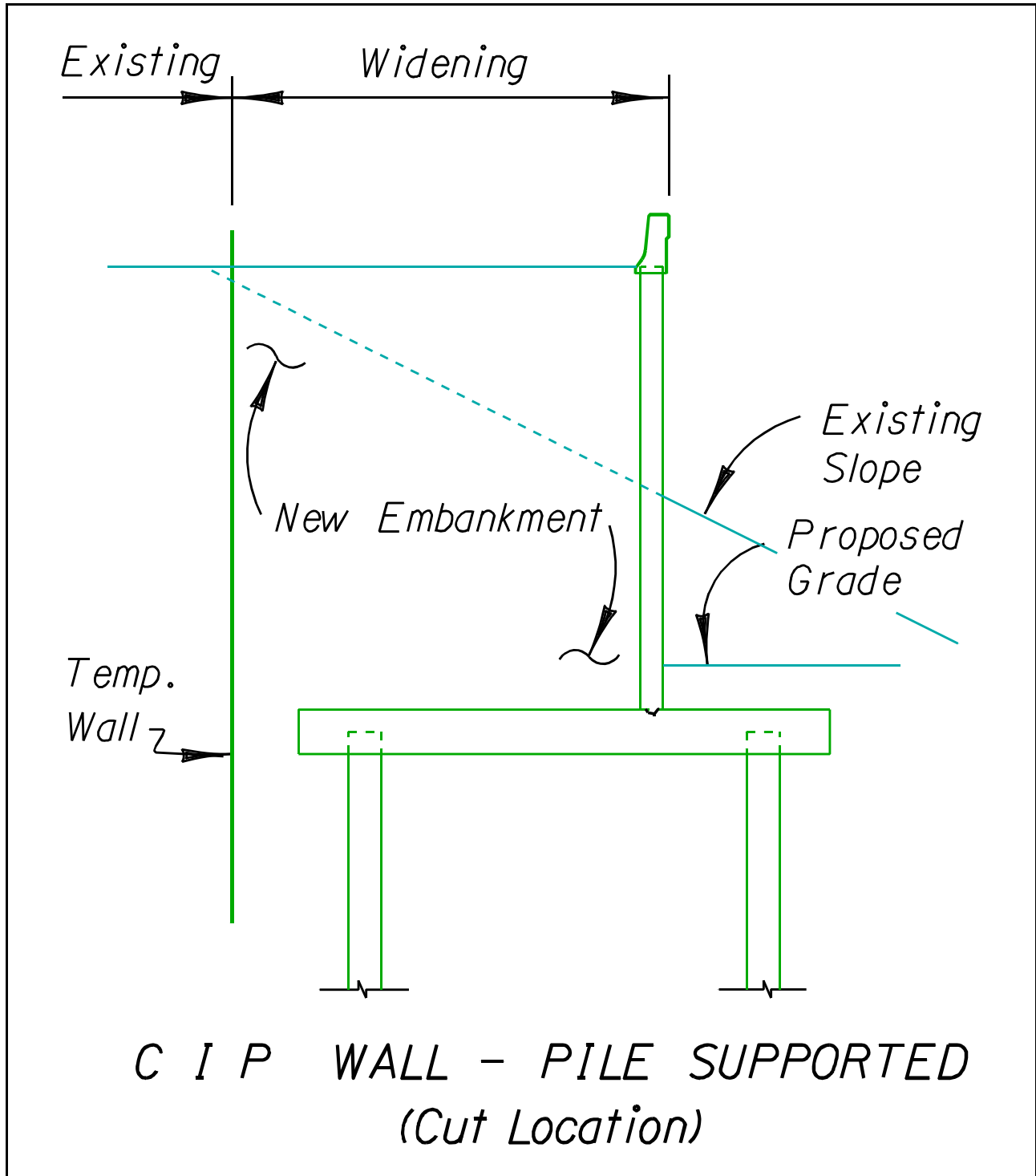


Figure 5-5

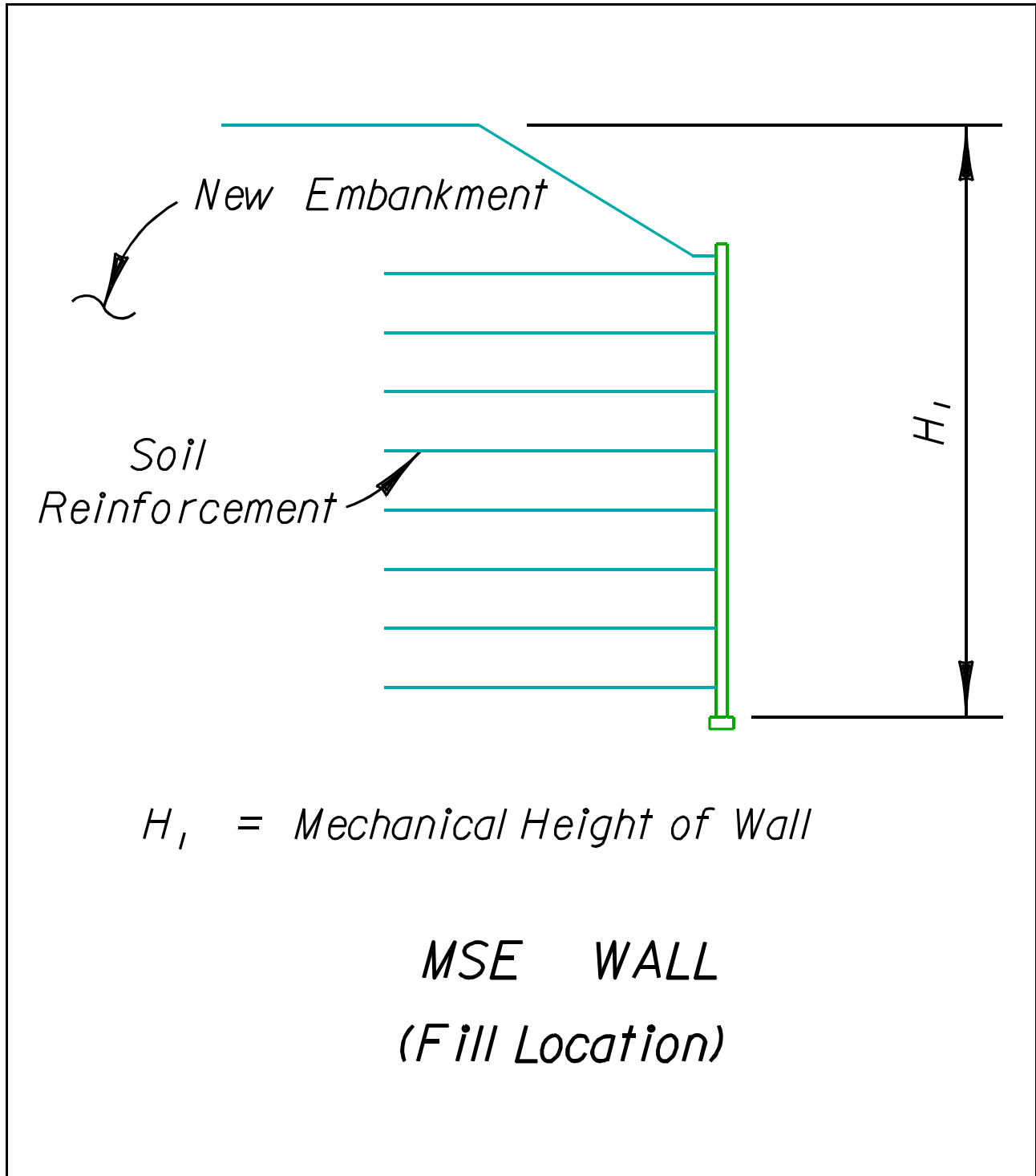


Figure 5-6

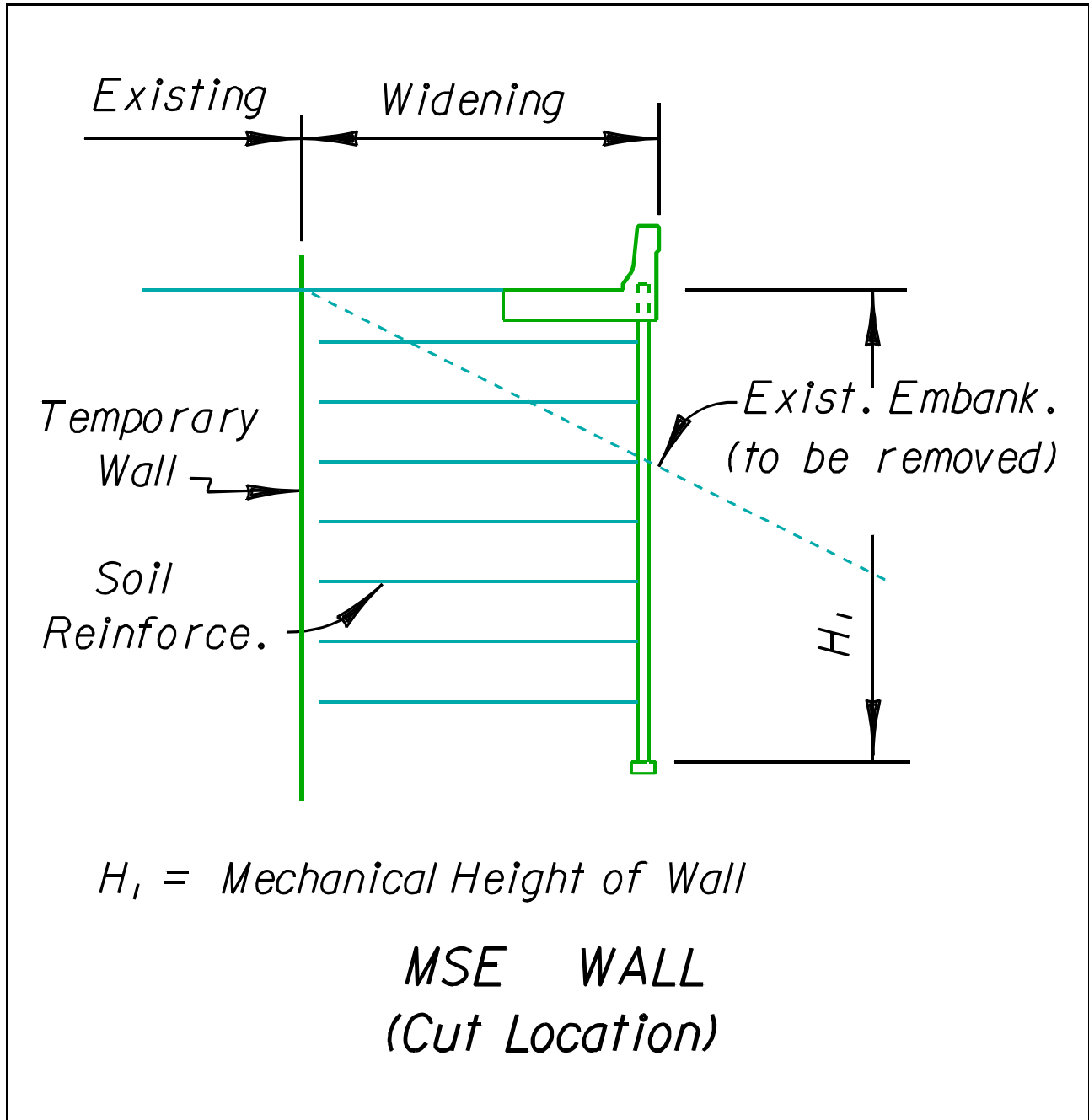


Figure 5-7

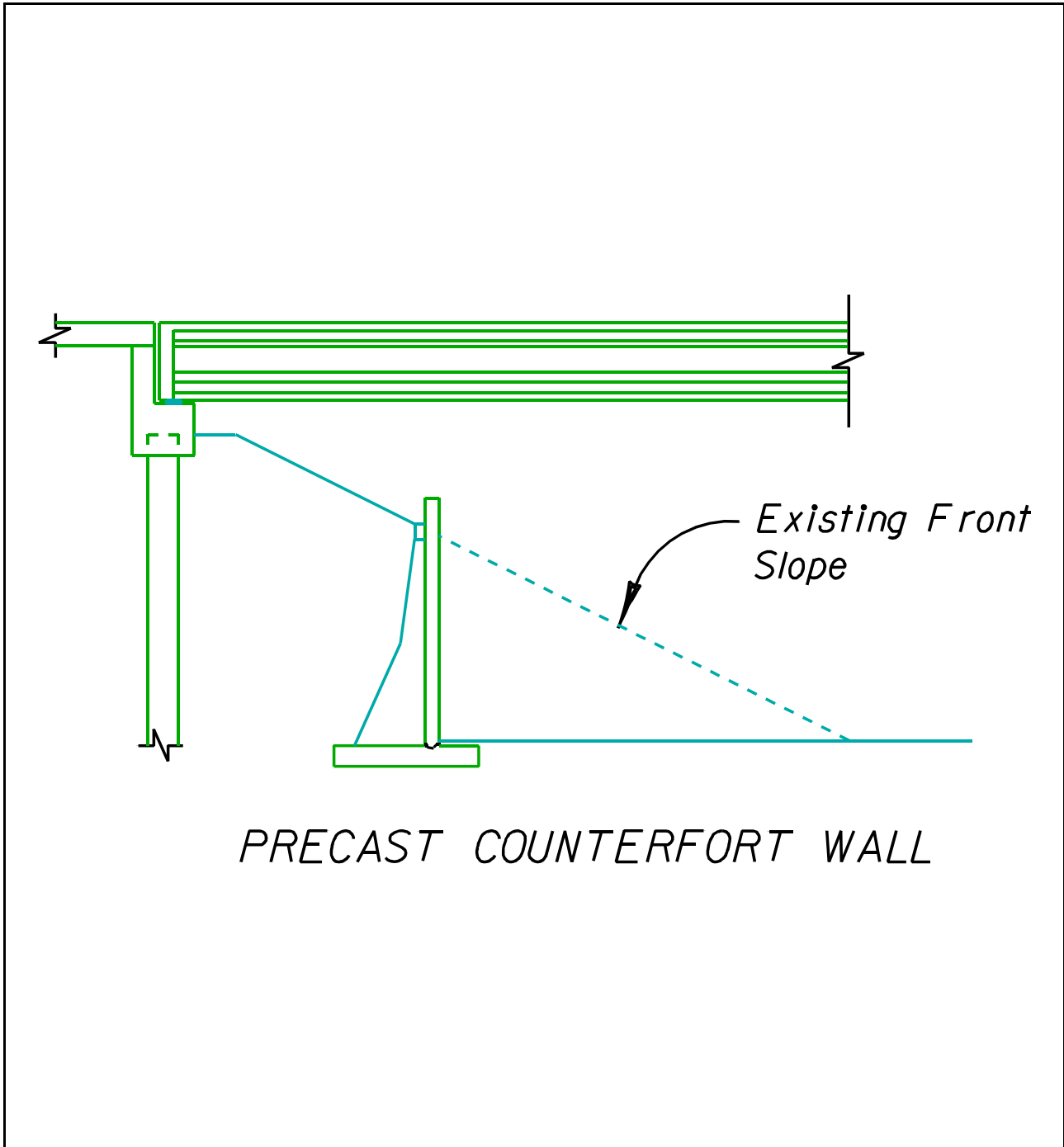


Figure 5-8

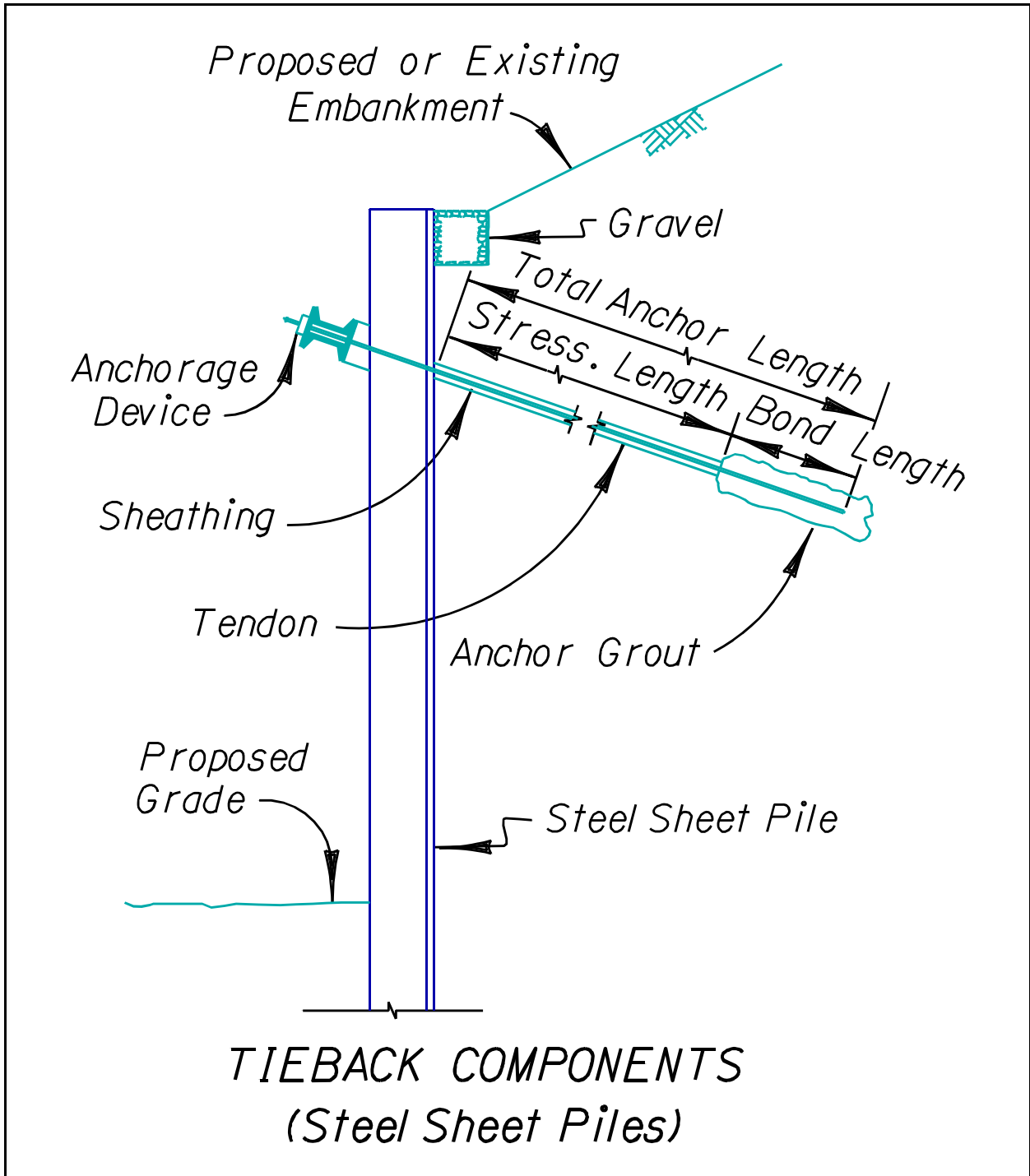


Figure 5-9

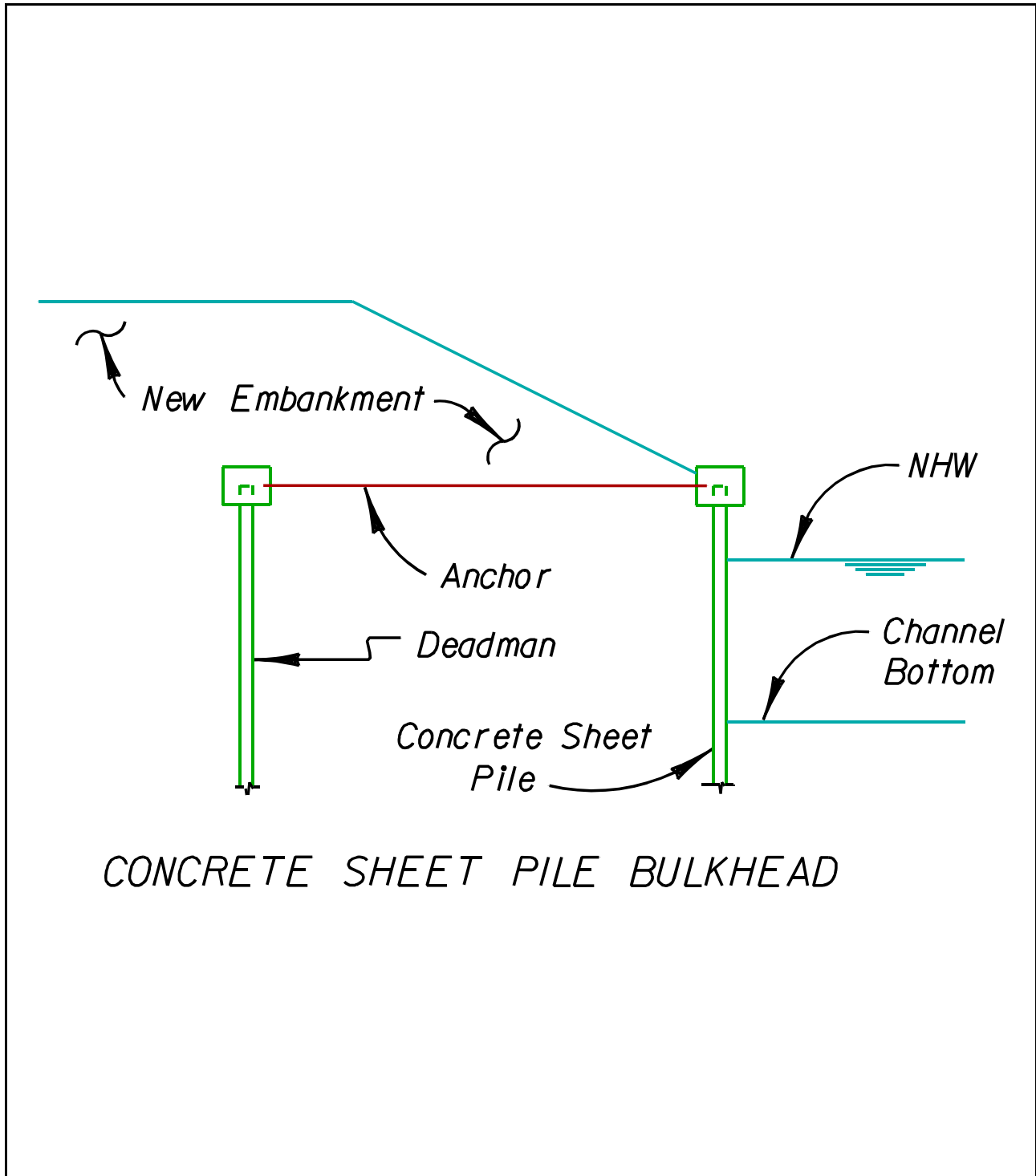


Figure 5-10

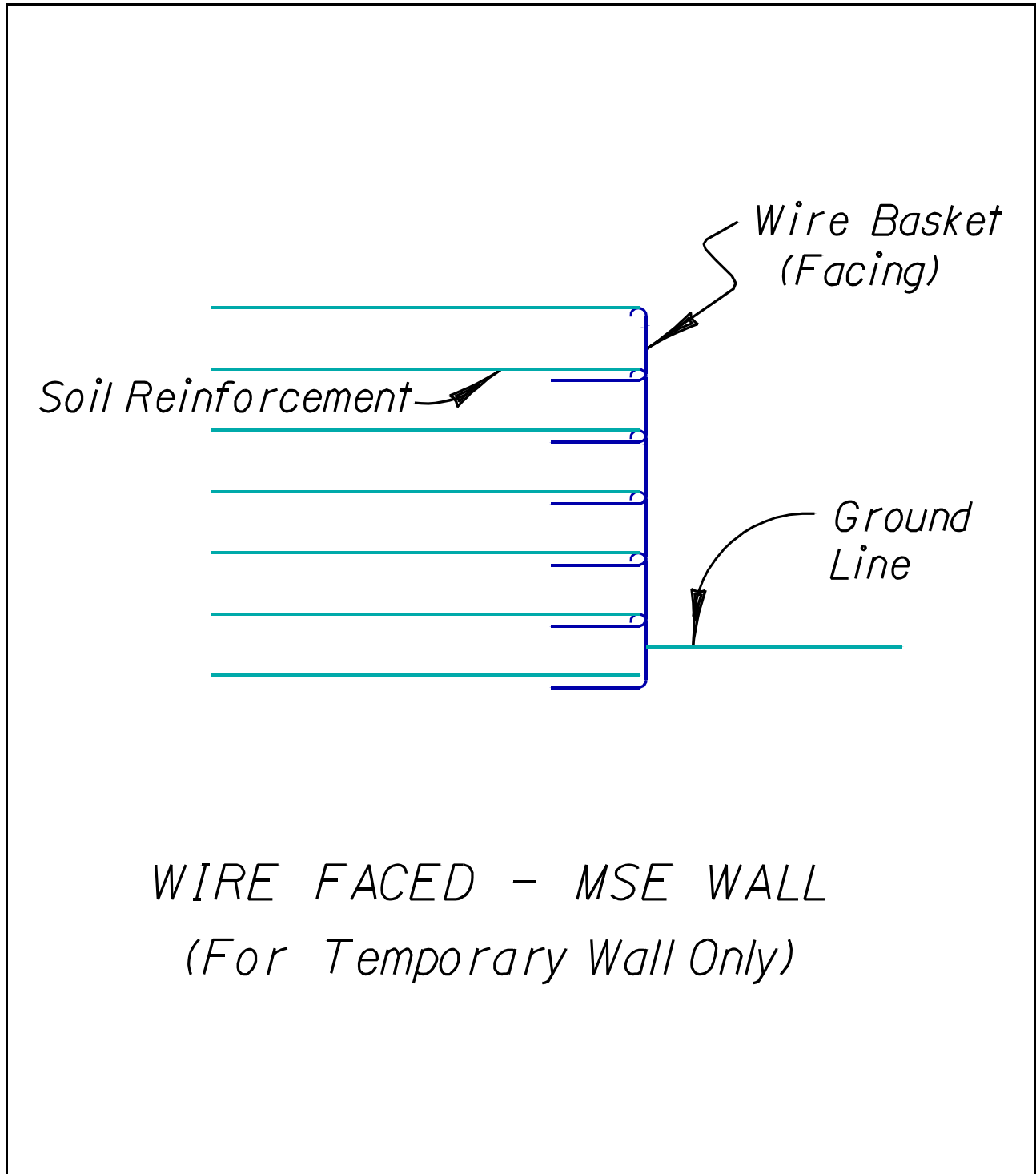


Figure 5-11

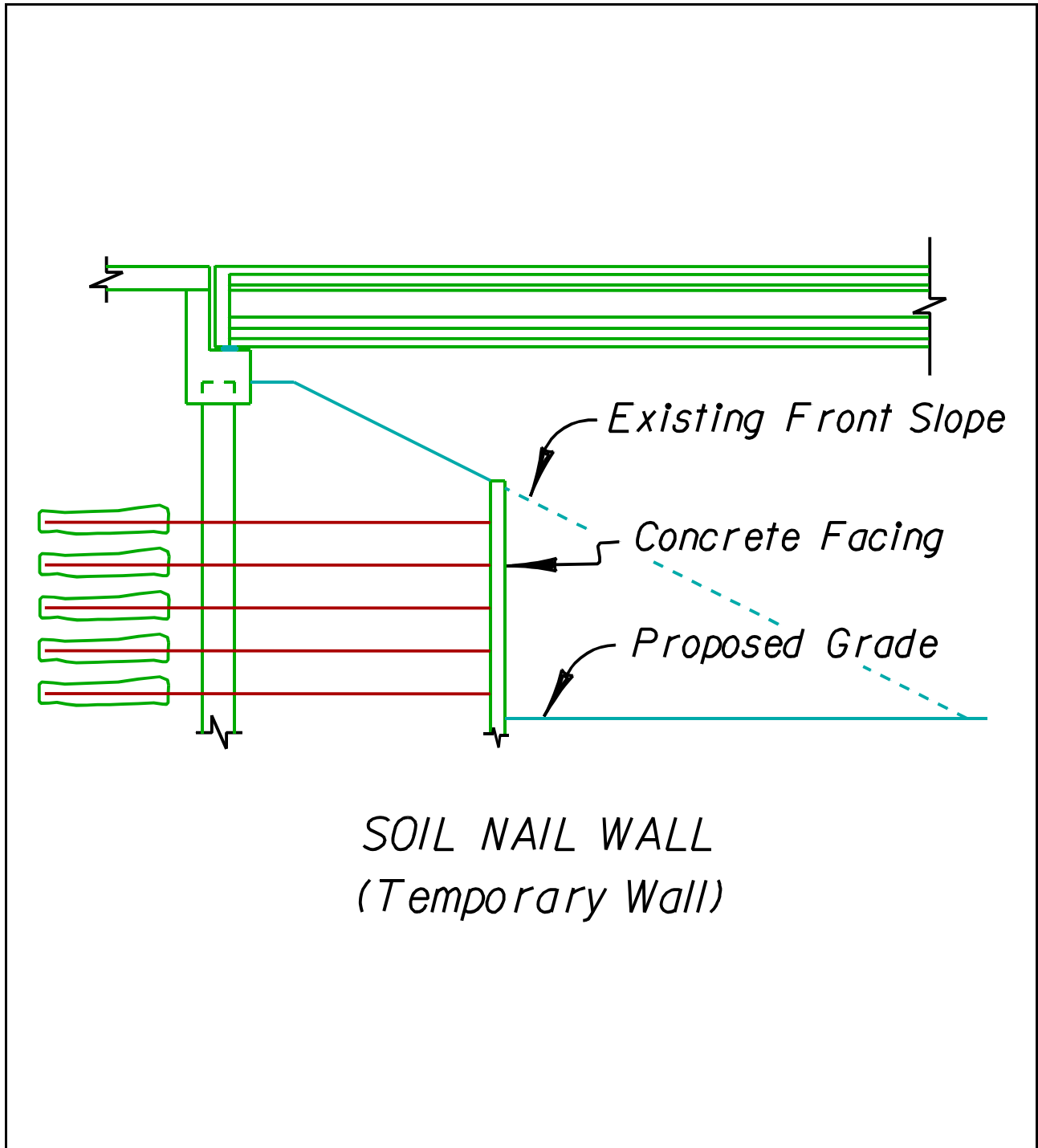


Figure 5-12

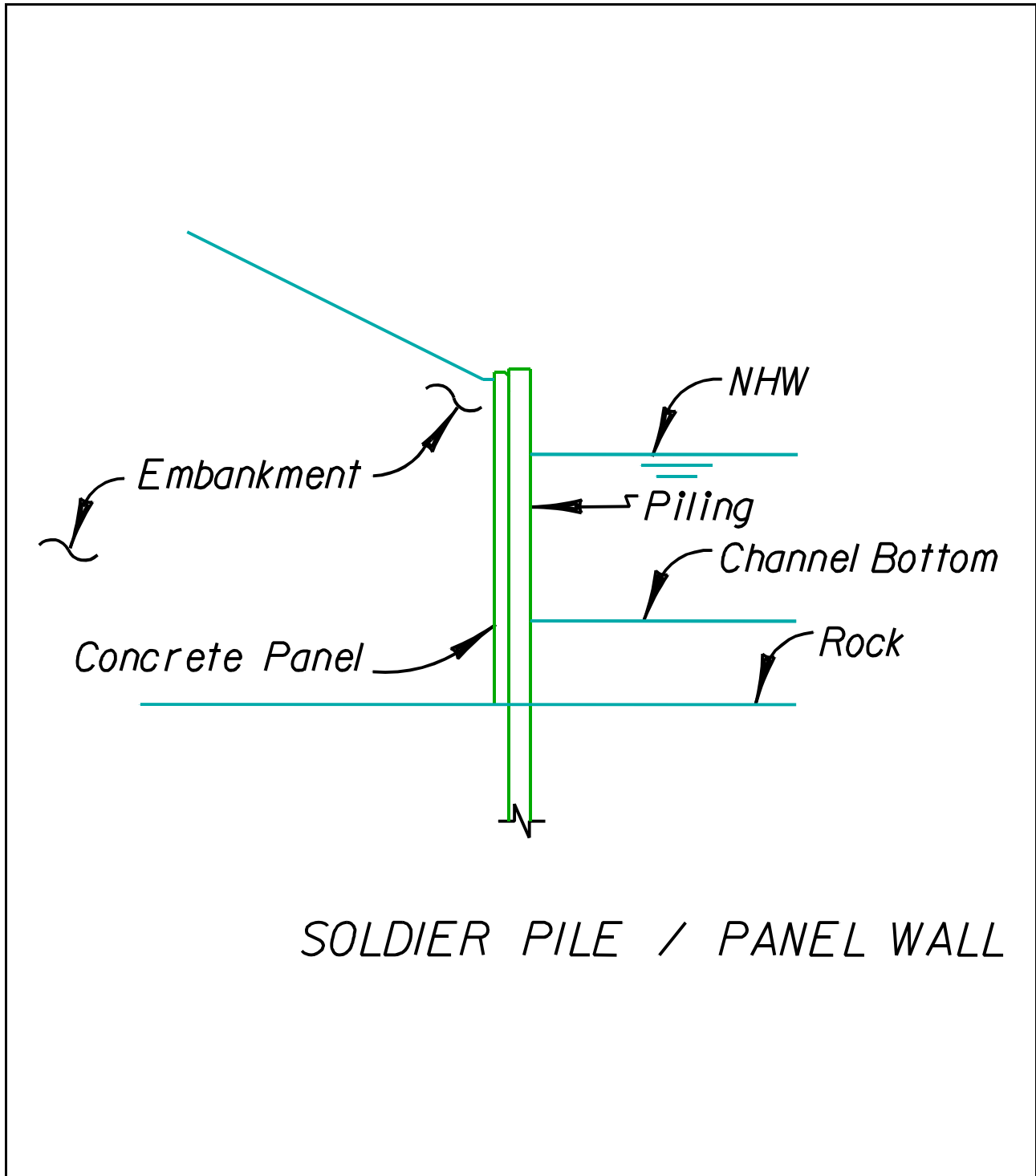


Figure 5-13

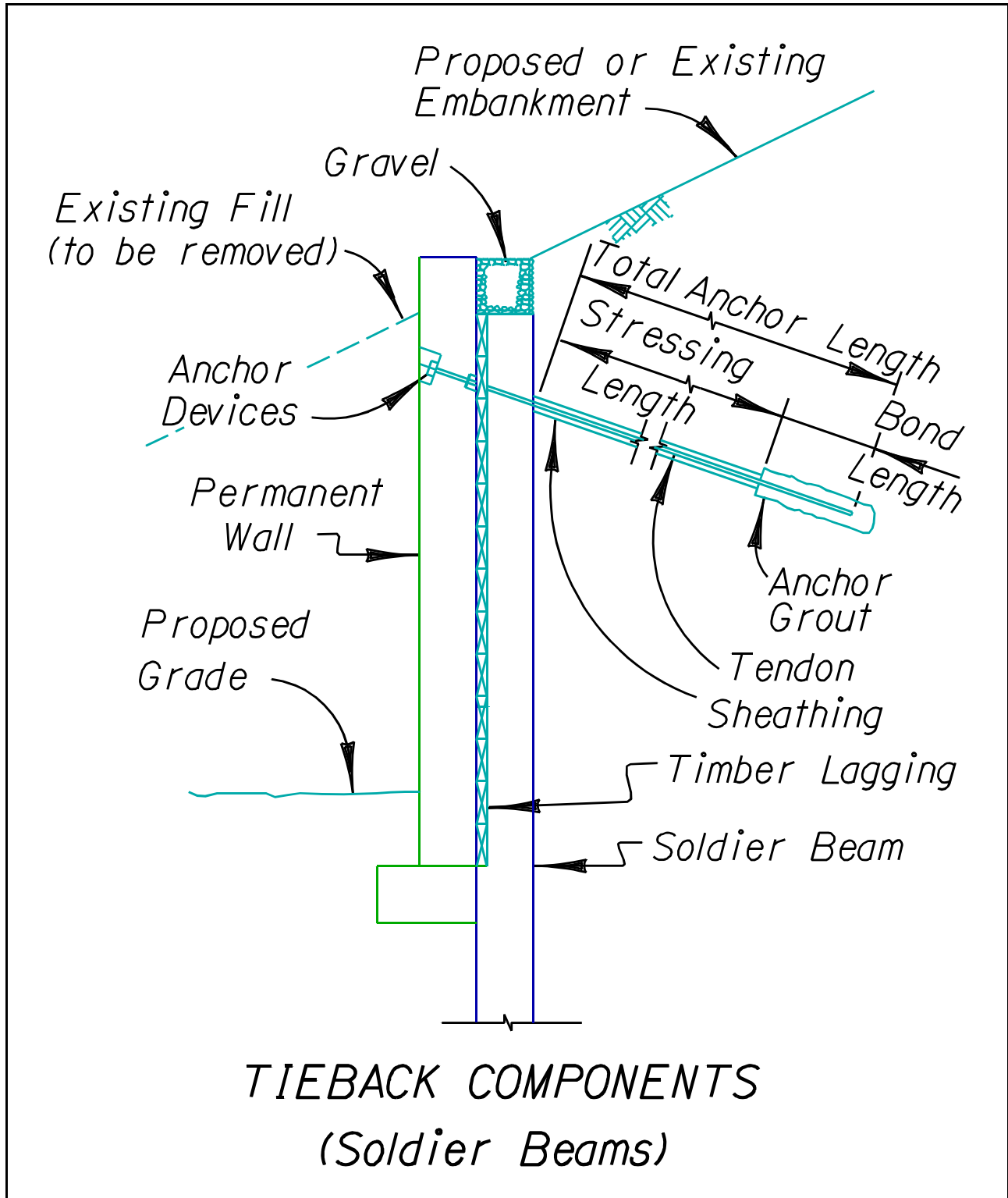


Figure 5-14

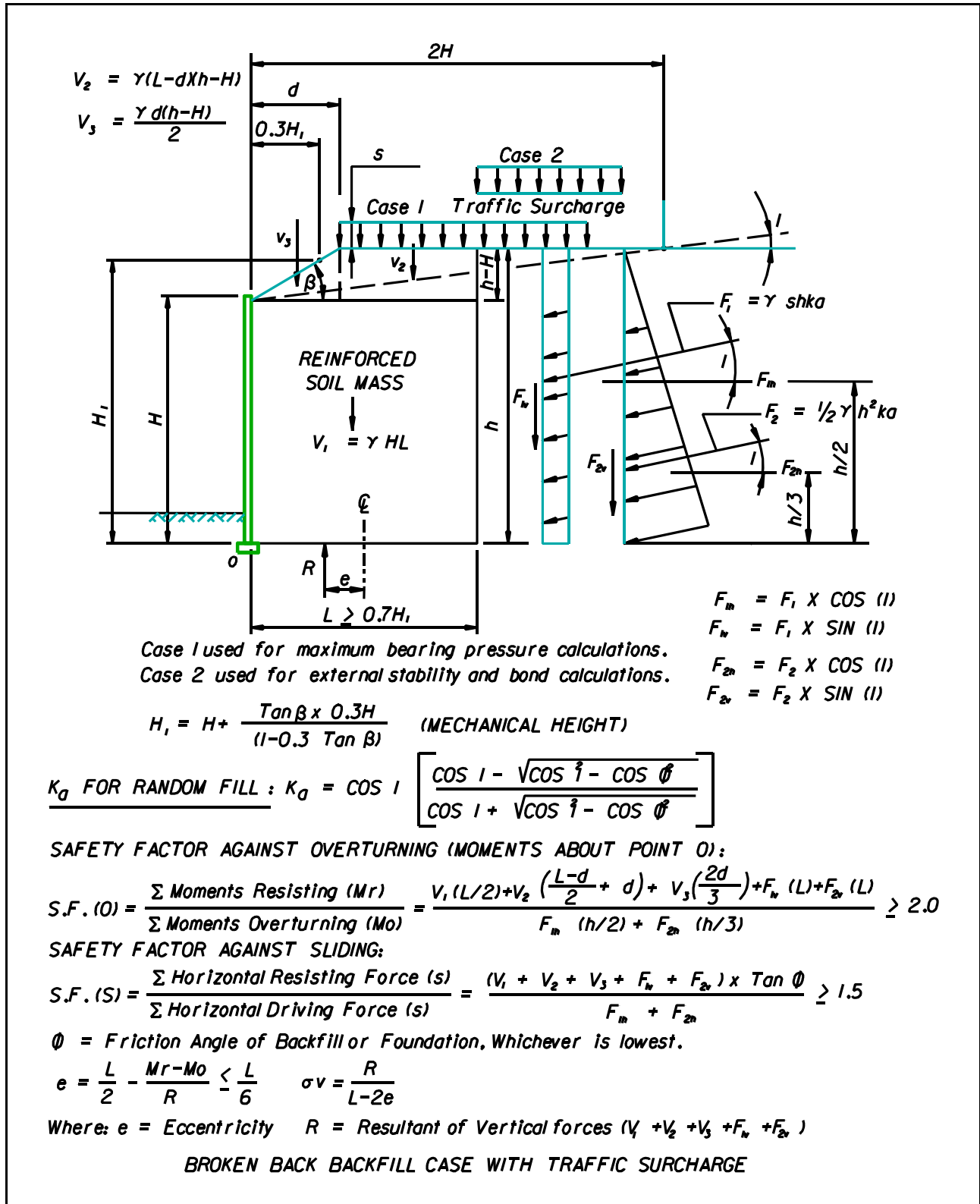


Figure 5-15

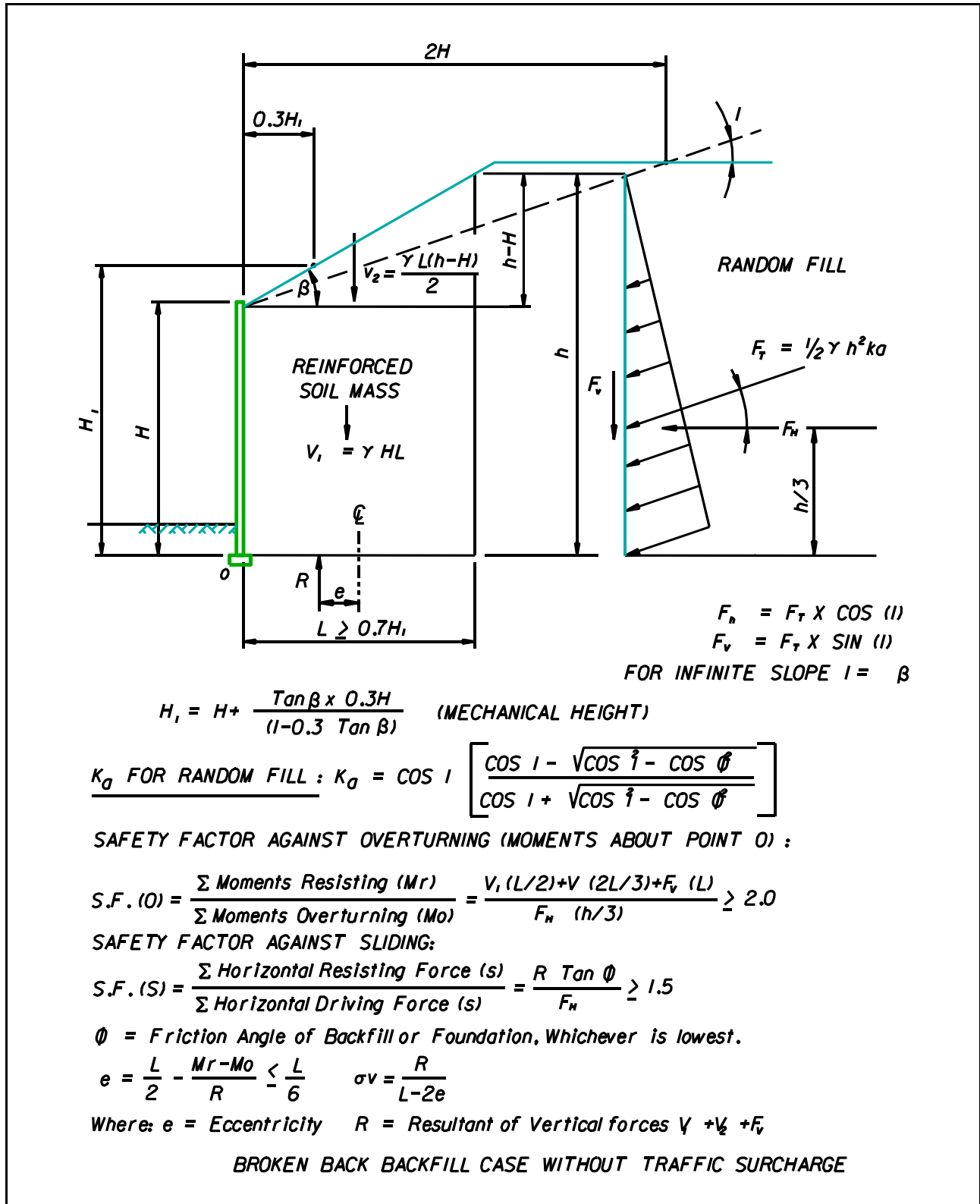
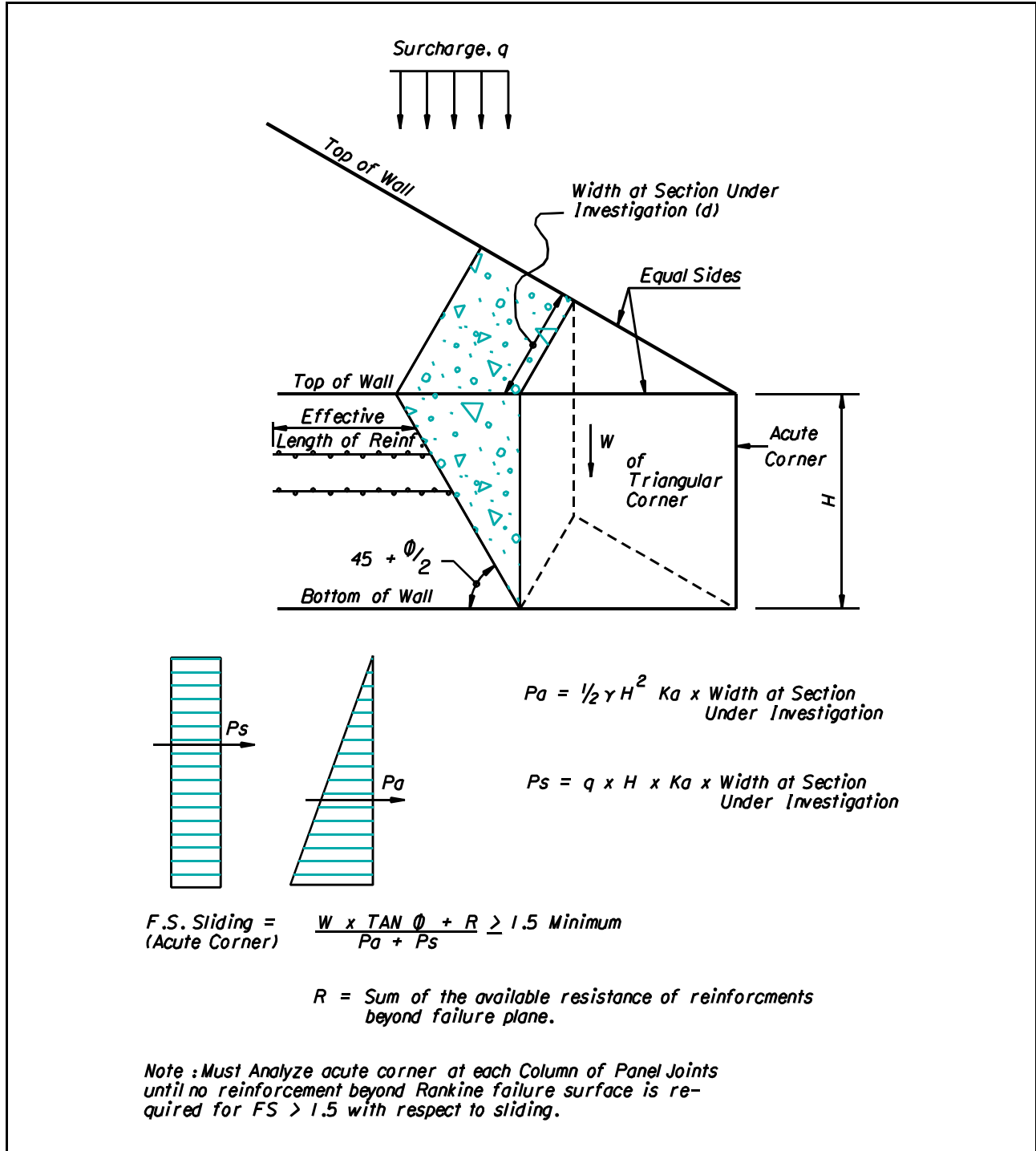


Figure 5-16



ACUTE CORNER PRESSURE DIAGRAM

Figure 5-17

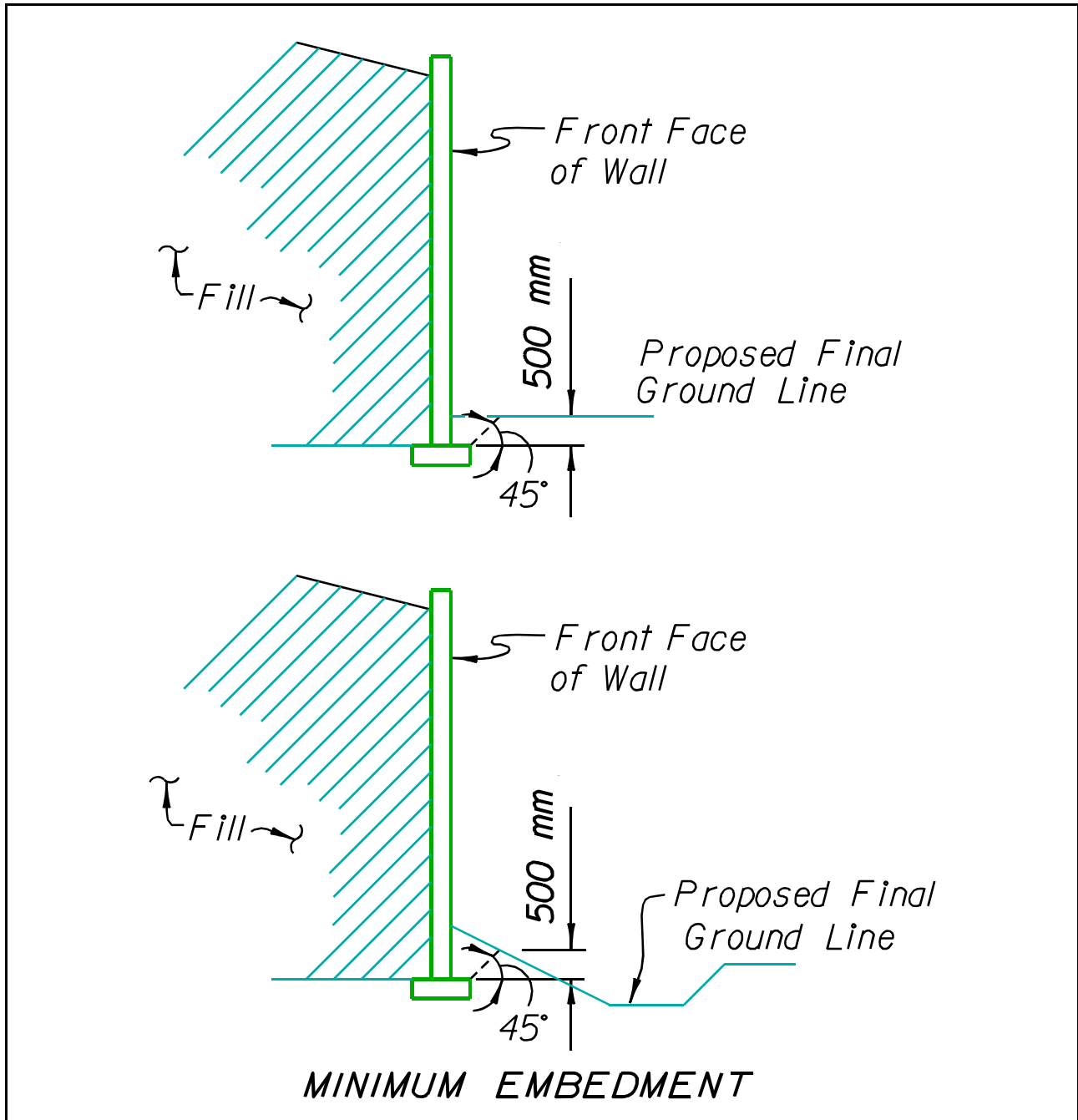


Figure 5-18

Chapter 6

SUPERSTRUCTURE ELEMENTS AND APPURTENANCES

6.1 General

This Chapter contains information and criteria related to the design, reinforcing, detailing, and construction of superstructure elements and appurtenances for bridge structures and includes deviations from LRFD that are required for these same elements and appurtenances. The Chapter includes elements that are part of the superstructure such as deck slabs, beams, barriers, curbs, and pilasters as well as appurtenances that are essential accessories to the superstructure such as joints, bearings, and deck drains.

6.2 Curb Heights on Bridges

For all bridge projects that utilize curbs, the bridge curb height and batter shall match the curb height and batter of the roadway approaches. Bridges with sidewalks are normally encountered in an urban environment, and the curb will normally be 150 mm in height. Where the roadway approaches the bridge with a raised median, the median on the bridge shall match that on the roadway.

6.3 Pilaster Details for Roadway Lighting

See the Standard Drawings (Topic No.: 625-020-300) for details to be used in designing bridges that require lighting standards.

6.4 Deck Drainage Systems [2.6.6]

6.4.1 Deck Drains

Simple deck drains that are encased in concrete, such as scuppers, shall be either PolyVinyl-Chloride (PVC) Schedule 80 UV-Resistant, Gray Cast Iron, or Ductile Cast Iron. Other deck drains that are encased in concrete, such as grates and inlets, shall be either Gray Cast Iron, Ductile Cast Iron, or Galvanized Steel; except that in Extremely Aggressive Environments, Gray Cast Iron or Ductile Cast Iron must not be used.

6.4.2 Drain Conveyances

Drain conveyances that are encased in concrete shall be PVC Type PSM, Polyethylene (PE),

or Ductile Cast Iron. Drain conveyances not encased in concrete shall be Machine-Made or Filament-Wound “Fiberglass” (Glass-Fiber-Reinforced-Thermosetting-Resin), or Ductile Cast Iron; except that in Extremely Aggressive Environments, Ductile Cast Iron must not be used.

6.5 Traffic Railing Barriers [13.7]

Unless otherwise approved, all new traffic railing barriers proposed for use in new construction, rehabilitation, reconstruction, and widening projects, as well as all traffic railing barrier systems proposed as a retrofit for existing traffic railing barriers, shall be proven effective through successful crash testing. Crash testing shall be performed in accordance with the National Cooperative Highway Research Program (NCHRP) Report No. 350 and LRFD.

The traffic railing barrier shown on Index 700 of the Standard Drawings, and the retrofit detail shown as Scheme 1 in the Roadway and Traffic Design Standards, Index No. 401 (Topic No.: 625-010-003), have been determined to meet the applicable crash testing requirements.

6.5.1 General Requirements

In addition to the previous requirements, all traffic railing barriers shall:

- A. Meet the requirements for Test Level 4 (TL-4) in NCHRP Report No. 350.
- B. Meet the strength and geometric requirements of LRFD **[13.6] [13.7] [13.8]** in accordance with the test levels and crash test criteria.
- C. Be upgraded on both sides of a structure when widening work is proposed for only one side and the traffic rail barrier on the other side does not meet the criteria for new traffic rail barriers.

6.5.2 Existing Bridges with Sub-Standard Traffic Railing Barriers

When work is proposed on an existing structure with traffic railing barriers that do not meet the criteria for new railing barriers as provided above, the existing traffic railing barriers shall be replaced or retrofitted to meet the crash-worthy criteria unless an variation is approved. As a minimum, the traffic railing barriers and portions of the bridge deck to which they are anchored shall meet the “General Requirements” of this article.

