

STRUCTURES DESIGN GUIDELINES FOR LOAD FACTOR DESIGN

for use with

AASHTO Standard Specifications for Highway Bridges

Topic No.: 625-020-152-a
Effective: January 1, 2000

February 25, 2000

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

TO: All FDOT and Consultant Designers Performing Bridge and/or Structures Design for the Florida Department of Transportation

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

COPIES: R. E. Nichols, Freddie Simmons

SUBJECT: ***Structures Design Guidelines (LFD)*** for Load Factor Design for use with ***AASHTO Standard Specifications for Highway Bridges*** (Topic No.: 625-020-152-a)
Effective: January 1, 2000

By receipt of this letter, please be advised that the ***Structures Design Guidelines (LFD)*** for Load Factor Design in SI Units for Use with ***AASHTO Standard Specifications for Highway Bridges*** (Topic No.: 625-020-152-a) is currently available. This document is a complete reprint of the former version of the document that was effective July 1, 1997 (Topic No.: 625-020-150-b).

The following specific administrative and technical modifications are particularly noteworthy:

Chapter 1, 2, 3, 6, 13, 14, 17, 19, 20, and 21:

1. These ten chapters have been removed from the ***Structures Design Guidelines*** and now reside in the ***Plans Preparation Manual*** (Topic No.: 625-00-005) with the exception of Chapter 17 which now resides on the Structures Design Office's homepage at <http://www.dot.state.fl.us/structures>.

Chapter 4 - Structural Loadings and Material Properties:

1. Table 4.2 "Miscellaneous Loads" has been expanded.
2. Guidance for applying the future wearing surface allowance in concert with profilograph requirements is provided.

Chapter 5 - Foundations and Geotechnical Data:

1. Article 5.6 clarifies the roles of the Structures, Hydraulic, and Geotechnical Engineers in considering the effects of scour for foundation design.

Chapter 7 - Concrete Design, Environmental Classification, and Construction Criteria:

1. The chapter has been rewritten to be consistent with the ***Structures Design Guidelines for Load and Resistance Factor Design*** (Topic No.: 625-020-150-c).

Chapter 8 - Design Criteria for Segmental Box Girder Bridges:

1. Only minor clarifications have been made.

Chapter 9 - Vessel Collision:

1. This chapter has been rewritten in a manner similar to the ***Structures Design Guidelines for Load and Resistance Factor Design*** (Topic No.: 625-020-150-c).

Chapter 10 - Superstructure and Approach Slab Design:

1. Article 10.4 now provides bridge length definitions and the effect of bridge length on bridge deck thickness and profilograph requirements.
2. Article 10.20 provides more detailed guidance and requirements regarding the design of bridge traffic railing barriers for new and existing construction.
3. Article 10.23 provides guidance and requirements for lateral restraint of superstructures.

Chapter 11 - Structural Steel Design:

1. Articles 11.1 and 11.3 differentiate the certification and FCM requirements for highway bridges and pedestrian bridges, respectively.

Chapter 15 - Movable Bridge Requirements:

1. Chapter 15 has been rewritten in a manner similar to that of the ***SDG-LRFD***.

Chapter 16 - Bridge Widening:

1. New Article 16.8 clarifies Bridge Deck Grooving requirements for different types of widenings.

Chapter 18 - Retaining Walls:

1. Article 18.2.1.M provides guidance in the use of Hybrid-Gravity/MSE Walls.
2. A number of minor editorial changes have been incorporated to make Chapter 18 better coordinated with the ***PPM***.

STRUCTURES DESIGN GUIDELINES

Topic No.: 625-020-152-a

Effective: January 1, 2000

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Florida Department of Transportation
Structures Design Office
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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 the provisions of the **AASHTO Standard Specifications for Highway Bridges**, Sixteenth Edition, 1996, 1997, and the 1998 Interims thereto, published in SI Units by the American Association of State Highway and Transportation Officials (AASHTO) for Load Factor Design (LFD), and hereinafter simply referred to as "LFD." These exceptions may be in the form of deletions from, additions to, or modifications of the LFD specifications. These two documents must always be used jointly and must be utilized in concert with the FDOT's **Plans Preparation Manual**, Volumes I and II (Topics Nos.: 625-000-005 and 625-000-006, respectively) 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 or other entities or governmental agencies requiring compliance with FDOT's policies, procedures, and standards.

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 LFD and includes article-by-article commentary when appropriate. In addition to its companion documents referenced 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 at 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:

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 **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 superceded 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. D99-2 would be the second Bulletin issued in 1999 for **SDG** Topic Number 625-020-150-*d*.

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 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 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: State

Construction Office, 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 DSDO'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

FDOT STRUCTURES DESIGN

NOTE:

The provisions of this chapter have been relocated to the *Plans Preparation Manual* (Topic No.: 625-000-005), Chapter 26.

Chapter 2

GEOMETRIC DESIGN CRITERIA

NOTE:

The provisions of this chapter have been relocated to the *Plans Preparation Manual* (Topic No.: 625-000-005), Chapter 2.

Chapter 3

BRIDGE PROJECT DEVELOPMENT

NOTE:

The provisions of this chapter have been relocated to the *Plans Preparation Manual* (Topic No.: 625-000-005), Chapters 16, 26 & 29.

Chapter 4

STRUCTURAL LOADINGS AND MATERIAL PROPERTIES

4.1 Special Loads

4.1.1 Dead Loads

A. Future Wearing Surface:

See Table 4.2 regarding the allowance for a Future Wearing Surface.

B. Sacrificial Concrete:

Bridge decks subject to the profilograph requirements of Chapter 10 require an added thickness of sacrificial concrete which must be accounted for as added Dead Load but which cannot be utilized for bridge deck section properties. See "Deck Thickness Determination" in Chapter 10.

4.1.2 Live Loads

A. Prestressed I-Girder Bridges:

In addition to AASHTO loadings, when the skew angle is 30E or less, a 5% increase shall be applied to AASHTO Live Loads in the design of prestressed I-Girders (See also "Intermediate Diaphragms for Prestressed I-Girder Construction" in Chapter 10).

B. Cast-in-Place Slab Bridges, Precast Multi-Beam Deck Bridges, and Box Culverts with Less than 600 mm of Cover:

The superstructure of cast-in-place slab bridges and precast, multi-beam-deck bridge construction meeting the requirements of Chapter 10, as well as cast-in-place or precast box culverts with less than 600 mm of cover, shall be designed for MS22.5 truck loading. The MS22.5 truck has axle loads 25% greater than those of the MS18 truck, but the axle spacing is identical to that of the MS18 truck loading. Similarly, the lane load for the MS22.5 live load is 25% greater than that for the MS18 lane load. Substructures of these bridges may be designed for the MS18 truck loading.

4.2 Seismic Provisions

4.2.1 General

The majority of Florida bridges will not require seismic design or the design of seismic restrainers. For such bridges, only the minimum bearing support dimensions need to be satisfied as outlined in Procedure 1 of AASHTO Seismic Performance Category "A." 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 will 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 Method" specified in AASHTO Division I-A, Seismic Design, Article 4.3, 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.

It can be shown that neoprene bearing pads utilized for the majority of Florida bridges have sufficient shearing strength and slip resistance to transfer the calculated seismic forces to the bridge substructures. 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. An example would be, a long, continuous superstructure, supported on a system of sliding bearings with one short, stiff pier used to 'fix' the superstructure. In the example, the fixed connection must be designed to transmit the calculated seismic force as it likely will govern over other load cases.

4.2.2 Seismic Design for New Construction

When required for new construction as described above, use Procedure 1 of AASHTO Seismic Performance Category "B" to determine the seismic design forces to be used for the design of the connection between the superstructure and substructure **only**. In the event that the horizontal load calculated using Procedure 1 of Seismic Performance Category "B" exceeds the 20% of the dead load calculated under Seismic Performance Category "A," then Seismic Performance Category "A" shall be used.

The acceleration coefficient in Florida varies from 1% to 3.75%, and shall be determined from the Map of Horizontal Acceleration contained in the latest edition of AASHTO's **Standard Specifications for Highway Bridges**.

Minimum bearing lengths shall be calculated by the provisions of Performance Category "A."

4.2.3 Seismic Design for Widening

All bridge elements for the widening of an existing bridge structure classified as a major widening by the definitions of Chapter 16 shall comply with the seismic provisions for new construction described above.

Seismic provisions for the widening of an existing structure classified as a minor widening in accordance with Chapter 16 will be considered by the FDOT on an individual basis.

4.2.4 Lateral Restraint

When lateral restraint of the superstructure is required because of seismic loading, comply with the provisions and requirements of "Lateral Restraint of Superstructures" in Chapter 10.

4.3 Wind Loads

4.3.1 Wind Loads on Bridges

Bridges shall be designed for wind pressures that increase with the height of the superstructure above the average existing ground (or water) elevation in the immediate vicinity of the bridge. The elevation of the superstructure shall be taken as the profile grade elevation at the centerline of the substructure. All substructure elements for a given superstructure unit may be designed for the average elevation of the piers in that unit. The design wind pressures for various heights above the average existing ground (water) elevation are shown in Table 4.1:

Table 4.1 Wind Load on Bridges

Height (meters)	Pressure (kN/m ²)*
$0 < H \leq 10$	2.4
$10 < H \leq 15$	2.7
$15 < H \leq 20$	3.0
$20 < H \leq 25$	3.2

*NOTE: The wind pressure on the structure applies to both substructure and superstructure elements.

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

shown above shall be increased by 20% for bridges located in Palm Beach, Broward, Dade, and Monroe counties. Wind pressures for bridges over 25 meters high, or with unusual structural features, shall be proposed by the EOR for FDOT approval on an individual basis.

4.3.2 Wind Loads on Other Structures

Wind speeds for miscellaneous highway related structures are specified in Chapter 29 of the *Plans Preparation Manual*, Volume I (Topic No. 625-000-005).

4.3.3 Wind Loads During Construction

During the design phase of a bridge project, the stability and lateral buckling of beams and girders shall be analyzed for wind loading and handling/erection during construction and prior to casting the concrete deck slab. When deemed necessary by the EOR, adequate temporary bracing, shoring, tie-downs, strongbacks, or other means of insuring stability and resistance to lateral buckling shall be detailed in the design plans.

When analyzing the stability of beams and girders, the Factor of Safety against overturning, taken about the extreme lateral point of contact bearing, shall not be less than 1.5, and the wind pressure shall be taken as one-half (50%) of the value given in Table 4.1.

4.4 Miscellaneous Loads

The loadings in Table 4.2 may be used unless a more refined analysis is performed.

Table 4.2 Miscellaneous Loads

Item	Unit	Load
Barrier, Traffic (F-Shape)	kN/m	6.1
Barrier, Median Traffic (F-Shape)	kN/m	7.1
Barrier, Traffic (F-Shape) w/Railing	kN/m	6.2
Railing, 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 bridges not subject to the profilograph requirements of Chapter 10. The allowance also applies to widenings unless otherwise authorized by the Department.
- ** The value shown in the unit load of metal forms and concrete required to fill the form flutes and is to be applied over the projected plan area of the metal forms.

4.5 Material Properties

Unless otherwise stipulated or obtained from testing, the modulus of elasticity of concrete at 28 days shall be taken from Table 4.3 for Florida Limerock Aggregate.

Table 4.3 Modulus of Elasticity (GPa)

(Showing loads of the concrete mix itself excluding any and all embedded items such as reinforcing, ties, inserts, etc.)

Strength (Mpa)	Normal Aggregate Concrete Load (kN/m ³)				Florida Limerock Aggregate Concrete Load (kN/m ³)			
	22.0	22.8*	23.6	24.3	22.0	22.8*	23.6	24.3
23	21.8	23.0	24.2	25.3	19.6	20.7	21.8	22.8
28	24.0	25.4	26.7	27.9	21.6	22.8	24.0	25.1
31	25.3	26.7	28.1	29.4	22.8	24.0	25.3	26.4
35	26.9	28.3	29.8	31.2	24.2	25.5	26.8	28.1
38	28.0	29.5	31.1	32.5	25.2	26.6	28.0	29.2
41	29.1	30.7	32.3	33.8	26.2	27.6	29.1	30.4
45	30.5	32.1	33.8	35.4	27.4	28.9	30.5	31.8
48	31.5	33.2	35.0	36.5	28.3	29.9	31.5	32.9

* These columns shall normally apply.

4.6 Thermal and Shrinkage Loads on Structures

Shrinkage values stated herein shall be used for sizing joints and bearings in accordance with the provisions of Chapter 12.

Structures shall be designed for the effects of thermal and shrinkage strains where appropriate. Thermal effects shall be based upon the temperature ranges given in Table 12.1

except for post-tensioned segmental construction which shall comply with the requirements of Chapter 8.

Conventional reinforced concrete structures shall be designed for a shrinkage strain of 0.0002, in accordance with AASHTO. Traditional prestressed beam structures with reinforced concrete decks shall be designed for a shrinkage strain of 0.0001. Continuous post-tensioned segmental or prestressed beam structures shall be designed for the thermal and shrinkage movements reported by the design program used.

Chapter 5

FOUNDATIONS AND GEOTECHNICAL DATA

5.1 Design Methodology

The entire foundation design shall be performed using the AASHTO Strength Design Method (Load Factor Design). The soil performance factors for foundation systems as specified in AASHTO, Article 4.10.6 (1.0 for lateral analysis), shall be used for all load cases.

5.2 Load Cases

5.2.1 Load Case 1 (Always required - Scour may be "0")

Conventional AASHTO loadings (using load factor combination groups as specified in AASHTO Table 3.22.1A), but utilizing the most severe case of scour up to and including that from a 100 year flood event.

5.2.2 Load Case 2 (Applies only if vessel collision force is specified)

Extreme event of Vessel Impact (using load factor combination groups as specified in the AASHTO **Guide Specification and Commentary for Vessel Collision Design of Highway Bridges**, Section 3.14) utilizing scour depths described in Chapter 9, "Scour with Vessel Collision."

5.2.3 Load Case 3 (Applies only if scour is predicted)

Stability check during the superflood (most severe case of scour up to and including that from the 500-year flood) event.

$$\text{Load Case 3} = 1.0(DL) + 0.5(L) + 1.0(SF) + 1.0(B)$$

[Eq. 5-1]

Where:

DL	=	Dead Load
L	=	MS Live Load
SF	=	Stream Flow
B	=	Buoyancy

5.3 Foundation Considerations

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

The minimum size of prestressed concrete piles shall be 455mm, and the minimum size drilled shaft shall be 915mm (preferably larger). An exception to this minimum size requirement are piles for fender systems. Minimum pile/shaft spacing center-to-center shall be at least three (3) times the least width of the element measured at the ground line.

When cylinder piles are proposed for use on a project, an alternate foundation type shall be developed and included in the Contract Documents. Concrete cover on cylinder pile reinforcement shall comply with the requirements for cover on prestressed concrete pile reinforcement shown in Table 7.2.

The design engineer 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, effect on adjacent structures, etc. shall be considered in evaluating foundation design alternatives.

5.4 Geotechnical Report

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 as well as background information that may assist the Structural Engineer in determining appropriate pile lengths. The Geotechnical Engineer will contact the District Construction Office and District Geotechnical Engineer, as needed, to obtain local, site-specific foundation construction history.

The Geotechnical 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 the Office of 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 (See the ***Plans Preparation Manual***, Volume I, Chapter 26 (Topic No.: 625-00-005) for category definitions). 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 shall 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 Structures Engineer and the District Geotechnical Engineer prior to commencing any operations.

5.5 Downdrag Considerations for Pile and Drilled Shaft Foundation Design

For pile or drilled shaft foundations, the downdrag load shall be shown on the plans.

For pile foundations, the term “downdrag” is defined as the ultimate skin friction above the neutral point (the loading added to the pile due to settlement of the surrounding soils) plus the dynamic resistance above the neutral point (the resistance that must be overcome during the driving of the pile) minus the live load. The dynamic resistance typically equals 0.25 to 1.0 times the ultimate skin friction above the neutral point.

For drilled shaft foundations, the term “downdrag” is defined as the ultimate skin friction above the neutral point (the loading added to the drilled shaft due to settlement of the surrounding soils) minus the live load.

Scour may or may not occur as predicted; however, discounting of scourable soil layers shall not be used to reduce the predicted downdrag.

5.6 Scour Considerations for Foundation Design

This is a multi-discipline effort involving Geotechnical, Structures, and Hydraulics Engineers. The process listed below will often require 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. These three engineering disciplines have specific responsibilities in considering scour as a step in the foundation design process.

5.6.1 The Structures Engineer

The Structures Engineer determines the preliminary design configuration of a bridge structure utilizing all available geotechnical and hydraulic data and performs lateral stability evaluations for the applicable loading(s) described in “Load Cases” at the beginning of this Chapter (arbitrary deflection limits shall not be imposed except on movable bridges). A preliminary lateral stability analysis normally will occur during the BDR phase of the project, and a final

evaluation will occur subsequent to the selection of the final configuration(s). The Structures Engineer needs to apply sound engineering judgement in comparing results obtained from scour computations with available hydrological, hydraulic, and geotechnical data to achieve a reasonable and prudent design.

5.6.2 The Hydraulics Engineer

Utilizing good engineering judgement as required by HEC-18, the Hydraulics Engineer provides the worst case scour elevation through a 100-year flood event (100-Year Scour), a 500-year flood event (500-Year Scour), and for 'Long-Term Scour.' 'Long Term Scour' is defined and described in Chapter 4 of the FDOT *Drainage Manual* (Topic No. 625-040-001).

5.6.3 The Geotechnical Engineer

The Geotechnical Engineer provides the factored axial (compression and tension) curves, mechanical properties of the soil and foundation recommendations based on construction methods, pile availability, similar nearby projects, site access, etc.

5.7 Minimum Pile or Shaft Tip Elevation

The minimum pile or shaft tip elevation must be the deepest of the minimum elevations that satisfy lateral stability requirements for the three load cases including the appropriate scour condition. The minimum tip may be set lower to satisfy any unique soil conditions. The minimum tip must be established by the Structures Engineer and concurred with by the Geotechnical Engineer.

5.8 Anticipated Pile Lengths

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.

5.9 Test Piles

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. Test piles may 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. 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 in end bents are not normally required and should not be specified unless specific justification is provided in the Geotechnical Report.

Test piles shall be required for all projects unless, in the opinion of the District Geotechnical Engineer, pile driving records for an existing, adjacent structure include enough information (i.e., stroke length, hammer type, cushion type, etc.) to adequately determine the authorized pile lengths and driving criteria.

When determining the location of test piles, traffic maintenance requirements, required sequence of construction, geological conditions, and pile spacing must be considered. For phase construction, preferably locate all test piles in the first portion 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 designer shall take these facts into consideration in preparing estimated lengths. Test piles should be at least 5m longer than the estimated production piles. Additional length may be required by the load frame geometry when static load tests are used. The designer shall coordinate his recommended test pile lengths and locations with the District Geotechnical Engineer, or Geotechnical Consultant, prior to finalization of the plans.

5.10 Pile Load Tests

Load tests include static load tests and dynamic load tests. In general, the more variable the subsurface profile, the less cost effective static load tests are. When site variability is an issue, other test options to be considered include additional field exploration, more laboratory samples, in-situ testing, and pullout 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.

5.10.1 Static Load Test

Test piles 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 developing the required pile capacity at the desired depths, and/or the possibility exists that considerable cost savings will result if higher soil capacities can be obtained. Also, static load tests will reduce driving effort because a higher Performance Factor is applied to the Ultimate Bearing Capacity formula. If static load test piles are called for on the design plans, the number of required tests, pile type, pile size, and load 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 in Section 455 of the Specifications.

5.10.2 Dynamic Load Test

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. 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.

5.10.3 Special Considerations

Load testing of foundations which will be subjected to subsequent scour activity require special attention. The necessity of isolating the resistance of the scourable material from the load test results must be addressed in the plans or Technical Special Provisions for the project.

5.11 Battered Piles

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 Designer should evaluate the effects of length and batter on the selected pile size. 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 adverse 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 of batter causing pile breakage in bending. The ability to install battered piles must be determined during the design phase. The following maximum batters, measured as vertical to horizontal (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

5.12 Fender Piles

See Section 455 of the Specifications.

5.13 Pile Driving Resistance

The Geotechnical Engineer calculates a Required Driving Resistance (RDR) which is the same value as the ultimate bearing capacity, or ultimate resistance, by combining the net scour value and downdrag value with the factored design load as follows:

$$RDR = \frac{\text{FACTORED DESIGN LOAD} + \text{NET SCOUR} + \text{DOWNDRAG}}{f} \quad [\text{Eq. 5-2}]$$

Where: f = 0.80 with static load tests
0.70 with dynamic load tests
0.60 with Wave Equation (WEAP) but no static or dynamic load tests

The required driving resistance shall not exceed the following values unless specific justification is provided and accepted by the Department:

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

5.14 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.

If jetting or preforming is utilized to EI_{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 designer must verify that it will not violate any of the environmental permits.

5.15 Spread Footings

In the event that the Geotechnical Report recommends a spread footing foundation, the maximum soil pressures, the minimum footing widths, the minimum footing embedments, and the Performance Factors (f) used will be indicated in the Geotechnical Report. The design 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 designer 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 600mm of the bottom of the footing or higher, dewatering must be specified.

5.16 Plan Information - Piles

All plans for projects utilizing driven piles shall contain the Pile Data Table (Table 5.1) 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.

The Pile Data Table (Table 5.1) is not required in the Geotechnical Report; however, the information shown on the plans for Table 5.1 shall be reviewed by the Geotechnical Engineer of Record.

In the Pile Data Table, the values for driven piles for the column entitled Ultimate Bearing Capacity (UBC) shall be determined from the following formula:

$$UBC = \frac{FACTORED\ DESIGN\ LOAD\ \% \ NET\ SCOUR\ \% \ DOWNDRAG}{f} \quad [Eq. 5-3]$$

Table 5.1 Pile Data Table

PILE DATA TABLE														
INSTALLATION CRITERIA							DESIGN CRITERIA							
Pier or Bent No.	Pile Size (mm)	Ult. Brg. Cap. (kN)	Ten. Cap. (kN)	Min. Tip Elev. (m)	Test Pile Len. (m)	Req'd Jet Elev. (m)	Req'd Preform Elev. (m)	Fact'd Des. Load (kN)	Down Drag (kN)	Total Scour Resist. (kN)	Net Scour Resist. (kN)	Long-Term Scour Elev. (m)	100-Year Scour Elev. (m)	f

- 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).
- 100-Year Scour** - defined in Chapter 4, *FDOT Drainage Manual* (Topic No. 625-040-001).