If the bridge structure is scheduled for major rehabilitation or replacement within five years, the traffic railing barrier may remain, as is, unless a known safety problem exists. This situation should be documented as a variation.

For movable bridges only, if the sole, major work is related to rehabilitation of the mechanical

and/or electrical equipment, the railing will not have to be retrofitted.

6.5.3 Traffic Railing Barriers for Historic Bridges

With respect to historic bridge sites and areas of great aesthetic concern, case-by-case variation to the crash-worthy criteria will be considered within the following limits:

- A. A variation request shall include written justification explaining the special historic or aesthetic concerns at the site, and why a crash-worthy traffic railing barrier is not compatible.
- B. The design speed and operating speed shall be no greater than 65 km/h.
- C. The general strength and geometric requirements stated above are met.

6.5.4 Requirements for Test Level 5 (TL-5)

Consideration should be given to providing a traffic railing barrier that is in conformance with the requirements of Test Level 5 (TL-5) as included in the NCHRP Report No. 350 when the traffic railing barrier meets the Criteria for TL-5 [13.7.2].

Commentary: There is no Standard Index Sheet for TL-5, and at this time there are no plans for including TL-5 in the Standard Drawings.

6.6 General Design Policy for Expansion Joints

Expansion joints with elastomeric neoprene seals and other joint devices shall be designed in accordance with LRFD, the Department's Standard Specifications for Road and Bridge Construction, and this Chapter.

The EOR shall minimize the use of joints by providing deck slab continuity at intermediate bents or piers. Deck joints shall be designed to resist loads and accommodate movements at the service and strength limit states and to satisfy the requirements of the fatigue and fracture limit states.

Indices Nos. 400 thru 403 of the Standard Drawings include strip seal expansion joints with two alternate cross-sections. Whenever possible and after confirming their adequacy, these standard designs shall be used when a strip seal is determined to be the best joint type.

Shop Drawings are required for all expansion joints except poured rubber seals and open joints.

Commentary: Bridge deck expansion joints are a continuous maintenance problem at best

and, at worst, may be a source of structural deterioration resulting in a major bridge maintenance problem.

6.7 Joint Types and Specifications [14.5.6]

6.7.1 Joint Types by Group

Joint Types by Group that may be encountered are:

A. Group 1:

Strip seal, compression seal, poured rubber, open joint, silicon seal, copper water-stop, Jeene, or other proprietary joints with a maximum opening not exceeding 75 mm.

B. Group 2:

Sliding Plate, finger joint, modular, or other proprietary joints with a maximum opening exceeding 75 mm.

6.7.2 Specifications by Joint Type

Strip seals shall be in accordance with the Standard Drawings (Sheet Indices No. 400 thru 403) and the manufacturer's specifications. The anchor studs' **geometric specifications only** shall be in accordance with Index No. 400 of the Standard Drawings. Generally, fabrication of strip seal expansion joints shall conform to the notes of Index No. 400.

Poured rubber seals shall be in accordance with FDOT Specifications, Section 932-1.

In some instances open joints may be acceptable; however, such joints must have provision for diverting drainage without causing damage to the bridge bearings, other structural elements, or the environment and must have the prior approval of the Department.

All other proprietary joints shall be in accordance with the manufacturer's specifications.

Strip seals, silicon seals, or joints other than the compression seals (Group 1) should be used to the maximum extent possible. Sliding plates, finger joints, etc. (Group 2) should be used only for very large movements; i.e., bridges with span lengths greater than 60 meters or where movements of continuous units result in a maximum joint opening width greater than 75 mm.

Commentary: Group 2 joints are normally more complicated to install than Group 1 joints and require closer coordination with the joint manufacturer during installation.

General requirements, selection, design requirements, fabrication, and installation of joints are covered in LRFD [14.5.1] through [14.5.5], respectively.

6.8 Expansion Joint Criteria [14.5.1]

When an expansion joint is required, the joint shall satisfy the following criteria as applicable:

- Proprietary joints shall be designed and installed in strict accordance with LRFD and the manufacturer's specifications unless directed otherwise by the Department.
- The joint components shall accommodate the full range of structural movements without exceeding the manufacturer's limitations and/or design values, particularly at maximum joint opening.
- The joint must provide a good riding surface, relatively free from vibration and noise.
- Frame rails must be designed as necessary to resist all anticipated loads, including dynamic load allowance.
- The joint shall not transfer undue stresses to the structure.
- Elastomers for joint seals shall provide a service life warranty for a period of five(5) years minimum.
- The joint shall be designed for minimum maintenance and ease of access and parts replacement.
- The joint shall be leak-proof with a continuous sealing element for the entire joint length.
- The joint materials shall be resistant to both corrosion and ultraviolet (UV) rays and shall not be a catalyst or vehicle for electrolytic action.

6.9 Temperature Variation Effects on Joints and Bearings [14.4]

Bridge movement due to temperature variation (range) shall be calculated from an assumed mean temperature of 21 degrees Celsius (°C) at the time of construction. The design of the joints and bearings shall be based on the expansion and contraction of the superstructure due to the design range.

The temperature ranges of Table 6.1 shall be used for designing joints and bearings:

Table 6.1 Temperature Range by Superstructure Type

Structural Material of Superstructure	Temperature (Degrees Celsius)			
	Mean	High	Low	Range
Concrete Only	21	35	7	28
Concrete Deck on Steel Girder	21	41	1	40
Steel Only	21	49	1	48

The thermal coefficients of LRFD [5.4.2.2] and [6.4.1] for normal density concrete and structural steel, respectively, apply to the material of the main longitudinal supporting beams or girders.

6.10 Expansion Joint and Bearing Design to Accommodate Movement [14.4]

The moving surfaces of the joint shall be designed to work in concert with the bearings to avoid binding the joints and adversely affecting force effects imposed on bearings. Thus, whether in an expansion or a contraction mode, the movement range of the joint must be compatible with that of the bearings supporting that joint.

Besides movement due to temperature variation, the design of expansion joints and bearings must consider the effects of creep, shrinkage, skew, rotation, lateral shear, and vertical shear.

Joints and bearings for conventional, reinforced and prestressed concrete superstructures shall be designed for the calculated movement due to temperature change plus creep and shrinkage, or 115% of the calculated movement due to temperature change alone, whichever is larger. Joints and bearings for longitudinally post-tensioned superstructures shall be designed for creep and shrinkage plus 120% of the calculated movement due to temperature change. The minimum joint opening shall be specified in accordance with the manufacturer's recommendations; however, the joint opening must not be less than 15 mm at 21 degrees Celsius (°C).

The width, "W," of the joint shall meet the requirements of LRFD [14.5.3.2], except that "W" for the different joint types shall not exceed the appropriate value from Table 6.2. When setting the joint, either the design width, "W," shall be decreased by the amount of anticipated movement due to creep and shrinkage, or the joint shall be set to the minimum width for installing the seal, whichever results in the wider opening.

Table 6.2 Joint Width Limitation by Joint Type

JOINT TYPE	MAXIMUM JOINT WIDTH
Poured Rubber	20 mm
Silicon Seal	55 mm
Strip Seal	75 mm
Modular Joint	*
Finger Joint	None

* Set width in accordance with the manufacturer's recommendation.

6.11 Joints for Bridge Widening

Whether the existing joint is extended along its length or totally replaced, it is essential to have all existing and new surfaces cleaned, protected, and finished in accordance with the Specifications. Any chipped concrete, debris, or dust shall be removed prior to and after placement of the new seal or joint system.

Concrete spalls adjacent to existing joints shall be repaired prior to joint lengthening. All seals shall be continuous and shall generally extend to the face of the barrier at the high side of deck surface, and past the gutter line with a 100 mm upturn at the low side of deck surface.

An "approved" seal for use in the widening of Group 1 or 2 joints described below is a seal that either is listed on the Department's QPL and/or is approved by the DSDE, District Structures and Facilities Engineer, or SDO, as appropriate.

6.11.1 Widening of Bridges with Group 1 Joints

- A. If the existing joint consists of armored strip seal and the steel armor is in good condition, remove the existing seal, extend the armor into the widening by welding new sections to existing armor and provide new strip seal continuously across the entire deck.
- B. If the existing joint consists of armored strip seal and the steel armor is in poor and irreparable condition, or if the existing joint system consists of armored or non-armored compression seal in good or bad condition, remove the existing seal and armor, repair the damaged concrete, and install a new Group 1 Joint other than a compression seal.
- C. If the existing joint consists of copper water-stop, poured rubber, or silicon, and

is performing satisfactorily, remove all existing joint material, extend joint gap into the widening, and install a new silicon seal.

- D. If the existing joint is an open joint, is performing satisfactorily, and the joint gap is not wider than 25 mm at 21 degrees Celcius (°C), extend the open joint into the widening. If it is not performing satisfactorily, extend the gap into the widening and seal the entire joint with silicon sealer.
- E. If the existing joint is an open joint that is wider than 25 mm at 21 degrees Celcius (°C), provide a blockout for a Group 1 Joint (other than a compression seal) that can accommodate the existing joint opening. The new bridge deck shall provide a blockout for the new joint system.
- F. If the existing joint is a Jeene Joint and is performing satisfactorily, extend the joint gap into the widening, remove the existing Jeene Joint and provide a new Jeene Joint. If it is not practicable to install a new Jeene Joint, provide a blockout for a Group 1 Joint other than a compression seal. The new bridge deck shall provide a blockout for the new joint system.

Commentary: Generally, field splices of strip seal expansion joints shall conform to the notes of Index No. 400 of the Standard Drawings.

6.11.2 Widening of Bridges with Group 2 Joints

- A. If the existing joint is in good condition, or repairable, the joint shall be lengthened using the same type of joint after performing any needed repairs. Lengthening shall be performed in conformance with the joint manufacturer's recommendations.
- B. If the existing joint material is proprietary and no longer available, it preferably shall be replaced with a Group 2 Joint that will accommodate the same calculated movement.

6.12 General Design Policy for Bridge Bearings

Bridge bearings must accommodate the movements of the superstructure and transmit loads to the substructure supports. The type of bearing to be used depends on the amount and type of movement as well as the magnitude of the reaction.

Shop Drawings are required for all bearings except Composite Neoprene Bearing Pads shown in the Standard Drawings, Index No. 200.

Commentary: In general, simple-span prestressed concrete beams, simple-span steel girders and some continuous beams can be supported on composite neoprene

bearing pads (elastomeric bearings). Heavier reactions can be accommodated by using external steel load plates, and larger movements can be accommodated by using PTFE (Teflon) bearing surfaces.

Some structures with large bearing loads and/or multi-directional movement might require other bearing devices such as pot, spherical, or disc bearings. Index No. 200 of the Standard Drawings includes four (4) composite neoprene bearing pad designs. Whenever possible and after confirming their adequacy, these standard designs of Index No. 200 should be used.

6.13 Bearing Types and Specifications [14.6.1] [14.6.2] [14.7]

The choice of a type of bearing shall depend on a suitability analysis. The EOR shall follow the selection process of LRFD Table 14.6.2-1, Bearing Suitability, for an appropriate bearing type.

Currently available bearing types by Group are:

- Group 1: Plain Elastomeric Pads, Steel-Reinforced Elastomeric Pads, Resilient Pads and other proprietary pads that are usually associated with a maximum joint opening not exceeding 75 mm or a maximum span length not exceeding 60 m.
- Group 2: Plane PTFE Sliding Surfaces, Curved Sliding Spherical or Cylindrical Bearings, Pot Bearings, Bronze or Copper Alloy Sliding Surfaces, Disc Bearings, and other proprietary bearing systems that are usually associated with a maximum joint opening exceeding 75 mm or a maximum span length exceeding 60 m.

Elastomeric bearings (Group 1) shall be used to the maximum extent possible. Group 2 Bearings shall be used only for very large loads, on bridges with maximum span length greater than 60 meters, where movements of continuous units dictate the use of these bearings, or on bridges with a maximum joint opening greater than 75 millimeters.

Generally, the plane PTFE and the bronze or copper alloy sliding surfaces are mated with polished stainless steel surfaces, and the curved sliding spherical or cylindrical bearings are mated with PTFE or bronze sliding surfaces.

For general requirements for bearings, see FDOT Specifications, Sections 400 and 932.

The special design requirements of LRFD cover specific material properties, mating surfaces, and design requirements such as coefficient of friction, load resistance, compressive stress, compressive deflection, and shear deformation, as applicable to the various bearing systems.

Commentary: If the resistance factor for bearing is other than 1.0, the design calculations shall include the method for obtaining such a factor.

6.14 Criteria for Bearings

When a bearing is required, the bearing design shall satisfy the following criteria as applicable:

- If the bearing is proprietary, it shall be designed in accordance with LRFD and installed in strict conformance with the manufacturer's recommendations, unless directed otherwise by the Department.
- The bearing components shall accommodate the full range of structural movements without exceeding the manufacturer's limitations and/or design values particularly at maximum joint opening.
- The bearing must provide shock absorbing qualities relatively free from rocking and noise.
- Bearings must be designed as necessary to resist all anticipated loads including dynamic load allowance.
- The bearing shall absorb and dissipate the stresses and transfer a portion thereof without causing overstress to the substructure.
- Elastomers used in bearings such as pots, discs, etc., shall provide a service life not less than that claimed by the bearing manufacturer.
- The bearing shall be designed for minimum maintenance and for ease of access in order to avoid excessive jacking of the superstructure during parts replacement.
- The bearing shall seal off moisture from the inner components of the bearing assembly.
- The bearing materials shall be resistant to both corrosion and ultraviolet (UV) rays and shall not be a catalyst or vehicle for electrolytic action.

6.15 Bearings for Bridge Widening

When the project involves the widening of a structure that does not already include provisions for bearing replacement, the District Structures and Facilities Engineer shall be consulted and will decide if provisions for the replacement of the bearings shall be made on the plans for the widening.

It is essential to have all new surfaces cleaned, protected and finished in accordance with the

Specifications prior to and after placement of the new bearing pads or bearing systems.

6.15.1 Widening of Bridges with Group 1 Bearings

- A. If the existing bearings are in good condition, keep the bearings in place and use matching type new bearings. The Shape Factor and Hardness of both the existing and new bearings play a major role in the determination of the size of the new bearings.

Commentary: The EOR must exercise caution in order to avoid bearing designs and/or details that result in final elevation differentials that could cause geometric incompatibility and/or overstressing in the superstructure, substructure, and/or the bearings.

- B. If the existing bearings are in poor condition, remove and replace the bearings. All bearings at a joint location shall be of matching type and shall be designed to allow for the same movement.
- C. If the existing bearings are in good condition but have shifted from their initial locations, relocate them to their original positions then proceed as noted above.

6.15.2 Widening of Bridges with Group 2 Bearings

- A. If the existing bearings are in good condition, keep the bearings in place and use matching-type new bearings.

Commentary: The EOR must exercise caution in order to avoid bearing designs and/or details that result in final elevation differentials that could cause geometric incompatibility and/or overstressing in the superstructure, substructure, and/or the bearings.

- B. If the existing bearings are in poor condition, proceed as noted for Group 1 Bearings.
- C. If the existing bearings are in good condition but have shifted from their locations, proceed as noted for Group 1 Bearings.
- D. If the existing bearings are in good condition but the bearing material is proprietary and is no longer available, keep the bearings in place for the “existing” structure, and use the closest possible match from the Group 2 Bearings for the “new” structure that will be compatible with the existing bearings.

6.16 Provisions for Maintainability of Bearings

The following provisions shall apply to all bridges except for flat slab or multi-beam superstructures (cast-in-place or precast) resting on thin bearing pads.

Superstructure design and details shall result in the use of bridge bearings that are reasonably accessible for inspection and maintenance. Provisions shall be made on all new designs for the replacement of bearings without causing undue damage to the structure and without having to remove anchorages or other devices permanently attached to the structure. The provisions for the removal of bearings shall be made in the design and shown on the plans (i.e., jacking locations, jacking sequence, jack load, etc.).

The replacement of bearings for conventional girder structures, particularly concrete beams, is relatively simple as jacking can be accomplished between the end diaphragms and substructure. For these bridges, a note describing the jacking procedure for replacing bearings will usually suffice; however, the plans should clearly state that the jacking equipment is not part of the bridge contract.

Provisions for the replacement of bearings for certain non-conventional structures, such as steel box girders or segmental concrete box girders, will require separate details and notes describing the procedures.

6.17 Control of Shrinkage Cracking in Continuous Superstructures [5.10.8]

Continuous superstructures require the use of a designated casting sequence to minimize shrinkage cracking. Generally, the sequence should result in construction joints spaced at not more than 25.0 meters. Camber diagrams shall be constructed using the casting sequence impact on the varying stiffness of the superstructure. The plans shall contain a note which indicates that the casting sequence may not be changed unless a new structural analysis is performed by the Contractor's Specialty Engineer, and new camber diagrams are calculated.

6.18 Keyways

Keyways shall be provided in superstructure decks at formed surfaces of construction joints and elsewhere as necessary to transfer applied loads from one cast section to an adjacent, second cast section. Keyways shall be trapezoidal in shape for ease of forming and stripping, and, typically, shall be about 50 mm deep and about 150 mm wide (or one third the thickness of the member for members less than 450 mm in thickness) running the full length.

Chapter 7

CONCRETE STRUCTURES

7.1 General

This Chapter contains information related to the design, reinforcing, detailing, and construction of concrete components. It also contains deviations from LRFD that are required in such areas as deck slab reinforcing and construction, post-tensioning design and detailing, and the design and use of adhesive anchors.

The EOR's attention is drawn to the requirements of this Chapter regarding the design of prestressed, pretensioned concrete components. In particular concerning strand transfer and development and shear design.

7.2 Concrete Deck Slabs [5.13.1] [9.7]

7.2.1 Thickness

The thickness of bridge decks that are cast-in-place (CIP) on beams or girders shall be a minimum of 215 mm for all new construction and for major widenings as defined in Chapter 9. The deck thickness includes a 15 mm additional, sacrificial thickness to be included in the dead load of the deck slab but which shall be omitted from its section properties.

The thickness of CIP bridge decks on beams or girders for widenings defined as minor widenings in accordance with Chapter 9 shall be handled on an individual basis but, generally, shall match the thickness of the adjoining existing deck.

7.2.2 Grooving Bridge Decks

All bridge plans for new construction, and for widenings noted below, that utilize C-I-P bridge deck (floors) that will not be surfaced with asphaltic concrete shall include the following item in the Summary of Pay Items:

Item No. 2400-7 - Bridge Floor Grooving - Sq. Meters

A. Quantity Determination:

The quantity of bridge floor grooving shall be determined in accordance with the provisions of Article 400-20.4 of the Specifications.

B. Traffic Lane Widening:

For widened superstructures where at least one traffic lane is to be added, the entire deck area as described in Article 400-20.4 of the Specifications shall be grooved.

C. Shoulder Widening:

For projects consisting of shoulder widening only, a note shall be added to the plans specifying that the bridge floor finish shall match that of the existing bridge.

7.2.3 Empirical Deck Slab Design [9.7.2]

Deck slabs for Category 1 Structures that meet the criteria set forth in LRFD [9.7.2.4] shall be designed by the Empirical Design method of LRFD [9.7.2].

Deck slabs for all Category 2 Structures and for Category 1 Structures that do not meet the requirements of LRFD [9.7.2.4] shall be designed in accordance with the Traditional Design method of LRFD [9.7.3].

7.3 Reinforcing Steel [5.4.3]

Steel reinforcing for concrete design shall be ASTM A615M, Grade 420.

7.3.1 Reinforcing Steel over Intermediate Piers or Bents

When CIP slabs are made composite with simple span concrete beams, and such slabs are cast continuous over intermediate piers or bents, supplemental longitudinal reinforcing shall be provided in the tops of slabs. Such reinforcing shall be sized, spaced, and placed in accordance with the following criteria:

- ! No. 16 Bars placed between the continuous, longitudinal reinforcing bars.
- ! A minimum of 12.0 m in length.
- ! Placed symmetrically about the centerline of the pier or bent.

In structures made continuous for live load, such reinforcing shall extend beyond the point of contraflexure for maximum positive or minimum negative moment, whichever is most distant from the piers or bents.

7.3.2 Special Requirements for Concrete Decks on Continuous Steel Girders

Longitudinal reinforcing steel within the negative moment regions of continuous, composite steel girder superstructures shall comply with the requirements of LRFD [6.10.1.2].

In addition, the remainder of the deck shall have longitudinal steel supplied at the rate of 0.5% of the total cross-sectional area of the slab of which two-thirds (2/3) shall be placed in the top layer of the slab reinforcing and one-third (1/3) in the bottom layer.

7.3.3 Transverse Slab Reinforcement [5.8.2.4]

A. Reinforcing Placement When the Slab Skew is 15 Degrees or Less:

When the end of a slab is skewed 15 degrees or less, place the transverse reinforcement parallel to the skew for the entire length of the slab.

B. Reinforcing Placement When the Slab Skew is More Than 15 Degrees:

When the end of a slab is skewed more than 15 degrees, place the required transverse reinforcement perpendicular to the centerline of span. In this case, because the typical required transverse reinforcement cannot be placed full-width in the end, triangular portions of the slab, the required amount of transverse reinforcing in these areas shall be increased by twenty-five percent (25%). In addition, four (4) No. 13 Bars, full-width, shall be placed parallel to the end skew in the top of each end of the slab.

Note: Regardless of the angle of skew, the barrier reinforcement cast into the slab need not be skewed.

7.4 Prestressed, Pretensioned Components

7.4.1 General Design Requirements for Prestressed Construction

Prestressed, pretensioned concrete design, details, and construction, commonly referred to as "prestressed," shall conform with LRFD Section [5] and the requirements of this article.

A. Strand Transfer and Development [5.8.2.3] [5.11.4] [5.11.4.1] [5.11.4.2]:

The strand transfer requirements of LRFD [5.8.2.3] apply; however, in LRFD [5.11.4], delete Equation 5.11.4.1-1 for bonded strand development and substitute in lieu thereof the following Equation 7-1:

$$l_d \geq 1.6 (0.15 f_{ps} - 0.097 f_{pe}) d_b \quad [Eq. 7-1]$$

Where: All terms have the definitions of LRFD [5.11.4.1].

Additionally, delete the fourth line of the first paragraph of LRFD [5.11.4.2] and add the following:

“specified in Equation 7-1 shall be multiplied by the factor of 1.25.”

Commentary: The 1.6 multiplier on LRFD Equation 5.11.4.1-1 that is shown in Eq. 7-1 above is required by the Federal Highway Administration (FHWA) to be incorporated into the strand development calculation for all prestressed, pretensioned concrete applications. The 1.6 multiplier does not apply to strand transfer calculations, however. The multiplier is based upon the results of prestressed concrete shear and bond research conducted in the United States, particularly that conducted by North Carolina State University (Ref: Bond of Epoxy Coated Prestressing Strand, Final Report to NCDOT, Research Project No. 23241-85-3, Center for Transportation Engineering Structures, North Carolina State University, Raleigh, NC, December, 1986, 191 pages, by T. E. Cousins, D. W. Johnston, and Paul Zia).

B. Strand Spacing [5.10.3.3.1]:

Table 5.10.3.3.1-1 in LRFD [5.10.3.3.1] showing the minimum center-to-center spacing of strands is replaced by Table 7.1 shown below. The spacings shown are minimum values, and larger spacings are permitted.

Table 7.1 Strand Spacing (c/c)

STRAND DESIGNATION	NOMINAL STRAND DIAMETER (mm)	MINIMUM CENTER-TO-CENTER SPACING (mm)
No. 9	9.5	38
No. 11	11.1	44
No. 13	12.7	44
No. 13S	13.1	50
No. 14	14.3	50
No. 14S	14.6	50
No. 15	15.3	50

7.4.2 Requirements For Prestressed Piling

For prestressed piling not subjected to significant flexure under service or impact loading,

strand development shall be in accordance with LRFD [5.11.4] and [5.8.2.3]. Bending that produces cracking in the pile, such as that resulting from ship impact loading, is considered significant. A pile embedment of 1.2 meters into a footing is not considered adequate for this required strand development.

7.4.3 Requirements for Prestressed Beams

The use of ASTM A416/416M, Grade 1860, low-relaxation, straight, prestressing strands is preferred for the design of prestressed beams. However, the requirements stipulated hereinafter apply to simply supported, fully pretensioned beams, whether of straight or depressed (draped) strand profile, except where specifically noted otherwise.

- A. Bridges that contain varying span lengths, skew angles, beam spacing, beam loads, or other design criteria may result in very similar individual designs. The designer should consider the individual beams designs as a first trial subject to modifications by combining similar designs into groups of common materials and stranding based upon the following priorities:
1. 28-Day Compressive Concrete Strength (fN_c)
 2. Stranding (size, number, and location)
 3. Compressive Concrete Strength at Release (fN_{ci})
 4. Shielding (Debonding)*

*Note: Full length shielding of strands in some beams to facilitate casting bed utilization of beams with slightly different strand patterns is prohibited.

Commentary: Groupings of beam designs as far down the priority list as possible maximizes casting bed usage and minimizes variations in materials and stranding.

- B. In analyzing stresses and/or determining the required length of debonding, stresses shall be limited to the following values:
1. Tension at top of beam at release (straight strand only):
 - a. Outer 15 percent of design span = $1.0\sqrt{fN_{ci}}$
 - b. Center 70 percent of design span = $0.5\sqrt{fN_{ci}}$
 2. Tension at top of beam at release (depressed strands only) = $0.5\sqrt{fN_{ci}}$
- C. In order to achieve uniformity and consistency in designing strand patterns,

the following parameters shall apply:

1. Strand patterns utilizing an odd number of strands per row (a strand located on the centerline of beam) and a minimum side cover (centerline of strand to face of concrete) of 75 mm are required for all AASHTO and Florida Bulb-Tee beam sections except AASHTO Type V and VI beams for which a strand pattern with an even number of strands per row shall be utilized.
2. Use of "L-shaped" longitudinal bars in the webs and flanges in end zone areas.
3. The minimum compressive concrete strength at release shall be 28 MPa or $0.6 f'_c$, whichever is the greater. Higher release strengths may be used and specified when required by the designer but generally should not exceed $0.8 f'_c$.
4. Prestressed beams shall be designed and specified to conform to one of the following classes and related strengths of concrete:

Table 7.2 Beam Concrete Classes and Strengths

Class of Concrete	28-Day Compressive Strength (f'_c) MPa	Modulus of Elasticity (E_c) GPa**
Class III*	35	25.5
Class IV	38	26.6
Class V	45	28.9
Class VI	59	33.1

* Class III concrete may be used only when the superstructure environment is classified as Slightly Aggressive in accordance with the criteria in Chapter 2.

** Florida Limerock Aggregate Concrete

5. All prestressed concrete components shall be designed for stay-in-place (SIP) metal forms unless such forms are prohibited. The weight to be used in the design shall be in accordance with Chapter 3.
6. The use of time-dependent, inelastic creep and shrinkage is not allowed in the design of simple span, pretensioned components either during design or construction. Prestress loss must be calculated in accordance with LRFD **[5.9.5]**.

Commentary: The FDOT cannot practically control, nor require the Contractor to control, the construction sequence and materials for simple span precast, prestressed beams. To benefit from the use of refined time-dependent analysis, literally every prestressed beam design would have to be re-analyzed using the proper construction times, temperature, humidity, material properties, etc. of both the beam and the yet-to-be-cast composite slab.

7. Stress and camber calculations for the design of simple span, pretensioned components shall be based upon the use of transformed section properties. The stresses at release of the prestress must be based upon the section of concrete that is reduced by the area of the prestressing strands.
8. When wide-top beams such as bulb-tees and AASHTO Types V and VI beams are used in conjunction with stay-in-place metal forms, the designer shall evaluate the edges of flanges of those beams to safely and adequately support the self-weight of the forms, concrete, and construction load specified in Section 400 of the FDOT Standard Specifications for Road and Bridge Construction.
9. The design thickness of the composite slab shall be provided from the top of the stay-in-place metal form to the finished slab surface, and the superstructure concrete quantity shall not include the concrete required to fill the form flutes.

7.5 Precast Prestressed Slab Units

To control the maximum camber expected in the field for precast prestressed slab units, the design camber shall not exceed 6 mm. Unless otherwise specified on the plans, the design camber shall be computed for slab concrete with an age of 120 days. The design camber shown on the plans shall be the value of camber due to prestressing minus the dead load deflection of the slab unit after all prestress losses.

7.6 Florida Bulb-Tee Beams

The minimum web thicknesses for Florida Bulb-Tee beams are:

Pretensioned Beams	=	166 mm
Post-Tensioned Beams	=	203 mm

7.7 Precast Prestressed Double-Tee Beams

All bridge structures utilizing precast, prestressed double-tee beams shall conform with the

design criteria and details provided in the Standard Drawings (Topic No.: 625-020-300).

7.8 Stay-in-Place (SIP) Slab Forms

Bridge plans shall be designed and detailed for stay-in-place metal forms for prestressed beam and steel girder superstructures except where restricted for use by Chapter 2.

7.9 Prestressed Beam Camber/Build-Up Over Beams

Unless otherwise required as a design parameter, beam camber for computing the build-up shown on the plans shall be based on an age of beam concrete of 120 days. In all cases, the age of beam concrete used for camber calculations shall be shown on the build-up detail as well as the value of camber due to prestressing minus the dead load deflection of the beam.

Commentary: In the past, the FDOT has experienced significant slab construction problems associated with excessive prestressed, pretensioned beam camber. The use of straight strand beam designs, higher strength materials permitting longer spans, stage construction, long storage periods, improperly placed dunnage, and construction delays are some of the factors that have contributed to camber growth. Actual camber at the time of casting the slab equal to 2 to 3 times the initial camber at release is not uncommon.

7.10 Post-Tensioning Strand

Unless otherwise permitted by FDOT, post-tensioning strand, either for longitudinal or transverse prestressing applications, shall be ASTM A416/416M, low-relaxation strand.

7.11 Dimension and Location Requirements for Post-Tensioning Ducts [5.4.6] [5.10.3.3]

Show offset dimensions to post-tensioning duct trajectories from fixed surfaces or clearly defined reference lines at intervals not exceeding 1.5 meters. When the rate of curvature of a duct exceeds one-half ($\frac{1}{2}$) degree per meter, offsets shall be shown at intervals not exceeding 750 mm. In regions of tight reverse curvature of short sections of tendons, offsets shall be shown at sufficiently frequent intervals to clearly define the reverse curve.

Curved ducts that run parallel to each other or around a void or re-entrant corner shall be sufficiently encased in concrete and reinforced as necessary to avoid radial failure (pull-out into the other duct or void). In the case of approximately parallel ducts, the EOR shall consider the arrangement, installation, stressing sequence, and grouting in order to avoid potential problems with cross-grouting of ducts.

Ducts for post-tensioned bulb-tee beams shall be round, galvanized metal ducts only.

7.12 Prestressed Beam Standards and Semi-Standards

Prestressed beam standards and semi-standards for use in simple span applications are included in the Standard Drawings (Topic No.: 625-020-300) available through the Office of Maps and Publications Sales in Tallahassee.

7.13 Pretensioned/Post-Tensioned Beams

In designing pretensioned beams made continuous by field-applied post-tensioning, the pretensioning shall be designed such that the following minimum criteria are satisfied:

- The pretensioning acting alone shall comply with the minimum steel requirements of LRFD [5.7.3.3.2].
- The pretensioning shall be capable of resisting all loads applied prior to post-tensioning, including a superimposed dead load equal to 50% of the uniform weight of the beam, without exceeding the stress limitation for pretensioned concrete construction.
- The pretensioning shall be of such a level that the initial midspan camber at release, including the effect of the dead load of the beam, is at least 13 mm. In computing the initial camber, the value of the modulus of elasticity of the concrete, E_c , shall be in accordance with Table 7.2 for the minimum required strength of concrete at release of the pretensioning force. And, the pretensioning force in the strands shall be reduced by losses due to elastic shortening and steel relaxation.
- Anchorage zones of post-tensioning ducts as well as beam lengths in which ducts deviate both horizontally and vertically require integrated drawings.

7.14 Special Requirements For Box Culverts [5.7.3.4]

The crack width parameter (Z) of LRFD [5.7.3.4] may be increased to 23,000 N/mm for inspectable portions of box culverts.

7.15 Adhesive Anchor Systems

7.15.1 General

Adhesive Anchor Systems are used to attach new construction to existing concrete structures. Adhesive Anchor Systems incorporate an adhesive bonding material and steel bar anchors installed in clean, dry holes drilled in hardened concrete. Anchors may be either reinforcing bars or threaded rods depending upon the application. Adhesive Anchor Systems shall not be used to splice with existing reinforcing bars in either non-prestressed or prestressed concrete applications unless special testing is performed and special, proven construction techniques are utilized. **Adhesive Anchor Systems shall not be used to splice prestressed piling.**

Adhesive Anchor Systems pre-approved for use by the Department and not requiring any submittal data are listed in the Qualified Products List (QPL) as “Adhesive Bonding Material Systems for Structural Applications” and comply with Section 937 of the Specifications. Additionally, Adhesive Anchors shall be installed in accordance with manufacturer’s recommendations for hole diameter and hole cleaning technique and must meet the requirements of Section 416 of the Specifications.

Commentary: Installation of Adhesive Anchor Systems in saturated, surface-dry holes; i.e., holes with damp surfaces but with no standing water, is not pre-approved or recommended by the Department. However, in the event such a condition is encountered during construction, the Department may consider approving continued installation, but only on an adjusted, case-by-case basis. The damp hole strength of products on the QPL has been determined to be approximately 75% of the required dry hole strength.

Unless special circumstances dictate otherwise, Adhesive Anchor Systems shall be designed for a ductile failure. A ductile failure requires embedment sufficient to ensure that failure will occur by yielding or fracture of the steel. In order to produce ductile failure, the following embedments may be assumed:

A. For Anchors in Tension:

That length of embedment which will develop 125 percent (125%) of the specified yield strength of 100 percent (100%) of the specified tensile strength, whichever is less.