Additional Plan Notes:

- ! Minimum Tip Elevation is required _____ (reason must be completed by designer, for example: “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 actual 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, a driving resistance higher than that required for load capacity will be required whenever the Ultimate Bearing Capacity is shown to be greater than the Factored Design Load.

5.17 Plan Information - Drilled Shafts

All plans for projects utilizing drilled shafts shall contain the Drilled Shaft Data Table (Table 5.2) and notes included hereinafter, as applicable. For items that do not apply, place “N/A” in the column, but the table itself should not be revised or modified.

The Drilled Shaft Data Table (Table 5.2) is not required in the Geotechnical Report; however, the information shown on the plans for Table 5.2 shall be reviewed by the Geotechnical Engineer of Record.

Table 5.2 Drilled Shaft Data Table

DRILLED SHAFT DATA TABLE								
INSTALLATION CRITERIA					DESIGN CRITERIA			
Pier or Bent No.	Shaft Size (mm)	Tip Elevation (m)	Minimum Tip Elevation (m)	Minimum Rock Socket Length (m)	Factored Design Load (kN)	Downdrag (kN)	Long-Term Scour Elevation (m)	100-Year Scour Elevation (m)

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 the Engineer permits adjustments to the tip elevation specified.

5.18 Cofferdams and Seals

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 required for bidding purposes.

When showing seal dimensions in the bridge plans, the EOR shall show the maximum 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 following maximum allowable stresses at the time of complete dewatering of the cofferdam:

Table 5.3 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	1.725 MPa
Cohesive shear stress between seal concrete and concrete piles or shafts	0.520 MPa*
Cohesive shear stress between seal concrete and steel piles or casings	0.250 MPa*

* Values shall not exceed one-half (1/2) the Ultimate Side Friction of the sheet piles, concrete piles, or drilled shafts.

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.

Chapter 6

HYDRAULIC DATA AND U.S. COAST GUARD PERMITS

NOTE:

The provisions of this chapter have been relocated to the *Plans Preparation Manual* (Topic No.: 625-000-005), Chapters 26 & 27.

Chapter 7

ENVIRONMENTAL CLASSIFICATION, CONCRETE AND STEEL STRUCTURES, AND CONSTRUCTION CRITERIA

7.1 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 16. 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 plans' 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, other than for 'Slightly Aggressive' shall be contained in parentheses after the classification.

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

7.2 Environmental Classification Parameters

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

following criteria.

A. For substructure components and systems based upon 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 7-1).

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 7-1.

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 7-1.

7.2.1 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 proprietary walls), and bulkheads shall be considered as substructures.
- B. Abutments/end bents shall always be governed by the most stringent condition.
- C. Approach slabs shall be considered as superstructure; however, Class II Concrete (Bridge Deck) will be used for all environmental classifications.

7.2.2 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 7.1 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 7.1 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

7.3 Concrete Structures

7.3.1 Concrete Cover

The requirements for concrete cover over reinforcing steel are listed in Table 7.2. Examples of concrete cover are shown in Figures 7.2 through 7.5.

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 7.2.

7.3.2 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 7.3 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. The following example is given:

Component - submerged piling
Environment - Extremely Aggressive over saltwater
Concrete Class - Table 7.3 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. Single Column or Wall Piers:

In all columns or walls exposed to the splash zone.

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

7.3.3 Penetrant Sealers

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

Refer to Chapter 16 for use of penetrant sealers on bridge widening projects.

7.3.4 Stay-in-Place Forms

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. The use of **composite** Stay-in-Place Forms is not permitted.

A. Metal Stay-in-Place Forms:

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

B. Concrete Stay-in-Place Forms:

Concrete Stay-in-Place forms may be used for all environmental classifications without exception; however, the bridge plans must be specifically designed, detailed and prepared for their use.

7.3.5 Prestressed Concrete Piles

Fender piles generally have a short life expectancy. Fender piles are considered to be sacrificial; therefore, no additional corrosion protection requirements are needed beyond the use of concrete class as shown in Table 7.3.

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.

7.3.6 Epoxy Coated Reinforcing Steel

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

7.3.7 Construction Joints in Drilled Shafts

When drilled shafts are used in bents and extend through the water and wet-dry (splash) zone, shafts shall be detailed to extend without a construction joint a minimum of 4.0 meters above the Mean High Water elevation or bottom of the bent cap, whichever is lower. It is preferred that taller shafts extend to the bottom of the bent cap without a construction joint.

7.3.8 Secondary Reinforcing Requirements for Pier Footings

A two-way cage of reinforcement must be provided in all faces of pier footings,. 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 minimum reinforcement used to form the cage **in each direction** shall be sized and spaced according to the most stringent of the following criteria:

- ! Reinforcing consisting of not less than No. 16 bars spaced at 300 mm.
- ! When the footing qualifies as mass concrete, which is the usual condition,

reinforcing not less than the minimum reinforcing steel necessary to satisfy the following equation:

$$3A_b \geq \frac{s(2d_c + d_b)}{100} \quad [\text{Eq. 7-1}]$$

Where:

- A_b = minimum area of bar (mm^2)
- s = spacing of the bars (mm)
- d_c = concrete cover measured to center of bar (mm)
- d_b = diameter of the reinforcing bar (mm)

NOTE: The quantity $(2d_c + d_b)$ need not be taken as greater than 75 mm.

When the value of $(2d_c + d_b)$ is 75 mm, which is the usual case, Equation 7-1 simplifies to:

$$3A_b \geq 0.75 s \quad [\text{Eq. 7-1a}]$$

7.3.9 Mass Concrete

The EOR shall consider Mass Concrete requirements in selecting member sizes and shall avoid Mass Concrete if practicable; however, when its use is unavoidable, the EOR shall be responsible for indicating which portions of the concrete in a bridge shall be designated as Mass Concrete.

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 the concrete exceeds 1.0 meter and the ratio of volume of concrete to the surface area is greater than 0.3 meters, then mass concrete shall be required. (The surface area for this ratio shall include the summation of all the surface areas of the concrete component being considered, including the full underside (bottom) surface of footings, caps, construction joints, etc.). Volume and surface area calculations shall be in units of meters.

Estimated bridge pay quantities shall be arranged so that Mass Concrete is a separate quantity. Mass concrete quantities shall be separated into Mass Concrete (Substructure) and Mass Concrete (Superstructure). Seal Concrete shall not be considered as Mass Concrete.

The EOR shall also be responsible for taking precautionary measures to reduce the likelihood of cracking of the concrete. These precautionary measures are especially needed in the design of bascule bridge substructures and other structural components requiring the casting of large volumes of concrete. To prevent or control cracking, some considerations to be taken

in all designs involving Mass Concrete include more and/or better reinforcing steel distribution, the judicious placing of construction joints, as well as other methods as outlined in ACI 207, ACI 244, and ACI 308.

7.4 Steel Structures

7.4.1 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.

7.4.2 Three-Coat Inorganic Zinc Paint System

Unless otherwise specifically approved by the FDOT, all structural steel bridge members that

require coatings will receive a three-coat paint system in accordance with the specifications.

7.4.3 Steel Box Girder Interior Painting

Refer to Chapter 10, "Special Requirements for Box Girder Inspection and Access," for interior painting of steel box girders.

7.4.4 Existing Structures with Hazardous Material

Refer to Chapter 11, "Existing Structures with Lead Based Paint," for the handling and disposal of hazardous materials encountered on existing structures.

7.4.5 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 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, etc., shall be ASTM A 325M, Type 1 and shall be hot-dip galvanized.

7.5 Construction Joints in Tall Piers

Construction joints shall be detailed on the plans for tall piers or columns to limit the concrete lifts to 8.0 meters; except, 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.

7.6 Control of Shrinkage Cracking

For long walls and similar construction, the EOR is advised to consider the following:

- ! The minimum percentage of reinforcement to control shrinkage/thermal cracking shall be at least 0.1% of the full cross-section area, or 265 mm² per meter per face, whichever is larger. This minimum reinforcement applies in each direction and in each face.
- ! Limit vertical lifts to 8.0 meters between horizontal construction joints.
- ! Placing vertical construction and/or contraction joints at a maximum spacing of 9.0 meters, and extending the joints from the top of wall to the top of footing.
- ! Constructing expansion joints in walls at a maximum spacing of 36 meters. Keyways for expansion joints shall be detailed in accordance with the shape and dimensions described hereinafter.
- ! Clearly detail required construction joints on the plans.
- ! Provide a water barrier in construction and/or expansion joints where water leakage has to be prevented.

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

Pier caps and decks should 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.

Continuous superstructures require the use of a designated casting sequence. Camber diagrams shall be constructed using the impact of the casting sequence up on the varying stiffness of the superstructure. The plans shall contain a note that the casting sequence may not be changed unless a new structural analysis is performed and new camber diagrams calculated by the Specialty Engineer.

Keyways shall be provided at construction joints at the discretion of the EOR. Where the

initially-cast surface is roughened and adequate reinforcing passes through the construction joint, keyways generally are not required. Keyways also are not required for reinforced bridge decks. When keyways are required, they shall be detailed as trapezoidal in shape for ease of forming and stripping, have a minimum depth of 50 mm, and a width of about one-third (1/3) the thickness of the member in the direction of the applied shear force.

7.7 Adhesive Anchor Systems

7.7.1 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, designed in accordance with the requirements of this Article, shall not be used to splice prestressed piling; however, Adhesive Bonding Material complying with Section 937 of the Specifications and installed in accordance with Section 416 may be used for the adhesive joint and to fill the holes shown in the pile splice detail on the *Standard Drawings*.**

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. 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.

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 or 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.**

7.7.2 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 ϕ_{gn} (See Figure 7-6). [mm²]
- A_n = effective area of a group of Adhesive Anchors in tension, used in calculating ϕ_{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 7-6). [mm²]
- A_{v0} = $4.5(c^2)$, effective breakout area of a single Adhesive Anchor in shear, used in calculating ϕ_{gv} (See Figure 7-7). [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 ϕ_{gv} (See Figure 7-7). [mm²]
- c = anchor edge distance (distance from free edge to centerline of anchor). [mm]
- d = nominal diameter of Adhesive Anchor. [mm]
- f'_{lc} = 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. [N]
- N_n = nominal tensile strength of Adhesive Anchor. [N]
- N_o = nominal tensile strength as controlled by concrete embedment for a single Adhesive Anchor. [N]
- N_s = design strength as controlled by Adhesive Anchor steel. [N]
- N_u = factored tension load. [N]

- s = Adhesive Anchor spacing (measured from centerlines of anchors). [mm]
 V_c = shear design strength as controlled by the concrete embedment for Adhesive Anchors. [N]
 V_n = nominal shear strength of Adhesive Anchor. [N]
 V_s = design shear strength as controlled by Adhesive Anchor steel. [N]
 V_u = factored shear load. [N]
 t' = 7.5 MPa nominal bond strength for products on the QPL.
 f_c = 0.85, capacity reduction factor for an Adhesive Anchor that is controlled by the concrete embedment.
 f_s = 0.90, capacity reduction factor for an Adhesive Anchor that is controlled by anchor steel.
 ϕ_e = modification factor, for strength in tension, to account for anchor edge distance less than $10d$ (1.0 when $c \geq 10d$).
 ϕ_{gn} = strength reduction factor for Adhesive Anchor groups in tension (1.0 when $s \geq 2.0h_e$).
 ϕ_{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 $h \geq 1.5c$).

7.7.3 Design Requirements for Tensile Loading

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

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

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

$$N_c = f_c \phi_e \phi_{gn} N_o \quad [\text{Eq. 7-3}]$$

where:

$$N_o = t' p d h_e \quad [\text{Eq. 7-4}]$$

For anchors with a distance to a free edge of concrete less than $10d$, but greater than or equal to $3d$, a reduction factor, ϕ_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 $10d$, ϕ_e may be taken as 1.0.

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

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

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

7.7.4 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:

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

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:

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

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

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

7.7.5 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:

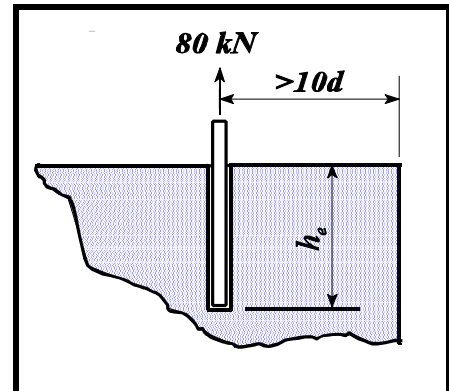
$$\frac{N_u}{f N_n} + \frac{V_u}{f V_n} \leq 1.0 \quad [\text{Eq. 7-10}]$$

In Equation 7-10, $f N_n$ 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). $f V_n$ 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).

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.**

7.7.6 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:

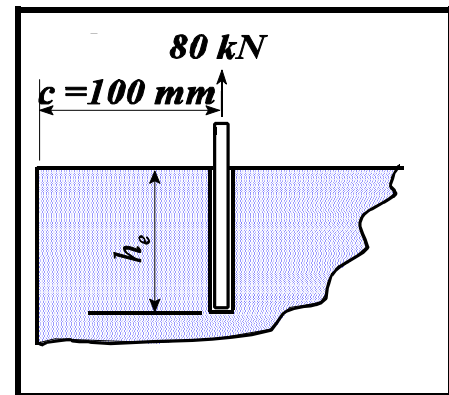
N_u	=	80.0 kN
f_y	=	720.0 MPa
f_u	=	860.0 MPa
t'	=	7.5 MPa

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.	$N_s = f_s A_e f_y$ where: $f_s = 0.9$ $A_e = 0.75 (p d^2/4)$ $f_y = 720 \text{ MPa}$
Substituting and solving for d :	$80000 = (0.9)[0.75 (p d^2/4)](720)$ $d = 14.5 \text{ mm}$ 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 edge or spacing concerns, ϕ_e and ϕ_{gn} may be taken as unity.	$N_c = f_c \phi_e \phi_{gn} N_o$ (for embedment) where: $f_c = 0.85$ $\phi_e, \phi_{gn} = 1.0$ (no edge/spacing concern) $N_o = t' p d h_e$
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.	$N_{c(\text{req'd})} = 1.25 A_e f_y \# A_e f_u$

Determine the effective area for a 16 mm threaded rod:	$A_e = 0.75 (p \ 16^2/4)$ $A_e = 151 \text{ mm}^2$
Determine the required tension force, $N_{c(\text{req'd})}$, to ensure ductile behavior.	$N_{c(\text{req'd})} = 1.25 A_e f_y \# A_e f_u$ $N_{c(\text{req'd})} = 1.25 (151)(720) \# (151)(860)$ $N_{c(\text{req'd})} = 136 \text{ kN} > 130 \text{ kN}$ <i>therefore, use $N_{b(\text{req'd})} = 130 \text{ kN}$</i>
Substituting and solving for h_e :	$130,000 = 0.85 (1.0) (1.0) (7.5) p (16) h_e$ $h_e = 406 \text{ mm}$

7.7.7 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:

$$\begin{aligned}
 N_u &= 80.0 \text{ kN} \\
 f_y &= 720.0 \text{ MPa} \\
 f_u &= 860.0 \text{ MPa} \\
 t' &= 7.5 \text{ MPa}
 \end{aligned}$$

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.	$N_s = f_s A_e f_y$ <i>where:</i> $f_s = 0.9$ $A_e = 0.75 (p d^2/4)$ $f_y = 720 \text{ MPa}$
Substituting and solving for d :	$80000 = (0.9)[0.75 (p d^2/4)](720)$ $d = 14.5 \text{ mm}$ <i>therefore, use a 16 mm threaded rod</i>

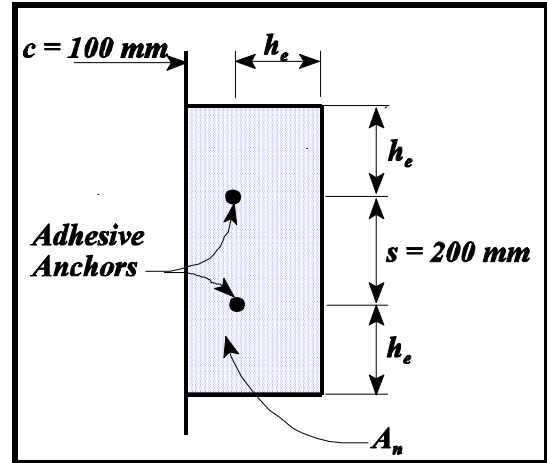
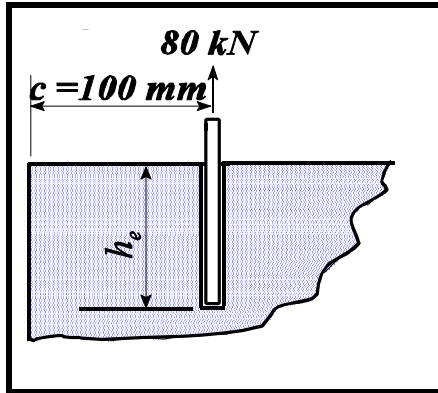
Step 2 - Determine required embedment length to ensure steel failure	
Basic equation for embedment length calculation. Since there are no spacing concerns, ϕ_{gn} may be taken as unity, and, since the edge distance (100 mm) is less than $10d$ (160 mm), the edge effect, ϕ_e , will need to be evaluated.	$N_c = \phi_c \phi_e \phi_{gn} N_o \text{ (for embedment)}$ <p>where:</p> $f_b = 0.85$ $\phi_e, \phi_{gn} = 1.0 \text{ (no edge/spacing problem)}$ $N_o = t' p d h_e$
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.	$N_{c(REQ'D)} = 1.25 A_e f_y \# A_e f_u$
Determine the effective area for a 16 mm threaded rod:	$A_e = 0.75 (p 16^2/4)$ $A_e = 151 \text{ mm}^2$
Determine the required tension force, $N_{c(REQ'D)}$, to ensure ductile behavior.	$N_{c(REQ'D)} = 1.25 A_e f_y \# A_e f_u$ $N_{c(REQ'D)} = 1.25 (151)(720) \# (151)(860)$ $N_{c(REQ'D)} = 136 \text{ kN} > 130 \text{ kN}$ <p>therefore, use $N_{c(REQ'D)} = 130 \text{ kN}$</p>
Determine edge effect factor, ϕ_e . Note: $c_{cr} = 10d$	$\phi_e = 0.60 + 0.40 (c \div 10d)$ $\phi_e = 0.60 + 0.40 [100 \div (10)(16)]$ $\phi_e = 0.85$
Substituting and solving for h_e :	$130,000 = 0.85 (1.0)(0.85)(7.5) p (16) h_e$ $h_e = 477 \text{ mm}$

7.7.8 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:

$$\begin{aligned}
 N_u &= 80.0 \text{ kN} \\
 c &= 100.0 \text{ mm} \\
 s &= 200.0 \text{ mm} \\
 f_y &= 720.0 \text{ MPa} \\
 f_u &= 860.0 \text{ MPa} \\
 t' &= 7.5 \text{ MPa}
 \end{aligned}$$



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.	$N_s = f_s A_e f_y$ where: $f_s = 0.9$ $A_e = (2) 0.75 (p d^2/4)$ $f_y = 720 \text{ MPa}$
Substituting and solving for d :	$80000 = (0.9)(2)[0.75 (p d^2/4)](720)$ $d = 10.2 \text{ mm}$ although a 13 mm threaded rod is OK, use a 16 mm threaded rod to minimize embedment length
Design steel strength	$N_s = 0.9(2)(0.75)(p 13^2/4)(720)$ $N_s = 129,000 \text{ kN} > 80,000 \text{ kN} \sim \text{OK}$
Step 2 - Determine required embedment length	
Basic equation for embedment length calculation. Since there are edge or spacing concerns, γ_e and γ_{gn} will need to be determined.	$N_c = f_c \gamma_e \gamma_{gn} N_o$ (for embedment) where: $f_c = 0.85$ γ_e and γ_{gn} are calculated below $N_o = t' p d h_e$

Determine edge effect factor, λ_e .	$\lambda_e = 0.60 + 0.40 (c \div c_r)$ $\lambda_e = 0.60 + 0.40 [100 \div (10)(16)]$ $\lambda_e = 0.85$
Determine group effect factor, λ_{gn} . Note: λ_{gn} is best determined by assuming a value for h_e and iterating to find the required embedment. Therefore, assume: $h_e = 360$ mm	$\lambda_{gn} = A_n / A_0$ $\lambda_{gn} = \frac{(100 + h_e)[200 + 2(h_e)]}{4.0(h_e)^2}$ $\lambda_{gn} = \frac{(100 + 360)[200 + 2(360)]}{4.0(360)^2}$ $\lambda_{gn} = 0.82$
Substituting and solving for h_e .	$80,000 = (0.85)(0.82)(0.85)(7.5)p(16)h_e$ $h_e = 358 \text{ mm} \cdot 360 \text{ mm} \quad \sim \text{OK}$
Design adhesive bond strength.	$N_c = (0.85)(0.82)(0.85)(7.5)p(16)(360)$ $N_c = 80,400 > 80,000 \quad \sim \text{OK}$
Step 3 - Final Design Strength	
Strength as controlled by steel.	$N_s = 129,000 \text{ kN} > 80,000 \quad \sim \text{OK}$
Strength as controlled by adhesive bond.	$N_c = 80,400 > 80,000 \quad \sim \text{OK}$
Final Design.	2 - 16 mm anchors embedded 360 mm

**Table 7.2 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.	65 ⁽⁴⁾	65 ⁽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 Short Bridges. ⁽⁵⁾	50	50
Long Bridges. ⁽⁵⁾	65 ⁽⁴⁾	65 ⁽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 15-mm allowance for milling.
(5) See Short and Long Bridge definitions in Chapter 10.

Table 7.3 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."