B. For Anchors in Shear:

An embedment equal to seventy percent (70%) of the embedment length determined for anchors in tension.

In circumstances where ductile failure is not required, the design may be based upon the design strength of either the steel anchor or the adhesive bond, whichever is less.

Adhesive Anchor Systems meeting the specifications and design constraints of this article are permitted for horizontal, vertical, or downwardly inclined installations, only. **Overhead or upwardly inclined installations of Adhesive Anchors are prohibited.**

7.15.2 Notation

The following notation is used in this Article:

A_e = effective tensile stress area of steel anchor (may be taken as 75% of the gross area for threaded anchors). [mm²]

A_{n0} = $4.0(h_e)^2$, effective area of a single Adhesive Anchor in tension, used in

- calculating Ψ_{gn} (See Figure 7-1). [mm²]
- A_n = effective area of a group of Adhesive Anchors in tension, used in calculating Ψ_{gn} , defined as the rectangular area bounded by a perimeter spaced $1.0h_e$ from the center of the anchors and limited by free edges of concrete (See Figure 7-1). [mm²]
- A_{v0} = $4.5(c^2)$, effective breakout area of a single Adhesive Anchor in shear, used in calculating Ψ_{gv} (See Figure 7-2). [mm²]
- A_v = effective area of a group of Adhesive Anchors in shear and/or Adhesive Anchors loaded in shear where the member thickness, h , is less than $1.5c$, used in calculating Ψ_{gv} (See Figure 7-2). [mm²]
- c = anchor edge distance (distance from free edge to centerline of the anchor). [mm]
- d = nominal diameter of Adhesive Anchor. [mm]
- f'_c = minimum specified concrete strength. [MPa]
- f_y = minimum specified yield strength of Adhesive Anchor steel. [MPa]
- f_u = minimum specified ultimate strength of Adhesive Anchor steel. [MPa]
- h = concrete member thickness. [mm]
- h_e = embedment depth of anchor. [mm]
- N_c = tensile design strength as controlled by bond for Adhesive Anchors. [kN]
- N_n = nominal tensile strength of Adhesive Anchor. [kN]
- N_o = nominal tensile strength as controlled by concrete embedment for a single Adhesive Anchor. [kN]
- N_s = design strength as controlled by Adhesive Anchor steel. [kN]
- N_u = factored tension load. [kN]
- s = Adhesive Anchor spacing (measured from centerlines of anchors). [mm]
- V_c = shear design strength as controlled by the concrete embedment for Adhesive Anchors. [kN]
- V_s = design shear strength as controlled by Adhesive Anchor steel. [kN]
- V_u = factored shear load. [kN]
- τ' = 7.5 MPa nominal bond strength for products on the QPL.
- ϕ_c = 0.85, capacity reduction factor for an Adhesive Anchor that is controlled by the concrete embedment.
- ϕ_s = 0.90, capacity reduction factor for an Adhesive Anchor that is controlled by anchor steel.
- Ψ_e = modification factor, for strength in tension, to account for anchor edge distance less than $10d$ (1.0 when $c \geq 10d$).
- Ψ_{gn} = strength reduction factor for Adhesive Anchor groups in tension (1.0 when $s \geq 2.0h_e$).
- Ψ_{gv} = strength reduction factor for Adhesive Anchor groups in shear and single Adhesive Anchors in shear influenced by member thickness (1.0 when $s \geq 3.0c$ and $h \geq 1.5c$).

7.15.3 Design Requirements for Tensile Loading

The design tensile strength for Adhesive Anchor steel is determined by Equation 7-2:

$$N_s = \phi_s A_e f_y \quad [\text{Eq. 7-2}]$$

The design tensile strength for Adhesive Anchor bond is determined by Equation 7-3:

$$N_c = \phi_c \psi_e \psi_{gn} N_o \quad [\text{Eq. 7-3}]$$

where:

$$N_o = \tau' \pi d h_e \quad [\text{Eq. 7-4}]$$

For anchors with a distance to a free edge of concrete less than $10d$, but greater than or equal to $3d$, a reduction factor, ψ_e , as given by Equation 7-5 shall be used. For anchors located less than $3d$ from a free edge of concrete, an appropriate strength reduction factor shall be determined by special testing. For anchors with an edge distance greater than $10d$, ψ_e may be taken as 1.0.

$$\psi_e = 0.60 + 0.40 (c \div 10d) \quad [\text{Eq. 7-5}]$$

For anchors loaded in tension and spaced closer than $2.0h_e$, a reduction factor, ψ_{gn} , given by Equation 7-6 shall be used. For anchor spacings greater than $2.0h_e$, ψ_{gn} shall be taken as 1.0.

$$\psi_{gn} = A_n \div A_{n0} \quad [\text{Eq. 7-6}]$$

7.15.4 Design Requirements for Shear Loading

Adhesive Anchors loaded in shear shall be embedded a distance of not less than $6d$.

For Adhesive Anchors loaded in shear, the design shear strength controlled by anchor steel is determined by Equation 7-7:

$$V_s = \phi_s 0.7 A_s f_y \quad [\text{Eq. 7-7}]$$

For Adhesive Anchors loaded in shear, the design shear strength controlled by concrete breakout for shear directed toward a free edge of concrete is determined by Equation 7-8:

$$V_c = \phi_c \psi_{gv} 6.0 c^{1.5} \sqrt{f'_c} \quad [\text{Eq. 7-8}]$$

For anchors spaced closer than $3.0c$, a reduction factor, ψ_{gv} , given by Equation 7-9 shall be used. For anchor spacings greater than $3.0c$, ψ_{gv} shall be taken as 1.0.

$$\psi_{gv} = A_v \div A_{v0} \quad [\text{Eq. 7-9}]$$

7.15.5 Interaction of Tensile and Shear Loadings

The following linear interaction between tension and shear loadings given by Equation 7-10 shall be used unless special testing is performed:

$$\frac{N_u}{\phi N_n} + \frac{V_n}{\phi V_n} \leq 1.0 \quad [\text{Eq. 7-10}]$$

In Equation 7-10, ϕN_n is the smaller of the design tensile strength controlled by the Adhesive Anchor steel (Equation 7-2) or the design tensile strength as controlled by Adhesive Anchor bond (Equation 7-3). ϕV_n is the smaller of the design shear strength controlled by the Adhesive Anchor steel (Equation 7-7) or the design shear strength as controlled by concrete breakout (Equation 7-8).

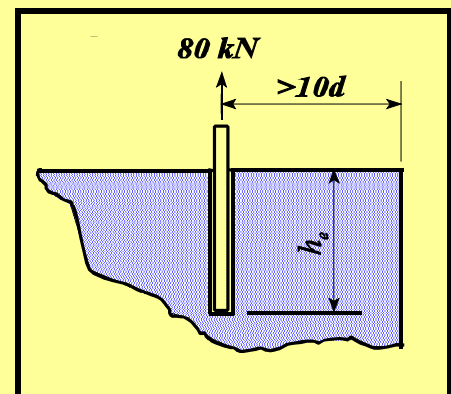
*Commentary: If Adhesive Anchor Systems are required to act as dowels from existing concrete components such that the existing reinforcing steel remains fully effective over its length, then the Adhesive Anchor System must be installed to a depth equal to the development length of the existing reinforcing steel. In this case, the required reinforcing steel spacings, covers, etc. apply to both the existing reinforcing steel and the Adhesive Anchor System. **There is, however, no additional benefit to the Adhesive Anchor System to install anchors to a greater depth than required by this Article.***

7.15.6 Example 1 - Single Adhesive Anchor Away from Edges and Other Anchors

Design an adhesive anchor using threaded rod (ASTM A193M, Grade B7) for a factored tension load of 80 kN. The anchor is located over 10 anchor diameters from edges and is isolated from other anchors. The anchor embedment length is to be sufficient to ensure steel failure.

Given:

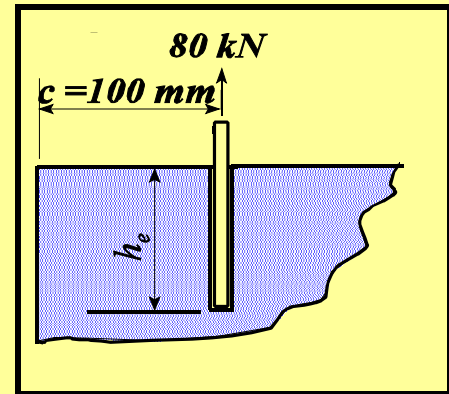
$$\begin{aligned} N_u &= 80.0 \text{ kN} \\ f_y &= 720.0 \text{ MPa} \\ f_u &= 860.0 \text{ MPa} \\ \tau' &= 7.5 \text{ MPa} \end{aligned}$$



Design Procedure	Calculation
Step 1 - Determine required rod diameter	
Determine the required diameter of the threaded rod by setting the factored tension load equal to the design steel strength.	$N_u = N_s$
The effective area for the threaded rod may be taken as 75% of the gross area. As with reinforcing bars, the minimum specified yield strength of the rod is used to determine the required diameter.	$N_s = \phi_s A_e f_y$ <i>where:</i> $\phi_s = 0.9$ $A_e = 0.75 (\pi d^2/4)$ $f_y = 720 \text{ MPa}$
Substituting and solving for d :	$80000 = (0.9)[0.75 (\pi d^2/4)](720)$ $d = 14.5 \text{ mm}$ <i>therefore, use a 16 mm threaded rod</i>
Step 2 - Determine required embedment length to ensure steel failure	
Basic equation for embedment length calculation. Since there are no edge or spacing concerns, Ψ_e and Ψ_{gn} may be taken as unity.	$N_c = \phi_c \Psi_e \Psi_{gn} N_o$ (for embedment) <i>where:</i> $\phi_c = 0.85$ $\Psi_e, \Psi_{gn} = 1.0$ (no edge/spacing concern) $N_o = \tau' \pi d h_e$
For ductile behavior it is necessary to embed the anchor sufficiently to develop 125% of the yield strength or 100% of the ultimate strength, whichever is less.	$N_{c(\text{req'd})} = 1.25 A_e f_y \leq A_e f_u$
Determine the effective area for a 16 mm threaded rod:	$A_e = 0.75 (\pi 16^2/4)$ $A_e = 151 \text{ mm}^2$
Determine the required tension force, $N_{c(\text{req'd})}$, to ensure ductile behavior.	$N_{c(\text{req'd})} = 1.25 A_e f_y \leq A_e f_u$ $N_{c(\text{req'd})} = 1.25 (151)(720) \leq (151)(860)$ $N_{c(\text{req'd})} = 136 \text{ kN} > 130 \text{ kN}$ <i>therefore, use $N_{b(\text{req'd})} = 130 \text{ kN}$</i>
Substituting and solving for h_e :	$130,000 = 0.85 (1.0) (1.0) (7.5) \pi (16) h_e$ $h_e = 406 \text{ mm}$

7.15.7 Example 2 - Single Adhesive Anchor Away from Other Anchors but Near Edge

Design an adhesive anchor using threaded rod (ASTM A193M, Grade B7) for a factored tension load of 80 kN. The anchor is located 100 mm from an edge but is isolated from other anchors. The anchor embedment length is to be sufficient to ensure steel failure.



Given:

N_u	=	80.0 kN
f_y	=	720.0 MPa
f_u	=	860.0 MPa
τ'	=	7.5 MPa

Design Procedure	Calculation
Step 1 - Determine required rod diameter	
Determine the required diameter of the threaded rod by setting the factored tension load equal to the design steel strength.	$N_u = N_s$
The effective area for the threaded rod may be taken as 75% of the gross area. As with reinforcing bars, the minimum specified yield strength of the rod is used to determine the required diameter.	$N_s = \phi_s A_e f_y$ where: $\phi_s = 0.9$ $A_e = 0.75 (\pi d^2 / 4)$ $f_y = 720 \text{ MPa}$
Substituting and solving for d :	$80000 = (0.9)[0.75 (\pi d^2 / 4)](720)$ $d = 14.5 \text{ mm}$ therefore, use a 16 mm threaded rod
Step 2 - Determine required embedment length to ensure steel failure	
Basic equation for embedment length calculation. Since there are no spacing concerns, Ψ_{gn} may be taken as unity, and, since the edge distance (100 MM) is less than $10d$ (160 mm), the edge effect, Ψ_e , will need to be evaluated.	$N_c = \phi_c \Psi_e \Psi_{gn} N_o$ (for embedment) where: $\phi_b = 0.85$ $\Psi_e, \Psi_{gn} = 1.0$ (no edge/spacing problem) $N_o = \tau' \pi d h_e$
For ductile behavior it is necessary to embed the anchor sufficiently to develop 125% of the yield strength or 100% of the ultimate strength, whichever is less.	$N_{c(\text{req'd})} = 1.25 A_e f_y \leq A_e f_u$

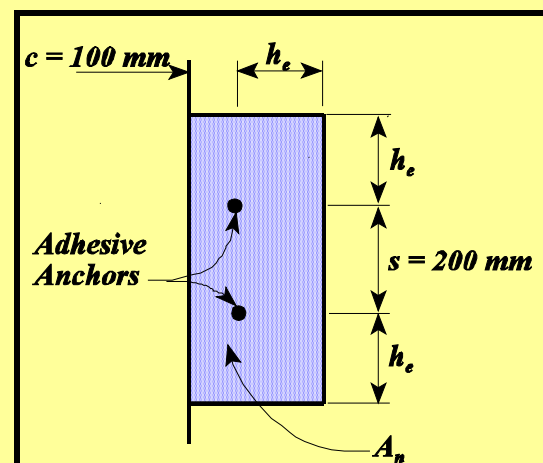
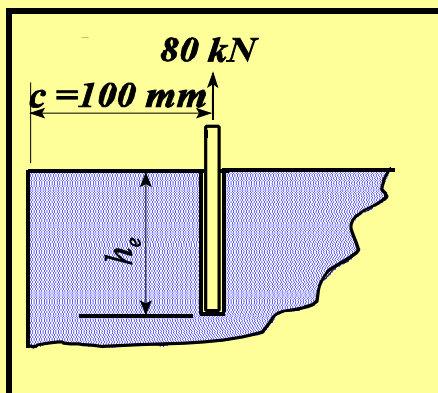
Determine the effective area for a 16 mm threaded rod:	$A_e = 0.75 (\pi 16^2/4)$ $A_e = 151 \text{ mm}^2$
Determine the required tension force, $N_{c(\text{req'd})}$, to ensure ductile behavior.	$N_{c(\text{req'd})} = 1.25 A_e f_y \leq A_e f_u$ $N_{c(\text{req'd})} = 1.25 (151)(720) \leq (151)(860)$ $N_{c(\text{req'd})} = 136 \text{ kN} > 130 \text{ kN}$ therefore, use $N_{c(\text{req'd})} = 130 \text{ kN}$
Determine edge effect factor, Ψ_e . Note: $c_{cr} = 10d$	$\Psi_e = 0.60 + 0.40 (c \div 10d)$ $\Psi_e = 0.60 + 0.40 [100 \div (10)(16)]$ $\Psi_e = 0.85$
Substituting and solving for h_e :	$130,000 = 0.85 (1.0)(0.85)(7.5) \pi (16) h_e$ $h_e = 477 \text{ mm}$

7.15.8 Example 3 - Two Adhesive Anchors Spaced 200 mm Apart, 100 mm from Edge

Design a group of two adhesive anchors using threaded rod (ASTM A193M, Grade B7) for a factored tension load of 80 kN. The anchors are located 100 mm from an edge and are spaced 200 mm apart. Steel failure is not required.

Given:

N_u	=	80.0 kN
c	=	100.0 mm
s	=	200.0 mm
f_y	=	720.0 MPa
f_u	=	860.0 MPa
T'	=	7.5 MPa



Design Procedure	Calculation
Step 1 - Determine required rod diameter	
Determine the required diameter of the threaded rod by setting the factored tension load equal to the design steel strength.	$N_u = N_s$
The effective area for the threaded rod may be taken as 75% of the gross area. As with reinforcing bars, the minimum specified yield strength of the rod is used to determine the required diameter.	$N_s = \phi_s A_e f_y$ <i>where:</i> $\phi_s = 0.9$ $A_e = (2) 0.75 (\pi d^2 / 4)$ $f_y = 720 \text{ MPa}$
Substituting and solving for d :	$80000 = (0.9)(2)[0.75 (\pi d^2 / 4)](720)$ $d = 10.2 \text{ mm}$ <i>although a 13 mm threaded rod is OK, use a 16 mm threaded rod to minimize embedment length</i>
Design steel strength	$N_s = 0.9(2)(0.75)(\pi 13^2 / 4)(720)$ $N_s = 129,000 \text{ kN} > 80,000 \text{ kN} \therefore \text{OK}$
Step 2 - Determine required embedment length	
Basic equation for embedment length calculation. Since there are edge or spacing concerns, Ψ_e and Ψ_{gn} will need to be determined.	$N_c = \phi_c \Psi_e \Psi_{gn} N_o$ (for embedment) <i>where:</i> $\phi_c = 0.85$ Ψ_e and Ψ_{gn} are calculated below $N_o = \tau \pi d h_e$
Determine edge effect factor, Ψ_e .	$\Psi_e = 0.60 + 0.40 (c \div c_r)$ $\Psi_e = 0.60 + 0.40 [100 \div (10)(16)]$ $\Psi_e = 0.85$
Determine group effect factor, Ψ_{gn} . Note: Ψ_{gn} is best determined by assuming a value for h_e and iterating to find the required embedment. Therefore, assume: $h_e = 360 \text{ mm}$	$\Psi_{gn} = A_n / A_o$ $\Psi_{gn} = \frac{(100 + h_e)[200 + 2(h_e)]}{4.0(h_e)^2}$ $\Psi_{gn} = \frac{(100 + 360)[200 + 2(360)]}{4.0(360)^2}$ $\Psi_{gn} = 0.82$
Substituting and solving for h_e .	$80,000 = (0.85)(0.82)(0.85)(7.5)\pi(16)h_e$ $h_e = 358 \text{ mm} \approx 360 \text{ mm} \therefore \text{OK}$

Design adhesive bond strength.	$N_c = (0.85)(0.82)(0.85)(7.5)\pi(16)(360)$ $N_c = 80,400 > 80,000 \quad \therefore \text{OK}$
Step 3 - Final Design Strength	
Strength as controlled by steel.	$N_s = 129,000 \text{ kN} > 80,000 \quad \therefore \text{OK}$
Strength as controlled by adhesive bond.	$N_c = 80,400 > 80,000 \quad \therefore \text{OK}$
Final Design.	2 - 16 mm anchors embedded 360 mm

7.16 Design Criteria For Segmental Box Girder Bridges [5.14.2]

The design criteria included in this chapter apply to the design of precast or cast-in-place segmental concrete bridges in Florida. These criteria are to be used by any authority, consultant, or contractor engaged in segmental design for the FDOT.

Contract drawings shall be prepared in accordance with LRFD. Specific requirements regarding post-tensioning duct size and alignment are described in this Chapter and Chapter 1 and shall be considered when selecting the depth of, and detailing for, concrete box girders.

Inspection and maintenance criteria to be used in the selection of the box girder height are included in Chapter 1.

7.16.1 Design Loading [5.14.2.3]

All loadings shall conform to the referenced Specifications except as provided hereinafter.

7.16.2 Thermal Effects [5.14.2.3.5]

A. Normal Mean Temperature:

For detailing purposes, the normal mean temperature shall be taken from Table 6.1 in Chapter 6.

B. Seasonal Variation (expansion/contraction):

Refer to Chapter 6 for temperature ranges and thermal coefficient.

7.16.3 Creep and Shrinkage [5.14.2.3.6]

Creep and shrinkage strains and effects shall be calculated in accordance with LRFD, and the following values shall apply:

A. Relative Humidity:

Use a Relative Humidity of 75% for all areas (inland and coastal).

B. Modulus of Elasticity:

Use a Secant Modulus of Elasticity at 28 days.

7.16.4 Prestress

The structure shall be designed for both initial and final prestress forces.

A. Secondary Effects:

Prestress forces in continuous structures can result in substantial secondary effects. Also, in curved structures, draped web tendons will produce transverse bending stresses in the box cross-section due to the lateral component of force arising from plan curvature. All such effects must be properly considered.

B. Deck Slab:

All box girder deck slab shall be transversely post-tensioned. Where draped post-tensioning is used in deck slabs, due consideration shall be given to the final location of the c.g.s. (**C**enter of **G**ravity of the prestressing **S**teel) within the duct. Critical eccentricities over the webs and at the centerline of box shall be reduced 6 mm from theoretical to account for adverse construction tolerances.

C. Required Prestress:

Prestress forces are required to be shown on the drawings only at locations where they can be verified during construction; i.e., at anchorages at the stressing ends of tendons.

7.16.5 Material

A. Concrete:

The minimum 28-day cylinder strengths of concrete shall be:

1. Precast superstructure (including CIP joints) = 38 MPa
2. Precast pier stems = 38 MPa

B. Reinforcement:

Reinforcing steel shall be ASTM A615M, Grade 420.

C. Concrete Cover to Reinforcement:

Concrete cover shall be in accordance with Chapter 2.

D. Post-Tensioning Steel:

1. Strand shall be ASTM A416, Grade 1860, low relaxation.
2. Parallel wires shall be ASTM A421, Grade 1655.
3. Bars shall be ASTM A722, Grade 1035.
4. Prestressing Parameters:
 - a. Apparent modulus of elasticity (strand) = 190,000 MPa
 - b. Modulus of elasticity (parallel wires & bars) = 200,000 MPa
 - c. Anchor set (to be verified during construction):
 - (1) Strand = 15 mm
 - (2) Parallel wires = 15 mm
 - (3) Bars = 0

7.16.6 Expansion Joints

Refer to Chapter 6 for design of expansion joints.

7.16.7 Erection Schedule and Construction System

A typical erection schedule and anticipated construction system shall be incorporated into the design documents in an outlined, schematic form. The assumed erection loads, along with times of application and removal of each of the erection loads, shall be clearly stated in the plans.

7.16.8 Construction Data Elevation and Camber Curve

A. General:

Construction Data Elevations shall be based on the vertical and horizontal highway geometry; whereas, the Camber Curve shall be calculated and based on the assumed erection loads used in the design and the assumed construction sequence.

B. Construction Data Elevations:

This information shall be based on the highway geometry and shall be given in 3D space with “x”, “y”, and “z” coordinates. The data points shall be located at the centerline of the box and over each web of the box(es).

C. Camber Curve:

Camber Curve data shall be provided at the centerline of the box. Camber curve data is the opposite of deflections. Camber is the amount by which the concrete profile at the time of casting must differ from the theoretical geometric profile grade (generally a straight line) in order to compensate for all structural dead load, post-tensioning, long and short term time dependent deformations (creep and shrinkage), and effects of construction loads and sequence of erection.

7.16.9 Final Computer Run

It is required that the final design shall be proven by a full longitudinal analysis taking into account the assumed construction process and final long term service condition, including all time related effects. The analysis shall be made using a computer program approved by FDOT.

7.16.10 Integrated Drawings

Congested areas of post-tensioned concrete structures shall be shown on integrated drawings with an assumed post-tensioning system. Such areas include, but are not necessarily limited to, anchorages zones, areas containing embedded items for the assumed post-tensioning system, areas where post-tensioning ducts deviate both in the vertical and transverse directions, and other highly congested areas as determined by the engineer and/or the Department.

The assumed post-tensioning system, embedded items, etc. shall be selected in a manner that will accommodate competitive systems. Integrated drawings utilizing the assumed system shall be defined to a scale and quality required to show double-line reinforcing and post-tensioning steel in two-dimension (2-D) and, when necessary, in complete three-dimension (3-D) drawings and details.

7.16.11 Special Requirements for Post-tensioning Anchorages in Slabs

Post-tensioning termini that are required to be located in either the top or bottom slab of box girder bridges, whether for permanent or temporary post-tensioning, shall be anchored by means of interior “blisters” or other approved means. Blockouts that extend either to the

interior or exterior surfaces of the slabs are not permitted.

7.17 Couplers in Post-Tensioning Tendons [5.4.5]

Strand couplers as described in [5.4.5] are **not** allowed.

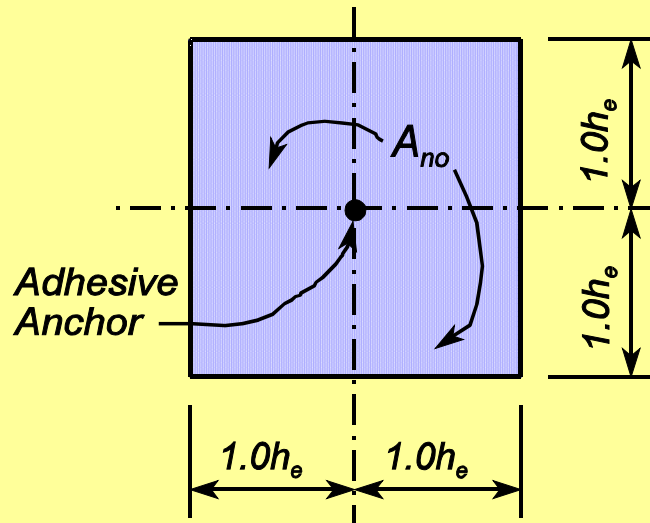
7.18 Design of Continuous Prestressed Concrete I-Girder Superstructures [5.4.2.3]

For the design of continuous prestressed concrete I-Girder superstructures, comply with the requirements of LRFD [5.4.2.3] by utilizing ACI 209 with the following design values:

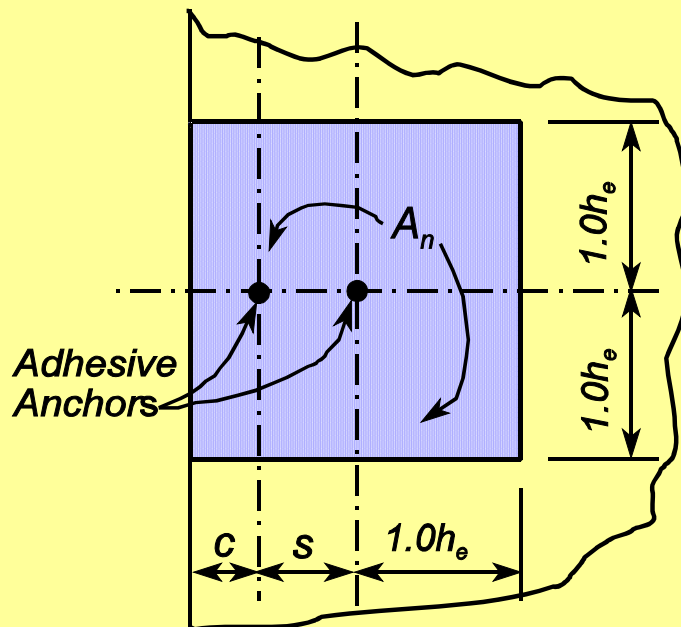
- Ultimate Creep Coefficient = 2.0
- Ultimate Shrinkage Strain = 0.0004
- Beam Age when Deck is Cast = 120 Days

The foregoing creep and shrinkage values include corrections for slump, humidity, and volume/surface ratio; and shall be used for both the beam and the deck slab.

Commentary: A parametric study conducted by the FDOT's Structures Design Office indicates that the above values predict losses consistent with the AASHTO lump-sum loss approach. The correction factors applied to the basic creep and shrinkage values are average values. The values given above are subject to change as future research results become available.



For Calculation of A_{no}



For Calculation of A_n

*Effective Tensile Stress Areas
of Adhesive Anchors*

Figure 7-1

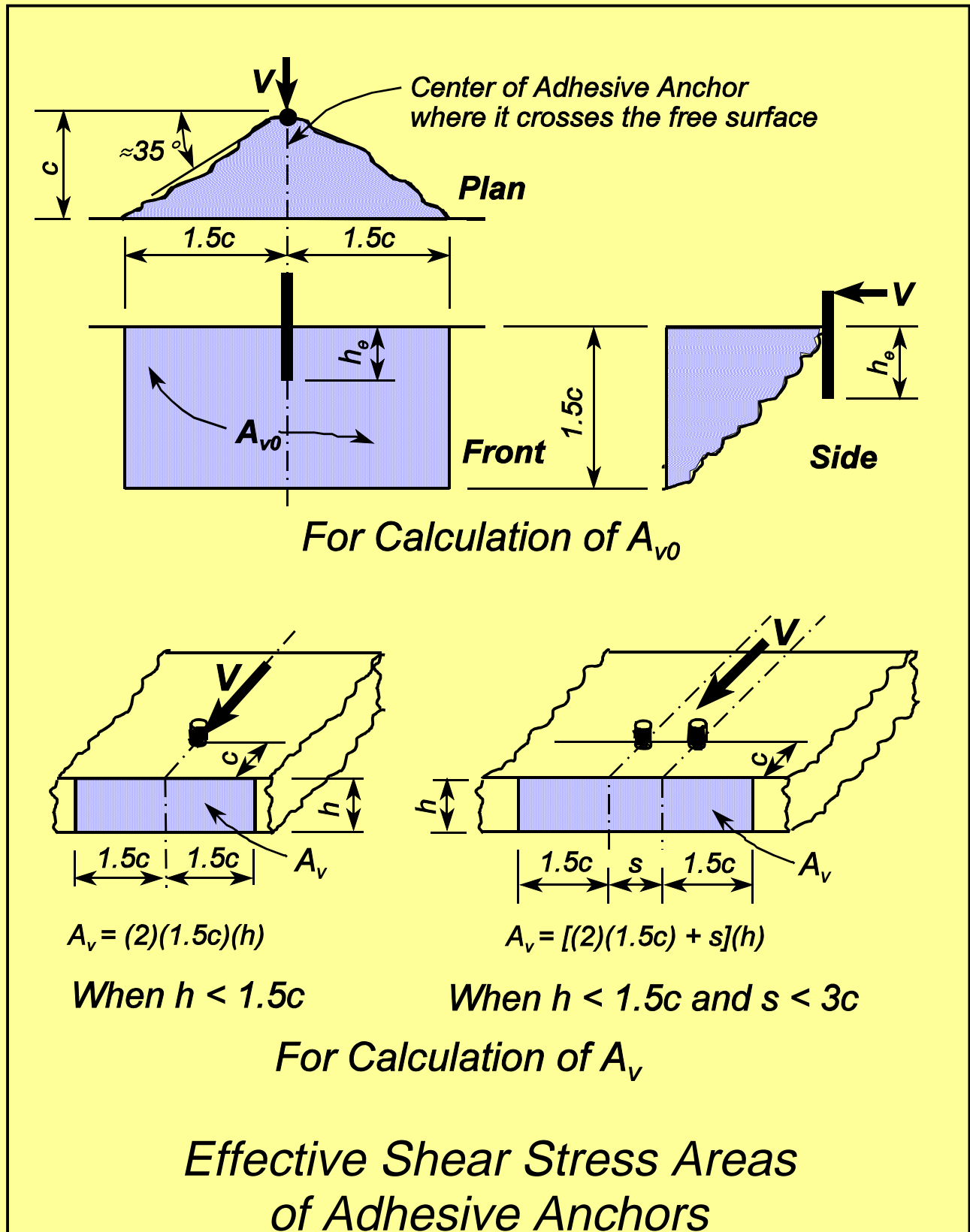


Figure 7-2

Chapter 8

STEEL BRIDGE STRUCTURES

8.1 General

This chapter includes guidance for the design and detailing of steel bridges. Steel bridge components shall be designed in accordance with LRFD and the requirements of this Chapter.