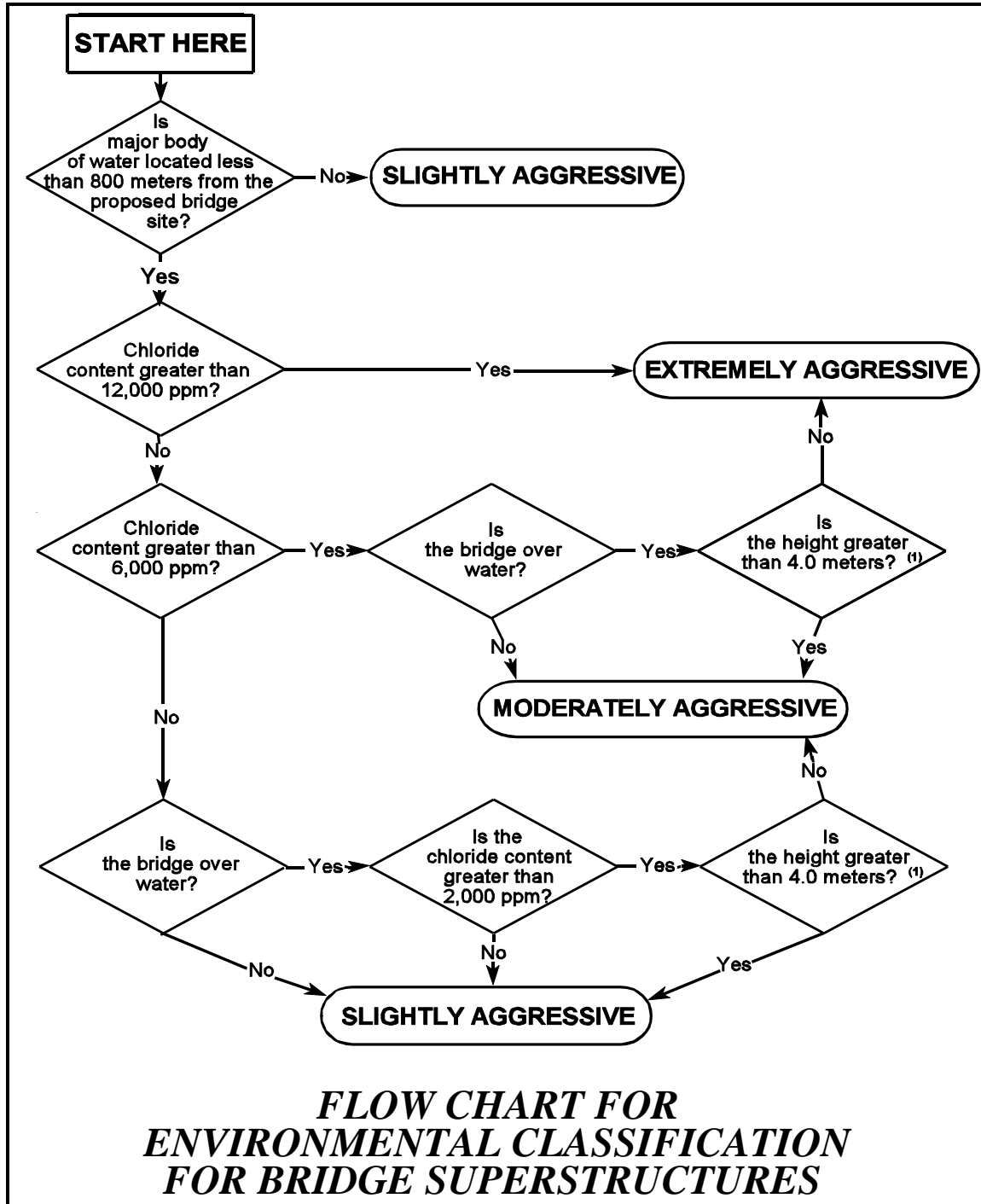


Figure 7-1

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).