A bridge that is curved for part or all of its length shall be designed in its entirety in accordance with the current AASHTO Guide Specifications for Horizontally Curved Highway Bridges, the AASHTO Standard Specifications for Highway Bridges, and Interim Specifications.

Structural steel for miscellaneous items such as ladders, platforms, and walkways shall be designed in accordance with the current AISC Manual of Steel Construction, Load and Resistance Factor Design.

Inspectability shall be considered when selecting the depth of, and details for, steel box girders.

Commentary: The LRFD Specifications for horizontally curved bridges is not currently available.

Refer to Chapter 1 for box girder inspection access requirements. Refer to Chapter 2 for the use of uncoated weathering steel and for lead-based paint requirements. Refer to the Structures Detailing Manual (Topic No.: 625-020-200) for notes for steel fabrication plants.

8.2 Structural Steel [6.4.1]

All structural steel shall be in accordance with AASHTO M270M (ASTM A709/A709M), Grade 250, 345, or 345W. Grade 485W, 690, or 690W may be approved by the Department for use in special cases. Show the AASHTO M270M or ASTM A709/A709M designation on the contract documents.

Miscellaneous hardware, including shapes, plates, and threaded bar stock, may conform to ASTM A709M, Grade 250.

Commentary: AASHTO M270M and ASTM A709/A709M are bridge steel specifications that include notch toughness and weldability requirements. These requirements may not be present in other ASTM Specification such as A36 or A588.

Steels meeting bridge steel specifications are prequalified for use in welded bridges.

8.3 Bolts [6.4.3.1]

Structural bolted connections shall be designed as slip-critical. ASTM A325M Type 1 high-strength bolts shall be used in painted connections, and Type 3 bolts shall be used for unpainted weathering steel connections. ASTM A490M bolts shall not be used. Non-high-strength bolts may conform to ASTM F568.

8.4 Load Indicating Devices [6.4.3.5]

Direct tension indicator (DTI) washers shall not be used.

Commentary: The FDOT has not approved DTI washers for use on its bridge projects.

8.5 Fracture [6.6.2]

Charpy V-Notch (CVN) testing for Temperature Zone 1 is required on the material in all main load carrying member components that are subjected to tension or stress reversal.

Fracture critical members are to be avoided if possible; however, any non-redundant member subject to tensile stress must be considered to be fracture critical.

The plans shall designate, with a symbol, all steel components that require CVN testing only, and, with another symbol, all steel components that are fracture critical. The plans shall also designate which members are considered main members and which are ancillary members for the purpose of shop inspection and testing as required by the ANSI/AASHTO/AWS Bridge Welding Code.

Commentary: CVN testing is required regardless of whether the structure is redundant or non-redundant, even though the base metal CVN values are more stringent for non-redundant members.

The failure of a non-redundant member could cause collapse of the structure. The bottom flange and connecting web plates in the positive moment region of the end spans of twin box girder bridges shall be considered as fracture critical. Negative moment regions are not fracture critical.

The designer must consider that fracture critical requirements are expensive due to the intensive welding procedures, base metal and welding material testing, and inspection. The steel industry should be consulted for advice when a choice is to be made between redundant and non-redundant systems.

8.6 Dead Load Camber [6.7.2]

The structure, including the slab, shall be designed for a sequence for placing the concrete deck which shall be shown on the plans. Camber diagrams shall account for the deck placing sequence. The designer shall consider the superstructure geometrics and properties and use the appropriate level of analysis to determine deflections and camber.

Commentary: Steel girders are to be fabricated to match the profile grade with an allowance for dead load deflection such that when the deck is placed, the amount of build-up is minimized. For bascule, skewed, or curved bridges, or for bridges with large overhangs on the exterior girder, a grid, 3-D or finite element analysis may be necessary to determine accurately the girder deflections and required camber.

8.7 Minimum Steel Dimensions [6.7.3]

The minimum thickness of plate girder and box girder webs is 11 mm.

The minimum flange size for plate girders and top flanges of box girders is 20 x 300 mm.

The minimum box girder bottom flange thickness is 12 mm.

Commentary: These minimum dimensions are selected to reduce distortion caused by welding and to improve girder stiffness for shipping and handling.

8.8 Thickness of Steel Plates

Specify flange plate widths and web plate depths in 25 mm increments.

Specify plates in accordance with the commonly available thicknesses of Table 8.1.

Commentary: On a given project, the number of different flange plate thicknesses should be minimized so that the fabricator is not required to order small quantities.

Table 8.1 Common Metric Plate Thicknesses

THICKNESS INCREMENT (mm)	PLATE THICKNESS (mm)					
0.5	5	5.5	6			
1.0	7	8	9	10	11	12
2.0	14	16	18	20	22	
2.0/3.0	25	28	30	32		
3.0	32	35	38			
5.0	40	All plate thicknesses greater than 40 mm				

8.9 Diaphragms and Cross Frames For “I-Girders” [6.7.4]

Diaphragms shall be connected with bolts at stiffener locations and shall be connected directly to stiffeners without the use of connection plates whenever possible. Generally, a “K-frame” with the members welded to each other, without connection plates, and detailed to eliminate variation from one diaphragm to another, is the most economical arrangement and shall be used. For straight bridges with a constant cross section, parallel girders, and a girder-spacing-to-girder-depth ratio less than two, an “X-frame” design is the most economical and shall be considered.

8.10 Diaphragms and Cross Frames for Box Girders [6.7.4]

External diaphragms shall be an “X-frame” or a “K-frame” as noted for “I-girders” above and shall be connected to girders with bolts at stiffener locations. Internal diaphragms may be connected by welding or bolting to stiffeners in the fabrication shop. For box girders, a “K-frame” internal diaphragm shall be used.

Commentary: An “X-frame” internal diaphragm is easier to fabricate and erect than a “K-frame,” but the “K-frame” allows easier inspection access in box girders.

8.11 Lateral Bracing [6.7.5]

For box girders, an internal lateral bracing system shall be used in the plane of the top flange. Bracing shall be configured as single diagonals and shall be connected to the underside of the top flange at internal diaphragm locations.

Commentary: A single diagonal member is preferred over an “X-diagonal” configuration for

ease of fabrication and erection. Details shall avoid interferences between the lateral bracing and stay-in-place metal forms.

8.12 Transverse Intermediate Stiffeners [6.10.8.1]

Intermediate stiffeners that provide diaphragm connections shall be attached to the compression flange by fillet welding and attached to the tension flange, or flanges subject to stress reversal, by fillet welding or bolting.

Commentary: On tension flanges, welded connections are preferred because of the lower cost, but the design of the flange must consider the appropriate fatigue detail category. A bolted connection is acceptable if the cost is justified.

8.13 Bearing Stiffeners [6.10.8.2]

All Bearing Stiffeners that do not have attached diaphragms shall be mill-to-bear on the bottom flange and shall tight fit at the top flange.

Bearing Stiffeners that also provide diaphragm connections shall be mill-to-bear on the bottom flange and shall be fillet welded to both the top and bottom flanges. In negative moment regions only, however, stiffeners with attached diaphragms may be bolted to the top flange.

Commentary: In negative moment regions, welded connections are preferred because of the lower cost, but the design of the flange must consider the appropriate fatigue detail category.

8.14 Minimum Negative Flexure Slab Reinforcement [6.10.3.7]

Negative flexural regions shall be considered as any location where the top of the slab is in tension under any combination of dead load and live load.

Commentary: See Chapter 7 for additional slab reinforcing requirements.

8.15 Longitudinal Stiffeners [6.10.8.3]

Whenever possible, longitudinal stiffeners shall not be used. However, if they must be used, they preferably shall be made continuous on one side of the web with transverse stiffeners located on the other side of the web. Generally, for aesthetic reasons, transverse stiffeners shall not be placed on the exterior face of exterior girders.

Commentary: If longitudinal stiffeners are being considered, an analysis of material and labor costs should be performed to justify their use. Their use may be justified on deep, haunched girders but normally cannot be justified on constant depth

deep, haunched girders but normally cannot be justified on constant depth girders. When longitudinal stiffeners are used on the same side of the web as the transverse stiffeners, the intersection of the stiffeners must be carefully designed with respect to fatigue.

8.16 Connections and Splices [6.13]

Field connections shall be bolted, not welded. Field welding is allowed only by prior written approval by the Department and then, only when bolting is impractical or impossible.

Where cantilever brackets are connected to exterior girders and tie plates are used to connect the top flange of the bracket to the top flange of the floorbeam, the tie plates shall not be connected to the girder top flange.

8.17 Size of Bolts [6.13.2.5]

Bolt diameters of 20, 22, 24, or 27 mm typically should be used. Larger bolts may be used with prior approval by the Department. Preferably, one size bolt shall be used for all structural connections on any given structure.

8.18 Slip Resistance [6.13.2.8]

Design bolted connections for Class B surface condition.

Commentary: This surface condition agrees with Florida fabrication practice.

8.19 Welded Connections [6.13.3]

The designer shall not show a specific, prequalified complete-joint penetration weld designation on the plans unless a certain type of weld; i.e., "V," "J," "U," etc., is required.

Commentary: The fabricator should be allowed to select the type of complete-joint penetration weld to use, and should show all welds on the shop drawings.

The plans shall identify areas that are subject to tension and those subject to stress reversal.

Commentary: This information will enable inspection personnel to identify the type and extent of testing required. Also, the shop drawings will further identify these areas.

When welding is required during rehabilitation or widening of an existing structure, the plans shall show the type of existing base metal. If the EOR cannot determine the base metal type, or if the type is not an approved base metal included in the ANSI/AASHTO/AWS Bridge

Welding Code, then a welding investigation shall be performed. The EOR shall advise the District Structures Design Office of the situation. The State Materials Office and the Department's independent inspection agency will then determine the welding procedures and welding inspection requirements for the project.

8.20 Welded Splices [6.13.6.2]

At flange transitions, the cross-sectional area shall not be reduced by more than one-half the area of the larger flange plate.

Commentary: These proportions will allow a smooth flow of stress through the splice.

Flange widths should be kept constant within each field-bolted section.

Commentary: By having constant width flange plates in a field section, the fabricator may order plates in multiples of the flange width, butt weld the plates full width, and then strip-out the flanges. Thus, the fabricator is required to make a minimum number of butt welds, handle a minimum number of pieces, and, thereby, minimize his fabrication costs.

The following criteria may be used to make a determination of the number of kilograms, Δw , of material that must be saved to justify the cost of introducing a flange transition:

- For 250 MPa material: $\Delta w = 135 + (0.0176) \times (\text{area of the smaller flange plate, mm}^2)$,
- For 345 MPa material: $\Delta w = 115 + (0.0150) \times (\text{area of the smaller flange plate, mm}^2)$,
- For 485 MPa material: $\Delta w = 100 + (0.0132) \times (\text{area of the smaller flange plate, mm}^2)$.

8.21 Steel Sheet Piling

The designer shall design and show on the plans the required sheet pile properties for both cold-rolled and hot-rolled sections. Design of the cold-rolled section shall be based on flexural section properties that are 85% of the full-section values. Wall components such as caps and tie-backs shall be detailed to work with both the hot-rolled and cold-rolled sections.

Commentary: Tests have shown that cold-rolled sheet pile sections fail in bending at about 85% of their full-section capacity, while hot-rolled sections develop full capacity. There is also some question on the availability of hot-rolled sheet piles; so, by showing the required properties for both types, the Contractor can furnish whichever is available.

Chapter 9

WIDENING AND REHABILITATION

9.1 General

Prior to widening any structure, the inspection report load analysis of the structure shall be reviewed. If the inventory load rating does not meet or exceed MS-18 (HS-20), a more refined analysis shall be performed to accurately load-rate the structure. If the rating is still below the required rating, the bridge should be strengthened to meet the required capacity or shall be considered for replacement.

For flat slab type structures that do not load-rate at MS-22.5 (HS-25), an analysis shall be performed to establish if adequate capacity exists for typical FDOT-permit trucks.

9.2 Widening Classifications and Definitions

Bridge structures to be widened shall be classified as either major or minor widenings according to the definitions of this Article.

9.2.1 Major Widening

A Major Widening is defined as new construction work to an existing bridge facility such that either the resulting total number of traffic lanes or bridge deck area is at least twice that of the unwidened bridge. The area shall be computed using the transverse coping-to-coping dimension.

9.2.2 Minor Widening

Any new construction work to an existing bridge facility that does **NOT** meet the requirements of a major widening as defined above.

Commentary: The term "facility" describes the total number of structures required to carry a transportation route over an obstruction. In this context, adding two lanes of traffic to one bridge of twin, two-lane bridges would be a minor widening because the total number of lanes of resulting traffic, six (6), in the finished "facility" is not twice the sum number of lanes of traffic, four (4), of the unwidened, existing twin bridges.

9.3 Analysis and Design

9.3.1 Aesthetics

The widening of a structure should be accomplished in a manner such that the existing structure does not look “added onto.” The aesthetic level of the widened structure shall be equal to or higher than the aesthetic level of the existing structure. When appropriate, consideration should be given to applying a Class V Finish to the existing structure.

9.3.2 Materials

Materials used in the construction of the widening shall preferably have the same thermal and elastic properties as the materials used in the construction of the existing structure.

9.3.3 Load Distribution

The main load carrying members of the widening shall be proportioned to provide longitudinal and transverse load distribution characteristics similar to the existing structure.

Commentary: Normally, this can be achieved by using the same cross-sections and member lengths as were used in the existing structure.

The construction sequence and degree of interaction between the widening and the existing structure after completion shall be fully considered in determining the distribution of the dead load for the design of the widening as well as for stress checks of the existing structure. The distribution of live load shall be in accordance with the applicable criteria of this Chapter.

9.3.4 Specifications

All widenings shall be designed in accordance with the LRFD except where otherwise superseded by FDOT Standard Specifications for Road and Bridge Construction, the Structures Design Guidelines, the Detailing Manual, the Standard Drawings, or as otherwise directed by the District Structures Design Engineer for Category 1 Structures or the State Structures Design Engineer for Categories 2 Structures.

9.3.5 Overlays

The elimination of asphalt overlays on bridge decks is preferred.

9.3.6 Stresses in the Existing Structure

Stresses in the main exterior member of the existing structure shall be reviewed for construction conditions and the final condition; i.e., after attachment of the widened portion of the structure. When computations indicate overstresses in the exterior member of the existing structure, the EOR and the appropriate FDOT Structures Design Office shall determine the amount of overstress that can be tolerated.

9.3.7 Substructure

As with any bridge structure, when selecting the foundation type for a widening, the EOR shall consider the recommendations of both the District Geotechnical Engineer and the District Drainage Engineer.

9.3.8 Other Special Considerations

- A. All widenings shall be designed for HL-93 loading as a minimum, regardless of the loading used in the original design.
- B. The amount of differential camber which will be present prior to placing the new deck must be considered in detailing connections and selecting or permitting construction methods.
- C. Open or sealed longitudinal joints in the riding surface are safety hazards and are to be avoided whenever possible.
- D. During deck pour and curing, it is preferable to minimize or eliminate live load vibrations from the existing structure.
- E. When a cantilevered deck slab is widened without additional support being provided, the proposed cantilever shall be carefully checked for the increased stresses.
- F. Refer to Chapter 6 for guidance regarding bearing replacement capabilities of widened structures.
- G. Ample clearance between proposed driven piles and existing piles, utilities, or other obstructions shall be provided. This is especially critical for battered piles.
- H. Fixity and expansion devices should be the same in both the widened and existing bridges.

9.4 Attachment of Widening to Existing Structure

9.4.1 Drilling into Existing Structure

When drilling into heavily reinforced areas, exposure of the main reinforcing bars by chipping shall be specified. Drilled holes shall have a minimum edge distance of three times the metal anchor diameter ($3d$) from free edges of concrete and 25 mm minimum clearance between the edges of the drilled holes and existing reinforcing bars. When holes larger than 40 mm in diameter, or deeper than 300 mm are required, core drilling shall be specified. If it is necessary to drill through reinforcing bars, core drilling should be specified.

If FDOT-approved Adhesive Anchor Systems are to be used, they shall comply in all respects with the criteria and requirements of Chapter 7.

9.4.2 Dowel Embedments

Reinforcing bar dowel embedments shall meet minimum development length requirements whenever possible. If this is not possible (e.g., traffic railing dowels into the existing slab), the following options are available:

- A. The allowable stresses in the reinforcing steel shall be reduced by the ratio of the actual embedment divided by the required embedment.
- B. If embedded anchors are to be used to develop the reinforcing steel, use Adhesive Anchor Systems designed in accordance with Chapter 7.

9.4.3 Surface Preparation

Preparation of existing surfaces for concreting shall be in accordance with the Specifications, Article 110-6.

9.4.4 Connection Details

Figures 9-1 through 9-4 are samples of details which have been used successfully for bridge widenings for the following types of bridge superstructures.

- A. Flat Slab Bridges (Figure 9-1):

It is preferred that a portion of the existing slab be removed in order to expose the existing transverse reinforcing for splicing. If the existing reinforcing steel cannot be exposed, the transverse slab reinforcing steel for the widening may be doweled directly into the existing bridge without meeting the normal splice

requirement. When splicing to the existing steel is not practical, Adhesive Anchor Systems, designed in accordance with Chapter 7, shall be utilized for the slab connection details as shown in Figures 9-1 and 9-4.

B. T-Beam Bridges (Figure 9-2):

The connection shown in Figure 9-2 for the slab connection is recommended. Limits of slab removal are at the discretion of the EOR but subject to the Department's approval.

C. Steel and Concrete Girder Bridges (Figure 9-3):

The detail shown in Figure 9-3 for the slab connection is recommended for either prestressed concrete or steel beam superstructures.

Commentary: These figures are for general information and are not intended to restrict the EOR in his judgement.

9.5 Expansion Joints

See Chapter 6.

9.6 Construction Sequence

A construction sequence shall be shown in the preliminary bridge plans submittal for all projects utilizing phase construction. In addition to this detail, the final plans shall include the anticipated construction phases.

9.7 Bridge Widening Rules

In order to minimize changing the characteristics of the deck slab supports and/or unduly affecting maintenance and aesthetics, the following criteria must be adhered to during the preparation of plans for bridge widening:

- The widened portion of a major widening shall be designed as if it were a new structure and, therefore, comply with the most current requirements. The widened portion of a minor widening shall match the existing structure as stipulated elsewhere in this Chapter.
- The mixing of concrete and steel beams in the same span shall be avoided.
- The use of beams of the same type as those in the existing structure is preferred; however, if the existing beams are cast-in-place concrete, the

widened deck shall be supported on prestressed beams.

- Providing the required vertical clearance on Interstate structures should always be considered and decision factors documented. To assist in meeting the minimum vertical clearance requirements on a widening, the standard beam depth may be decreased. Where the existing bridge does not satisfy current vertical clearance requirements and where the economics of doing so are justified, the superstructure shall be elevated and/or the underpassing roadway shall be lowered as part of the widening project.

Commentary: The stated clearance criteria is particularly important for bridges that have a history of frequent superstructure collisions from over-height vehicles.

- Generally, the transverse reinforcement in the new deck should be spaced to match the existing spacing. Different bar sizes may be used if necessary.
- Voided-slab bridges require special attention. If widening of a voided-slab bridge is being proposed, the DSDE shall be contacted for guidance. The DSDE will coordinate with the SDO to establish recommendations and criteria for the widening of the particular structure.
- For all widenings, the EOR must confirm that the available existing bridge plans depict the actual field conditions. Any discrepancies between existing plans and actual field conditions, which in the EOR's opinion are critical to the continuation of the widening design, should be brought to the attention of the FDOT's Project Manager at the earliest possible time.

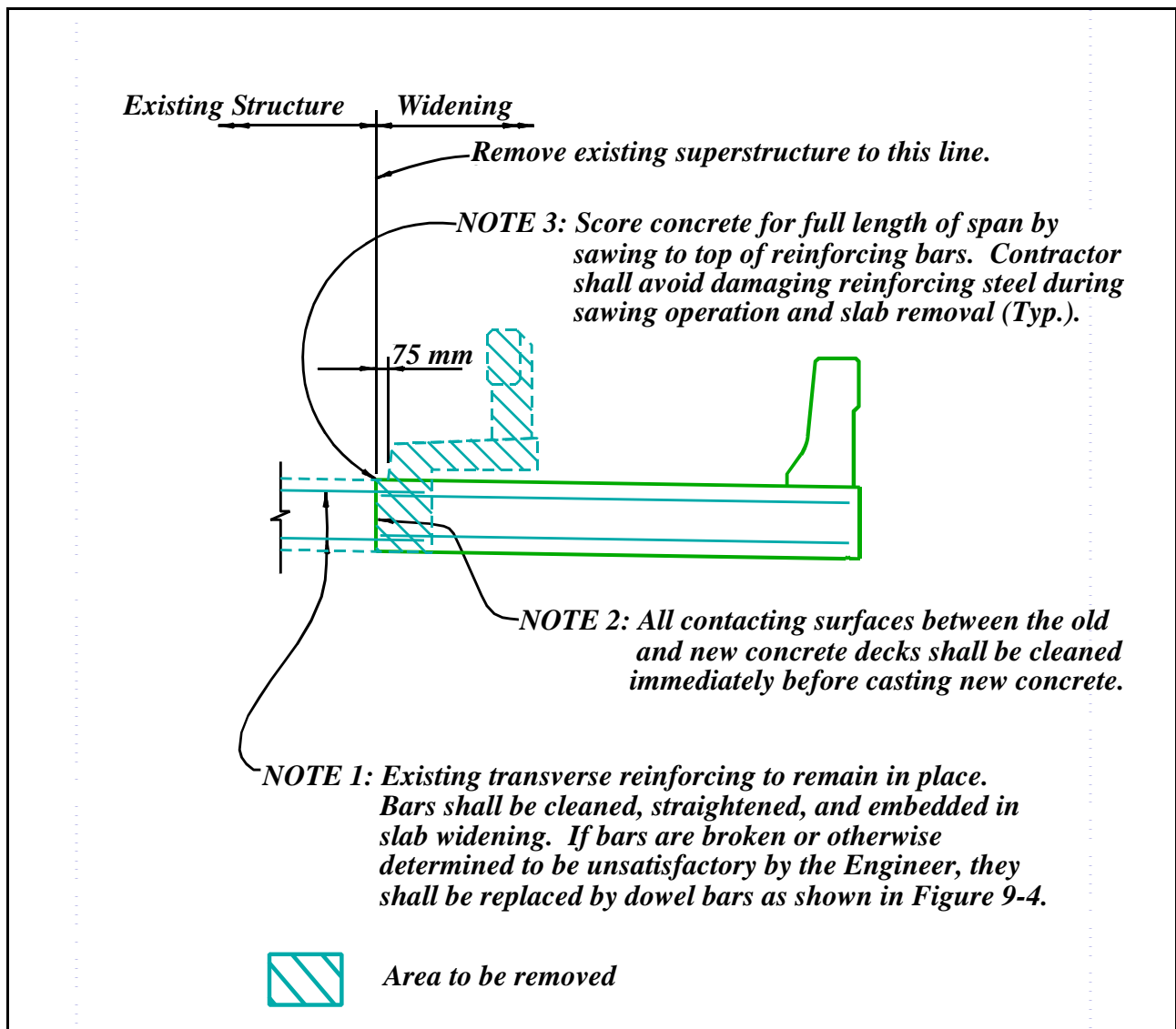
Commentary: In general, confirming the agreement of existing plans with actual field conditions should be included as part of any new survey. A structural engineer shall be involved in checking that the existing plans agree with actual field conditions for such items as:

- *Bridge location, pier location, skew angle, stationing*
- *Span lengths*
- *Number and type of beams*
- *Wingwall, pier, and abutment details*
- *Utilities supported on the bridge*
- *Finished grade elevations*
- *Vertical and horizontal clearances*

- *Other features critical to the widening*

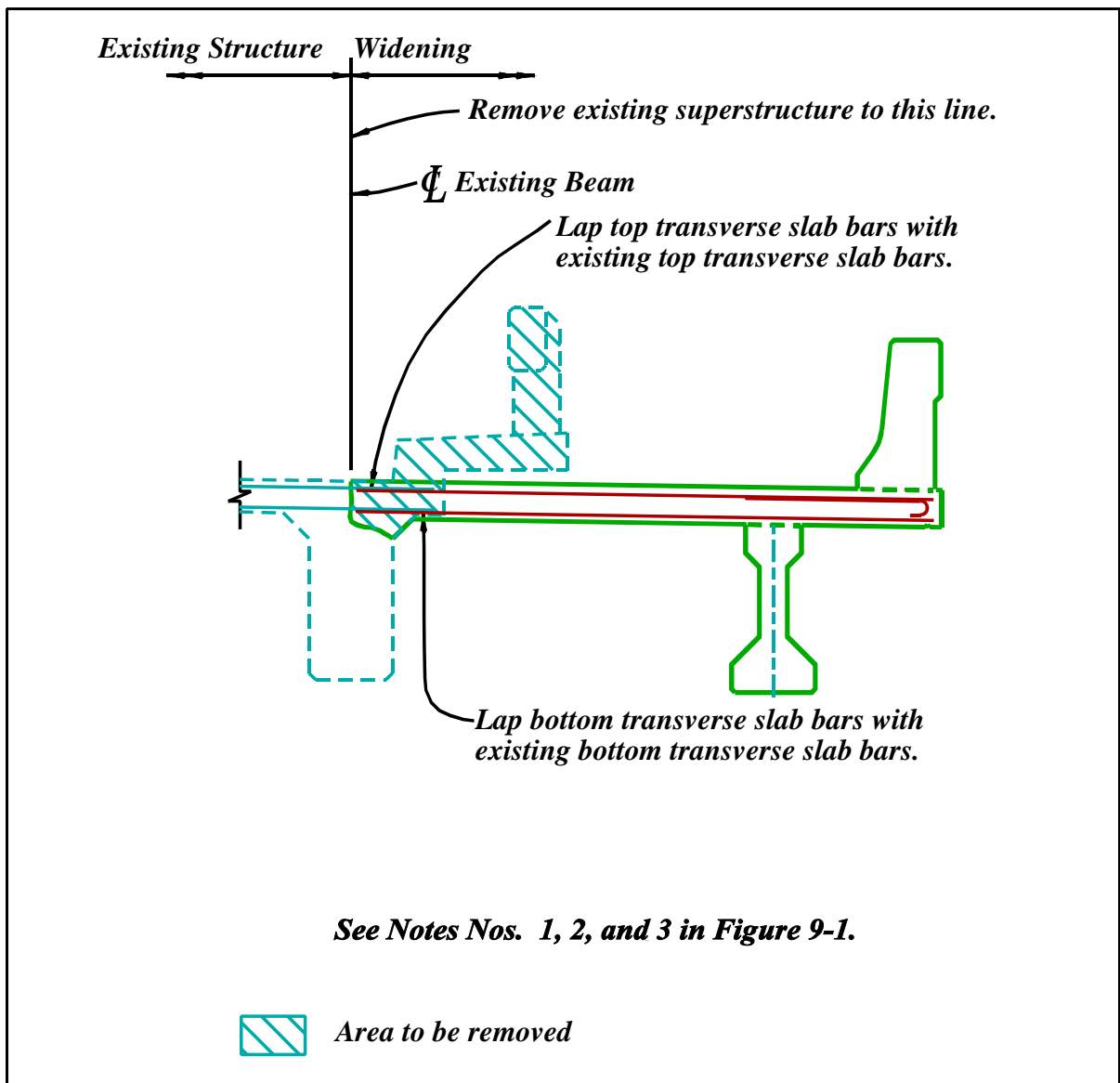
9.8 Upgrading Existing Bridges

The plans should include details for performing any necessary repairs to the existing bridge. M.O.T. plans should address traffic needs during construction activities on the existing structure such as installation of new joints, deck grooving, etc. For all widenings, when the existing deck has not been grooved, consideration should be given to grooving the existing deck to improve the skid resistance. For decks with insufficient cover over the top mat of steel, refer to Chapter 2 for guidance on the use of penetrant sealers.