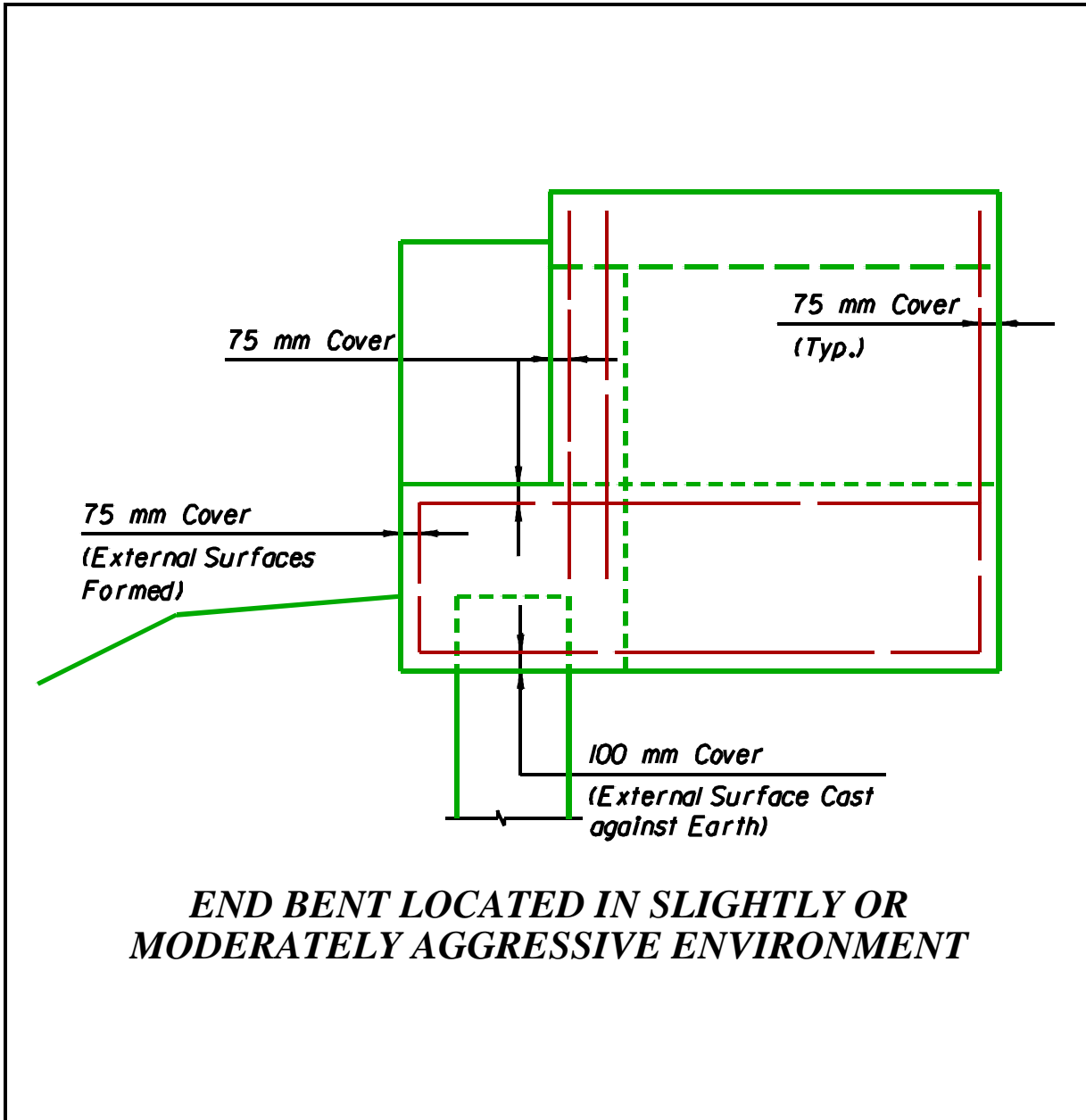


Figure 7-2

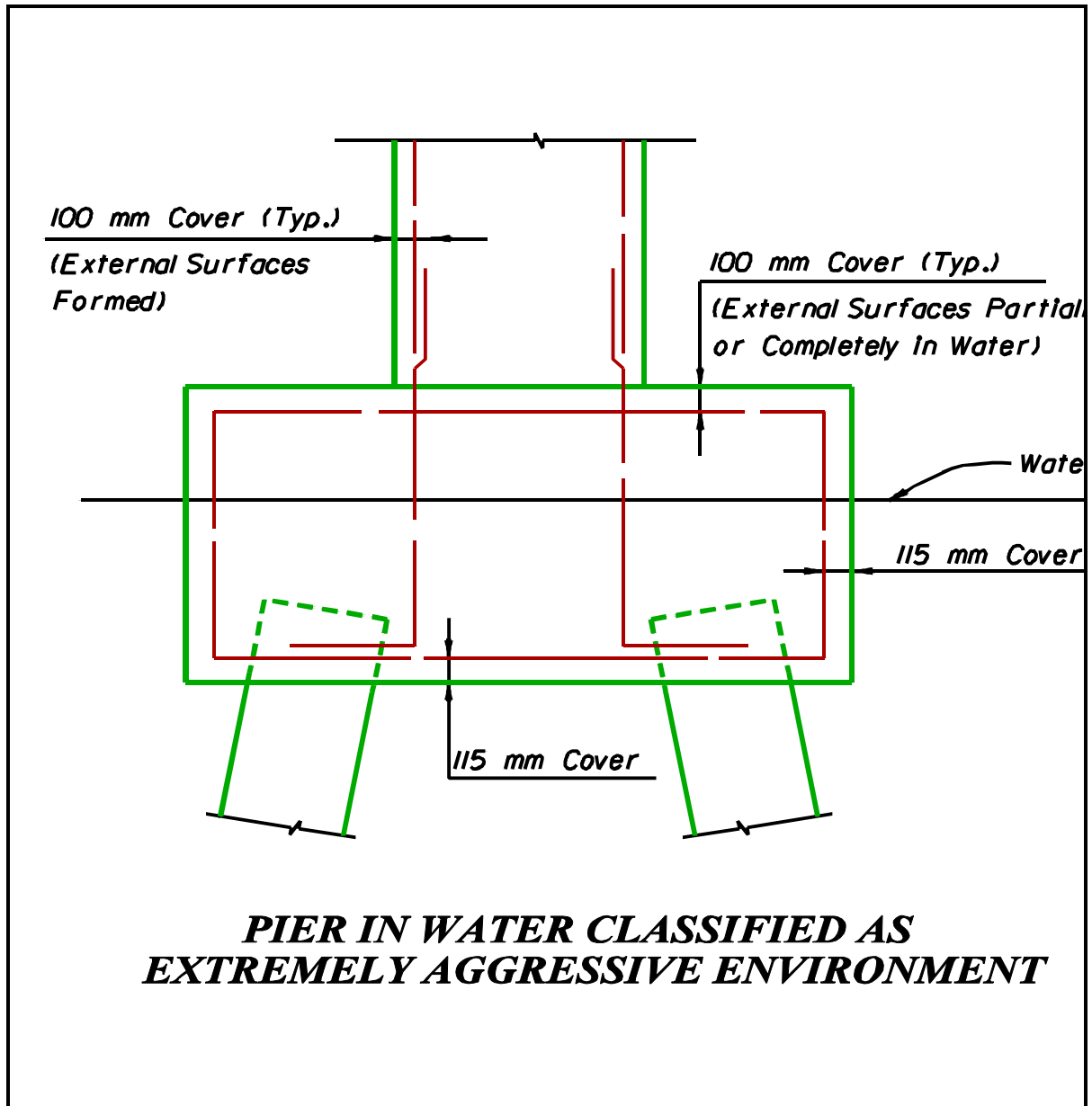


Figure 7-3

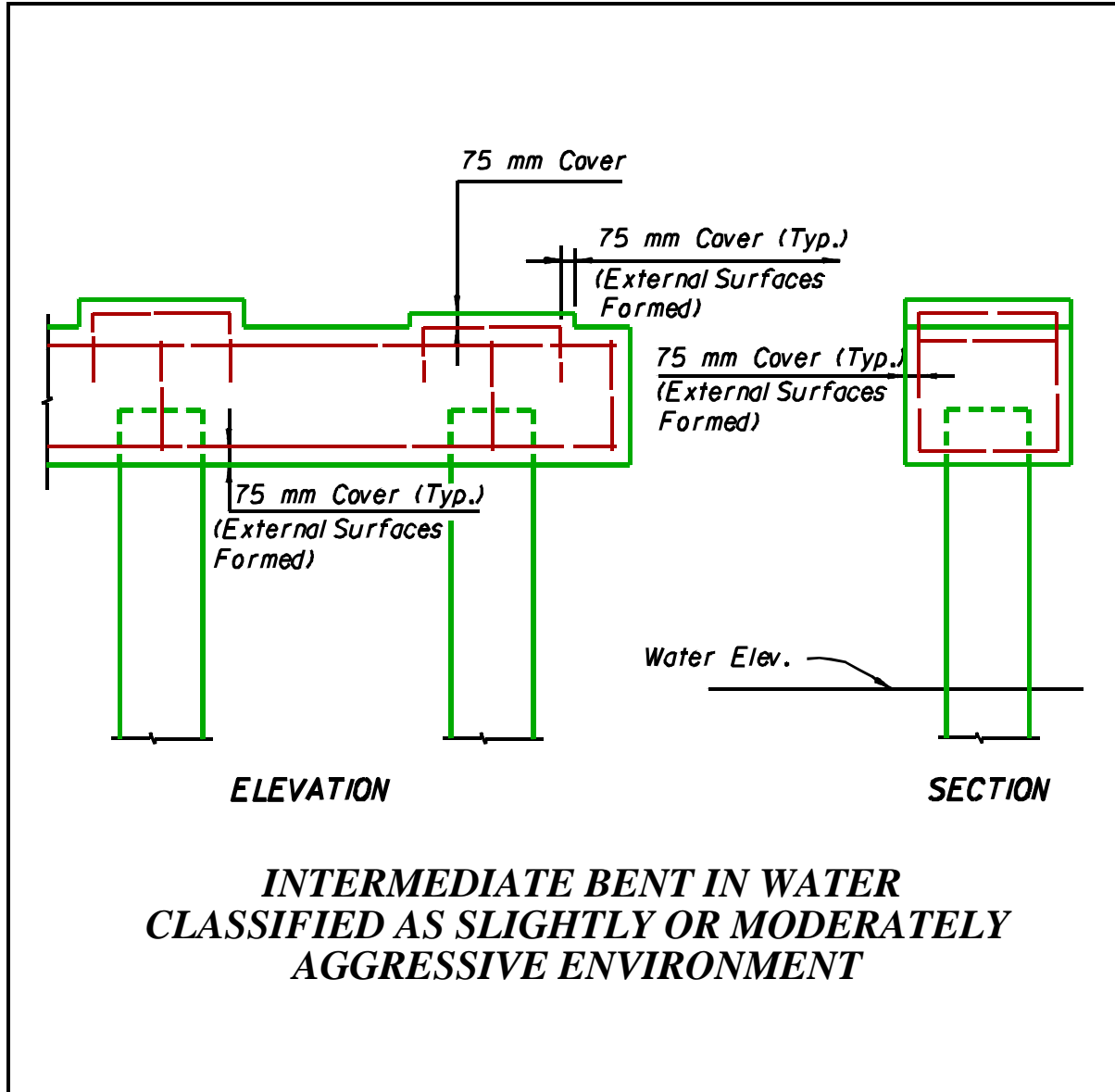


Figure 7-4

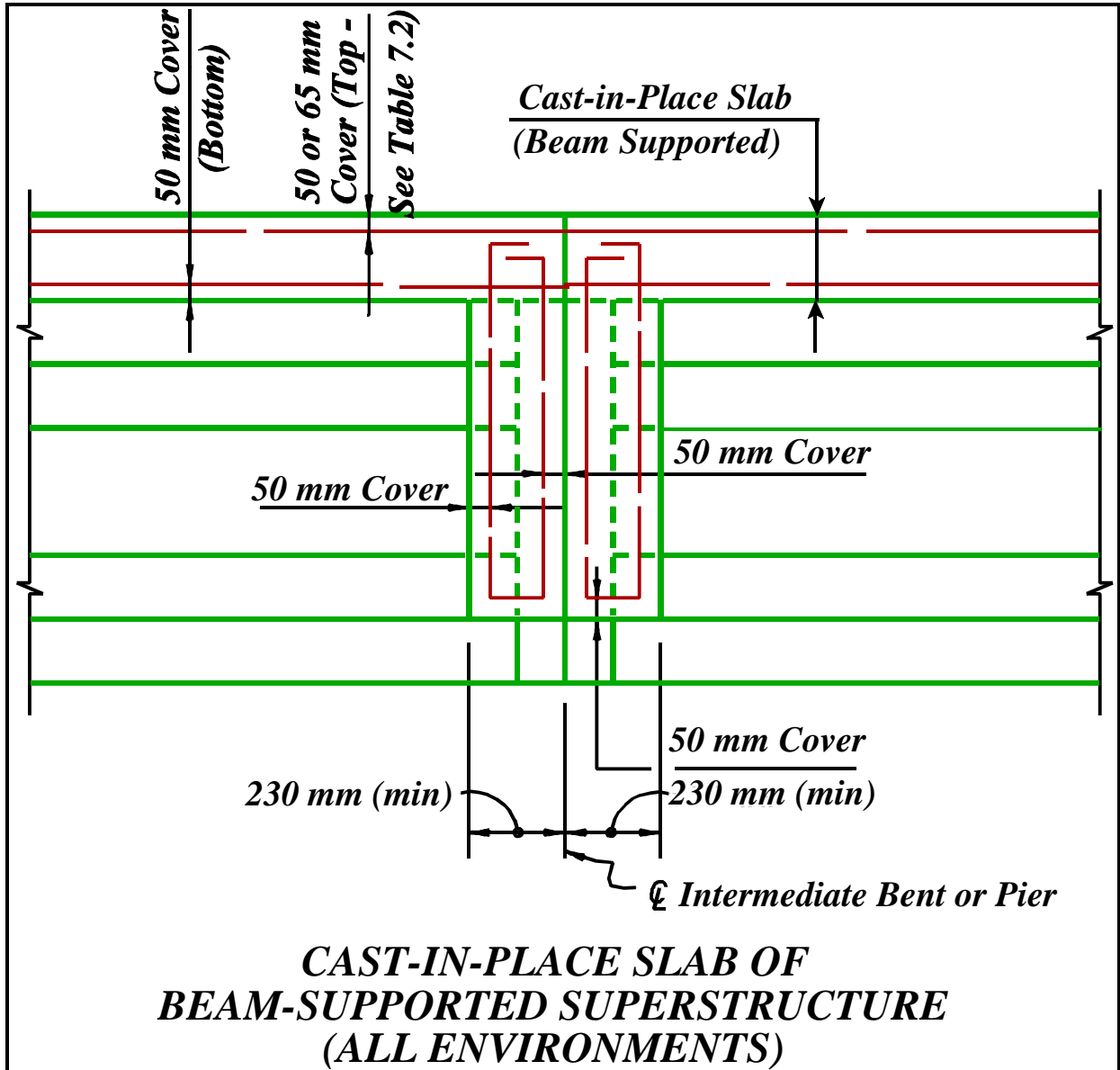


Figure 7-5

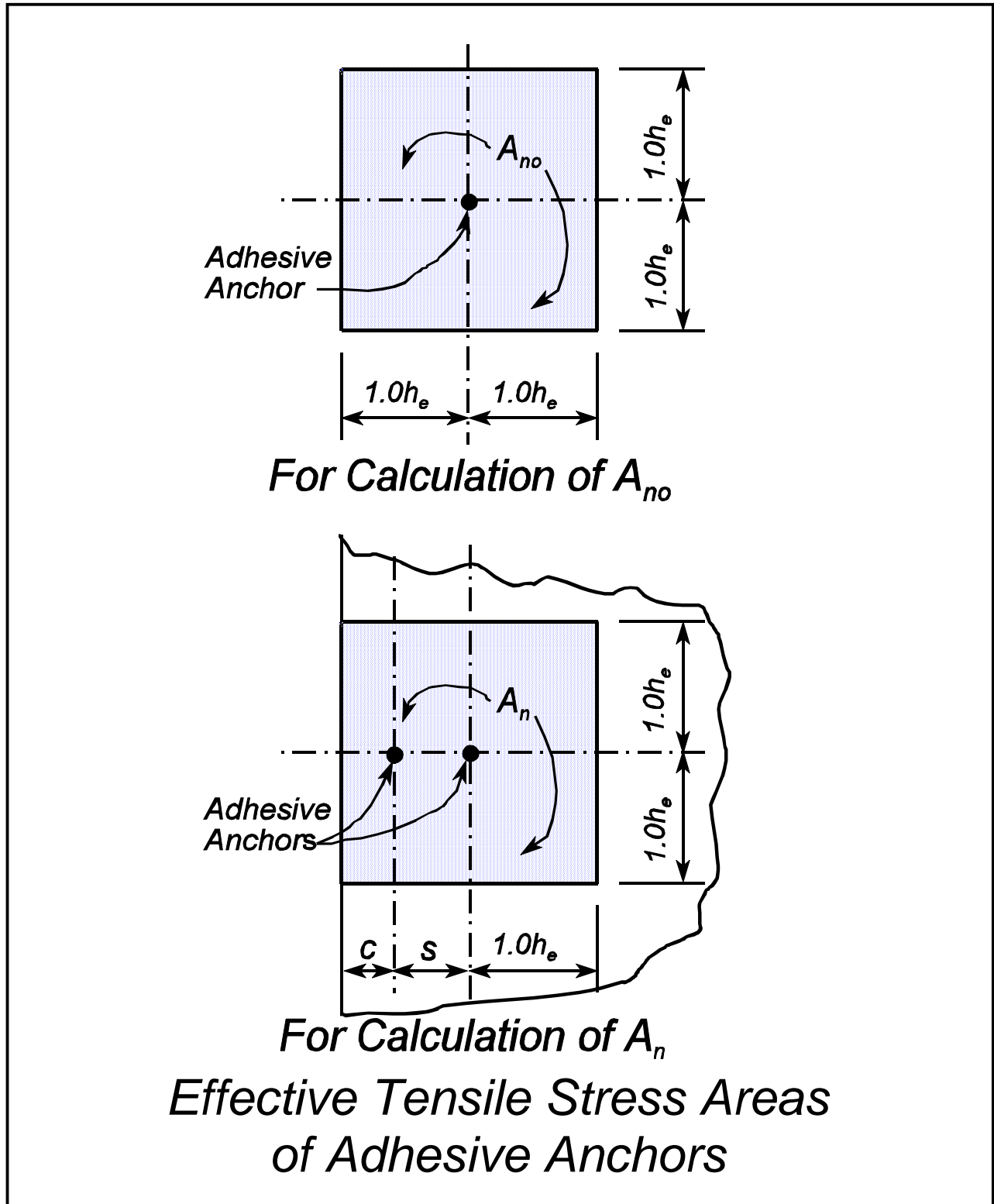


Figure 7-6

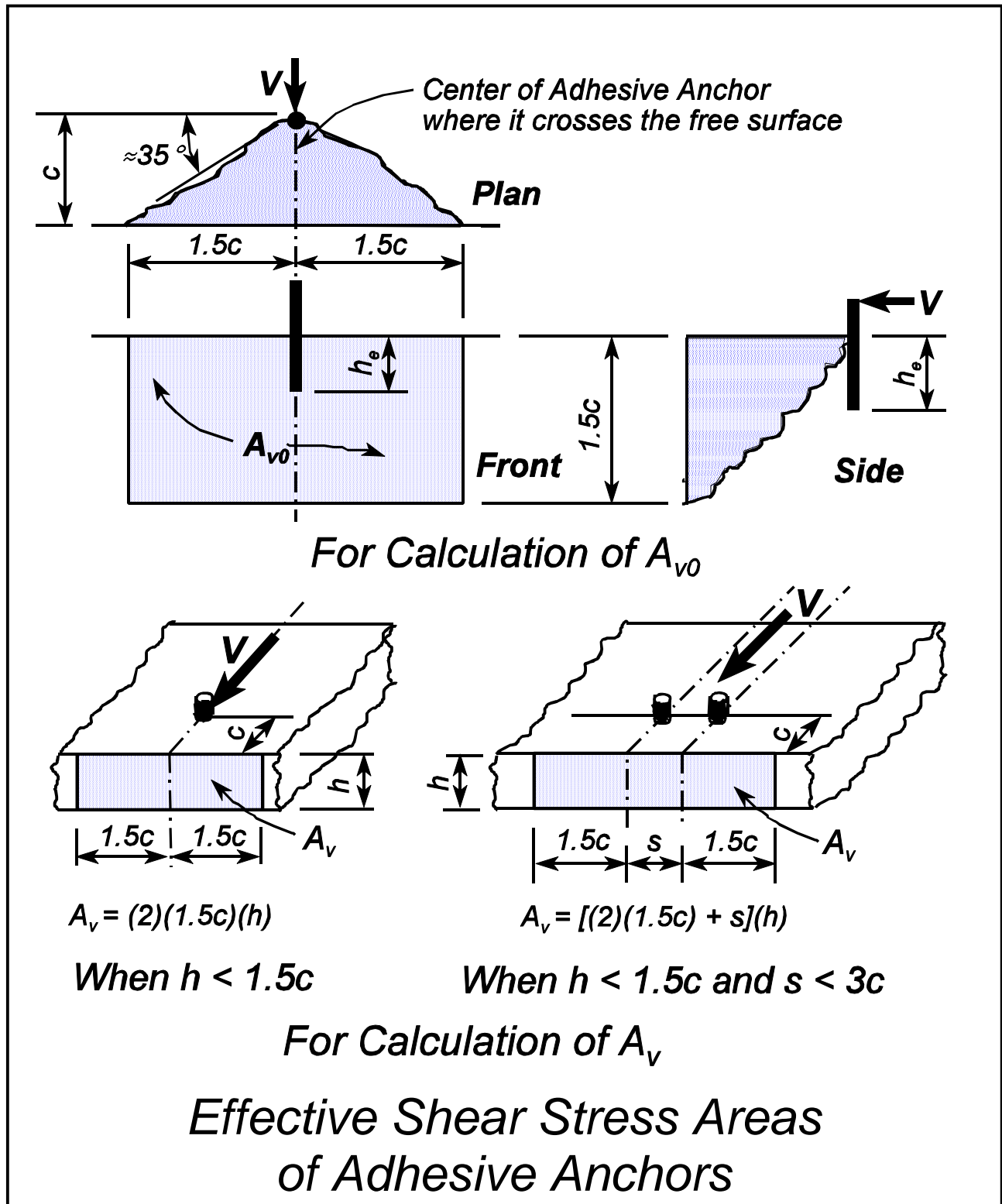


Figure 7-7

Chapter 8

DESIGN CRITERIA FOR SEGMENTAL BOX GIRDER BRIDGES

8.1 Introduction

The design criteria included in this chapter apply to the design of precast or cast-in-place segmental concrete bridges for the State of 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 AASHTO guide spec referenced below, Article 29.1.1, Method A. Specific requirements regarding post-tensioning duct size and alignment and criteria for box girder height are described in Chapter 10. Additionally, the inspectability requirements of Chapters 16 & 26 of the *Plans Preparation Manual*, Volume I (Topic No. 625-000-005), shall be considered when selecting the depth of, and detailing for, concrete box girders.

8.2 Specifications

The *Guide Specifications for Design and Construction of Segmental Concrete Bridges*, AASHTO, 1989 edition, and subsequent interim Specifications shall apply except where modified by the FDOT *Standard Specifications for Road and Bridge Construction*, latest edition and supplements thereto; the FDOT *Supplemental Special Provisions for Segmental Bridges and Post-Tensioning*; and the contents of this document. The AASHTO *Standard Specifications for Highway Bridges*, latest edition, shall be consulted for items not specifically addressed in the four referenced documents.

8.3 Design Loading

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

8.3.1 Thermal Effects

A. Normal Mean Temperature

For detailing purposes, the normal mean temperature taken from Figure 8-1 for the project's location shall be used.

B. Seasonal Variation (expansion - contraction)

1. For the purposes of design, the mean and range values taken from

Figure 8-1 shall be used. The expansion joint provisions of Chapter 12 shall also apply.

2. Thermal Coefficient (see Chapter 12).
3. The temperature setting variations for bearings and expansion joints shall be stated on the bridge plans.

C. Differential Temperature

The Differential Temperature shall conform to the requirements of the AASHTO ***Guide Specifications, Thermal Effects in Concrete Superstructures***, 1989, the AASHTO ***Guide Specifications for Design and Construction of Segmental Concrete Bridges***, and as amended herein.

Thermal gradients shall be applied **only** at service load combinations, and when combined with live load plus impact (LL+I), a beta factor of 0.5 will be applied to the gradient load.

8.3.2 Creep and Shrinkage

Creep and shrinkage strains and effects shall be calculated in accordance with Section 7.5 of the AASHTO ***Guide Specifications for Design and Construction of Segmental Concrete Bridges***. The following values shall apply.

- A. For all areas (inland and coastal), the relative humidity shall be taken as 75%.
- B. The secant modulus of elasticity at 28 days shall be taken from Table 4.3.

8.3.3 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 box cross-sections due to the lateral component of forces that result from plan curvature. All such secondary effects from prestressing must be properly considered.

B. Deck Slab

All box girder deck slabs 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 center of gravity of the prestressing steel (c.g.s.) within the duct. Critical eccentricities over the webs and at the centerline of the box shall be reduced 6mm from the theoretical value to account for adverse construction tolerances.

C. Tendon Geometry

When coordinating design calculations with detail drawings, the EOR shall take into account the fact that the center of gravity of the duct (c.g.) and the c.g.s. are not necessarily coincidental.

D. 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.

8.4 Material

8.4.1 Concrete

The minimum 28-day cylinder strength of concrete for precast pier stems and superstructure components shall be a minimum of 38 MPa.

8.4.2 Reinforcement

Reinforcing steel shall be ASTM A615M-96a, Grade 420, unless otherwise required.

8.4.3 Concrete Cover to Reinforcement

Concrete cover shall be in accordance with Table 7.2.

8.4.4 Post-Tensioning Steel

- A. Strand shall be ASTM A416M, Grade 1860, low relaxation.
- B. Parallel wires shall be ASTM A421M, Grade 1655.
- C. Bars shall be ASTM A722M, Grade 1035.
- D. Prestressing Parameters:

1. Apparent modulus of elasticity (strand): 190,000 MPa
2. Modulus of elasticity (parallel wires & bars): 200,000 MPa
3. Anchor set (to be verified during construction):
 - a. Strand: 15mm
 - b. Parallel wires: 15mm
 - c. Bars: 0
4. Level of stress in strands:
 - a. Maximum jacking stress:
 - (1) Stress Relieved Strands (S.R.S.): 0.90 f*y
 - (2) Low Relaxation Strands (L.R.S.): 0.90 f*y
 - b. Maximum anchoring stress:
 - (1) S.R.S.:
 - (A) At Anchor 0.70 f's
 - (B) At End of Seating Zone 0.71 flls
 - (2) L.R.S.:
 - (A) At Anchor 0.70 f's
 - (B) At End of Seating Zone 0.75 flls
 - c. Level of stress in wires and bars:
 - (1) Maximum jacking stress: 0.90 f*y
 - (2) Maximum anchoring stress: 0.70 flls

8.5 Epoxy Jointing of Segments

All precast segmental bridge structures shall contain epoxy joints between units. This requirement applies to substructure and superstructure units regardless of structure type or method of assembly.

8.6 Expansion Joints

8.6.1 Design

Expansion joints shall be designed for the full range of movement anticipated due to creep, shrinkage, elastic shortening, and temperature effects. The provisions of Chapter 12 shall also apply.

8.6.2 Settings

The setting of expansion joint recesses and expansion joint devices, including any precompression, shall be clearly stated on the drawings. Expansion joints shall be sized and set at time of construction for the following conditions:

- A. Allowance for opening movements based on the total anticipated movement resulting from the combined effects of creep, shrinkage, and temperature rise and fall. Creep and shrinkage shall be computed from the time the expansion joints are installed through day 4000. Temperature rise and fall shall be based on 120% of the maximum value given in Figure 8-1. All expansion joints shall be installed after superstructure segment erection and deck profiling is completed for the entire bridge. A note stating this requirement shall be placed on plans.
- B. To account for the larger amount of opening movement, expansion devices should be set precompressed to the maximum extent possible. Calculations shall allow for an assumed setting temperature of 29EC. The EOR shall provide a table on the plans giving precompression settings according to the prevailing conditions. Expansion devices shall be sized and set to remain in compression through the full range of design temperature from their initial installation until a time of 4,000 days.
- C. A table of setting adjustments shall be provided to account for temperature variation at the time of installation. The table shall indicate the ambient air temperature at time of installation, and adjustments shall be calculated for the difference between the ambient air temperature and the mean temperature given in Figure 8-1.

8.6.3 Armoring

Concrete corners under expansion joint devices shall be provided with adequate steel armoring to prevent spalling or other damage under traffic. It is recommended that the armor be at least 102x102x15.9 galvanized angles anchored to the concrete with welded studs or similar devices. Horizontal concrete surfaces supporting the expansion joint device and running flush with the armoring shall have a finish acceptable for the device. All armoring

shall have adequate vent holes to assure proper filling and compaction of the concrete under the armor.

8.7 Erection Schedule and Construction System

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

8.8 Construction Data Elevations and Camber Curve

A. General

Construction data elevations shall be based on the vertical and horizontal highway geometry.

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(s).

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.

8.9 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.

8.10 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, anchorage 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.

8.11 Special Requirements for Post-Tensioning Anchorages in Slabs

Post-tensioning terminuses 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.

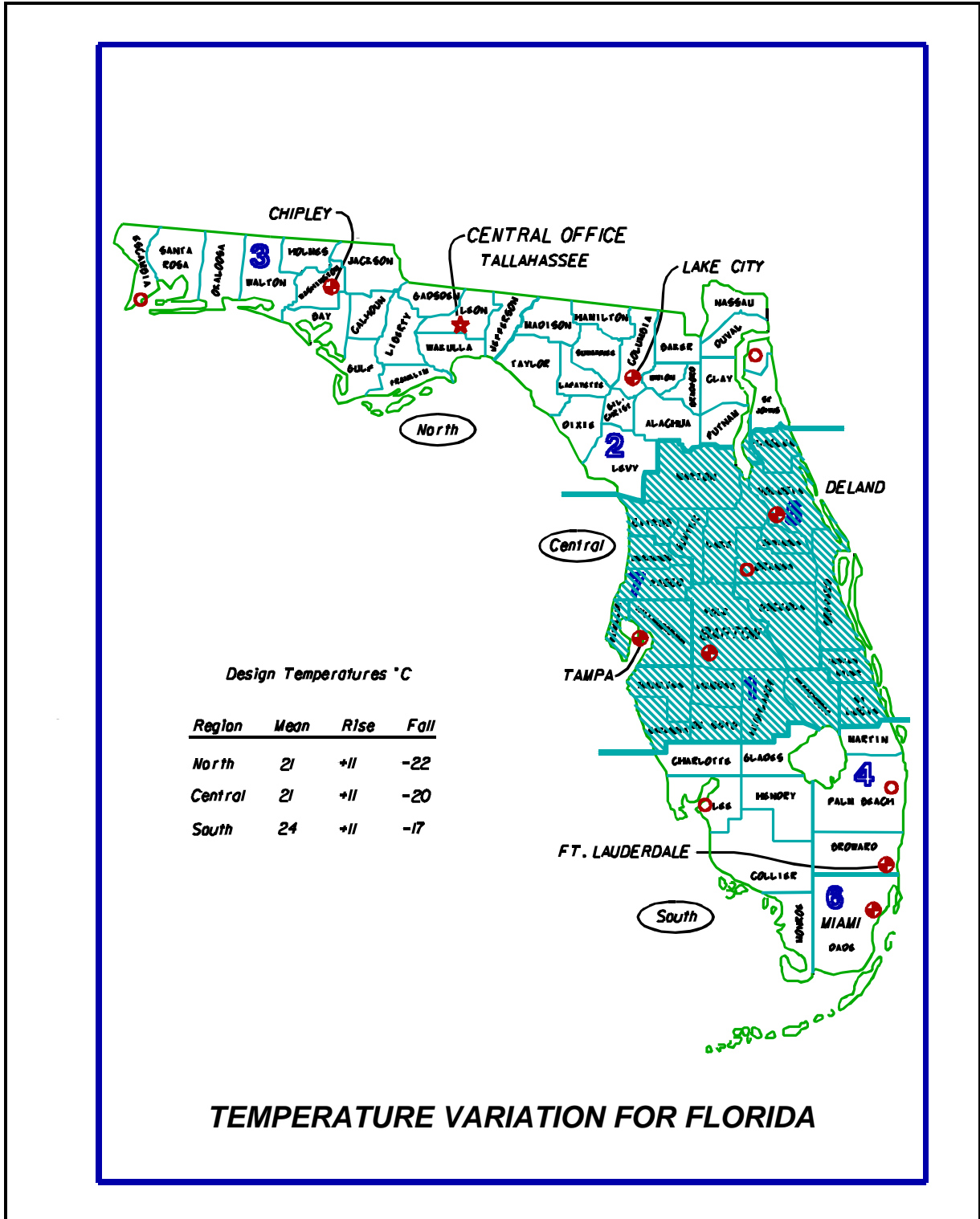


Figure 8-1

Chapter 9

VESSEL COLLISION

9.1 General

The design of all bridges over navigable waters must include consideration for possible Vessel Collision. Such collisions generally occur from barges or oceangoing ships. The Engineer of Record (EOR) shall conduct a vessel risk analysis to determine the most economical method for protecting the bridge. This shall include either designing the bridge to withstand Vessel Collision, or protecting it with dolphins and/or fenders. The Risk Analysis is conducted utilizing the Department's Mathcad-based computer program, "Vessel Impact Risk Analysis." The vessel configurations are also furnished for most sites. After conducting the Risk Analysis, the EOR shall utilize the AASHTO ***Guide Specification and Commentary for Vessel Collision Design***, Volume I, February 1991 and shall also comply with the procedure described hereinafter.

9.2 Research and Information Assembly (When not provided by the Department)

9.2.1 Data Sources

- A. U.S. Army Corps of Engineers, Waterborne Commerce Statistics Center, P.O. Box 61280, New Orleans, LA 70161. Telephone: (504) 862-1472.
- B. U.S. Army Corps of Engineers, "Waterborne Commerce of the United States (WCUS), Parts 1 & 2," Water Resources Support Center (WRSC), Fort Belvoir, VA.
- C. U.S. Army Corps of Engineers, "Waterborne Transportation Lines of the United States," WRSC, Fort Belvoir, VA.
- D. U.S. Army Corps of Engineers (COE), District Offices.
- E. U.S. Coast Guard, Marine Safety Office (MSO).
- F. Port Authorities and Water Dependent Industries.
- G. Pilot Associations and Merchant Marine Organizations.
- H. National Oceanic and Atmospheric Administration (NOAA), "Tidal Current Tables; Tidal Current Charts and Nautical Charts," National Ocean Service, Rockville, Maryland.

- I. Bridge tender record for bascule bridge at the District Maintenance Office.
- J. Local tug and barge companies.

9.2.2 Assembly of Information

The EOR shall assemble the following information.

- A. Characteristics of the waterway including the following:
 - 1. Nautical chart of the waterway.
 - 2. Type and geometry of bridge.
 - 3. Preliminary plan and elevation drawings depicting the number, size and location of the proposed piers.
 - 4. Navigation channel, width, depth and geometry.
 - 5. Average current velocity across the waterway.
- B. Characteristics of the vessels and traffic including the following
 - 1. Ship, tug and barge sizes (length, width and height)
 - 2. Number of passages for ships, tugs and barges per year (last five years and prediction to end of 25 years in the future).
 - 3. Vessel displacements.
 - 4. Cargo displacements (deadweight tonnage).
 - 5. Draft (depth below the waterline) of ships, tugs and barges.
 - 6. The overall length and speed of tow.
- C. Accident reports.
- D. Bridge Importance Classification.

9.3 Design Vessel

When utilizing the FDOT's Mathcad software for conducting the Vessel Collision risk analysis, a "Design Vessel," which represents all the vessels, is not required. The software computes the risk of collision for several vessel groups with every pier. When calculating the geometric

probability, the overall length of each vessel group (LOA) is used instead of the LOA of a single "Design Vessel."

9.4 Design Methodology - Damage Permitted

In addition to utilizing the general design recommendations presented in AASHTO's **Guide Specification and Commentary for Vessel Collision Design**, Volume I, the EOR shall also use the following design methodology:

- ! At least one iteration of secondary effects in columns shall be included; i.e., axial load times the initial lateral deflection.
- ! 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.
- ! 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 5. 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.
- ! 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.
- ! For the designer's Vessel Collision risk analysis, the FDOT will determine whether a bridge is critical or non-critical. A list is provided with the Department's software.

9.5 Pile Bents

Pile bents subject to minor vessel impact shall be designed to remain structurally adequate with any one pile removed and live load applied only within the designated, striped traffic lanes. The most critical Live Load type, pattern, and transverse location shall be used in considering the loading of one or more lanes. No direct impact analysis is required. The EOR shall use the Strength Design (Ultimate Strength) method for the following load combination:

$$L_{pr} = 1.3[DL + (LL + I)] \quad [Eq. 9-1]$$

Where:

$$\begin{aligned} L_{pr} &= \text{Load Combination with single pile removed} \\ DL &= \text{Design Dead Load} \end{aligned}$$

LL = Critical Live Load described above
 I = Live Load Impact

9.6 Widenings

Bridge structures that span over navigable waterways and that are classified as major widenings in accordance with Chapter 16 shall be designed for Vessel Collision. Bridge structures that span over navigable waterways and that are classified as minor widenings in accordance with Chapter 16 shall be considered for Vessel Collision design requirements on an individual basis.

9.7 Movable Bridges

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

9.8 Main Span Length

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.

9.9 Scour with Vessel Collision

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:

9.9.1 Load/Scour Combination 1

$$LS_{(1)} = \text{Vessel Collision} + \frac{1}{2} \text{Long-Term Scour} \quad [\text{Eq. 9-2}]$$

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).

9.9.2 Load/Scour Combination 2

$$LS_{(2)} = \text{Minimum Impact Vessel} + \frac{1}{2} \text{100-Year Scour} \quad [\text{Eq. 9-3}]$$

Where:

Min. Impact Vessel: Defined in AASHTO ***Guide Specification and Commentary for Vessel Collision Design*** (with related collision speed).

100-Year Scour: Defined in Chapter 4, FDOT ***Drainage Manual*** (Topic No. 625-040-001).

9.9.3 Criteria for Developing Soil Model

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.

9.10 Application of Impact Forces

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 9-1.

9.11 Impact Forces on Superstructure

Vessel Impact Forces on the superstructure shall be applied in accordance with the AASHTO ***Guide Specification and Commentary for Vessel Collision Design***.