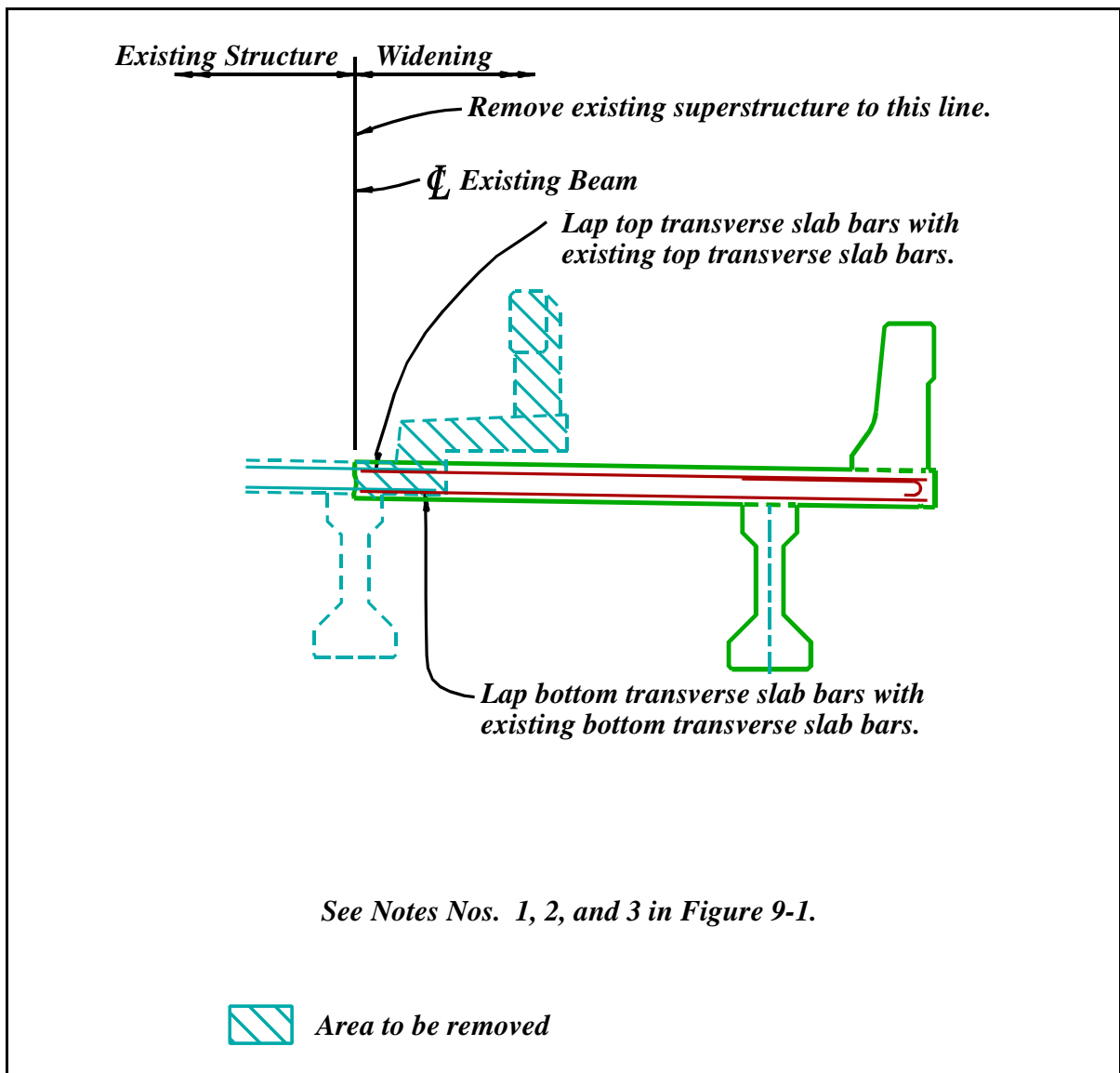
WIDENING DETAIL FOR FLAT SLAB SUPERSTRUCTURE

Figure 9-1



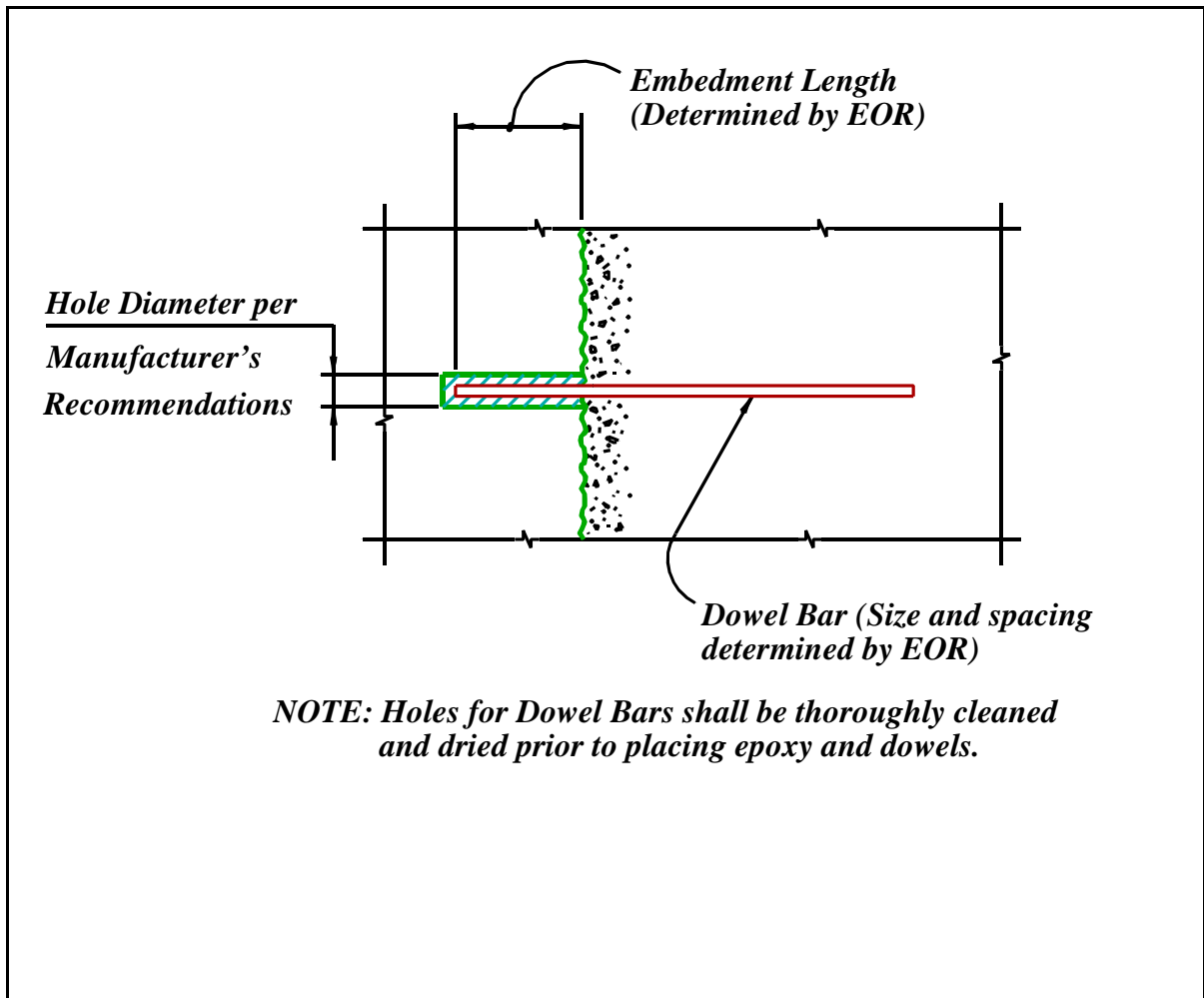
WIDENING DETAIL FOR MONOLITHIC BEAM AND SLAB SUPERSTRUCTURE

Figure 9-2



WIDENING DETAIL FOR AASHTO BEAM SUPERSTRUCTURE

Figure 9-3



WIDENING DETAIL SHOWING DOWEL INSTALLATION

Figure 9-4

Chapter 10

MOVABLE BRIDGE REQUIREMENTS

10.1 General

This Chapter contains information and criteria related to the design of movable bridge projects and is a supplement to the AASHTO Standard Specifications for Movable Highway Bridges. Because LRFD does not address design requirements for movable bridges, this Chapter shall be considered as the authority for such designs and as a complete, self-contained supplement to LRFD. Items and components not included in the AASHTO Standard Specifications for Movable Highway Bridges; i.e, span locks, live load shoes, etc., will be designed using LRFD.

10.2 Applicability and Requirements

The movable bridge design criteria of this Chapter are applicable to both new bridges and rehabilitation of existing bridges. Variations from these practices may only be authorized by the Mechanical/Electrical Section of the Structures Design Office (SDO). Projects for which the criteria are applicable shall result in designs that:

- Provide new bascule bridges with a two leaf per span configuration.
- Maximize assurance of reliable operation of movable bridges through incorporation of redundant features in movable bridge drive and control systems. This practice is applicable both to new bridge designs and to rehabilitation of existing bridges.

The EOR shall include recommendations for redundant drive and control systems in the BDR/30% plans submittal. For bridges having low rates of anticipated bridge openings or average daily traffic, application of redundant drive and control systems may not be cost effective. In this event, the EOR shall submit such information in the BDR and provide appropriate recommendations for omission of redundant systems.

Commentary: Redundant drive configurations include:

- *Hydraulic drive systems operated by multiple hydraulic cylinders. In these systems, a pump drive motor or its hydraulic pump can be isolated and bridge operations can continue while repairs are accomplished.*
- *Conventional gear driven systems, driven through one gear train into a single rack of a two-rack bridge.*

- *Trunnion supports on each side of the main girder shall be similar in stiffness vertically and laterally.*

Provide two parking spaces for bridge tenders in all new bridge designs.

All new gear driven movable bridges shall be designed with two rack drives actuated by dual motor drive systems, either of which will be capable of operating the bridge leaf. Normal operation of this configuration will involve operation of both drive systems.

Do not modify existing single motor/gear rack drive systems unless specifically directed by the Mechanical/Electrical Section of the SDO.

Design new movable bridge leaf(s) so that a leaf can be opened without excessive torsional deflection when unsymmetrically driving the racks.

Design new movable bridges leaf support systems to utilize either of the following trunnion support systems:

- Simple, rotating trunnion configuration, with bearing supports, or towers, on both inboard and outboard sides of the trunnion girder.
- Fixed trunnion configuration having a bearing located in the bascule girder and with trunnion supports located on both inboard and outboard sides of the bascule girder.

Design bascule leaf configurations such that the vertical clearance, in the full open position, will be unlimited between the fenders.

Specific practices required by the Department in considering the applicability and requirements of bascule bridges:

- Examination and evaluation of alternative bridge configurations offering favorable life-cycle cost benefits.
- Consideration of improved design or operational characteristics providing advantage to the traveling public.
- Incorporation of design and operational features which are constructible and which can be operated and maintained by the Department's forces.
- Maintaining a consistency of configuration, when feasible, for movable bridges throughout the State.

Design hydraulic cylinder actuated movable bridges to function in spite of loss of either a main

drive motor, hydraulic pump, or drive cylinder.

Commentary: For example, with a cylinder failure in a 4-cylinder system, the "matching" cylinder on the leaf will be deactivated and the leaf will be operated on two cylinders equally distant from the centerline of the bridge leaf. Design the system to include all necessary valving, piping, etc., to permit safe and expeditious changeover to the redundant mode. Specify a permanent plaque displayed in a convenient location on the machinery platform describing actions (valve closures and openings, electrical device deactivation, etc.) necessary for operation in the redundant mode.

10.3 Application of Metrics for Rehabilitation Projects

Rehabilitation plans for existing bridges will be produced in metrics. If portions of the project are to remain "as is," while other portions of the project employing like components are to be modified or replaced, retain the original configuration on modified or replacement hardware.

Commentary: For example, if the original configuration of the bridge utilized four hydraulic cylinders, and the rehabilitation plans required replacement of two of the cylinders, the replacement cylinders should exactly match the original cylinder configuration; plans for replacement cylinders will specify metric dimensions, soft converted to meet the same dimensions as the existing cylinders.

In rehabilitation projects, change to metric measurement systems when drive system components are changed. If hydraulic pumps in a cylinder drive system are to be replaced, pressure gauges and flow meters will be changed to read in metric units.

Commentary: For example, existing English system components, with pressure gauges and flow meters reading in PSI and Gallons/Minute, change instrumentation to read in kPa and Liters/Minute.

Incorporate original construction drawings with plans for rehabilitation of a movable bridge, if it is determined that existing drawings are necessary to clarify the project work. Existing plans shall **not** be redrawn to show metric equivalents.

10.4 General Requirements

10.4.1 Definitions and Terms

A. Auxiliary Drive:

Hand crank, gearmotor with disconnect-type coupling, portable hydraulic pump, drill, etc., that can be used to lower leaf(s) to open the bridge to

vehicular traffic or raise the leaf(s) to open the bridge to marine traffic if the main drives fail.

B. Control System:

Directs bridge operation.

C. Creep Speed:

Not more than 10% of full speed, final creep speed will be determined by bridge conditions.

D. Drive System:

Provides leaf(s) motion.

E. Emergency Stop:

Leaf(s) stop within (3±1) seconds of depressing the **EMERGENCY STOP** push-button or in the event of a power failure. **All** other rotating machinery stops instantly.

F. End-of-Travel Function:

Contact connection where a closed contact allows operation and an open contact stops operation (i.e., leaf limit switches).

G. Full Seated:

Leaf(s) is(are) at rest on live load shoes, interlock OK to drive span locks.

H. Full Open:

Tip of leaf clears fender on a vertical line.

I. Full Speed:

Maximum design speed at which the drive(s) move(s) the leaf(s).

J. Gate Down:

Gate arm is in the lowered position (closed to vehicular traffic), gate arm is horizontal.

K. Gate Up:

Gate arm is in the raised position (open to vehicular traffic), gate arm is vertical.

L. Hard Open:

Leaf opening such that counterweight bumper blocks come in contact with pier bumper blocks.

M. Indicating Function:

Contact connection where a closed contact indicates operation and an open contact indicates no operation (i.e., indicating lights).

N. Interlocks or Safety Interlocks:

Ensure events occur in sequence and no out-of-sequence events can occur.

O. Leaf:

Movable portion of bridge.

P. Mid-Cycle Stop:

Leaf(s) will stop following normal ramping after depressing the **STOP** push-button when in the middle of an opening or closing cycle.

Q. Near Closed:

A point 8 to 10 degrees (approximately, final position to be field determined) before **FULL CLOSED**, drive to creep speed.

R. Near Open:

A point 8 to 10 degrees (approximately, final position to be field determined) before **FULL OPEN**, drive to creep speed.

S. Normal Stop:

Leaf(s) go through the normal stop sequence, switching from full speed to creep speed when **NEAR OPEN** limit switch (or **NEAR CLOSED** limit switch) is operated, and continuing at creep speed until **FULL OPEN** (or **FULL CLOSED**).

T. Ramp:

Rate of acceleration or deceleration of leaf drive.

U. Span:

The movable portion of the bridge structure, center line of trunnion to center line of trunnion.

V. Stop:

When depressing STOP button on console, leaf decelerates to a full stop following acceleration/deceleration ramp.

10.4.2 Specifications and Requirements

Comply with the requirements of the latest edition of AASHTO Standard Specifications for Movable Highway Bridges. Specify electrical equipment conforming to IEEE, NEMA, or UL. Comply with the latest edition of the National Electrical Safety Code, National Electrical Code, and local ordinances.

Use the FDOT's Standard Specifications for Road and Bridge Construction and the "Technical Special Provisions" issued by the Mechanical/Electrical Section of the SDO. Additional specifications may be required.

Provide detailed calculations to justify all equipment and systems proposed. Calculations shall be submitted, for review, for all aspects of the systems and components. Substantiate recommended equipment and sizes. Submit calculations in an 8-1/2x11 binder.

10.4.3 Speed Control for Leaf-driving Motors

Specify a drive capable of developing 100% torque at zero speed. Drive shall be capable of opening or closing the bridge in no more than 69 seconds (see Figure 10-1) under AASHTO Condition A or Condition C (hydraulic drives will open or close the bridge in no more than 128 seconds under AASHTO Condition C).

Clearly indicate in the plans what the torque requirements are for each of the AASHTO Conditions: A, C, and E.

The drive shall be capable of operating at full speed under AASHTO Condition A, full speed (half speed for hydraulic drives) for drives under AASHTO Condition C, and hold (zero speed) for drives under AASHTO Condition C.

10.4.4 Bridge Terminology

See Figure 10-2 for standard bridge terminology.

10.5 Machinery Systems

10.5.1 Trunnions and Trunnion Bearings

A. Trunnions:

1. All shoulders shall have fillets of appropriate radius. Suitable provisions for expansion should be provided between shoulders and bearings.
2. Keyways should have a maximum width of $1/4$, and a maximum depth of $1/8$, the shaft diameter. All keys and keyways shall have at least an LC3 fit with a surface finish of $1.6 \mu\text{m}$ or better.
3. Do not use keyways between the trunnion and the trunnion hub/collar.
4. For shafts over 200 mm diameter, a hole $1/5$ the shaft diameter shall be bored lengthwise through the center of the shaft. The trunnion shall extend at least 15 mm beyond the end of the trunnion bearings. Provide a 50 mm-long counterbore concentric with shaft journals at each of the hollow shafts.
5. Shafts shall have a minimum shrink fit of FN2 with the hub and shall have a surface finish of $1.6 \mu\text{m}$ or finer throughout. In addition to the shrink fit, dowels of appropriate size shall be drilled through the hub into the shaft after the shaft is in place. When anti-friction roller bearings are used, the surface finish shall be compatible with the bearing manufacturer's requirements. For journals, the running clearance shall be at least an RC6 fit with a surface finish of $0.2 \mu\text{m}$ or better. Deflections of the trunnion with load must be calculated and compared with these clearances to insure the journals do not bottom out and bind, particularly on rehabilitations and Hopkins frame bridges. Clearances may have to be adjusted.
6. Lubrication for trunnion journals shall be through grooves cut into the bushing. The grooves shall be straight, parallel to the axis of the shaft, with a radius on all sides of approximately 4 mm. Grooves shall be accessible to cleaning by a wire up to 8 mm diameter. No fewer than four grooves shall be provided, located symmetrically in the area of contact in the shaft. The placement of the grooves shall cover at least 90 degrees of rotation and be able to supply lubrication to all portions of the trunnion journal swept by one movement of the span. Each groove shall be supplied with a pressure fitting and flush out port.
7. For rehabilitation of existing Hopkins trunnions, verify that trunnion eccentrics have capability for adjustment to accommodate required

changes in trunnion alignment.

B. Hubs and Rings:

1. Hubs and Rings shall have a mechanical shrink fit to both the trunnion and the structural bascule girder or truss. The shrink fit to the structural component shall be a minimum of FN2 between hub and ring and the structural steel. The minimum mounting surface finish of both hubs and rings shall be 3 μm or finer.
2. Hubs shall have a 13 mm diameter groove cut at the end of the bore for the hub in the structural steel. The length of the hub shall be at least equal to the bore diameter. The minimum ratio of the outer hub flange diameter to the bore shall be 2.8, and the minimum ratio of non-flanged hub diameter to the bore shall be 1.8. The minimum thickness of any portion of the hub flange shall be 50 mm. The minimum thickness of the ring shall be 50 mm.
3. Secure the hub and ring by bolts in addition to the shrink fit. The bolt circle shall be concentric to the axis of the trunnion and spaced at the maximum sealing pitch allowable in AASHTO Standard Specifications for Highway Bridges. The hub shall be spot faced at each bolt a diameter 13 mm larger than the appropriate washer dimensions. Bolts shall be at least 22 mm in diameter. The hub may have undersized cast bolt holes which shall be reamed to full size after the hub and ring are in place.
4. Detail the hub so that it can be installed on the trunnion in any position.
5. See Figure 10-3.

10.5.2 Span Balance

A. New Construction:

Design new bascule bridges so that the center of gravity may be adjusted vertically and horizontally. Design the bridge so that the center of gravity is forward (leaf heavy) of the trunnions or center of roll by an amount sufficient to produce a positive reaction at the live load shoes. Ensure the reaction is large enough to secure the bridge against undue vibration. Locate the leaf center of gravity so that the specified unbalance does not increase appreciably during operation or exceed that unbalance for which the drive mechanism was designed.

Commentary: A rule of thumb is to insure that the center of gravity is located at minus one-

half (-1/2) the total bridge opening angle with the leaf in the down position and rotates to plus one-half (+1/2) of the total bridge opening angle with the leaf in the full open position.

Example: If the total opening angle is 75 degrees, then the center of gravity would rotate from minus 37.5 degrees (-37.5°) to plus 37.5 degrees (+37.5°).

B. Rehabilitation Projects:

Bridge rehabilitations in which the leaf balance must be adjusted, the above provisions also apply. Include detailed leaf balance adjustment plans, including the location and weight of any ballast to be furnished and installed to achieve an acceptable balance condition.

C. Design Unbalance:

For new and rehabilitated bridges, state the design unbalance in the plans using "WL" and " α ."

Where:

α = angle of elevation of the center of gravity when the leaf is closed.

WL = total leaf weight, "W," times the distance, "L," from the trunnion axis to its center of gravity.

10.5.3 Speed Reducers

Specify and detail speed reducers to meet the requirements of the latest edition of ANSI/AGMA 6010 Standard for Spur, Helical, Herringbone, and Bevel Enclosed Drives. Specify and detail gearing to conform to AGMA Quality No. 8 or higher using following factors:

- Service Factor of 1.0 or higher indicating "Actual Input" and "Output Torque" requirements
- C_m and K_m = per the current AGMA 6010 Standard
- C_a and K_a = 1.0
- C_l and K_l = 1.0
- C_r and K_r = 1.0

A reverse bending factor of 0.8 is required in the strength rating (not required for vertical lift drives).

Allowable contact stress numbers, "Sac," shall conform to the current AGMA 6010 Standard for through-hardened and for case-hardened gears.

Allowable bending stress numbers, "Sat," shall conform to the current AGMA 6010 Standard for through-hardened and for case-hardened gears.

Commentary: These allowable contact and bending stress numbers are for AGMA Grade 1 materials. Grade 2 allowables are permitted only with an approved verification procedure and a sample inspection as required per Mechanical/Electrical Section of the SDO instructions.

All speed reducers on a bridge should be models from one manufacturer unless otherwise approved by the Mechanical/Electrical Section of the SDO. Include gear ratios, dimensions, construction details, and AGMA ratings on the Drawings.

The reducer shall be capable of withstanding an overload torque of 300 percent of full-load motor torque. This torque shall be greater than the maximum holding torque for the span under the maximum brake-loading conditions.

Gears shall have spur, helical, or herringbone teeth. Bearings shall be anti-friction type and shall have a B-10 life of 40,000 hours as defined in AASHTO, except where rehabilitation of existing boxes requires sleeve-type bearings. Housings shall be welded steel plate or steel castings. The inside of the housings shall be sandblast-cleaned prior to assembly, completely flushed, and be protected from rusting. Exact ratios shall be specified.

Each unit shall have means for filling and completely draining the case, have an inspection cover to permit viewing of all gearing (except the differential gears, if impractical), and both a dipstick and sight glass to show the oil level. Sight glasses shall be of rugged construction and protected against breakage. Drains shall have shutoff valves to minimize spillage. Each unit shall be furnished with a moisture trap breather of the desiccant type with color indicator to show desiccant moisture state.

Splash lubrication shall be used on the gears and bearings, unless otherwise required by the gear manufacturer. Pressurized lubrication systems shall not be used on speed reducer boxes unless approved in writing by the Mechanical/Electrical Section of the SDO. Provide remote indication of lubrication system malfunction.

Commentary: If a pressurized lubrication system is specified for the reducer, a redundant lubrication system shall also be specified. The redundant system shall operate at all times when the primary system is functioning.

10.5.4 Open Gearing

Limit the use of open gearing. When used, design open gearing per AGMA specifications. Design and specify guards for high speed gearing.

10.5.5 Span Locks

A. General:

1. Specify only rectangular lock bars.
2. Span locks should be located as close to the main bascule girders as possible, giving consideration to maintenance access.
3. Specify 100 x 150 mm lock bars unless analysis shows need for a larger size. Design calculations and the selection criteria are to be submitted for review and approval.
4. The bar is to be installed in the guides and receivers with bronze wear fittings top and bottom, properly guided and shimmed. Lubrication will be provided at the sliding surfaces. Both the front and rear guides are to have a "U" shaped wear-plate that restrains the bar sideways as well as vertically. The receiver is to have a flat wear-plate to give freedom horizontally to easily insert the lock-bar in the opposite span. The total vertical clearance between the bar and the wear-shoes shall be 0.254 to 0.635 mm. The side clearance on the guides shall be 1.5875 ± 0.794 mm.
5. Provide vertical stiffening behind the web for support of guides and receivers.
6. Mount guides and receivers with shims for adjusting without major fitting removal. Wear-plate shims shall be slotted for insertion and removal.
7. Provide lubrication at convenient locations for routine maintenance.
8. Actuation parts shall be mounted on the lock-bars in the proper location to intersect limit switches to control each end of the stroke. Incorporate a means to adjust the limit switch actuation. The receiver end of the lock-bar shall be tapered to facilitate insertion into the receivers of the opposite span.
9. Connection of the lock-bar to the hydraulic cylinder shall allow for the continual vibration due to traffic on the span. The connecting element must be free to move vertically to prevent excessive forces acting on the extended rod or the cylinder mounted on center trunnions and to swing with the movement.
10. The hydraulic fluid system is to be a reversing motor-driven pump and

pipng system connected to the cylinder. Specify relief valves to prevent over pressure should the lock-bar jam. Specify pilot operated check valves in the lines to the cylinder to lock the cylinder piston in place when pressure is removed to prevent undesirable motion. A hydraulic hand pump and quick-disconnect fittings on the piping are to be provided to allow setting or releasing of the lock-bar on loss of power. The time of each bar movement shall be 5 to 9 seconds.

11. Design and specify access platforms with access hatches located out of the roadway.

B. Lock Design Standards:

1. The empirical formula, Equation 10-1 listed below, can be used to determine double leaf bascule lock loads with acceptable results; however, more exact elastic analysis can be used if the solution thus obtained is not accurate enough.

$$S = \frac{P}{4} \left(\frac{A}{L}\right)^2 \left(3 - \frac{A}{L}\right) \quad [\text{Eq. 10-1}]$$

Where:

S = Shear in lock in Newtons for a given load on the span, "P."

A = Distance in meters from the support to the given load, "P."

L = Distance in meters from the support to the center lock.

(Note: See Figure 10-4 for diagrammatic sketch of "S," "A," and "L.")

2. A double-leaf bascule span expected to carry a substantial percentage of truck traffic should have locks designed on a more conservative basis because of the higher number of maximum load cycles expected over a given time period.

10.5.6 Brakes

Use thrustor type brakes. Provide a machinery brake and a motor brake. Submit calculations justifying the brake torque requirements. Show brake torque requirements on plans. Carefully consider machinery layout when locating brakes. Avoid layouts which require removal of multiple pieces of equipment for maintenance of individual components.

10.5.7 Couplings

Submit calculations justifying coupling sizes specified. Submit manufacturer's literature.

10.5.8 Clutches

Clutches for emergency drive engagement shall be rated for the maximum emergency drive torque. The engaging mechanism shall be positive in action and shall be designed to remain engaged or disengaged while rotating at normal operating speed. Provisions shall be made so that the main operating drive is fully electrically disengaged when the clutch is engaged.

10.5.9 Bearings (Sleeve and Anti-Friction)

- A. Sleeve Bearings shall be grease-lubricated bronze bushings 200 mm in diameter or less and shall have grease grooves cut in a spiral pattern for the full length of the bearings.
- B. Anti-Friction Bearing pillow block and flange-mounted roller-bearings shall be adaptor-mounting, self-aligning, expansion and/or non-expansion types. Housings shall be cast steel and capable of withstanding the design radial load in any direction, including uplift. Housing shall be furnished by the same manufacturer as the bearing. Roller bearings and housings shall be furnished by the same manufacturer. Bases shall be cast without mounting holes. Mounting holes shall be "drilled-to-fit" at the site at the time of assembly with the supporting steel work. Seals shall retain the lubricant and exclude water and debris. Cap bolts on pillow blocks shall be high-strength steel. The cap and cap bolts shall be capable of resisting the rated bearing load as an uplift force. Where clearance or slotted holes are used, the clearance space shall be filled with a non-shrink grout after alignment to insure satisfactory side load performance.

10.5.10 Anchors

For machinery supports anchored to concrete, design for the maximum forces generated in starting or stopping the leaf plus 100% impact. Design hydraulic cylinder supports for 150% of the relief valve setting or the maximum operating loads plus 100% impact, whichever is greater. Detail machinery supports anchored to the concrete by preloaded anchors such that no tension occurs at the interface of the steel and concrete under any load conditions.

Mechanical devices used as anchors shall be capable of developing the strength of reinforcement without damage to the concrete. All concrete anchors shall be undercut-bearing, expansion-type anchors. The anchorage shall be developed by expanding an anchor sleeve into a conical undercut so as to eliminate direct lateral stresses found in the setting of conventional anchors. The expansion anchors shall meet the ductile failure criteria of American Concrete Institute (ACI) Standard 349, Appendix B. The design shall insure an expansion anchoring system that can develop the tensile capacity of the bolt without slip or

concrete failure. The bolt shall consistently develop the minimum specified strength of the bolting material to provide a favorable plastic stretch over the length of the bolt prior to causing high energy failure. Require pullout testing of anchors deemed to be critical to the safe operation of the bridge machinery system. Pullout verification tests shall be performed at not less than 200% of maximum operational force levels.

Design the conical undercut and the nut to transfer the bolt tension load into direct bearing stress between the conical nut and expansion sleeve and the expansion sleeve and conical concrete surface. The depth and diameter of the embedment shall be sufficient to assure steel failure, with concrete cone shear strength greater than the strength of the bolting material.