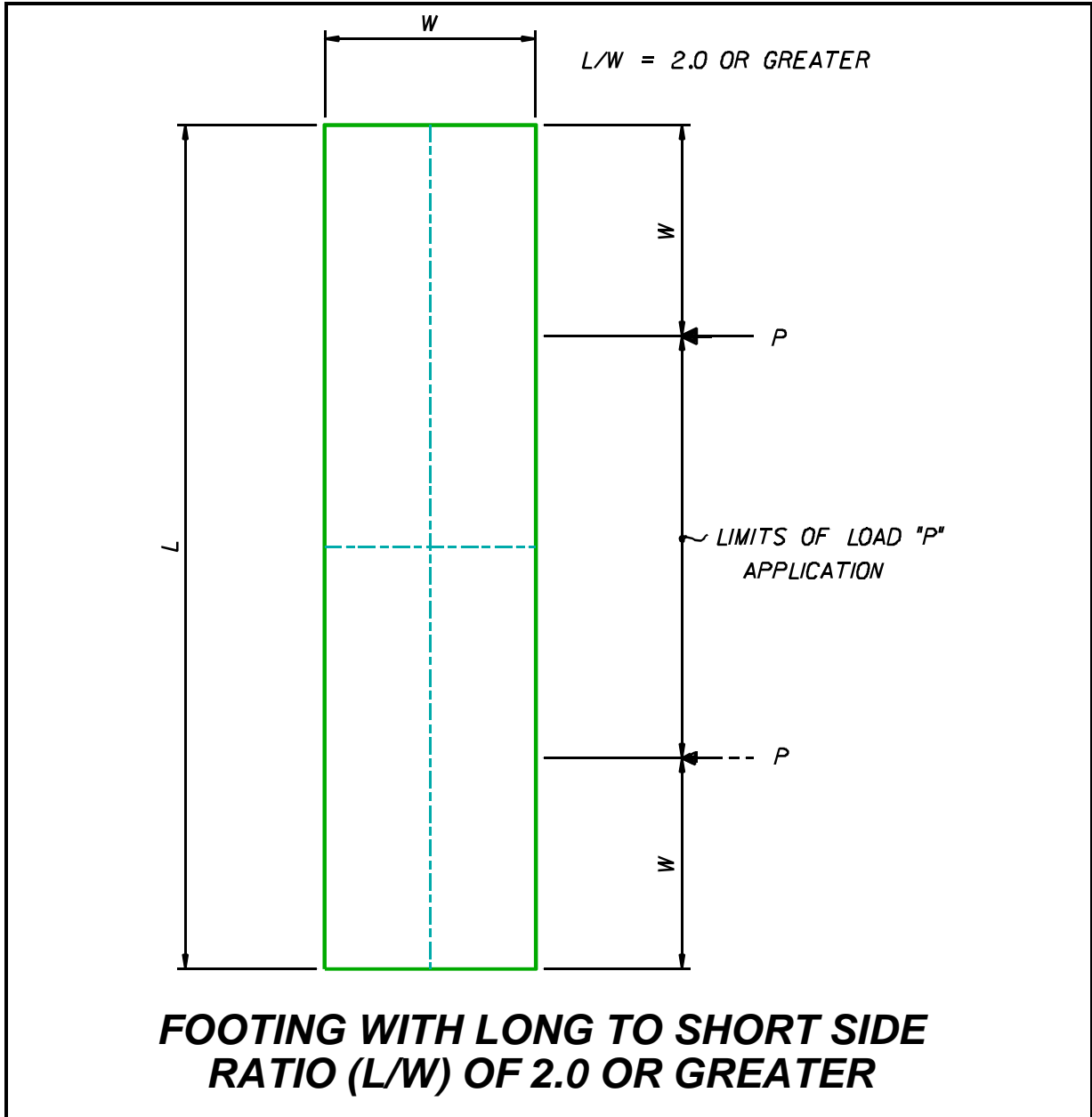


Figure 9-1

Chapter 10

SUPERSTRUCTURE AND APPROACH SLAB DESIGN

10.1 Curb Heights on Bridges

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

10.2 Pilaster Details for Roadway Lighting

See the *Standard Drawings* for details to be used in designing bridges requiring light standards.

10.3 Intermediate Diaphragms for Prestressed Concrete I-Girder Construction

Where the skew angle is greater than 30°, provide diaphragms per AASHTO, Article 8.12. Where the skew angle is equal to or less than 30°, intermediate diaphragms are not required.

10.4 Cast-in-Place Bridge Decks

10.4.1 Bridge Length Definitions for Deck Thickness and Finishing Requirements

For the purpose of establishing profilograph and deck thickness requirements, bridge structures shall be defined either as Short Bridges or Long Bridges. The determining length shall be the length of the bridge structure measured along the Profile Grade Line (PGL) of the structure and shall include the lengths of exposed concrete riding surface of the approach slabs. Based upon this established length, the following definitions shall apply:

- ! Short Bridges: Bridge structures less than or equal to 100 meters in PGL length.
- ! Long Bridges: Bridge structures more than 100 meters in PGL length.

10.4.2 Deck Thickness Determination

The thickness of bridge decks that are cast-in-place (CIP) on beams or girders of bridges meeting the definition of Long Bridges shall be 215 mm minimum for all new construction. The 215 mm deck thickness includes a 15 mm additional, sacrificial thickness to be included in the dead load of the deck slab but which is omitted from its section properties. The thickness of CIP bridge decks on beams or girders of bridges that meet the definition of Short Bridges shall be 200 mm minimum for all new construction.

The thickness of CIP bridge decks on beams or girders for widenings defined as Major Widenings in accordance with Chapter 16 shall be 200 mm. However, whenever a Major Widening is selected by the Department to meet profilograph requirements, a minimum deck thickness of 215 mm, meeting the requirements and design methodology for new construction of the preceding paragraph, shall be used.

The thickness of CIP bridge decks on beams or girders for widenings defined as Minor Widenings in accordance with Chapter 16 shall be handled on an individual basis but generally shall match the thickness of the adjoining existing deck.

The thickness of all other CIP or precast concrete bridge decks shall be based upon the reinforcing cover requirements of Chapter 7, Table 7.2.

The determining concrete deck thickness from the preceding requirements shall be utilized in establishing bearing elevations by deducting the determined thickness, before milling, from the Finish Grade Elevations required by the Contract Drawings.

10.4.3 Temperature Reinforcing

Temperature reinforcing for all cast-in-place bridge decks, including flat slabs, shall be No. 13 Bars spaced at not more than 300 mm.

10.4.4 Reinforcing Steel over Intermediate Piers or Bents

When cast-in-place 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 No. 16 Bars placed between the longitudinal temperature bars, shall be not less than 12.0 meters in length, and shall be placed symmetrically about the centerline of the pier or bent.

10.4.5 Transverse Reinforcement

When the end of the slab is skewed 15 degrees or less, the required transverse reinforcement shall be placed parallel to the skew for the entire length of the slab.

When the end of the slab is skewed more than 15 degrees, the required transverse reinforcement shall be placed perpendicular to the longitudinal reinforcement. 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 those end areas shall be increased by 25 percent. In addition, four No. 13 Bars, full-width, shall be placed parallel to the end skew in the top of each end of the slab and spaced at 100 mm.

10.4.6 Length of End Spans

When practicable, to accommodate standard slab support systems, the end spans of multiple-span, cast-in-place, flat slab superstructures shall be sized such that the clear, horizontal dimension from the face of the end bent to the face of the first intermediate bent is identical to that same clear, horizontal dimension between intermediate bents.

10.5 Grooving Bridge Decks

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

Item No. 400-7 - Bridge Floor Grooving - Sq. Meters

The area of Bridge Floor Grooving shall be determined as the area bounded by the gutter lines (at barrier rails, curbs, or median dividers) and the beginning and end of the bridge, or by the ends of approach slabs for those approaches, or portions thereof, that will not receive bituminous surfacing.

For widened superstructures where at least one traffic lane is to be added, the entire deck area within the bounds described in the preceding paragraph shall have a grooved finish. If the existing structure has a grooved deck finish, then only the widened portion is required to receive Bridge Floor Grooving.

For projects with shoulder widening only, a note should be added to the plans specifying that the bridge floor finish shall match that of the existing bridge. The SDO shall be contacted to determine the required bridge floor finish for unusual situations such as a poor riding surface condition of an existing bridge deck that is to be widened.

10.6 Prestressed Pretensioned Components, Beam Design, and Construction

Prestressed, pretensioned design and construction, herein referred to more commonly as "prestressed," shall conform with the requirements of this Article:

10.6.1 General Design Requirements for Prestressed Construction

- A. Unless otherwise specified or approved by the Department, the center-to-center spacing of all strands, except in piling, shall be 50 mm. The spacing of strands in piling varies with the number of strands required but shall not be less than 50 mm.
- B. Stress computations resulting from prestressing shall be based upon the following lengths of strand transfer and development:
 - 1. For all prestressed components except piling:
 - a. Transfer length
 - (1) Fully bonded strands = 80 strand diameters
 - (2) Debonded strands = 100 strand diameters (from the terminus of the debonding material).
 - b. Development length
 - (1) Fully bonded strands = 1.6 times AASHTO 9.28.1
 - (2) Debonded strands per AASHTO 9.28.1 and 9.28.3
 - 2. For prestressed piling not subjected to significant flexure under service or impact loading, strand development shall be in accordance with AASHTO 9.28.1. Bending that produces cracking in the pile is considered significant.

Prestressed piling subject to significant flexure (such as that resulting from vessel collision loading) and embedded a minimum of 1.2 meters into reinforced, cast-in-place concrete pile caps are to be considered capable of developing their full ultimate moment capacity without strand slippage (Reference: *PCI Journal*, Vol. 37, No. 6, November/December 1992, pp. 42-59).

10.6.2 Requirements for Prestressed Beams

The use of ASTM A416M, Grade 1860, low-relaxation, straight, prestressing strands is **preferred** for the design of prestressed, pretensioned beams. The requirements stipulated hereinafter apply to simply-supported, fully-pretensioned beams, either of straight or depressed (draped) strand profile, except where specifically noted otherwise.

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

Groupings of beam designs as far down this priority list as possible will help maximize casting bed usage and minimize variations in materials and stranding.

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

- B. 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. In the end regions of the beam, a maximum of 25 percent of the strands may be debonded to satisfy the allowable stress limits; however, debonded strands shall not exceed 37.5 percent of the total strands in any one row. In applying these percentage limitations the following methodology shall be used:
 - a. Strands shall be debonded in a pattern that is symmetrical about the vertical axis of the beam.
 - b. The theoretical number of debonded strands shall be rounded to the closest even number (pairs) of strands except that debonded strands will not be permitted in rows containing three (3) strands or less. It is the FDOT's preference not to debond strands located on the vertical centerline of beams so as to avoid any potential restriction of freedom of movement of the debonded

strands by the closely-spaced, side-alternating stirrups near beam ends. Therefore, debonding of strands located on beam centerlines must be specifically requested by the EOR and approved by the appropriate FDOT plans review office for Category 1 or 2 bridge projects.

- c. Not more than 4 strands, or 40% of the total shielded strands, whichever is the greater, shall be terminated at any section. The number of shielded strands being terminated at the section under consideration shall include those for which the strand force has not yet been fully transferred.
 - d. The exterior strands adjacent to side forms in any and all rows shall be fully bonded.
 - e. **Shielded strands shall be distributed laterally throughout the strand pattern as uniformly as practical.** Whenever possible, shielded strands shall be separated in all directions by at least one fully bonded strand.
- C. The critical stresses at release in the end regions of the beam may be neglected within a distance of 75 percent of the beam height from the centerline of bearing for top of beam tensile stresses, and within a distance from the end of the beam equal to the fully bonded strand transfer length for bottom of beam compressive stresses. Stresses beyond these distances shall comply with the requirements stated hereinafter.
- D. 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 strands only):
 - a. Outer 15 percent of design span: $f_t = 0.966\sqrt{f'c_i}$
 - b. Center 70 percent of design span: $f_t = 0.498\sqrt{f'c_i}$
 - 2. Tension at top of beam at release (depressed strands only):
 - a. Full length of design span: $f_t = \frac{0.498\sqrt{f'c_i}}{0.498\sqrt{f'c_i}}$
 - 3. Compression at bottom of beam (at release): $f_b = 0.6f_{ll}c_i$
 - 4. Compression at top of beam (service load) = AASHTO (1996), Article 9.15.2.2, "Stress at Service Load After Losses Have Occurred."

5. Tension at bottom of beam (service load):
 - a. Slightly or moderately aggressive environment: $f_t = 0.498\sqrt{f'_{ci}}$
 - b. Extremely aggressive environment: $f_t = 0.249\sqrt{f'_{ci}}$

- E. Reinforcement shall be provided in the top flange of all beams to control tensile cracks prior to, during, and after release of the prestressing force. The reinforcement shall consist of a combination of No. 16, ASTM A615M, Grade 420, deformed reinforcing bars and No. 9 mm (min.), ASTM A416M, Grade 1725 or 1860 prestressing strands initially tensioned to 44.5 kN each. The number of bars and strands varies with the beam type as shown in the **Standard Drawings**. Because most of the initial tension in the strands is lost prior to the application of the service live load, their effect on beam stresses after release and under service load may be ignored. The No. 16 reinforcing bars in each end of the beam, called "Bars A" on the **Standard Drawings**, shall be a minimum of 3.0 meters or 15 percent of the design span in length, whichever is greater.

- F. The strands in the bottom flange shall be confined for a distance from the beam end equal to one and one-half (1.50) times the depth of the beam. The confinement shall be provided by enclosed No. 10 deformed ties, ASTM A615M, Grade 420, spaced 150 mm center-to-center as shown in the **Standard Drawings**. The ties shall be of standard dimensioning and shape for a given beam type regardless of strand pattern. For straight strand designs only, the requirement of AASHTO 9.22.1 to provide 4% of the prestressing force as vertical stirrups at a unit stress of 90 kN is waived.

- G. "L-shaped" longitudinal bars are required in the webs and flanges in the end zone areas.

- H. Unless otherwise specifically directed or approved by the SDO, the vertical stirrup reinforcement for fully pretensioned beams shall be symmetrically spaced about the centerline of the beams. The stirrups shall be at least one No. 13 Bar and shall be spaced at not more than the following increments:

Table 10.1 Minimum Stirrup Requirements for Beams

Standard Beam Designation	Support to 1/4 of Span	1/4 of Span to Centerline of Span
Type II, III, IV, V, VI and all Bulb-Tee Beams	1 - No. 13 Bar at 150 mm o.c.	1 - No. 13 Bar at 300 mm o.c.

The preferred detailing practice is to provide several stirrups at a transition

spacing of 225 mm past the 1/4-Span to avoid an abrupt change in the stirrup spacing. This also provides a smooth transition when considering the effect of the stirrup's extension into the cast-in-place deck slab.

- I. The minimum compressive concrete strength at release shall be 28 MPa. Higher release strengths may be used and specified when required by the designer but generally should not exceed 31 MPa unless otherwise specifically approved by the SDO or DSDO.
- J. Prestressed beams shall be designed and specified to conform to one of the following classes and related strengths of concrete:

Table 10.2 Beam Concrete Classes and Strengths

Class of Concrete	28-Day Compressive Strength
Class III*	35 Mpa
Class IV	38 Mpa
Class V	45 MPa
Class VI	59 MPa

* Class III Concrete may be used only when the superstructure environment is classified as "slightly aggressive."

- K. A note shall be shown on the beam sheet in the General Notes stating "Optional stranding shall comply in all respects with the Department's **Structures Design Guidelines**."
- L. 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 4.
- M. 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 AASHTO, Article 9.16.2.
- N. Stress and camber calculations for the design of simple span, pretensioned components shall be based upon the section properties of the gross concrete section, only, neglecting any effect of mild reinforcing steel and/or prestressing strand. **The use of transformed section properties is not permitted.**

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 for the FDOT **Standard Specifications for Road and Bridge Construction**. When S-I-P metal forms are permitted, a note shall be added to the "General Notes" of the bridge plans stipulating the weight allowance included in the beam design for the stay-in-place metal forms and concrete required to fill the form flutes. Additionally, a note shall be added requiring all embedded items and accessories required for their use to be included on the Shop Drawing submittal for the forming system.

The design slab thickness shall be provided from the top of the stay-in-place metal form to the finished slab surface, and the estimated superstructure concrete quantity shall **not** include the concrete required to fill the form flutes.

10.7 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.

10.8 Precast Prestressed Double-Tee Beams

All bridge structures designed utilizing precast, prestressed double-tee beams shall conform with the design criteria and details provided in the **Standard Drawings**.

10.9 Stay-in-Place (SIP) Slab Forms

For stay-in-place metal and concrete forms see Articles 400-5.7 and 400-5.8 in the FDOT **Standard Specifications for Road and Bridge Construction**. SIP metal forms shall be designed and detailed for prestressed beam and steel girder superstructures in accordance with Chapter 10 and Chapter 11, respectively, except where restricted for use by Chapter 7.

10.10 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 drawing as well as the value of camber due to prestressing minus the dead load deflection of the beam.

10.11 Post-Tensioning Strand

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

10.12 Dimensional and Location Requirements for Post-Tensioning Ducts

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 (1/2) degrees 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 to avoid potential problems with cross-grouting of ducts.

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

10.13 Barrier/Railing Distribution

In lieu of AASHTO, Article 3.23, and in lieu of a more refined analysis and for superstructures not less than 12 nor more than 46 meters in length, the dead load of barriers and railings attributed to exterior beams or stringers may be determined by the following equation:

$$W_{ext} = \frac{(W)(C_1)(C_2)}{100} \quad [Eq. 10-1]$$

Where:

W_{ext} = Portion of the uniform dead load weight of the barriers or railings on the exterior beams (N/m).

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

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

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 (N/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 AASHTO, Article 3.23, or by individual analysis.

10.13.1 Example Problem 1:

(New Construction, 2-lane bridge)

$$L = 30.00 \text{ m}$$

$$K = 6$$

$$S = 2.44 \text{ m}$$

$$C_1 = 57.73$$

$$C_2 = 0.846$$

$$W = 6100 \text{ N/m}$$

$$W_{ext} = \frac{(6100)(57.73)(0.846)}{100} = 2979 \text{ N/m}$$

$$W_{int} = \frac{(2)(6100 + 2979)}{(6 + 2)} = 1561 \text{ N/m}$$

10.13.2 Example Problem 2:

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

$$W_{ext} = (2979)(0.75) = 2234 \text{ N/m (at exterior beam on barrier side)}$$

$$W_{int} = \frac{(6100 + 2234)}{(6 + 1)} = 773 \text{ N/m (all other beams)}$$

10.14 Design of Beams with Variable Spacings

In the design of simple span steel or concrete beams for the condition where the beam spacing at each end varies, and in lieu of a more refined analysis, the average beam spacing may be used for the beam design except for shear calculations as noted below.

For shear design only, and in lieu of a more refined analysis, the shear at the section under consideration, V_x , may be calculated as the shear at the section using the average beam spacing times the square root of the ratio of the actual beam spacing at the section divided by the average beam spacing such that:

$$V_x = V_{avg} \sqrt{\frac{\text{Beam Spacing at Section}}{\text{Average Beam Spacing}}} \quad [\text{Eq. 10-4}]$$

10.15 Prestressed Beam Standards and Semi-Standards

Prestressed beam standards and semi-standards for use in simple span applications are included in the **Standard Drawings** available through the Office of Maps and Publications Sales in Tallahassee and on the SDO's website in ".DGN" format.

10.16 Pretensioned/Post-Tensioned Beams

In the design of pretensioned beams made continuous by field-applied post-tensioning, the pretensioning shall be designed such that, as a minimum, the following conditions are satisfied:

- ! The pretensioning shall meet the minimum steel provisions of AASHTO Article 9.18.2.
- ! 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 limitations for pretensioned concrete construction.
- ! The pretensioning force shall be of such magnitude that the initial midspan camber at release, including the effect of the dead load of the beam, is at least 13mm. In computing the initial camber, the value of the modulus of elasticity shall be in accordance with Table 4.3 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, and beam lengths in which ducts deviate both horizontally and vertically, require integrated drawings in accordance with Chapter 8.
- ! The limitation on the percentage of debonded strands of the pretensioned strand group at the ends of beams may be increased to 37.5% provided post-tensioning is applied to the beams prior to casting the deck concrete and provided that the total number of debonded strands is equal to or less than 25% of the total area of pretensioned and post-tensioned strands at the time of placement of the deck concrete.

10.17 Special Requirements for Bulb-Tee Beams

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

- ! Pretensioned Beams = 166mm
- ! Post-tensioned Beams = 203mm

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

10.18 Special Requirements for Wide-Top Precast/Prestressed Beams

Wide-top precast/prestressed beams, including AASHTO Type V and VI beams, as well as the Florida Bulb-Tee series, shall comply with the requirements of AASHTO, Article 9.8.3, and the effective fillet for design shall be in accordance with Figure 10-1.

10.19 Distribution of Load for Flat Slabs and Precast Multi-Beam Decks

Flat slab and precast, multi-beam deck bridges, whether simple or continuous, shall be designed for MS22.5 loading according to Chapter 4. The MS22.5 loading shall be distributed over the width, E, according to the requirements of AASHTO, Article 3.24.3.2 Case B. The substructure for such bridges types, however, may be designed for MS18 loading.

Additionally, the superstructures of precast, multi-beam deck bridges, such as those using double-tees or solid slabs, shall conform to the minimum post-tensioning requirements of Table 10.3.

Table 10.3 Transverse Post-Tensioning of Multi-Beam Decks

Beam Type	Minimum Flange or Slab Thickness	Min. Transverse Post-Tensioning after all Losses
Double-Tee	180 mm	1.40 MPa
Solid Slab	300 mm	1.00 MPa

10.20 Traffic Railing Barriers

Unless otherwise approved, all new traffic railing barriers proposed for use for 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, and comply with, the recommended guidelines in the ***National Cooperative Highway Research Program (NCHRP) Report No. 350***.

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 401, have been determined to meet the applicable crash testing requirements.

10.20.1 General Requirements

In addition to the preceding criteria, all traffic railing barriers shall meet the following requirements:

- A. Structural evaluation to be equivalent to or greater in strength to other safety shape railings which have been crash tested to the TL-4 criteria of ***NCHRP Report No. 350***.
- B. Compliance with the strength requirements of Article 2.7 of the ***Standard Specifications for Highway Bridges*** with the exception that the Highway Design Loading, "P," in Figure 2.7.4B shall be increased to 72 kN distributed over a longitudinal distance of 1.5 meters. This load and distribution length apply **only** to FDOT's 815-mm, F-Shape barrier. The appropriate highway design loading and distribution length for any other traffic barrier must be approved by the DSDE or SSDE as appropriate.
- C. The traffic railing barrier on both sides of a structure shall be upgraded when widening work is proposed for only one side, and the traffic rail barrier on the non-widened side does not meet the criteria for new traffic rail barriers.

10.20.2 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 exception or variation is approved. Unless otherwise approved by the DSDE or SSDE, 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 an exception or variation, as appropriate.

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.

10.20.3 Traffic Railing Barriers for Historic Bridges

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

- A. An exception or 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. Design speed and operating speed shall be no greater than 65 km/hr.
- C. The strength requirements stated in “General Requirements” above are met.

10.20.4 Requirements for Test Levels TL-5A, TL-5 and TL-6

Consideration should be given to providing a traffic railing barrier that meets the requirements of Test Levels TL-5A, TL-5 and TL-6 as included in **NCHRP Report No. 350** when any of the following conditions exist:

- A. The volume of truck traffic is unusually high.
- B. The proposed structure is located such that a vehicle penetrating the railing barrier would cause high risk to the public or the surrounding facilities.
- C. Sharply-curved ramp structures with moderate to heavy truck traffic.

10.20.5 Exceptions and Variations

Exceptions or variations to the requirements of this Article shall be approved in accordance with Chapter 23 of the *Plans Preparation Manual*, Volume I (Topic No. 625-000-005), with concurrence of the DSDE or SSDE as appropriate.

10.21 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.

10.21.1 Box Girder Height

The minimum desirable interior 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 (DSFE) shall be consulted for a decision on the box height and access details.

10.21.2 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.