A. Standards:

1. ACI 349, Appendix B, Section B.7 - Steel Embedments.
2. ASTM A 193 - Alloy Steel and Stainless Steel Bolting Materials for High-Temperature Service. Stud Bolts and Conical Nuts shall be ASTM A193, Grade B7, with a minimum ultimate tensile strength of 862 MPa and a minimum yield strength (0.2% offset) of 724 MPa.
3. ASTM A 194 - Carbon and Alloy Steel Nuts for Bolts for High-Pressure and High-Temperature Service. Nuts shall be ASTM A194, Grade 2H heavy hexagonal nuts.
4. ASTM A 513 - Electric-Resistance-Welded Carbon and Alloy Steel Mechanical Tubing. Bolt sleeve/Distance tube (Expansion sleeve) shall be ASTM A513, Type 5, mandrel-drawn alloy steel mechanical tubing.
5. ASTM F 436 - Hardened Steel Washers. Washers shall be ASTM F 436, Type 3, weathering steel.
6. ASTM A 36 - Structural Steel. Plate washers shall conform to ASTM A 36.
7. FS QQ-Z-325C (Type II, Class 3) - Plating of Anchorage Components

B. Stainless Steel Anchor Bolt Standards:

1. ASTM A 269 - Seamless and Welded Austenitic Stainless Steel Tubing for General Service. Stainless Steel Distance Tube/Expansion Sleeve.
2. ASTM A193, Grade B8 - Threaded Rod/Stud.
3. ASTM A193, Grade B8 - Conical Nut.

4. ASTM A194, Grade 8 - Heavy Hexagonal Nut (Washer shall be 18-8 Stainless Steel).

C. Anchor Bolt Design:

Anchor bolts subject to tension shall be designed at 200% of the allowable basic stress and shall be shown by tests to be capable of developing the strength of the bolt material without damage to concrete.

The design strength of embedment is based on the following maximum steel stresses:

1. Tension, $f_{s_{max}} = 0.9f_y$
2. Compression and Bending, $f_{s_{max}} = 0.9f_y$
3. Shear, $f_{s_{max}} = 0.55f_y$ (shear-friction provisions of ACI, Section 11.7, shall apply)

The permissible design strength for the expansion anchor steel is reduced to 90% of the values for embedment steel.

For bolts and studs, the area of steel required for tension and shear based on the embedment criteria shall be considered additive.

The design pullout strength of concrete, P_c , in Newtons, shall be calculated as:

$$P_c = 0.33 \phi \sqrt{f'_c} A \quad [\text{Eq. 10-2}]$$

Where:

- Φ = Capacity reduction factor, 0.65
 A = Projected effective area of the failure cone, mm^2
 f'_c = Specified compressive strength of concrete, MPa

Steel strength controls when the design pullout strength of the concrete, P_c , exceeds the minimum ultimate tensile strength of the bolt material.

The effective stress area shall be defined by the projected area of the stress cone radiating toward the concrete surface from the innermost expansion contact surface between the expansion anchor and the drilled hole. The effective area shall be limited by overlapping stress cones, by the intersection of the cones with concrete surfaces, by the bearing area of anchor heads, and by the overall thickness of the concrete. The design pullout strength of concrete shall be equal to or greater than the minimum specified tensile strength or average tensile strength if the minimum is not defined for the bolting material.

To maintain 100% anchor tension capacity, the minimum center to center spacing between any two anchors shall be the sum of the embedments of the two anchors. In case of anchor bolts of the same length, the depth of embedment shall be equal to the diameter of the stress cone at the concrete surface minus the diameter of the drilled hole. The minimum embedment depth shall be determined such that the ultimate tensile capacity of the bolt material exceeds the design pullout strength of concrete.

10.5.11 Fits

Use ANSI Standard B32.4M which presents a Preferred Series of sizes and tolerances for metric sizes. The metric version for the parts shall be specified in two stages:

A letter shall be associated with each member which would indicate fundamental deviation of the part from the basic size. A number following the letter shall be used to indicate the range of tolerance or tolerance grade applied in addition to the fundamental deviation for the part.

10.5.12 Finishes

Call out surface roughness in metric units (micrometers). See the Industrial Press, "Machinery Handbook," 24th Edition, p. 672.

10.6 Hydraulic Systems

Perform complete analysis and design of hydraulic systems utilized for leaf drive and control, including evaluation of pressure drops throughout the circuit for all loading conditions. Pressure drops shall be calculated for all components of their circuits including valves, filters, hoses, piping, manifolds, flow meters, fittings, etc. Power requirements shall be determined based upon pressure drops at the required flows and conservative pump efficiency values.

Design the system so that normal operating pressure is limited to 17.2 MPa. During short periods of time in emergency operations, pressure can increase to 20.7 MPa, maximum. Hydraulic system strength calculations to be correlated with the structure loading analysis.

Design the power unit and driving units for redundant operation so that the bridge leaf(s) may be operated at a reduced speed with one power unit or one driving unit out of service. Design the power unit to permit its installation and removal in the bridge without removing any major components. Design the power unit to allow the removal of each pump, motor, filter, and main directional valves without prior removal of any other main components. Operation of the redundant components shall be possible with the failed component removed from the system.

Design all span operating hydraulic components within the pier enclosure to prevent any

escape of oil to the environment. Specify a drip pan extending beyond the outermost components of the power unit and flange connections to prevent spilling oil leakage on the machinery room floor. Specify sump pumps and other clean up devices suitable for safe collecting and removing of any spilled oil.

Design the hydraulic system to limit the normal operating oil temperature to 77 ° C during the most adverse ambient temperature conditions anticipated.

10.6.1 Hydraulic Pumps

Specify minimum pressure rating of pumps to be 1.5 times the maximum operating pressure. Specify pumps of the Pressure Compensated type. Variation of the pressure setting, including ± 50 cst viscosity change shall be $\pm 2.5\%$ maximum. Overall minimum efficiency shall be 0.86. Boost pumps of any power, and auxiliary or secondary pumps less than 3.7 kW, need not be pressure compensated.

10.6.2 Hydraulic Motors

Specify hydraulic motors of the radial piston or axial piston type depending on the application. For low RPM, high torque application, use the radial piston type. Limit the use of non-piston motors, such as gear, vane, etc., to applications other than main span operation. Specify a load-holding spring-set brake for motors that are permanently linked to a load.

10.6.3 Cylinders

Design the hydraulic cylinder drive systems to prevent sudden closure of valving, and subsequent sudden locking of cylinders, in the event of a power failure or emergency stop. Specify cylinders designed according to ASME Pressure Vessel Code, Section 8. Specify cylinders with a minimum theoretical static failure pressure rating of 11 (69 MPa) as defined by NFPA Standards; and designed to operate on bio-degradable based hydraulic fluid unless otherwise approved by the Mechanical/Electrical Section of the SDO.

Specify stainless steel rods with chrome plated finish 0.127 to 0.305 mm thick per QQ-C-320C, Class 2a or others as approved by the Mechanical/Electrical Section of the SDO. Specify that the rod end fitting shall have a permanent clevis pin that is instrumented for strain gauge readings by the Department's bridge balance instrumentation. The pin shall be captured to prevent turning and loosening. The pin shall be fixed relative to the cylinder rod and not with the bridge structure. Deviations from this must be approved by the Mechanical/Electrical Section of the SDO.

The main lift cylinders shall be provided with pilot operating counterbalance or other load protection valves. They shall be manifolded directly to ports of cylinder barrel and hold load in position if supply hoses leak or fail.

10.6.4 Control Components

A. Flow Control Valves:

Use of non-compensated flow control valves shall be limited to applications where feed rates are not critical and where load induced pressure is relatively constant. Where load induced pressure is variable, specify pressure compensated flow control valves.

B. Directional Valves:

Vertical mounting of solenoid Directional Valves where solenoids are hanging from the valve are to be avoided; horizontal mounting is recommended. Solenoid operated directional control valves provided with a drain connection to reduce response times must always be mounted horizontally.

C. Relief Valves:

Specify relief valves to protect all high pressure lines.

D. Check Valves:

Specify poppet type check valves on main circuits or located to hold loads.

10.6.5 Hydraulic Lines

A. Piping:

Specify stainless steel piping material conforming to ASTM A312 GRTP 304L or TP316L. For pipe, tubing, and fittings, the minimum ratio of burst pressure rating divided by design pressure in the line shall be 4.

Provide calculations indicating that the velocity of fluid is at or below 1.31 m/s in suction lines, 1.97 m/s in return lines, and 6.56 m/s in pressure lines.

B. Manifolds:

Specify the use of manifolded components.

C. Flexible Hose:

Specify flexible hose only in cases where motion or vibration make the use of rigid piping undesirable. The minimum ratio of burst pressure rating divided by design pressure in the line shall be 4.

D. Seals:

Specify all seals, including the ones installed inside hydraulic components, to be fully compatible with the hydraulic fluid being used and adequate for the maximum pressure and temperature operating at that point.

10.6.6 Instrumented Pins

Install an instrumented strain gage pin in the upper clevis joint on each cylinder. Specify instrumented pins for permanent installation if possible. Specify a safety factor of 2.0 and provide calculations indicating loads used for sizing the instrumented pins.

Commentary: The purpose of these pins is to evaluate the span balance performance, hydraulic system operation and timing, and to allow corrections where needed.

10.6.7 Miscellaneous Hydraulic Components

A. Receivers (Reservoirs):

Tanks in open-loop systems shall have a capacity greater than the maximum flow of three minutes operation of all pumps connected to the tank plus 10%, and/or the capacity of the total oil volume in the system. Tanks must have an adequate heat dissipation capacity to prevent temperatures above 77° C. Tanks in closed-loop hydrostatic systems shall circulate, filter, and cool enough oil to maintain a maximum oil temperature of 77° C. Specify suction port strainers with oil shut-off valves. Specify tanks with easy drainage and provided with adequate openings that allow easy cleaning of all surfaces from the inside. Specify sumps with magnetic traps to capture metal particles. Specify Stainless Steel ASTM A316L tank material. Specify the use of air bladders to avoid water contamination from air moisture condensation due to the breathing effect of the tank.

B. Filtration:

Design and specify a filtering system so that filters can be easily serviced and filter elements can be changed without disturbing the system. Do not specify valves that can be left accidentally closed. Strainers are allowed in the suction lines between the tank and the main pumps. Filters can be used if the system is designed to assure that there will be enough static head under all operating conditions at the pumps' inlets. Require absolute pressure (vacuum) sensors to stop the pumps if adequate suction head is not available at the pumps' inlets, and specify pressure line filters capable of at least 10 micron filtration between the pump outlet and the rest of the hydraulic system. The system shall have filters with relief-check, by-pass valve and visual clogged filter

indicators. Specify a remote sensing pressure switch to indicate a clogged filter. The relief-check, bypass-valve lines must also be filtered.

C. Hydraulic Fluids:

Hydraulic fluid must be approved by all the manufacturers of the major hydraulic components used in the bridge. Specify the use of bio-degradable hydraulic fluid.

10.7 Electrical

10.7.1 Electrical Service

Wherever possible, design bridge electrical service for 277/480 V, three-phase, "wye." Voltage drop from point of service to furthest load shall not exceed 5%. Do not apply a diversity factor when calculating loads.

10.7.2 Alternating Current Motors

Size and select motors per AASHTO requirements. On hydraulic systems provide 25% spare motor capacity. Motors shall comply with the following requirements:

A. Design Criteria For Start-Ups:

12 per hour, 2 per ten minute period.

B. Power Output, Locked Rotor Torque, Breakdown or Pullout Torque:

NEMA Design B Characteristics.

C. Design, Construction, Testing, and Performance:

ANSI/NEMA MG 1 for Design B Motors.

D. Testing Procedure:

ANSI/IEEE 112, Test Method B. Load test motors to determine freedom from electrical or mechanical defects and compliance with performance data.

E. Motor Frames:

NEMA Standard T-frames of steel or cast iron (no aluminum frames allowed) with end brackets of cast iron with steel inserts. Motors 7.5 KW and larger

shall be TEFC.

F. Thermistor System (Motor Sizes 19 KW and Larger):

Three PTC thermistors embedded in motor windings and epoxy encapsulated solid state control relay for wiring into motor starter.

G. Bearings:

Grease-lubricated, anti-friction ball bearings with housings equipped with plugged provision for relubrication, rated for minimum AFBMA 9, L-10 life of 20,000 hours. Calculate bearing load with NEMA minimum V-belt pulley with belt center line at end of NEMA standard shaft extension. Stamp bearing sizes on nameplate.

H. Sound Power Levels:

ANSI/NEMA MG 1.

I. Nominal Efficiency:

Meet or exceed values in ANSI Schedules at full load and rated voltage when tested in accordance with ANSI/IEEE 112.

J. Nominal Power Factor:

Meet or exceed values in ANSI Schedules at full load and rated voltage when tested in accordance with ANSI/IEEE 112.

K. Insulation System:

NEMA Class F or better.

L. Service Factor:

1.15 (except when calculating motor size, use 1.0).

10.7.3 Engine Generators

Design per the requirements of the latest edition of NFPA 110. Size generators so that one side of the channel (one side of the bridge) can be opened at a time concurrent with traffic lights, navigation lights, tender house air conditioners, and house lights.

Specify only diesel fueled generators. Provide fuel tank sized to hold enough fuel to run the generator, at 100% load, for 12 hours (minimum 190 L). Specify day tank with a minimum 28

L capacity. Submit calculations justifying generator size recommended.

10.7.4 Automatic Transfer Switch

Design switch in conformance with the requirements of the latest edition of NFPA 110.

Specify Automatic Transfer Switch with engine generator. The Automatic Transfer Switch shall be fully rated to protect all types of loads, inductive and resistive, from loss of continuity of power, without derating, either open or enclosed.

Specify withstand, closing, and interrupting ratings sufficient for voltage of the system and the available short circuit at the point of application on the drawings. Provide short circuit calculations to justify ATS proposed.

10.7.5 Electrical Control

Design an integrated control system. Develop a control interface that matches the operating needs and skill levels of the bridge tenders and maintenance personnel that will be using the system. Design a system configuration, select control devices, and program the Programmable Logic Controller (PLC) to produce the desired interface. Control system shall consist of a relay system (hard wired) and a PLC monitoring operation.

Design the bridge control system to be powered through an uninterruptible power supply.

EMERGENCY-STOP (E-STOP) stops **all** machinery in the quickest possible time but in no less than 3.0 seconds. In an emergency, hit this button to stop machinery and prevent damage or injury. The E-STOP button is reset by twisting clockwise (or counterclockwise) to release to normal up position.

At a minimum, provide alarms for the following events:

- All bridge control failures.
- All generator/Automatic Transfer Switch failures.
- All traffic signal failures.
- All navigation light failures.
- All traffic gate failures.
- All span-lock failures.
- All brake failures (if applicable).

- All leaf limit switch failures.
- All drive failures; including motor high temperature (motors larger than 19 KW) and all hydraulic system failures.
- Near and far-leaf total openings (not an alarm but part of the monitoring function).
- All uses of bypass functions, type and time (not an alarm but part of the monitoring function).

10.7.6 Motor Controls

Specify full-size NEMA rated starters. Do not use IEC starters unless space constraints require their use, and then, only by obtaining prior approval from the Mechanical/Electrical Section of the SDO.

Panelboards and transformers should not be included in the Motor Control Center (MCC) unless space constraints require it, and then, only by obtaining prior approval from the Mechanical/Electrical Section of the SDO.

10.7.7 Programmable Logic Controllers

Refer to the "Technical Special Provisions" issued by the Mechanical/Electrical Section of the SDO.

10.7.8 Limit and Seating Switches

Limit switches shall be labeled as indicated in the "Technical Special Provisions" issued by the Mechanical/Electrical Section of the SDO. Do not use electronic limit switches.

Design each movable leaf with FULL-CLOSED, NEAR-CLOSED, NEAR-OPEN, FULL-OPEN, and FULL-SEATED limit switches. Specify NEMA 4, corrosion resistant metallic housings which have a high degree of electrical noise immunity and a wide operating range. Specify that NEAR-OPEN and NEAR-CLOSED limit switches be mounted, initially, approximately eight (8) degrees from FULL-OPEN and FULL-CLOSED, respectively. Final adjustment of NEAR-OPEN and NEAR-CLOSED will depend upon bridge configuration, drive machinery, and bridge operation.

Commentary: The FULL-CLOSED switch controls the drive stop and the FULL-SEATED switch is the safety interlock to allow driving the locks.

Do not connect limit switches in series. Each limit switch will be connected to a relay coil or

a PLC input. Provide position encoder (potentiometer or other type) to drive leaf position indicators on control console. The position encoder will also provide a signal to the PLC to use as a reference to determine leaf limit switch failure. Connect limit switches in the following configurations:

- Traffic Gates: End-Of-Travel configuration.
- Span Locks: End-Of-Travel configuration.
- Leaf(s): End-Of-Travel configuration.
- Safety Interlocks: Indicating configuration.

10.7.9 Safety Interlocking

A. Traffic Lights:

Traffic gates LOWER is not enabled until traffic lights RED. Provide bypass capability labeled TRAFFIC LIGHT BYPASS to allow traffic gates LOWER without traffic lights RED.

B. Traffic Gates:

1. Bridge Opening:

Span locks PULL is not enabled until **all** traffic gates are fully down (or TRAFFIC GATE BYPASS has been engaged).

2. Bridge Closing:

Traffic lights GREEN is not enabled until **all** traffic gates are fully raised (or TRAFFIC GATE BYPASS has been engaged).

3. Provide bypass capability labeled TRAFFIC GATE BYPASS to allow span lock PULL without **all** traffic gates LOWERED or traffic lights GREEN without **all** traffic gates RAISED.

C. Span Locks:

1. Bridge Opening:

Leaf RAISE is not enabled until **all** span locks are fully pulled (or SPAN LOCK BYPASS has been engaged).

2. Bridge Closing:

Traffic gate RAISE is not enabled until **all** span locks are fully driven (or SPAN LOCK BYPASS has been engaged).

3. Provide bypass capability and label SPAN LOCK BYPASS to allow leaf RAISE without **all** span locks pulled or traffic gate RAISE without **all** span locks DRIVEN.

D. Leaf(s):

1. Span locks DRIVE is not enabled until leaf(s) is(are) FULLY SEATED (as indicated by the FULLY SEATED switch).
2. Provide bypass capability and label LEAF BYPASS to allow span lock DRIVE without leaf(s) FULLY SEATED.

10.7.10 Instruments

Provide wattmeter for each drive (pump) motor and provide leaf position indication for each leaf.

10.7.11 Control Console

The Control Console shall contain the necessary switches and indicators to perform semi-automatic and manual operations as required by the standard FDOT "Basic Sequence Diagram." All wiring entering or leaving the Control Console shall be broken across approved terminals.

No components other than push-buttons, selector switches, indicating lights, terminal blocks, etc., shall be allowed in the Control Console. Refer to the "Technical Special Provisions" issued by the Mechanical/Electrical Section of the SDO.

10.7.12 Electrical Connections between Fixed and Moving Parts

Use extra flexible wire or cable.

10.7.13 Electrical Connections across the Navigable Channel

Specify a submarine cable assembly consisting of the following three separate cables:

- Power Cable: Jacketed and armored with twelve (12) #4 AWG copper and twelve (12) #10 AWG copper conductors.

- Signal and Control Cable: Jacketed and armored with fifty (50) #12 AWG copper and five (5) pairs of twisted shielded #14 AWG copper conductors.
- Bonding Cable: A single #4/0 AWG copper conductor.

Determine the total number of conductors required of each size and the number of runs of each type of cable that is required for the project. Use only multiples of the standard cables listed above. Allow for at least 25% spare conductors for each size.

Specify quick-disconnect type terminals for terminating conductors. Specify terminals that isolate wires from a circuit by a removable bridge or other similar means. Specify NEMA 4X enclosures for all terminals.

Design all above the water line, vertical runs of cable with supports at every five feet. Specify and detail a protective sleeve (schedule 120 PVC) around the cable from a point 1.5 m below the mean low tide to a point 1.5 m above mean high tide. Specify a water-tight seal at both ends of the sleeve.

10.7.14 Service Lights

Provide minimum of 300 Lux in all areas of the machinery platform.

10.7.15 Navigation Lights

Design a complete navigation light and aids system in accordance with all local and federal requirements and Standard Drawings Nos. 510 and No. 511. Comply with the latest edition of the Code of Federal Regulations (CFR) 33, Part 118, and Coast Guard Requirements.

10.7.16 Grounding and Lightning Protection

Provide the following systems:

A. Lightning Protection System:

Design per the requirements of NFPA 780 Lightning Protection Code. The bridge shall be protected with Class II materials.

B. Surge Suppression System:

Design Transient Voltage Surge Suppressor (TVSS) system to protect all power, control, signaling, and communication circuits and all submarine conductors that enter or leave the control house. **It is imperative to maintain proper segregation of protected and non-protected wiring within the**

Bridge Tender House. Refer to the "Technical Special Provisions" issued by the Mechanical/Electrical Section of the SDO.

C. Grounding and Bonding System:

All equipment installed on the bridge/project shall be "bonded" together by means of a copper bonding conductor which runs the entire length of the project (Traffic Light to Traffic Light). All metal bridge components (i.e., handrail, roadway light poles, traffic gate housings, leafs, etc.) will be connected to the "copper bonding conductor." The "copper bonding conductor" shall remain continuous across the channel by means of the submarine bonding cable.

Require earth grounds at regular intervals with no less than two driven grounds at each pier and one driven ground at each overhead traffic light structure and traffic gate. All driven grounds shall be tested to a maximum of 5 ohms to ground.

All materials shall be corrosion resistant materials such as monel, silicon bronze, or stainless steel, when in water, and all "main connections" to the "copper bonding conductor" shall be cadwelded.

In areas where the "copper bonding conductor" is accessible to non-authorized personnel, it must be enclosed in Schedule 80 PVC conduit with stainless steel supports every 1.5 m.

10.7.17 Movable Bridge Traffic Signals and Safety Gates

Refer to Roadway Standard Index No. 17890 for Traffic Control Devices for Movable Span Bridge Signals.

10.7.18 Communications Systems

Design and specify a Public Address System, an Intercom System, and a Marine Radio System for each movable bridge. The three systems shall work independent of each other and shall meet the following criteria:

A. Public Address System:

One way hand-set communication from the operators console to three (3) zones (marine channel, roadway, and machinery platforms and other rooms). Specify an "all call" feature so that the operator may call all three zones at once. Specify and detail loudspeakers mounted on the pier wall facing in both directions of the channel, one loudspeaker mounted at each overhead traffic signal, facing the oncoming gate, and loudspeakers at opposite ends of the machinery platform.

B. Intercom System:

Two way communication system which will work similar to an office telephone system with "station-to-station" calling from any station on the system and "all call" to all stations on the system from the main intercom panel. Each station shall have a "hands free" capability. A call initiated from one station to another shall open a channel and give a tone at the receiving end. The receiving party shall have the capability of answering the call by speaking into the open speaker channel, or by picking up the local receiver and speaking into it. All intercom equipment shall be capable of operation in a high noise, salt air environment. A handset shall be mounted adjacent to the operator's console, in each room on the bridge and on each machinery platform.

C. Marine Radio System:

Hand held, portable, operable on or off the charger, tuned to the proper channels, and a 110 volt charger located adjacent to the operator's console.

10.7.19 Functional Checkout

Develop and specify an outline for performing system checkout of all mechanical/electrical components to insure contract compliance and proper operation. Specify in-depth testing to be performed by the Contractor.

Functional testing for the electrical control system shall consist of two parts. The first part shall be performed before delivery, and the second part shall be performed after installation on the bridge. Both tests shall be comprehensive. The off-site functional testing shall be performed to verify that all equipment is functioning as intended.

All repairs or adjustments shall be made before installation on Department property. All major electrical controls shall be assembled and tested in one place, at one time. The test shall include as a minimum: control console, PLC, relay back-up system, Motor Control Center, and all other equipment required, in the opinion of the Electrical Engineer of Record, to complete the testing to the satisfaction of the Mechanical/Electrical Section of the SDO.

If not satisfactory, the testing shall be repeated at no cost to the Department. All equipment shall be assembled and inter-connected (as they would be on the bridge) to simulate bridge operation. No inputs or outputs shall be forced. Indication lights shall be provided to show operation and hand operated toggle switches may be used to simulate field limit switches.

After the off-site testing is completed to the satisfaction of the Mechanical/Electrical Section of the SDO, the equipment may be delivered and installed. The entire bridge control system shall be re-tested before the bridge is put into service. The field functional testing shall include, but is not necessarily limited to, the off-site testing procedure.

As a minimum, the following tests of Control Functions shall be performed for both manual and semi-automatic operations:

Commentary: The Electrical Engineer of Record is encouraged to include tests for other equipment not included in the minimum tests listed below.

- A. Demonstrate the correct operation of the bridge sequence as described in the "Technical Special Provisions" and in the drawings.
- B. Demonstrate **EMERGENCY STOP** of each span (leaf) at, or during, each phase of opening and closing the bridge (phases include ramping up or down, full-speed, and creep-speed).
- C. Demonstrate **EMERGENCY STOP** does prevent energization of all rotating machinery in any mode of operation.
- D. Demonstrate that the leaf(s) do not come to a sudden stop on a power failure.
- E. Interlocks:
 - 1. Simulate the operation of each limit switch to demonstrate correct operation and interlocking of systems.
 - 2. Demonstrate **BYPASS** operation for each failure for each required bypass.
 - 3. Simulate each failure for which there is an alarm message to demonstrate correct message displays.
 - 4. Testing of interlocks shall be sufficient to demonstrate that unsafe or out of sequence operations are prevented.
- F. Observe Position Indicator readings with bridge closed and full open to assure correct readings.
- G. Navigation Lights:
 - 1. Demonstrate that all lamps are working.
 - 2. Demonstrate the operation of the transfer relays and indicators for each light.
 - 3. Demonstrate proper change of channel lights from red to green.
 - 4. Demonstrate Battery Backup by simulating a power outage.

H. Traffic Gates:

1. Demonstrate proper operation of each gate arm.
2. Demonstrate opening or closing times. Time should not exceed 15 seconds in either direction.
3. Demonstrate that gate arms are perpendicular to the roadway when RAISED and parallel to the roadway when LOWERED.

I. Span Locks:

1. Operate each span lock through one complete cycle and record, with chart recorder, motor power (watts) throughout the operation, record lockbar to guide and lockbar to receiver, clearances.
2. Demonstrate pulling and driving times. Time should not exceed 10 seconds in either direction.
3. Operate each lock with hand crank or manual pump for one complete cycle.

J. Emergency Power:

1. The complete installation shall be initially started and checked-out for operational compliance by a factory-trained representative of the manufacturer of the generator set and the Automatic Transfer Switch. The supplier of the generator set shall provide the engine lubrication oil and antifreeze recommended by the manufacturer for operation under the environmental conditions specified.
2. Upon completion of initial start-up and system checkout, the supplier of the generator set shall notify the Engineer in advance and perform a field test to demonstrate load-carrying capability, stability, voltage, and frequency.
3. A dielectric absorption test shall be made on generator winding with respect to ground. A polarization index shall be determined and recorded. Submit copies of test results to the Engineer.
4. Phase rotation test shall be made to determine compatibility with load requirements.
5. Engine shut-down features such as low oil pressure, over-temperature, over-speed, over-crank, and any other feature as applicable shall be function-tested.

6. In the presence of the Engineer, perform resistive load bank tests at one hundred percent (100%) nameplate rating. Loading shall be 25%-rated for 30 minutes, 50%-rated for 30 minutes, 75%-rated for 30 minutes, and 100%-rated for 2 hours. Records shall be maintained throughout this period to record water temperature, oil pressure, ambient air temperature, voltage, current, frequency, kilowatts, and power factor. The above data shall be recorded at 15 minute intervals throughout the test.

K. Automatic Transfer Switch:

Perform automatic transfer by simulating loss of normal power and return to normal power. Monitor and verify correct operation and timing of: normal voltage sensing relays, engine start sequence, time delay upon transfer, alternate voltage sensing relays, automatic transfer operation, interlocks and limit switch function, timing delay and retransfer upon normal power restoration, and engine shut-down feature.

L. Programmable Logic Controller (PLC) Program:

Demonstrate the completed program's capability prior to installation or connection of the system to the bridge. Arrangements and scheduling for the demonstration shall be coordinated with the Engineer and the Electrical Engineer of Record.

A detailed field test procedure shall be written and provided to the Electrical Engineer of Record for approval. It shall provide for testing as listed below:

- Exercise all remote limit switches to simulate faults including locks, gates, traffic lights, etc. Readouts shall appear on the Alpha-Numeric display.
- When the local testing of all individual remote components is completed, check all individual manual override selections for proper operation at the console. When all override selections have been satisfactorily checked-out, switch the system into semi-automatic (PLC) mode and exercise for a full raise and lower cycle. Verify that operation is as diagrammed on the plan sheet for the sequence of events.
- Initiate a PLC sequence of operation interweaving the by-pass functions with the semi-automatic functions for all remote equipment.
- Remove the power from the input utility lines. The Automatic Transfer Switch (ATS) should activate the engine-generator to supply power. Raise and lower the bridge again. The bascule leafs should operate

in sequence; i.e, two adjacent bascule leafs at a time. Upon completion of the test, re-apply utility power to ATS. The load should switch over to utility for normal operation.

- Certify that all safety features are included in the program, and that the program will not accept commands that are contrary to the basic sequence diagram. Submit failure mode testing as part of the written field test procedure.

10.8 Tender House Architectural Design

These guidelines are intended primarily for use in the design of new control houses, but many items apply to renovations of existing houses. In general, comply with the requirements of the Southern Building Code; except in Dade and Broward Counties comply with the South Florida Building Code.

Do **not** design sealed, soundproof houses such as pipe column and curtain wall "control towers."

Commentary: The operator must be able to hear all traffic as well as see it.

Avoid the over use of windows and glazing. Where sight considerations permit, solid, insulated walls shall be used. Provide 1.0 m roof overhangs.

Commentary: Heat gain can be a problem.

The preferred wall construction is reinforced concrete of 150 mm minimum thickness and with architectural treatment such as fluted corner pilasters, frieze ornamentation, arches, horizontal banding, or other relief. Finish exterior of house with stucco, FDOT Class V Coating, or spray-on granite or cast stone.

Design the Bridge Tender's Room with a minimum of 19 m² of usable floor space. Add additional floor area for interior stairwells.

Commentary: This allows enough room for a toilet, kitchenette, and coat/mop closet as well as wall-hung desk and control console.