10.21.3 Inspection and Maintenance Access

Exterior access to box girders shall be provided through the bottom flange near the abutments, and in the end of each continuous unit. Exterior access openings and interior openings in diaphragms, etc. shall have least rectangular dimensions of 750 mm by 1220 mm. If openings of these dimensions cannot be obtained, the DSDE and DSFE shall be notified for proper guidance and resolution. 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. Additionally, openings shall be spaced at not more than 200 meters and located adjacent to piers that are convenient to lift truck access. Access locations over traffic lanes and locations that will require extensive maintenance of traffic operations shall be avoided.

10.21.4 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. All exterior holes in box girders not covered by a door shall be covered by a screen to prevent birds 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.

10.21.5 Interior Paint

When structural steel is to be painted, the interior of box girders shall be painted white or gray. The color of zinc primer is satisfactory.

10.22 Special Requirements for Concrete Decks of Continuous Steel Girder Superstructures

The provisions of AASHTO Article 10.38.4.3 are revised as stipulated hereinafter.

The negative moment region of a continuous, composite steel girder superstructure is defined as the portion of superstructure over an intermediate (interior) pier over which the top flange is in tension under any loading condition. Within this distance, provide longitudinal distribution reinforcement that is equal to at least one percent (1%) of the gross area of the concrete deck. Two-thirds ($2/3$) of this reinforcement shall be placed in the top layer. For stress calculations within the defined negative moment region, the slab shall be considered to be cracked and the longitudinal distribution reinforcement to be active.

For all other portions of the superstructure, provide longitudinal distribution reinforcement that is equal to at least one-half of one percent (0.5%) of the gross area of the concrete deck. Two thirds ($2/3$) of this reinforcement also shall be placed in the top layer.

Longitudinal distribution reinforcement also shall be provided in the bottom layer. The amount of reinforcement shall be independently calculated in accordance with the requirements of AASHTO Article 3.24.10, but shall not be less than the one-third ($1/3$) balance of the reinforcement defined above.

10.23 Lateral Restraint of Superstructures

The EOR shall always determine when lateral restraint of the superstructure of a bridge is required and make necessary provisions to assure that the bridge will function as intended. These provisions shall include considerations for the effects of geometry, creep, shrinkage, temperature, and/or seismic on the structure. When lateral restraint of the superstructure is required, then the appropriate method of restraint described hereinafter shall be employed.

10.23.1 For Elastomeric Bearings

When the required restraint exceeds the capacity of the bearing pad, the following appropriate restraint shall be provided:

A. Concrete Girder Superstructures:

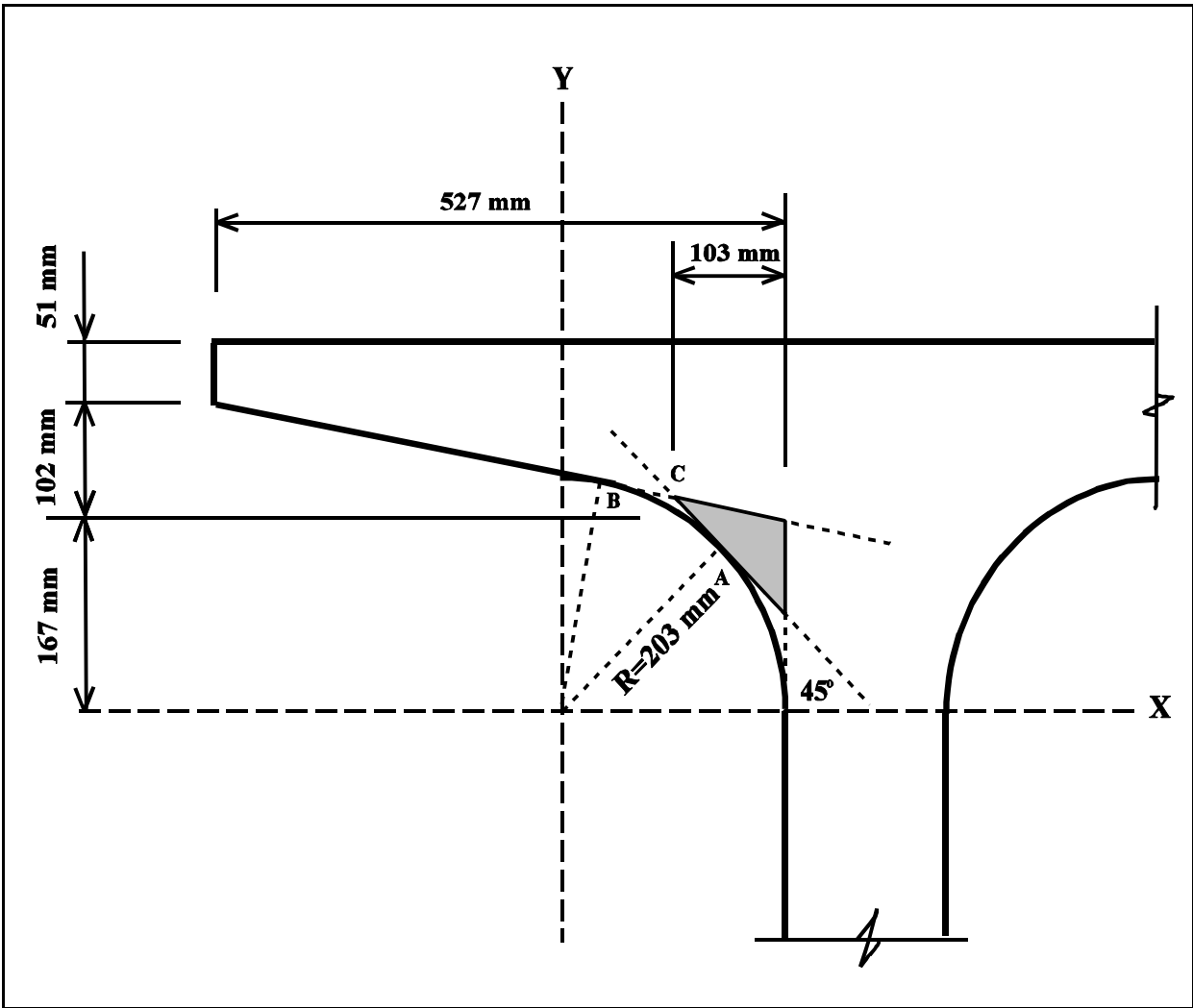
Provide concrete blocks cast on the substructure, positioned so as not to interfere with bearing pad replacement.

B. Steel Girder Superstructures:

Provide extended sole plates and anchor bolts.

10.23.2 For Mechanically Restrained Bearings

For bearings that provide restraint through guide bars or pintles, such as is the situation with pot bearings, the bearing itself shall be designed to provide the required lateral restraint.



***EFFECTIVE FILLET FOR DESIGN OF
FLORIDA BULB-TEE BEAMS***

Figure 10-1

Chapter 11

STRUCTURAL STEEL DESIGN

11.1 Design Criteria for Steel Bridges

Design of steel bridge components shall be in accordance with AASHTO **Standard Specifications for Highway Bridges**, latest edition, and interim specifications thereto. Horizontally curved plate girder and box girder designs and details shall be in accordance with AASHTO **Guide Specifications for Horizontally Curved Highway Bridges**, and interim specifications thereto.

The following general note shall be placed on the plans for all structural steel **highway** bridges:

“Structural steel girders and girder framing shall be fabricated by a shop that has the AISC Quality Certification for Major Steel Bridge. Fabrication and non-destructive testing shall be performed in accordance with the current applicable edition of the ANSI/AASHTO/AWS D1.5 **Bridge Welding Code**. Welding procedures shall be submitted and approved prior to welding on the project.”

For steel **pedestrian** bridges, use the identical note provided above, except change the AISC Quality Certification requirement to “**Minor** Steel Bridge.”

Design of new steel girder bridges shall be in accordance with either the AASHTO Strength Design Method - Load Factor Design, or AutoStress design. Refer to Chapter 16 for design methods applicable to widening of existing structures.

The design of structural steel for miscellaneous items such as ladders, platforms, and walkways shall be in accordance with the AISC **Manual of Steel Construction, Load Factor Design**.

Inspectability shall be considered in selecting the depth and in the detailing of steel box girders. Refer to the constructability and maintainability requirements of Chapters 16 and 26 of Volume I of the **Plans Preparation Manual**.

11.2 Steel Strengths and Thicknesses

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.

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 Specifications such as A36M or A588M.

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

To reduce distortion caused by welding and to improve girder stiffness for shipping and handling, the minimum thickness of plate girder and box girder webs shall be 10 mm. The minimum flange size shall be 20 mm by 300 mm, and the minimum box girder bottom flange thickness shall be 12 mm.

The designer should keep in mind that the cost of a fabricated steel plate or box girder is usually comprised of about 1/3 material costs and 2/3 labor costs; therefore, a design utilizing minimum steel weight as the governing criteria is not necessarily the most economical design. For example, it is recommended that a thicker web plate be used in lieu of a thinner plate that requires longitudinal stiffeners, and flanges of plate girders should be sized such that compression buckling is not a problem during transportation and erection of an individual girder.

11.3 Tension Member and Fracture Critical Requirements

11.3.1 For Structural Steel Highway Bridges

Main load carrying member components subjected to tension and stress reversal require Charpy V-Notch (CVN) impact testing on the material used. The 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 structures.

A non-redundant load path member is a component subjected to tensile stress such that failure of that component could cause collapse of the structure. These members are to be designated as fracture critical. Examples are defined in AASHTO, Article 10.3.1. Also, bottom flanges and connecting web plates in the positive moment regions of the end spans of **twin box girder** structures shall be considered as fracture critical (negative moment regions are not fracture critical).

The plans shall designate with a symbol all girder components that require CVN testing only (tension component, redundant member), and they shall designate with a different symbol all girder components that are fracture critical (tension component, non-redundant member). Splice plates attached to tension components shall also be designated by the appropriate symbol. The plans shall indicate which components of the structure are to be considered main

members for the purpose of the shop inspection and testing required by Chapter 6 of the ANSI/AASHTO/AWS **Bridge Welding Code**.

The designer must consider that fracture critical requirements are expensive due to the intensive welding procedures, base metal and welding material testing, and inspection after fabrication. The steel industry should be approached for advice when a choice may exist between a redundant and non-redundant system such as a three-versus-two plate girder bridge ramp. Fracture critical members are to be avoided if possible.

11.3.2 For Structural Steel Pedestrian Bridges

Because of the nature of the live loadings, none of the members in a steel pedestrian bridge shall be designated as fracture critical.

11.4 Connection Details and Hardware

The designer shall design and detail bolted attachments for all field connections rather than welded attachments. **Field welding is not allowed** unless specifically approved by the Department prior to the work. All bolted connections, other than anchor bolts, shall utilize ASTM A325M (Type 1) high-strength bolts designated as "slip-critical" connections. Bearing-type connections shall be used only with prior approval from the Department. The allowable load for slip-critical connections should be based on a Class B (Slip Coefficient 0.55) contact surface of bolted parts for typical steel components using the three-coat painting system.

ASTM A325M bolts with diameters of 20 mm, 22 mm, and 27 mm diameters typically shall be utilized. Larger diameter bolts may be used with prior approval of the Department. ASTM A490M bolts or load indicating (DTI) washers shall not be used unless authorized by the Department.

Anchor bolts and miscellaneous tie-down hardware may conform to ASTM A709M, Grade 250 for rolled shapes, plate, and threaded bars, or to ASTM F568M for standard bolt diameters and lengths.

For typical bascule bridge and girder/stringer superstructure systems, tie plates for transverse floor beams are normally connected to both the floor beam bracket and the interior floor beam in order to transmit the moment capacity of the bracket to the floor beam for moment continuity. When this detail is used, under no circumstances shall the tie plate be connected to the longitudinal girder.

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.

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.

Intermediate stiffeners that do not provide diaphragm connections shall have a tight fit tight against the compression flange and shall be shown with a cutback from the tension flange (no connection). Intermediate stiffeners that provide diaphragm connections shall be rigidly attached to both flanges by fillet welding to the compression flange and fillet welding or bolting to the tension flange of plate girders, or by bolting to the tension flange of box girders.

For additional information regarding connection details, refer to "Diaphragm and Lateral Bracing Requirements" in this Chapter.

11.5 Welding and Nondestructive Testing Requirements

All welding design, fabrication, and inspection shall be in accordance with the current AASHTO/AWS **Bridge Welding Code**. The contract plans shall use welding symbols as described in Section 2.1 of that code. The designer should not show a specific prequalified joint designation on the plans unless a certain type of groove weld ("V", "J", "U", etc.) is required. The shop drawings shall show the exact weld type proposed by the fabricator, and shall be approved by the EOR and the Department at that time.

The plans shall identify areas subject to tension and those areas subject to stress reversal. This will enable the inspection personnel to identify the type and extent of testing that is necessary in accordance with the welding specifications. Also, all shop drawings shall clearly identify these components. The contract plans shall also designate all welds to be performed during fabrication, and, when permitted, those to be performed in the field.

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 ascertain the type, or if the type is not an approved base metal included in the Bridge Welding Code, then a welding investigation must be performed. The EOR is responsible for contacting the District Structures Design Office so that the District Materials Office and the Department's independent testing agency can obtain the information necessary to define the welding and welding inspection requirements for the project.

As noted previously, the EOR may only use field welding connections with prior written approval from the Department.

11.6 Diaphragm and Lateral Bracing Requirements

Internal and external diaphragms typically shall be detailed such that the diaphragm is a shop fabricated "frame" (or rolled shape) that is field-connected to the steel beams or girders at stiffener locations. "K-Frame" diaphragms are preferred; however, an "X-Frame" configuration may be considered for deeper girders.

External diaphragms shall be attached by bolting at all connection locations. Internal diaphragms for box girders, which are installed during fabrication, shall be connected to the vertical stiffeners by welding to the stiffener when that portion of the stiffener is in a compression zone; however, when that portion of the stiffener is in a tension or stress reversal zone, diaphragms shall be bolted to the stiffener.

For box girders, lateral bracing shall be designed to provide rigidity of the section during handling, transportation and erection. Lateral bracing for curved box girders shall be designed to prevent warping of the section during shipping, erection and concrete deck placement.

For ease of fabrication, the designer shall consider the use of a single diagonal member lateral brace between panel points (diaphragm locations) rather than an "X" configuration. The connection of lateral bracing shall be bolted directly to the underside of the top flange. Additional buildup over the girder flange may be required with a stay-in-place metal form system in order to avoid conflicts with the lateral bracing.

11.7 Provisions for Stay-in-Place Metal Forms

The superstructure and all supporting structural steel components including beams, plate girders, and box girders **shall** be designed and detailed for the use of stay-in-place (S-I-P) metal forms except where prohibited by Chapter 7. The weight to be used in the design shall be in accordance with Chapter 4. When the clear span length between edges of beam flanges exceeds 3.0 meters, the plans shall note that cellular, metal S-I-P forms shall be used; however, **no** additional dead load over that of Chapter 4 is required. Stay-in-place metal forms of the type required herein shall also be designed and detailed for use as the interior (unexposed) soffit of all steel box girders even when otherwise normally prohibited.

A note shall be added to the "General Notes" of the bridge plans stipulating the use of stay-in-place metal forms, and the weight allowance included in the component's design for the metal forms and additional concrete weight to fill the form flutes. The superstructure shall be detailed to accommodate the use of stay-in-place metal forms yet perform equally as well for conventional, removable forms.

The design slab thickness 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.

11.8 Corrosion Protection of Structural Steel and Hardware

All structural steel bridge members shall receive a three-coat paint system in accordance with the Department's specifications.

All anchor bolts, tie-down hardware, and miscellaneous steel (ladders, platforms, grating, etc.) are usually hot-dip galvanized; however, high-strength bolts shall be uncoated when installed

and subsequently field painted.

The EOR shall contact the SDO regarding corrosion protection for individual items.

11.9 Deck Pouring Sequence and Girder Camber

The structure shall be designed for, and the plans shall show a sequence for placing the concrete deck. Steel girders shall be fabricated to match the profile grade and with a camber equal to the dead load deflection such that when the deck is placed, the amount of build-up over each girder is minimized.

11.10 Existing Structures with Lead Based Paint

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.

Chapter 12

EXPANSION JOINT DEVICES AND BRIDGE BEARINGS

12.1 General Policy for Joint Devices

Bridge deck joints are a continuous minor maintenance problem at best and at worst may be a source of structural deterioration resulting in a major bridge maintenance problem. For these reasons, it is the Department's policy to minimize the use of joints.

Expansion joints in bridge decks can be avoided by providing deck slab continuity at intermediate bents or piers. Generally, this is accomplished by designing continuous superstructures, or by allowing continuity of the deck slab, even when the supporting beams or girders are simply supported. In either case, the structures must be designed to accommodate the movements, or to resist the stresses, due to factors such as: temperature, elastic shortening, creep, and shrinkage.

For bridge structures defined as "Long Bridges" by the criteria of Chapter 10, expansion joints shall be detailed for installation as an independent operation performed after deck planing requirements have been met.

12.2 General Policy for Bridge Bearings

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

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 polytetrafluoroethylene [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.

12.3 Movement Due to Temperature Variation

For all bridges other than longitudinally post-tensioned segmental concrete bridges, movement due to temperature variation (range) shall be calculated from an assumed mean temperature of 21 degrees Celsius (EC) 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 appropriate temperature ranges of Table 12.1.

Movements of longitudinally post-tensioned segmental concrete bridges due to temperature variation shall be calculated according to the requirements of Chapter 8.

Table 12.1 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 following thermal coefficients shall be used based upon the material of the main longitudinal supporting beams or girders:

- ! Steel: 0.0000117 per Degree Celsius
- ! Concrete: 0.0000090 per Degree Celsius (with Florida limerock aggregate)

12.4 Expansion Joint Provisions

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

- ! If the joint device is proprietary, it shall be designed in accordance with AASHTO Standard Specifications and installed in strict accordance with the manufacturer's specifications unless directed otherwise by the Department.
- ! The joint components shall accommodate the full range of structure movement 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 to resist all anticipated loads, including impact.
- ! The joint shall not transfer undue stresses to the structure.
- ! Elastomers for joint seals shall provide a service life warranty for a minimum

period of five (5) years.

- ! The joint shall be designed for minimum maintenance and ease of access and parts replacement.
- ! The joint shall be reasonably 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.

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

12.5 Design of Expansion Joints to Accommodate Movement

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

Joints for conventional, reinforced and/or prestressed concrete superstructures shall be designed for the calculated movement due to temperature change plus creep and shrinkage, or for 115% of the calculated movement due to temperature change alone, whichever is the larger; however, the joint opening must not be less than 15 mm at 21 degrees Celsius (EC).

Joints for longitudinally post-tensioned superstructures shall be designed for creep and shrinkage plus 120% of the calculated movement due to temperature change.

The recommended width limitations for different joint types are shown in Table 12.2.

Table 12.2 Joint Width Limitation by Joint Type

Joint Type	Maximum Joint Width
Poured Rubber	20 mm
Silicone Seal	55 mm
Strip Seal	75 mm
Modular Joint	*
Finger Joint	Unlimited

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

12.6 Joints for Bridge Widening

The District Structures and Facilities Office shall be contacted to determine the type and condition of all existing joints to be widened. Existing joint types by group are:

- ! Group 1: Strip seal, compression seal, poured rubber, open joint, silicone seal, copper water-stop, and "Jeene."
- ! Group 2: Sliding Plate, finger joint, and modular.

Existing joints to be widened shall be carefully examined to determine in advance the extent of existing concrete deck to be removed. Removal of existing concrete shall be limited to that necessary to remove the existing joint armor and to permit proper anchorage of the new joint armor. Detail the existing joint removal, and/or note that extreme care shall be taken by the Contractor, to assure that the existing deck reinforcing steel will not be damaged by the new joint installation.

12.6.1 Lengthening of 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 silicone, and is performing satisfactorily, remove all existing joint material, extend joint gap into the widening, and install a new silicone 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 silicone sealer.
- E. If the existing joint is an open joint that is wider than 25 mm at 21 degrees Celcius (°C), provide a blackout for a Group 1 Joint (other than a compression seal) that can accommodate the existing joint opening. The new bridge deck shall provide a blackout 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 blackout for a Group 1 Joint other than a compression seal. The new bridge deck shall provide a blackout for the new joint system.

- Notes:
- (1) 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 deck surface.
 - (2) An 'approved' seal is a seal that either is listed on the Department's QPL or is approved by the DSDE, District Maintenance Engineer, or SDO, as appropriate.
 - (3) Generally, field splices of strip seal expansion joints shall conform to the notes of Index No. 400 of the **Standard Drawings**.

12.6.2 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.

12.7 Design of Bearings

Composite neoprene bearing pads and other bearing devices shall be designed in accordance with the latest edition of AASHTO **Standard Specifications for Highway Bridges**, the Department's **Standard Specifications for Road and Bridge Construction**, and the **Structures Design Guidelines**.

Index No. 200 of the **Standard Drawings** includes four (4) composite neoprene bearing pad designs. Whenever possible, and after confirming their adequacy, the standard designs of Index No. 200 should be used. Only when the neoprene capacities of the standard pads have been exceeded or when site conditions or constraints dictate provisions for special designs (such as multi-rotational capability) should other bearing systems or components be considered.

12.8 Maintainability Requirements - Bearings

The following provisions shall apply to all bridges except for flat slab 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, such as jacking locations, jacking sequence, jack load, etc., shall be made in the design and shown on the plans, and the substructure width shall be sized, or verified as satisfactory, to accommodate the jacks and any other required provisions.

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

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 or segmental concrete box girders, will require separate details and notes describing the procedures. Again, it should be noted that the jacking equipment is not part of the bridge contract.

Typically, the design and provision for bearing replacement shall include live load, unless directed otherwise by the Department. Generally, the provision for live load is excluded for bridges with horizontal curvature or other complex geometry. When live load is to be included, the following adjusted AASHTO group loads may be used:

- ! Service Load Design: Group I, except 140% allowable stress
- ! Load Factor Design: Group I, except $\beta_L (L + I)_n = 1.00$.

Chapter 13

RAILROAD CROSSINGS

NOTE:

The provisions of this chapter have been relocated to the *Plans Preparation Manual* (Topic No.: 625-000-005), Chapter 6.