Window sills shall be not more than 864 mm from the floor.

Commentary: This allows for operator vision when seated in a standard task chair.

For the condition of an operator standing at the control console, verify sight lines to:

1. Traffic gates for both directions of automobile traffic.

2. Marine traffic for both directions of the navigable channel.
3. Locations where pedestrians will normally stop.

10.8.1 Plumbing

- A. Ensure that the specifications call for a licensed plumber to install all water, waste, and vent piping.

- B. Pipes and Fittings:

Specify pipe fittings, valves, and corporation stops, etc. Provide an ice-maker supply line and a hose bib outside the house. Adjacent to the hose bib, provide a wall-mounted, corrosion-resistant (fiberglass or plastic) hose hanger and 15 m nylon-reinforced, 19 mm garden hose.

- C. Plumbing fixtures:

Specify a double bowl, enameled, cast-iron, self-rimming kitchen counter sink; a sink faucet; a lavatory; a lavatory faucet; and an elongated toilet. Do not specify low-flow fixtures unless the bridge has a marine-digester system.

- D. Site Water Lines:

Specify pipe and fittings for site water lines including domestic water line, valves, fire hydrants, and domestic water hydrants. Size to provide adequate pressure, and detail drawings as necessary to show location and extent of work. Specify disinfection of the potable water distribution system and all water lines in accordance with the requirements of the American Water Works Association (AWWA).

- E. Site Sanitary Sewage System:

Gravity lines to manholes are preferred; avoid the use of lift stations whenever possible. However, if lift stations are required, special consideration must be given to daily flows as well as pump cycle times. Do not use ultra low-flow toilets except as permitted hereinafter in this article. Low daily flows result in long cycle times and associated odor problems. Include pump and flow calculations and assumptions in the 60% Plans submittal.

For bridges that are not served by a local utility company, where connection is prohibitively expensive, or where septic tanks are not permitted or practical, ultra-low flow toilets (compressor assisted) and U. S. Coast Guard-approved marine sanitation devices are acceptable.

10.8.2 Stairs, Landings, Grating, and Ladders

Stairs shall comply fully with OSHA requirements. Stairs shall be at least 1.0 m wide and shall comply with applicable codes in regard to riser and tread dimensions. The use skid-resistant, open grating for treads, landings, and floor gratings is preferred.

Stairs and landings may be on the exterior of the house. Spiral stairs are acceptable for interior stairwells but are not preferred. Special attention must be paid to clearances for moving equipment into or out of a Control House.

Commentary: This reduces heating and cooling requirements and provides more usable floor space.

Design the stair assembly to support a live load of 4.8 kN/m² with the stringer deflection limited to 1/180 of span. Include design calculations in the 60% Plans submittal.

Avoid the use of ladders or stair-ladders whenever possible. However, in situations where stairs cannot be accommodated, stair-ladders should be used. Vertical ladders should be used only as a last resort and, even then, only for very limited heights.

10.8.3 Handrails and Railing

Use standard I.P.S.-size, Schedule 40, 38.1 mm diameter steel or aluminum pipe with corrosion-resistant, slip-on structural fittings that permit easy field installation and removal. Welded tube rails are not preferred.

Design railing assembly, wall rails, and attachments to resist a lateral force of 730 N/m without damage or permanent set. Include design calculations in the 60% Plans submittal.

10.8.4 Framing, Sheathing, and Finish Carpentry

Include a specification section, if necessary, for the following items.

- Structural floor, wall, and roof framing.
- Built-up structural beams and columns.
- Diaphragm trusses fabricated on site.
- Floor, wall, and roof sheathing.
- Sill gaskets and flashings.
- Preservative treatment of wood.

- Fire-retardant treatment of wood.
- Miscellaneous framing and sheathing.
- Telephone and electrical panel back boards.
- Concealed wood blocking for supporting toilet and bath accessories, wall cabinets, and wood trim.
- Finish Carpentry.

10.8.5 Desktop and Cabinet

Specify and detail a wall-hung desktop with drawer. Show desktop mounted 750 mm from finished floor. Specify and detail a minimum length of 2.1 m of 1.1 m base cabinets and a minimum length of 2.1 m of 610 mm or 914 mm wall cabinets.

Specify cabinet hardware and solid-surfacing material counter-tops and desktop.

10.8.6 Insulation

A. "R-Values:"

Design the Control House with insulation to meet the following minimum requirements:

- Walls: R11.
- Ceiling: R30.

B. Insulation Materials:

Board Insulation may be used at underside of floor slabs, exterior walls, and between floors separating conditioned and unconditioned spaces. Batt insulation may be used in ceiling construction and for filling perimeter window and door shim spaces, crevices in exterior wall, and for the roof/ceiling.

10.8.7 Fire-stopping

Design and detail fire-stops at all wall and floor penetrations according to the following ratings:

- Main floor fire walls: 1 hour

- Stair walls: 2 hours
- Room-to-room partitions: 3/4 hour

10.8.8 Roof

Use a "hip-roof" with a 6 in 12 pitch, or steeper, and with 1.0 meter eaves. Specify and detail either standing-seam, 18 gauge metal or glazed clay tiles. Avoid flat roofs, shed roofs, or any other configuration utilizing a "built-up" roofing system.

Soffits shall be ventilated aluminum or vinyl. Fascia material shall be aluminum, vinyl, or stucco.

Design for hurricane force uplift (as per 50 year wind speed recurrence, Table 29.1, PPM) on roof, roof framing, decking, anchors, and other components. Include design calculations in the 60% Plans submittal. During design, properly use and detail underlayment, eave and ridge protection, nailers, and associated metal flashing.

Commentary: Some manufacturers may not warrant metal roof systems in coastal environments. Before specifying a manufacturer's metal roof, determine if the manufacturer will warrant the roof in the project's environment. If not, use tiles meeting or exceeding the Grade 1 requirements of ASTM C1167.

10.8.9 Doors And Hardware

A. Exterior Doors:

Specify and detail armored aluminum entry doors ballistics-rated for 0.357 Magnum fired at point blank range.

B. Interior Doors:

- Passage: Solid-core or solid wood.
- Closets: Pine, louvered.

C. Hardware:

Specify corrosion resistant, heavy-duty, commercial ball-bearing hinges, levered locksets, and dead-bolts for entry doors. Specify adjustable thresholds, weatherstripping, seals, and door gaskets. Select and specify interior locksets. Require all locks to be keyed alike, and require spare keys.

10.8.10 Windows

Windows shall comply with the American Architectural Manufacturers Association (AAMA) standards for heavy commercial windows. Windows shall be double-hung, marine-glazed, heavy commercial (DHHC) type, extruded aluminum windows. Lites shall be ballistics-rated for 0.357 Magnum fired at point blank range. Windows shall be counter-balanced to provide 60% lift assistance. Solar film in a light tint is acceptable if compatible with glazing. Require perimeter sealant and specify operating hardware and insect screens.

Windows shall comply with ASTM E330 for 2.87 kPa exterior uniform load and 2.87 kPa interior load applied for 10 seconds without glass breakage and with no permanent damage to fasteners, hardware parts, actuating mechanisms, or any other damage.

Air leakage shall be no more than $0.002 \text{ m}^3/\text{s}/\text{m}^2$ of wall area at a reference differential pressure across assembly of 75 Pa when measured in accordance with ASTM E283.

No water leakage when measured in accordance with ASTM E331 with a test pressure of 287 Pa applied at 20.0 Liters per hour per square meter.

10.8.11 Interior Walls

Veneer Plaster shall be 6.4 mm plaster veneer over 12.7 mm gypsum "Blueboard," masonry, and/or concrete surfaces.

Gypsum Board shall be 12.7 mm "Blueboard" for veneer plaster. Require 12.7 mm fiberglass reinforced cement backer board for tile. All gypsum board shall have taped and sanded joint treatment and smooth texture finish.

10.8.12 Tile

Floor shall be non-skid quarry tile on tender's level. Do **not** use vinyl floor tiles or sheet goods.

Wall tile, if used, obtain Mechanical/Electrical Section of the SDO approval of wall tile selection.

10.8.13 Epoxy Flooring

Specify fluid applied non-slip epoxy flooring for electrical rooms, machinery rooms and machinery platforms. Ensure that the specified product is warranted by the manufacturer for the product as well as the installation.

10.8.14 Painting

Select and specify paint for woodwork and walls. Do not specify painted floors.

10.8.15 Wall Louvers

Design Wall Louvers with minimum 40% free area to permit passage of air at a velocity of 160 L/sec without blade vibration or noise and with a maximum static pressure loss of 6 mm measured at 175 L/sec. Use rainproof intake and exhaust louvers, sized to provide the required free area.

10.8.16 Toilet and Bath Accessories

Specify a mirror, soap dispenser, tissue holder, paper towel dispenser, and a waste paper basket for each bathroom. Provide each bathroom with an exhaust fan.

10.8.17 Equipment, Appliances, and Furnishings

Provide a shelf-mounted or built-in 0.0425 m³ microwave oven with digital keypad and users manual. Detail for and specify an under-counter, frost-free ice-maker refrigerator with user's manual. Specify a fire extinguisher for each room. Specify two gas-lift, front-tilt task chairs. Provide one 1 m x 1.5 m cork bulletin board. Specify blinds or shades for window treatment.

10.8.18 Heating, Ventilating, and Air-Conditioning (HVAC)

Use central, split-unit heat pump. If required for rehabilitation projects, multiple, packaged units may be acceptable if properly justified. Design HVAC system with indoor air-handler, duct-work, and out-door unit(s). Perform load calculations and design the system accordingly. Include load calculations in the 60% Plans submittal. For highly-corrosive environments, use corrosive resistant equipment.

When permitted by the Mechanical/Electrical Section of the SDO, specify packaged-terminal heat pump units. Specify controls for all HVAC equipment. Specify and show ceiling fans on floor plan.

10.8.19 Interior Luminaires

Avoid the use of heat producing fixtures. Specify fixtures under Section 508 of the "Technical Special Provisions" issued by the Mechanical/Electrical Section of the SDO.

10.9 Maintainability

10.9.1 General

The following guidelines for maintainability of movable bridges are applicable to both new bridges and to rehabilitation plans for existing bridges on which construction has not been initiated. Variations from these practices for the rehabilitation of existing bridges may be authorized by the Mechanical/Electrical Section of the SDO, but only by approval in writing.

10.9.2 Trunnion Bearings

Trunnion bearings shall be designed so that replacement of bushings can be accomplished with the span jacked 13 mm and in a horizontal position. Suitable jacking holes or puller grooves are to be provided in bushings to permit extraction. Jacking holes shall utilize standard bolts pushing against the housing which supports the bushing.

Trunnion bushings and housings shall be of a split configuration. The bearing cap and upper-half bushing (if an upper-half bushing is required) shall be removable without span-jacking or removal of other components.

10.9.3 Span-Jacking of New Bridges

Stationary stabilizing connector points are located on the bascule pier. These points provide a stationary support for stabilizing the span, by connection to the span stabilizing connector points. One set of span-jacking surfaces shall be located under the trunnions (normally, this will be on the bottom surface of the bascule girder). A second set shall be located on the lower surface at the rear end of the counterweight.

Span stabilizing connector points shall be located on the bascule girder forward and back of the span jacking surfaces. The stationary stabilizing connector point (forward) shall be in the region of the Live Load Shoe. Stationary stabilizing connector points (rear) are to be located on the cross girder support at the rear of the bascule pier. Connector points shall be designed to attach stabilizing structural steel components.

Commentary: The stationary jacking surface shall be positioned at an elevation as high as practical so that standard hydraulic jacks can be installed.

The following definitions of terms used above describe elements of the span-jacking system:

A. Span-jacking Surface:

An area located under the trunnion on the bottom surface of the bascule girder.

B. Span Stabilizing Connector Point (forward):

An area adjacent to the live load shoe point of impact on the bottom surface of the bascule girder.

C. Span Stabilizing Connector Point (rear):

An area at the rear end of the counterweight on the lower surface of the counterweight girder. (NOTE: For bascule bridges having tail locks, the span stabilizing connector point may be located on the bottom surface of the lockbar receiver located in the counterweight.)

D. Stationary Jacking Surface:

The surface located on the bascule pier under the span jacking surfaces. The stationary jacking surface provides an area against which to jack in lifting the span.

10.9.4 Trunnion Alignment Features

Center holes shall be installed in trunnions to allow measurement and inspection of trunnion alignment. Span structural components shall not interfere with complete visibility through the trunnion center holes. Trunnions shall be individually adjustable for alignment.

A permanent walkway or ladder with work platform shall be installed to permit inspection of trunnion alignment.

10.9.5 Lock Systems

Span locks are to be accessible from the bridge sidewalk through a suitable hatch or access door. A work platform suitable for servicing of the lockbars shall be provided under the deck and in the region around the span locks.

Lock systems shall be designed so that an individual lock may be disabled for maintenance or replacement without interfering with the operation of any of the other lockbars on the bascule leaf.

Tail locks, when required, shall be designed so that the lockbar mechanism is accessible for repair without raising the leaf. The lockbar drive mechanism shall be accessible from a permanently installed platform within the bridge structure.

Lockbar clearances shall be adjustable for wear compensation.

10.9.6 Machinery Drive Systems

All machinery drive assemblies shall be individually removable from the drive system without removal of other major components of the drive system.

Commentary: For example, a speed-reducer assembly shall be removable by breaking flexible couplings at the power input and output ends of the speed-reducer.

10.9.7 Lubrication Provisions

Bridge system components requiring lubrication shall be accessible without use of temporary ladders or platforms. Permanent walkways and stairwells shall be installed to permit free access to regions requiring lubrication. Lubrication fittings shall be visible, clearly marked and easily reached by maintenance personnel.

If specified by the Mechanical/Electrical Section of the SDO, automatic lubrication systems shall be provided for bearings and gears. Designs for automatic lubrication systems shall provide for storage of not less than three (3) months supply of lubricant without refilling. Refill shall be accomplished within a period of 15 minutes through a vandal-proof connection box located on the bridge sidewalk, clear of the roadway. Blockage of one traffic lane during this period is permitted.

10.9.8 Drive System Bushings

All bearing housings and bushings in open machinery drive and lock systems shall utilize split-bearing housings and bushings and shall be individually removable and replaceable without affecting adjacent assemblies.

10.9.9 Local Switching

"Hand-Off-Automatic" switching capability shall be provided for maintenance operations on traffic gate controllers and brakes and motors for center and tail-lock systems.

"On-Off" switching capability shall be provided for maintenance operations on span motor and machinery brakes, motor controller panels, and span motors.

Remote switches shall be lockable for security against vandalism.

10.9.10 Service Accessibility

A service area not less than 750 mm wide shall be provided around system drive components.

10.9.11 Service Lighting and Receptacles

Machinery and electrical rooms shall be lighted as necessary to assure adequate lighting for maintenance of equipment, but with a minimum lighting level of 300 Lux. Switching shall be provided so that personnel may obtain adequate lighting without leaving the work area for switching. Master-switching shall be provided from the control tower.

Each work area shall be provided with receptacles for supplementary lighting and power tools such as drills, soldering and welding equipment.

10.9.12 Communications

Permanent communications equipment shall be provided between the control tower and areas requiring routine maintenance (machinery drive areas, power and control panels locations traffic gates and waterway).

10.9.13 Wiring Diagrams

Wiring diagrams shall be provided for each electrical panel inside the panel door. Diagrams shall be enclosed in glass or plastic of optical quality.

10.9.14 Diagnostic Reference Guide for Maintenance

Diagnostic instrumentation and system fault displays shall be installed for mechanical and electrical systems. Malfunction information shall be presented on a control system monitor located in the bridge control house. Data shall be automatically recorded. System descriptive information, such as ladder diagrams and wiring data, shall be available on the system memory to enable corrective actions on system malfunctions and to identify areas requiring preventative maintenance.

10.9.15 Navigation Lights

Dual lamps and transfer relays shall be installed on fenders and center of channel positions to reduce effort required for maintenance of navigation lights.

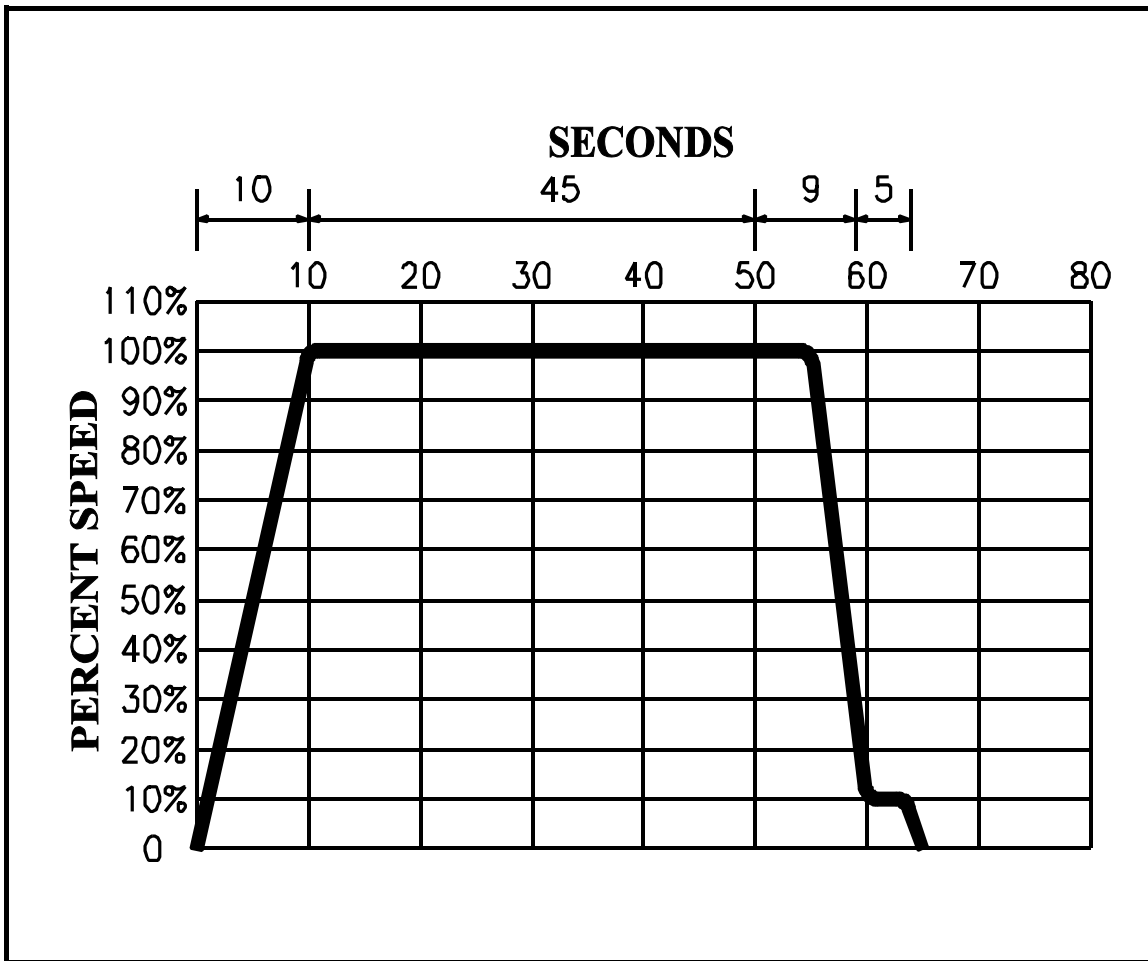
10.9.16 Working Conditions for Improved Maintainability

When specified by the Department, for either new or rehabilitated bascule bridge designs, use enclosed machinery and electrical equipment areas. Areas containing electronic equipment

shall be air-conditioned to protect the equipment as required by the equipment manufacturer and the Mechanical/Electrical Section of the SDO.

10.9.17 Weatherproofing

New and rehabilitated bascule bridge designs shall incorporate details to prevent water drainage and sand deposition into machinery areas. Avoid details that trap dirt and water; provide drain holes, partial enclosures, sloped floors, etc., to minimize trapping of water and soil.



Speed Ramp

Figure 10-1

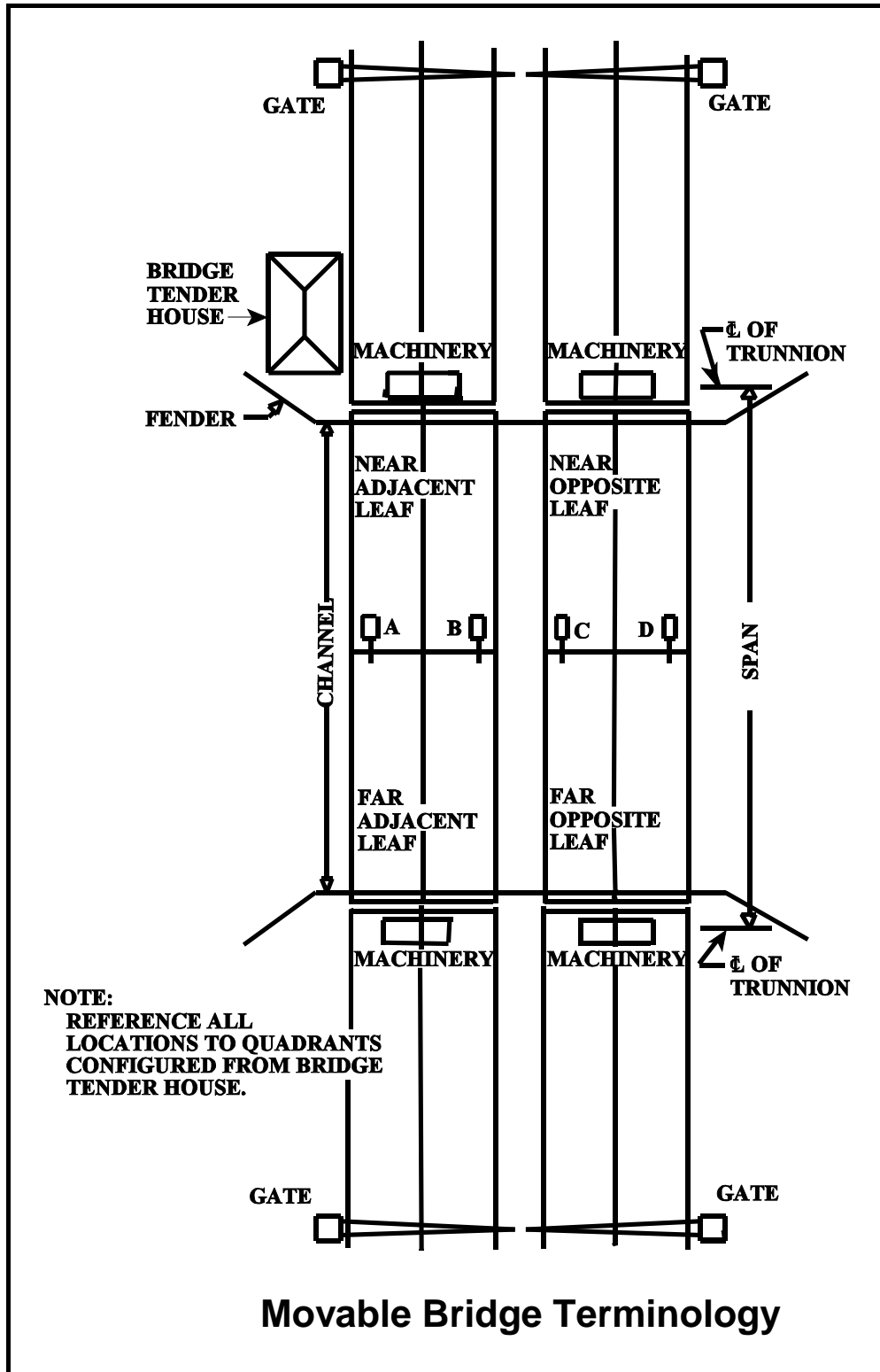
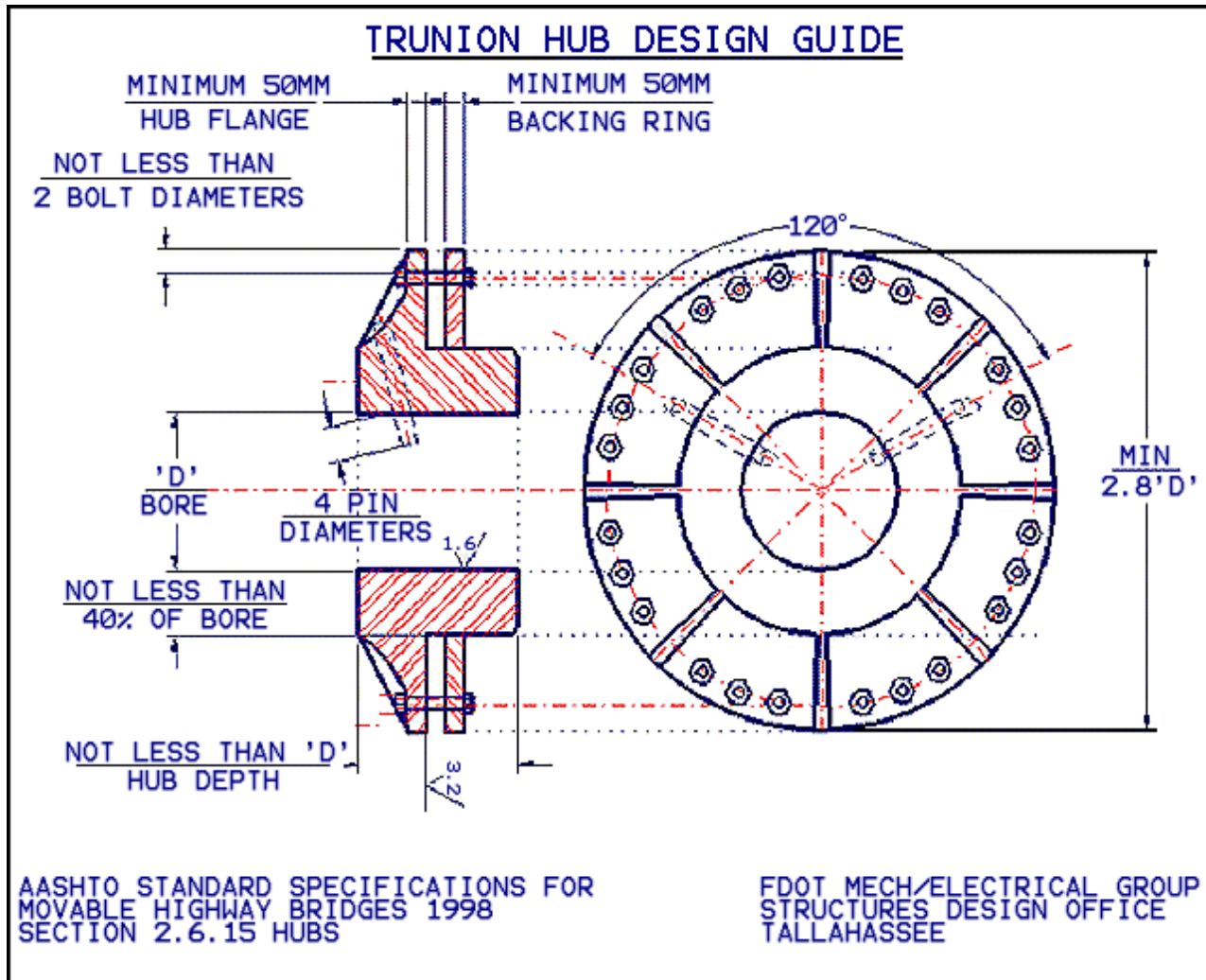


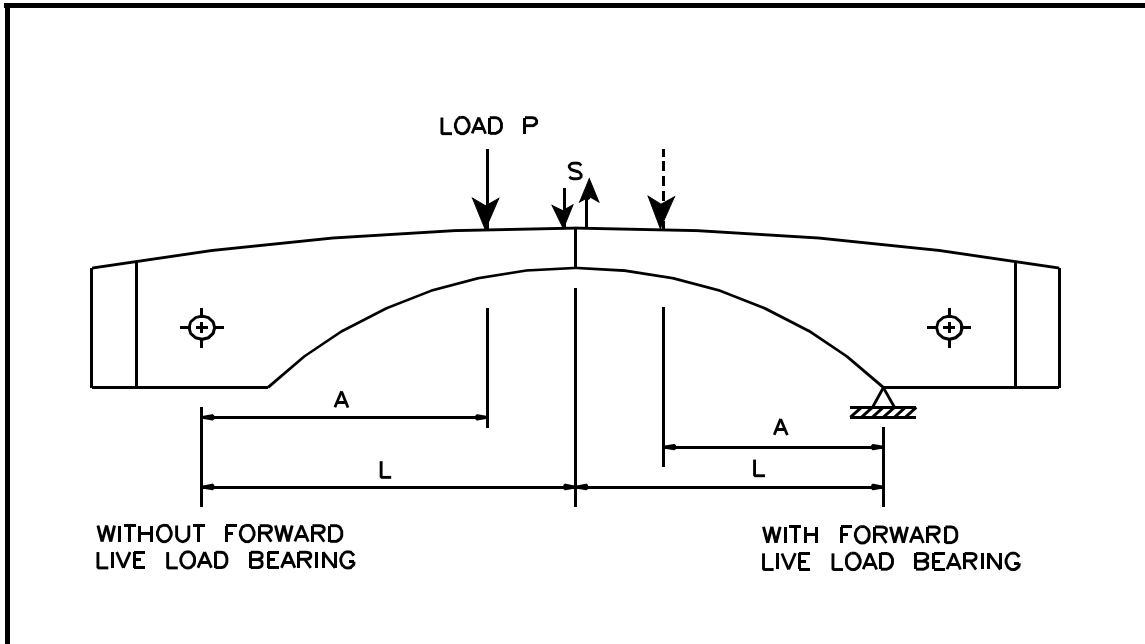
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Trunnion Hubs

(See "Trunnion and Trunnion Bearings" under "Machinery Systems" this Chapter)

Figure 10-3



Lock Design Criteria

Figure 10-4

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STRUCTURES DESIGN GUIDELINES

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STRUCTURES DESIGN GUIDELINES

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