Chapter 14

PEDESTRIAN OVERPASSES AND BIKEWAY BRIDGES

NOTE:

The provisions of this chapter have been relocated to the *Plans Preparation Manual* (Topic No.: 625-000-005), Chapter 8.

Chapter 15

MOVABLE BRIDGE REQUIREMENTS

15.1 General

This Chapter contains criteria related to the design of movable bridge projects and is a supplement to the AASHTO ***Standard Specification for Movable Highway Bridges***. The structural components shall be designed using the AASHTO ***Standard Specifications for Highway Bridges*** with due consideration to limiting deflections for both Service Load and operational conditions.

15.2 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 State Structures Design Engineer (SSDE). Projects for which the criteria are applicable shall result in designs that maximize assurance of reliable operation of movable bridges by incorporating redundant features in movable bridge drive and control systems.

New bascule bridges preferably shall incorporate two leafs per span as a redundant drive system except as provided below.

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 which case the EOR, during the BDR phase, shall provide appropriate recommendations for omission of redundant systems.

Redundant drive configurations include:

- ! Hydraulic drive systems operated by multiple hydraulic cylinders. In these systems, a hydraulic cylinder, a pump drive motor, or a 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 both 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 SSDE.

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

Design new movable bridges leaf support systems to utilize one 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.
- ! Simple, rotating trunnion configuration, with spherical roller bearings supported on the outboard side of the girder, and an inboard trunnion girder framed into the leaf structure.

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.

For example, with a cylinder failure, the failed cylinder will be deactivated, and the leaf will be operated on the remaining cylinders. 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.

15.3 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.

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 utilize metric dimensions, soft converted to meet the dimensions of 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.

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.

15.4 General Requirements

15.4.1 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) for vehicular traffic or raise the leaf(s) for 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.

15.4.2 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, including the requirements of the State Fire Marshall.

Use the FDOT's ***Standard Specifications for Road and Bridge Construction*** and the Developmental Specifications or Special Provisions issued by the Department. Additional Technical Special Provisions as may be required shall be sealed by the EOR.

Provide detailed calculations to justify all equipment and systems proposed. Calculations shall be submitted, for review, for all aspects of the systems and components. Submit calculations in an 8-1/2 x 11 binder.

15.4.3 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 15-1) 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.

15.4.4 Bridge Terminology

See Figure 15-2 for standard bridge terminology.

15.5 Machinery Systems

15.5.1 Trunnions and Trunnion Bearings

A. Trunnions:

1. All shoulders shall have fillets and rounds of appropriate radius. Flanges of bushings on sleeve bearings shall bear against a thrust surface provided by a step shoulder on the trunnion. The thrust surface shall be designed to resist an axial load equal to 15% of the maximum radial load on the bearing. 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 H7/p6 fit with a surface finish of $1.6 \mu\text{m}$ or better. Do not use keyways between the trunnion and the trunnion hub/collar.
3. In the case of sleeve bearings, the trunnion shall extend at least 15 mm beyond the end of the bearings. Provide a 50 mm-deep counterbore concentric with shaft journals at each of the hollow shafts.
4. Shafts shall have a minimum shrink fit of H7/s6 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 installed in holes drilled through the hub into the shaft after the shaft is in place. Dowel pins shall have an H7/s6 fit in the finished hole. The length of engagement shall be 1.5 to 2.0 times the pin diameter in each doweled part. When anti-friction roller bearings are used, the surface finish shall conform to the bearing manufacturer's requirements. 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.
5. 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.
6. 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 $H7/s6$ between hub and ring and the structural steel. The minimum mounting surface finish of both hubs and rings shall be $3\ \mu\text{m}$ or finer.

2. Adequate radii shall be provided at fillets on the hub and at shoulders on the bascule girder to prevent fillet-to-shoulder bearing. 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 flange should be stiffened by equally spaced, integral ribs. 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 radial position.
5. See Figure 15-3.

15.5.2 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 for which the drive mechanism was designed.

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 (-37.5E) to plus 37.5 degrees (+37.5E).

B. Rehabilitation Projects:

The above provisions for New Construction also apply to Bridge Rehabilitations in which the leaf balance must be adjusted. 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 "a."

Where:

- a = angle of elevation of the center of gravity when the leaf is closed.
- WL = total leaf weight, "W," times the distance from the trunnion axis to its center of gravity, "L."

15.5.3 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. 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.

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.

15.5.4 Open Gearing

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

15.5.5 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 150 x 200 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 to facilitate insertion and removal.
7. Provide lubrication fittings at locations that are convenient for routine maintenance.
8. Actuation elements shall be mounted on the lock-bars to activate 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 leaf.
9. Connection of the lock-bar to the hydraulic cylinder shall allow for the continual vibration due to traffic on the leaf. This may be accomplished by providing self-aligning rod-end couplers or cylinders with elongated pin holes on male clevises.
10. The hydraulic fluid system is to be a reversing motor-driven pump and piping 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. A hydraulic hand pump with hoses and quick-disconnect fittings shall 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 travel lanes.

B. Lock Design Standards:

1. The empirical formula, Equation 15-1 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.

$$S = \frac{P}{4} \left(\frac{A}{L}\right)^2 \left(3 + \frac{A}{L}\right) \quad [\text{Eq. 15-1}]$$

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 15-4 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.

15.5.6 Brakes

Use thrustor type brakes. Provide a machinery brake and a motor brake. Submit calculations for 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.

15.5.7 Couplings

Submit calculations and manufacturer's literature for coupling sizes specified.

15.5.8 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.

15.5.9 Bearings (Sleeve and Anti-Friction)

- A. Sleeve Bearings shall be grease-lubricated. Bronze bushings 200 mm in diameter or less 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. 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.

15.5.10 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 193M - Alloy Steel and Stainless Steel Bolting Materials for High-Temperature Service. Stud Bolts and Conical Nuts shall be ASTM A193M, 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 194M - Carbon and Alloy Steel Nuts for Bolts for High-Pressure and High-Temperature Service. Nuts shall be ASTM A194M, 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 36M - Structural Steel. Plate washers shall conform to ASTM A 36M.
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 A193M, Grade B8 - Threaded Rod/Stud.
3. ASTM A193M, Grade B8 - Conical Nut.
4. ASTM A194M, 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:

$$P_c = 0.33f \sqrt{f'_{lc}} A \quad \text{[Eq. 15-2]}$$

Where:

- F = Capacity reduction factor, 0.65
- A = Projected effective area of the failure cone, mm^2
- f'_{lc} = 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.

15.5.11 Fits

Use ANSI Standard B32.4M which presents a Preferred Series of sizes and tolerances for

metric sizes.

15.5.12 Finishes

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

15.6 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 60E C during the most adverse ambient temperature conditions anticipated.

15.6.1 Hydraulic Pumps

Specify minimum pressure rating of pumps to be 1.5 times the maximum operating pressure. 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.

15.6.2 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.

15.6.3 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 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 SSDE.

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 SSDE.

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.

15.6.4 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.

15.6.5 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.

B. Manifolds:

Specify valves mounted on manifolds as far as practical.

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 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.

15.6.6 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 60E C. Tanks in closed-loop hydrostatic systems shall circulate, filter, and cool enough oil to limit the maximum oil temperature to 60E 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 A240M for 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:

Require absolute pressure (vacuum) sensors to stop the pumps if adequate suction head is not available at the pump inlets. The system shall have filters with relief-check, by-pass valve and visual clog indicators. Specify remote sensing pressure switches to indicate clogged filters. Do not use valves that can accidentally be left closed.

C. Hydraulic Fluids:

The hydraulic fluid must be approved by the manufacturer of the hydraulic components in which it is to be used. Specify the use of bio-degradable hydraulic fluid.

15.7 Electrical

15.7.1 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.

15.7.2 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).

15.7.3 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 capacity of 38 L. Submit calculations justifying generator size recommended.

15.7.4 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 the available short circuit at the point of application in the system. Provide short circuit calculations to justify ATS proposed.

15.7.5 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).

15.7.6 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 SSDE.

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 SSDE.

15.7.7 Programmable Logic Controllers

Refer to the "Technical Special Provisions" issued by the SSDE.

15.7.8 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. 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.

15.7.9 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.

15.7.10 Instruments

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

15.7.11 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 SSDE.

15.7.12 Electrical Connections between Fixed and Moving Parts

Use extra flexible wire or cable.

15.7.13 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.

15.7.14 Service Lights

Provide minimum of 300 Lux in all areas of the machinery platform.

15.7.15 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.

15.7.16 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 SSDE.

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.

15.7.17 Movable Bridge Traffic Signals and Safety Gates

Refer to ***Roadway and Traffic Design Standards***, Index No. 17890, for Traffic Control Devices for Movable Span Bridge Signals.

15.7.18 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.

15.7.19 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 SSDE.

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 SSDE, 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 (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. On generators, a 100 KW or above 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.

15.8 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." 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 1.0m roof overhangs since 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. 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. 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:

- A. Traffic gates for both directions of automobile traffic.
- B. Marine traffic for both directions of the navigable channel.
- C. Locations where pedestrians will normally stop.

15.8.1 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 a hose bib outside the house.

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.

15.8.2 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 of 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. 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/m^2 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. Use of vertical ladders must be approved by the SSDE.

15.8.3 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.

15.8.4 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.

15.8.5 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 meters of 1.1 meters base cabinets and a minimum length of 2.1 meters of 610 mm or 914 mm wall cabinets.

Specify cabinet hardware and solid-surfacing material counter-tops and desktop.

15.8.6 Insulation

A. "R-Values:"

Design the Control House with the following minimum insulation 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.

15.8.7 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

15.8.8 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 tile roofing. 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.

It is noteworthy that 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.

15.8.9 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 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.

15.8.10 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.

15.8.11 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.

15.8.12 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, shall be approved by the SSDE.

15.8.13 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.

15.8.14 Painting

Select and specify paint for woodwork and walls. Do not specify painted floors.

15.8.15 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.

15.8.16 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.

15.8.17 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 refrigerator with user's manual. Specify a fire extinguisher for each room. Specify two gas-lift, front-tilt task chairs. Provide one 1m x 1.5m cork bulletin board. Specify blinds or shades for window treatment.

15.8.18 Heating, Ventilating, and Air-Conditioning (HVAC)

Use central, split-unit heat pump. If required, 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.

15.8.19 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.

15.9 Maintainability

15.9.1 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.

15.9.2 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.

15.9.3 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.

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.

15.9.4 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.

15.9.5 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.

15.9.6 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. For example, a speed-reducer assembly shall be removable by breaking flexible couplings at the power input and output ends of the speed-reducer.

15.9.7 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.

15.9.8 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.

15.9.9 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.

15.9.10 Service Accessibility

A service area not less than 750 mm wide shall be provided around system drive components.

15.9.11 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.

15.9.12 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.

15.9.13 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.

15.9.14 Navigation Lights

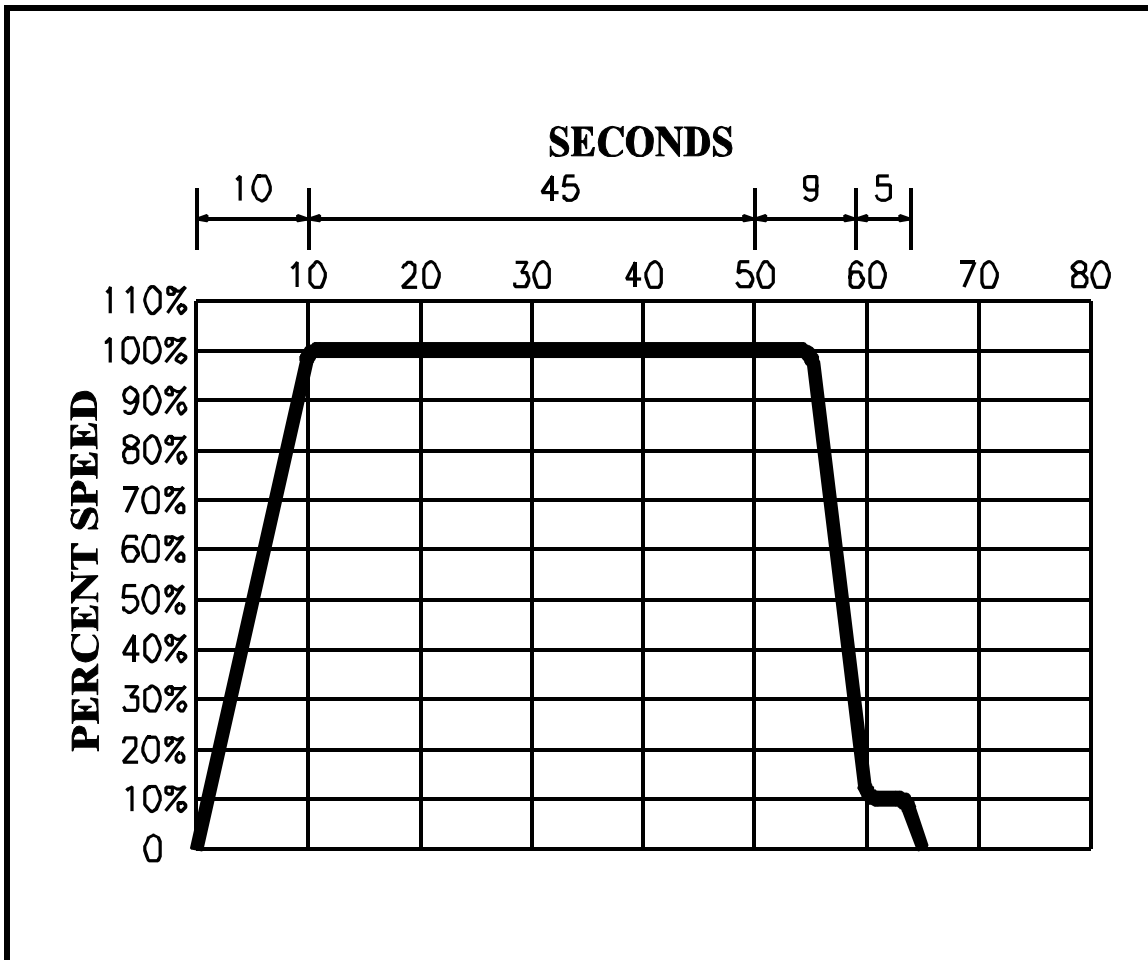
Dual lamps and transfer relays shall be installed on fenders and center of channel positions to reduce effort required for maintenance of navigation lights.

15.9.15 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 SSDE.

15.9.16 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 15-1

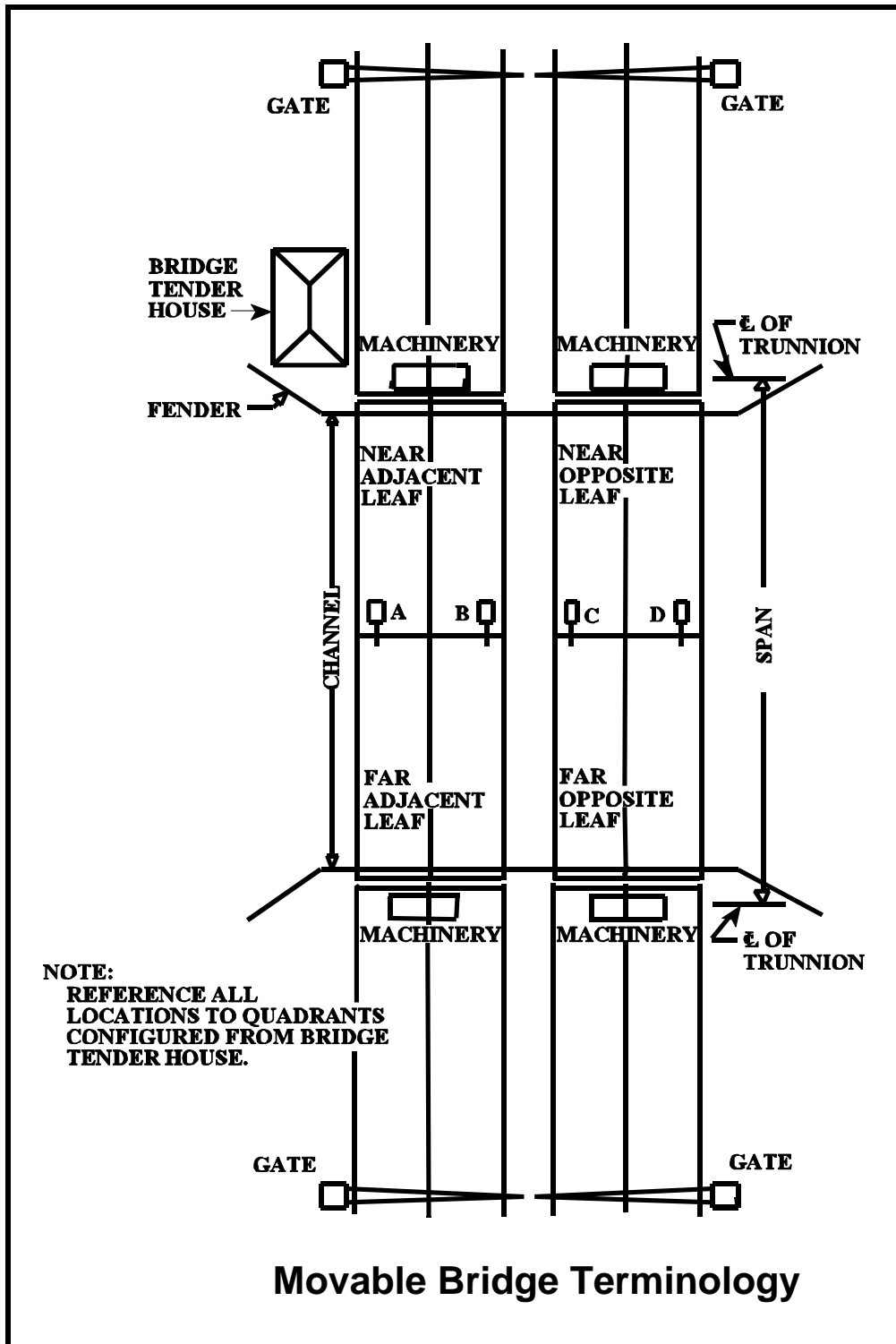
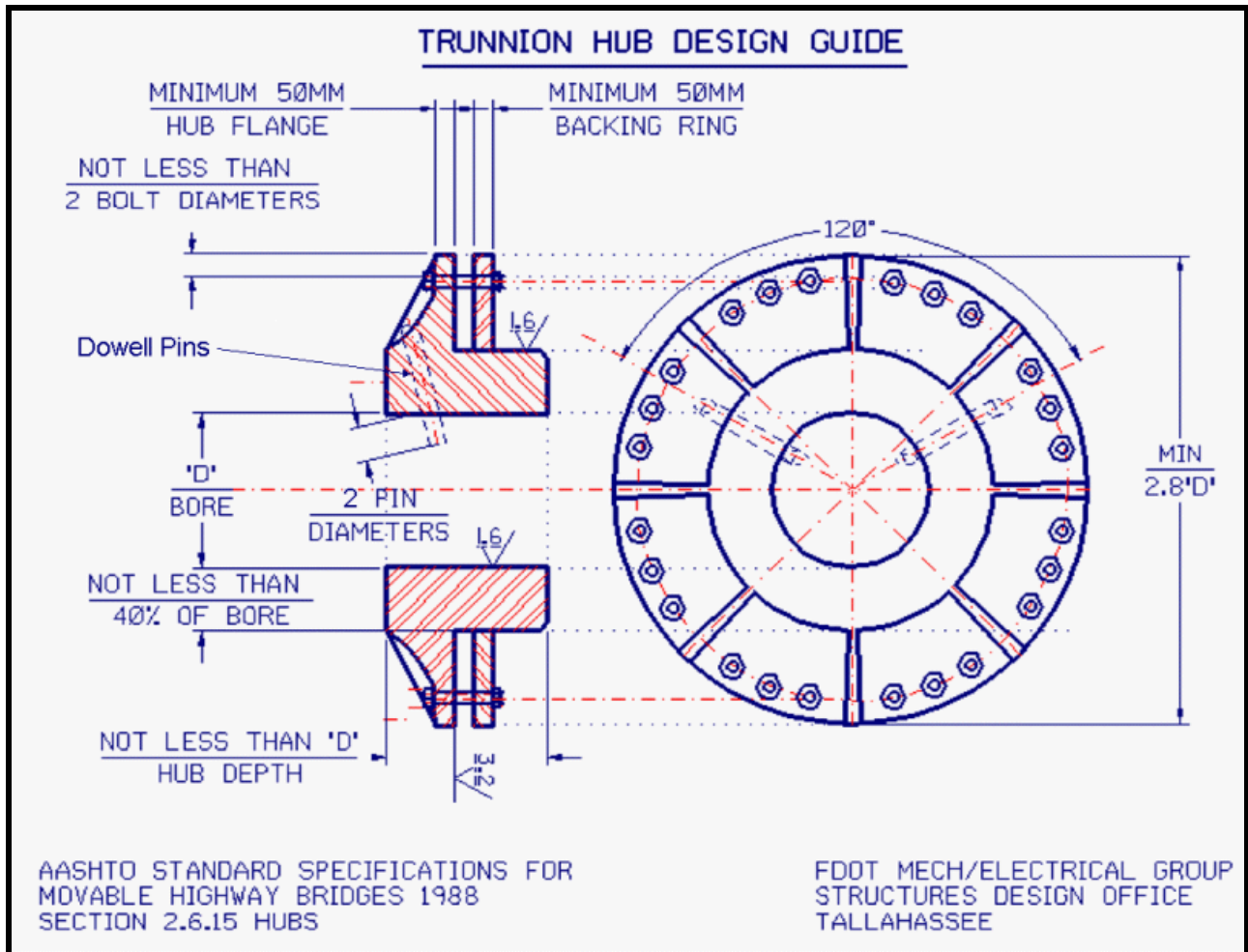


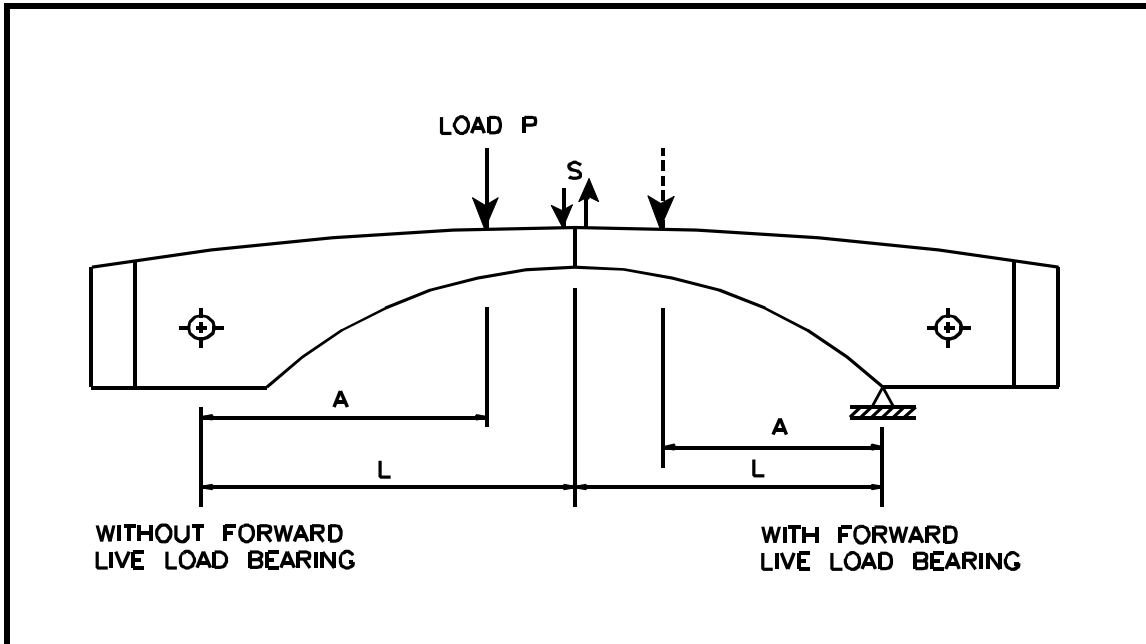
Figure 15-2



Trunnion Hubs

(See “Trunnions and Trunnion Bearings” under “Machinery Systems,” this Chapter)

Figure 15-3



Lock Design Criteria

Figure 15-4

Chapter 16

BRIDGE WIDENINGS

16.1 Bridge Widening Definitions

Bridge structures to be widened shall be classified either as major or minor widenings according to the definitions provided hereinafter. In these definitions the term "facility" describes the total structure or structures required to carry a transportation route over an obstruction. Also, the widening may occur on one side only, both sides, between separate, parallel structures, or any combination thereof. In this context, adding a lane of traffic each side of each of twin, two-lane bridges would be a "Major Widening" as defined hereinafter because the total number of lanes of traffic of the finished facility is twice that of the sum of the existing twin bridges.

16.1.1 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 the total bridge deck area will be at least twice that of the unwidened bridge. The area shall be computed using the transverse coping-to-coping dimension.

16.1.2 Minor Widening

A Minor Widening is defined as new construction work to an existing bridge facility that results in an extent of widening less than described above for Major Widenings.

16.2 Analysis and Design

16.2.1 General

A. Appearance:

The widening of a structure should be accomplished in a manner such that the existing structure does not look "added onto;" i.e, the structure appearance should be improved by the widening.

B. Materials:

Materials used in the construction of the widening preferably shall have the

same thermal and elastic properties as those used in the existing structure.

C. Load Distribution:

The main load carrying members of the widening should be proportioned to provide longitudinal and transverse load distribution characteristics similar to those of the existing structure. 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 design of the widening and for stress checks of the existing structure. The distribution of live load shall be in accordance with AASHTO criteria. When prestressed beams are used to widen an existing cast-in-place concrete box girder or T-Beam bridge, the factor for live load distribution for interior beam(s) of the widening shall be $S/1.8$ with "S" in meters.

D. Specifications:

The design of the widening shall conform to the latest edition of AASHTO ***Standard Specifications for Highway Bridges*** and interims thereto, FDOT ***Standard Specifications for Road and Bridge Construction***, these ***Structures Design Guidelines***, and any applicable Structures Design Office criteria. In general, the method of design for minor widenings shall be the same as the original design. For example, if the original design was performed by the Service Load method (Working Stress), then the design of the widening shall be performed by the Service Load method. Major widenings shall always be designed by the Load Factor method (Strength Design).

E. Overlay

The elimination of asphalt overlays on bridge decks is preferred.

F. Stresses in the Existing Structure

A review of stresses in the main members of the existing structure shall be made for construction conditions and the final condition; i.e., after attachment of the widened portion. When computations indicate large overstresses in members of the existing structure (primarily in columns) designed by Service Load methods, a review by Load Factor methods should be made. Using this comparison and good engineering judgment, a decision can be reached by the EOR and the appropriate FDOT review office, SDO or DSDE, as to the amount of overstress that can be tolerated.

G. Other Special Considerations

1. All widenings shall be designed for MS-18 loading as a minimum,

regardless of the loading used in the original design.

2. Large differential cambers between the existing structure and the proposed widenings are not allowed.
3. Open or sealed longitudinal joints in the riding surface are safety hazards to be avoided whenever possible.
4. During deck pour and curing, it is preferable to minimize or eliminate live load vibrations from the existing structure.
5. When a deck slab cantilever is widened with no additional support, the proposed cantilever should be carefully checked for increased stresses.
6. Diaphragms for widenings shall be in accordance with Chapter 10. Refer to Chapter 12 for guidance regarding bearing replacement capabilities of widened structures.
7. Ample clearance between proposed driven piles and existing piles, utilities, or other obstructions shall be assured. This is especially critical of battered piles.
8. Consideration should be given to require use of the Profilograph on Major Widenings. If the Profilograph is to be used, the deck thickness must be increased to accommodate its use as required in Chapter 10.

16.2.2 Substructure (Selection of Foundation)

When making a selection of foundation, the EOR shall consider the recommendations of both the District Geotechnical Engineer and the District Drainage Engineer.

16.3 Attachment of Widening to Existing Structure

16.3.1 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 clearance of 75 mm from the edge of the concrete and a minimum clearance of 25 mm from the existing reinforcing bars in the structure.

When it is required to drill holes larger than 40 mm, deeper than 30 mm, or through reinforcing bars, core drilling shall be specified.

16.3.2 Dowel Embedment

The depths of embedment of reinforcing bar dowels shall meet minimum development length requirements whenever possible. If this is not possible (e.g., traffic railing dowels into the existing slabs), 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. Adhesive anchors designed in accordance with Chapter 7 may be used.

16.3.3 Preparation of Contact Surface

Preparation of existing surfaces for concreting shall be in accordance with "Removal of Existing Structures" in Sections 110 and 400 of the FDOT's ***Standard Specifications for Road and Bridge Construction***.

16.3.4 Connection Details

The figures at the end of this chapter are samples of details that have been successfully used for bridge widenings. They are for general information and are not intended to restrict the EOR's judgment.

A. Flat Slab Bridges:

It is preferred that a portion of the existing slab shall be removed to expose the existing transverse reinforcing for splicing with new reinforcement. The transverse slab reinforcing for the widening may be doweled directly into the existing bridge without meeting the normal splice requirement if the existing reinforcing steel cannot be exposed. Adhesive anchors designed in accordance with Chapter 7 may be considered for the slab connection details (see Figures 16-1 and 16-4).

B. T-Beam Bridges:

The connection detail shown in Figure 16-2 for the slab connection is recommended. Limits of slab removal is at the discretion of the EOR, subject to the FDOT's approval.

C. Steel and Concrete Girder Bridges:

The detail shown in Figure 16-3 for the slab connection is recommended for either prestressed concrete or steel beam superstructures.

16.4 Expansion Joints

Expansion joint requirements for widenings are provided in Chapter 12.

16.5 Construction Sequence

A construction sequence, taking into account the Traffic Control requirements, shall be shown on the preliminary bridge plans submittal for all projects. Traffic Control Plans shall address traffic needs during construction activities on the existing structure such as installation of new joints, deck grooving, etc.

The final plans shall include a complete outline of the order of construction along with the approved Traffic Control Plans. The plans shall also include details for performing any necessary repairs to the existing bridge.

16.6 Bridge Widening Rules

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

16.6.1 Mixing of Materials

The mixing of concrete and steel beams in the same span shall be avoided.

16.6.2 Matching of Existing Beams

The use the same type of beams as those existing is preferred; however, if the existing beams are cast-in-place concrete, the widened deck should be supported on prestressed beams.

16.6.3 Vertical Clearance

Providing the AASHTO-required vertical clearance on Interstate structures must be considered and decision factors documented. As a minimum, the existing vertical clearance must be maintained. This may be accomplished by decreasing the standard beam depth. Where the existing bridge does not satisfy current vertical clearance requirements, and where the economics of doing so are justified, the superstructure should be elevated and/or the roadway beneath should be lowered as part of the widening project. This is particularly important for bridges that have a history of frequent impacts to the superstructure from over-height vehicles.

16.6.4 Matching Reinforcement

Generally, the transverse reinforcement in the new deck should be spaced to match the spacing of the existing reinforcing. Different bar sizes may be used if necessary.

16.6.5 Verifying Existing Bridge Plans

For all widenings, the EOR must confirm that the available existing bridge plans depict the actual field conditions. In general, this should be included as part of any new survey, but should be extended to involve a structural engineer checking that the following items, as a minimum, are in agreement with the existing plans:

- A. Bridge location, pier location, skew angle, and stationing.
- B. Span lengths.
- C. Number and type of beams.
- D. Wingwall, pier, and abutment details.
- E. Utilities (if any) supported on the bridge and those adjacent to the bridge if new piling will be required.
- F. Finished grade elevations and, if possible, exterior beam seat elevations.
- G. Vertical and horizontal clearances.
- H. Type(s) and condition of existing superstructure joints.
- I. Other features critical to the widening.

16.6.6 Resolution of Discrepancies

Any discrepancies which, in the designer's opinion, are critical to the continuation of the widening design should be brought to the FDOT Project Manager's attention at the earliest possible time. Critical discrepancies must be resolved by the EOR and the DSDE (or SDE for Category 2 Bridges), or the District should replace the structure rather than continue the widening.

16.6.7 Special Bridge Structures

Bridges with voided slabs such as prestressed "sonovoid" slabs require special attention. If widening of such a structure is being proposed, the SDO shall be contacted for guidance.

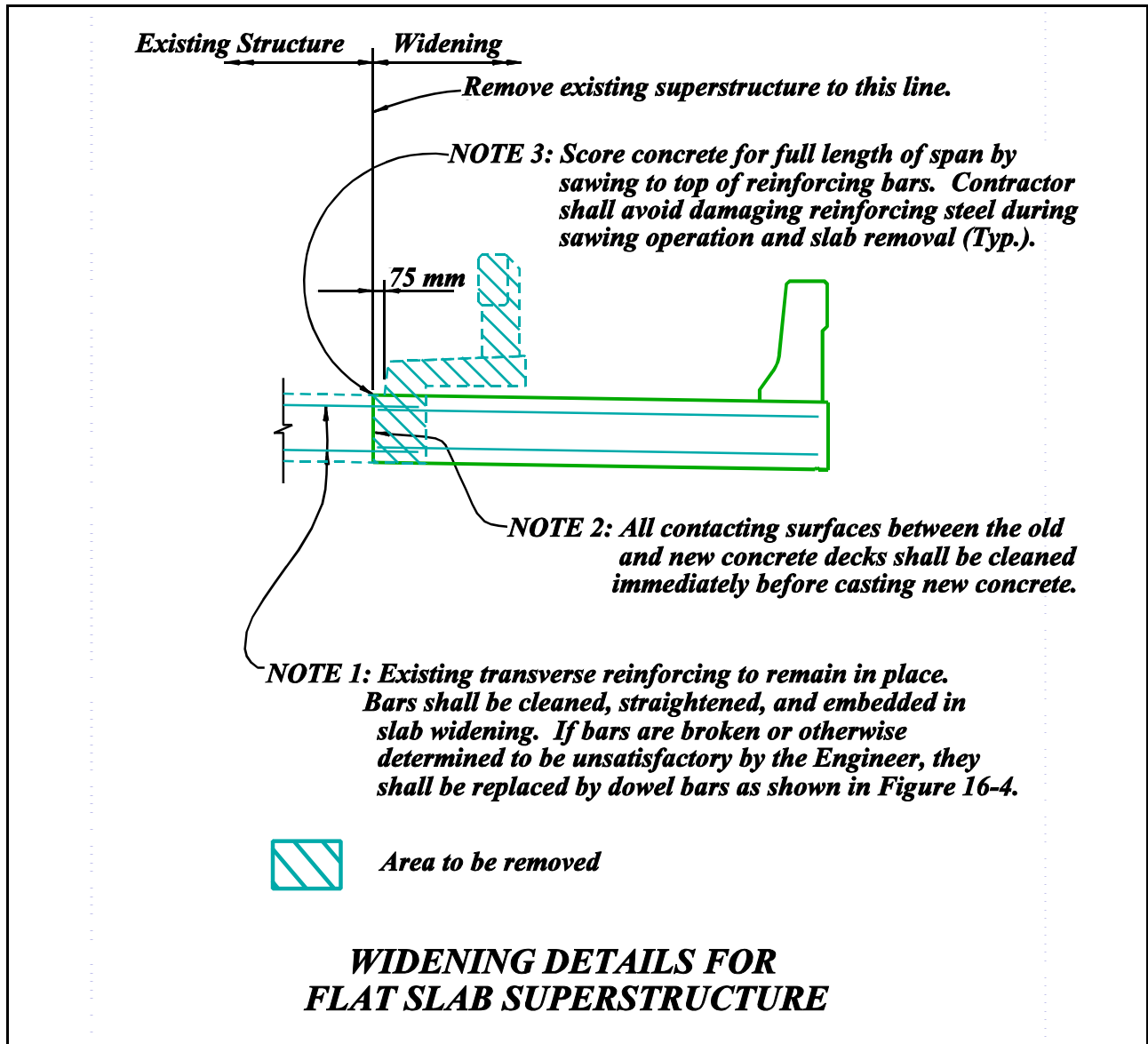
16.7 Bridge Deck Grooving

For widened superstructures where at least one traffic lane is to be added, the entire deck area described in Chapter 10, "Grooving Bridge Decks," shall be grooved.

For projects with shoulder widening only, a note should be added to the plans specifying that the bridge floor finish shall match that of the existing bridge deck surface. However, if the existing bridge deck surface is in poor condition, the DSDO shall be contacted for direction.

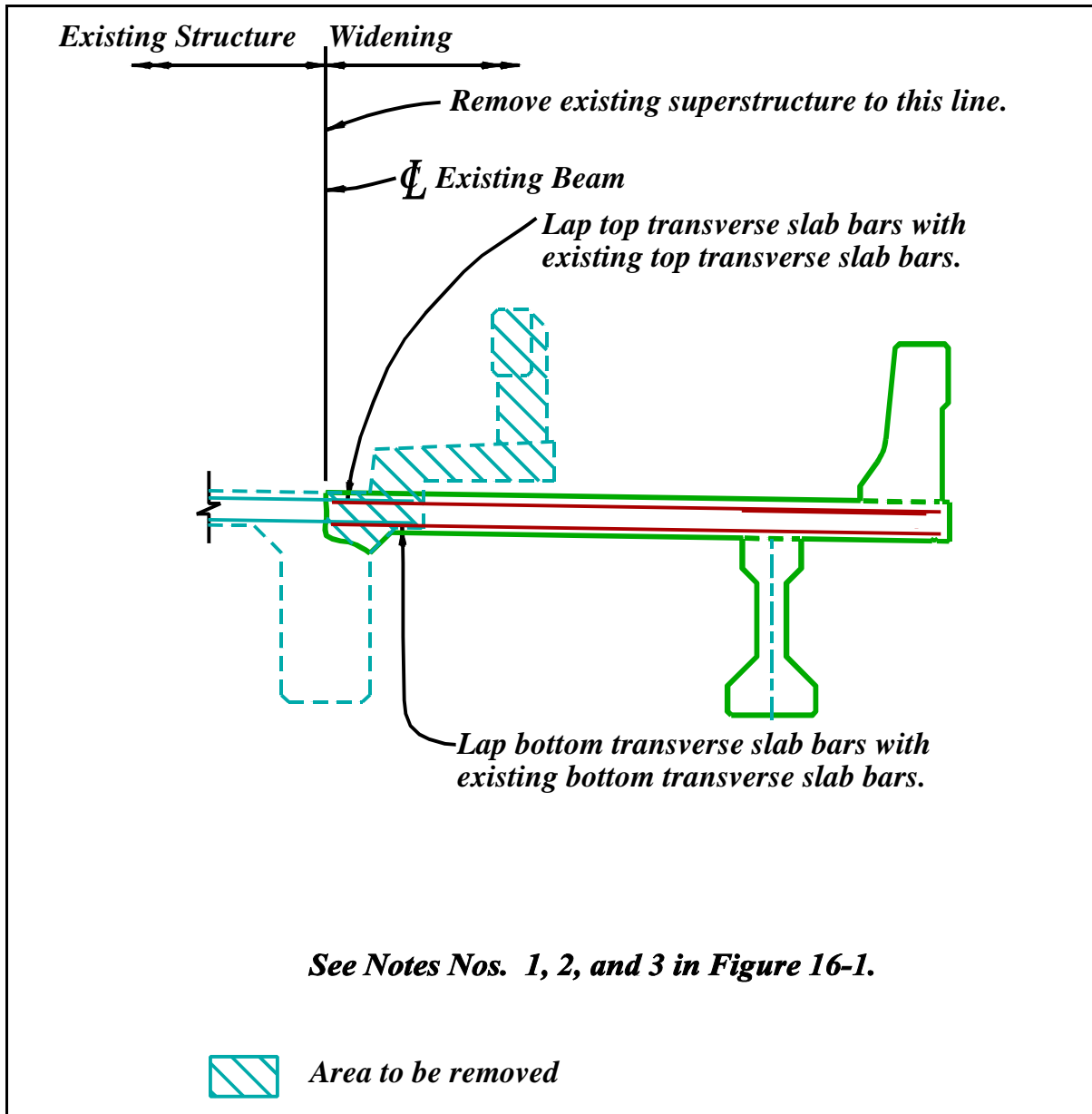
On bridge widening projects where the existing bridge deck does not conform with the current reinforcing steel cover requirement and the superstructure environment is classified as Extremely Aggressive due to the presence of chlorides, the existing bridge deck shall be sealed after grooving. If the existing 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.

Contact the SDO for guidance for the required bridge surface finish for unusual situations or for bridge deck surface conditions not covered above.



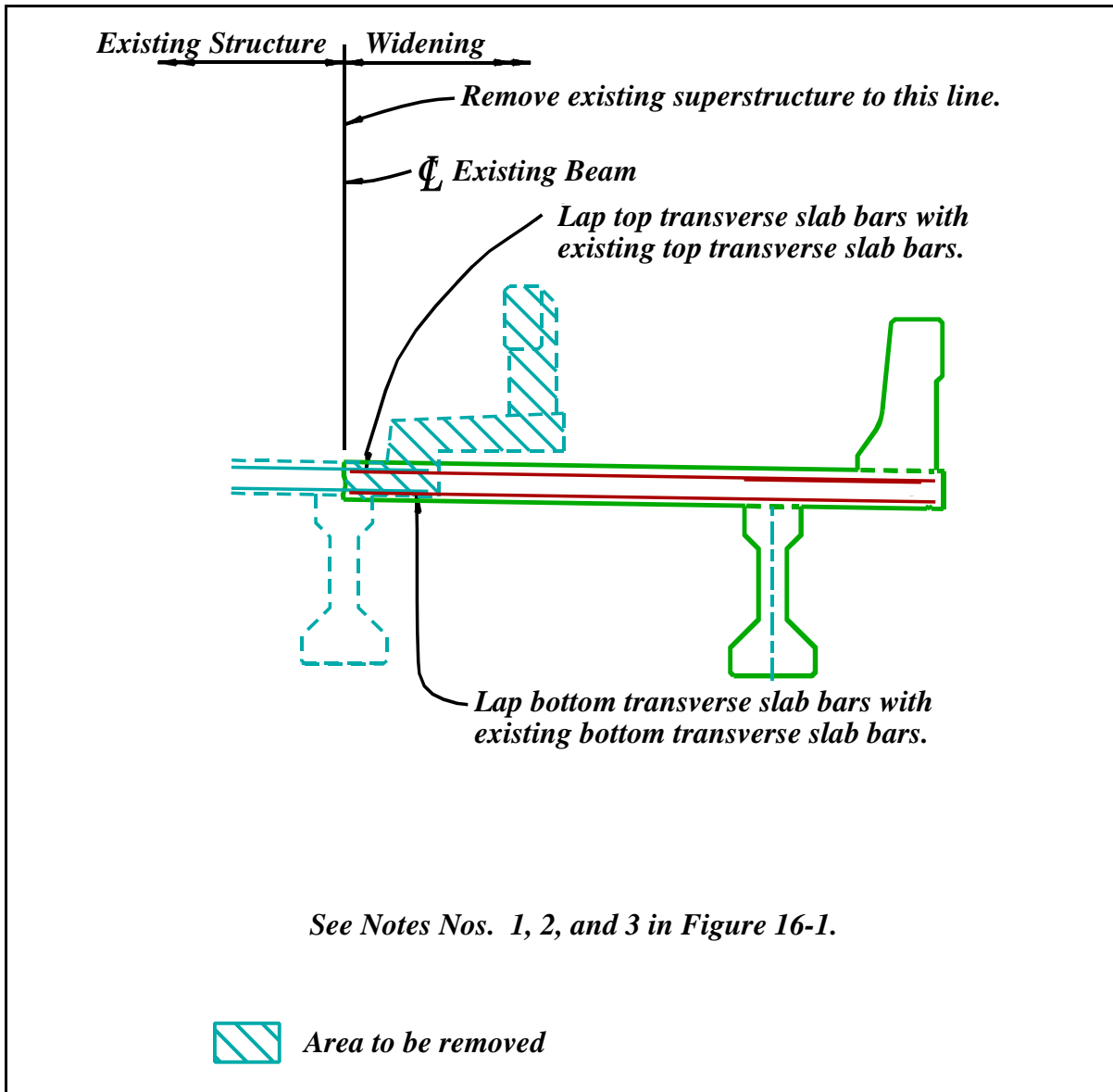
WIDENING DETAIL FOR FLAT SLAB SUPERSTRUCTURE

Figure 16-1



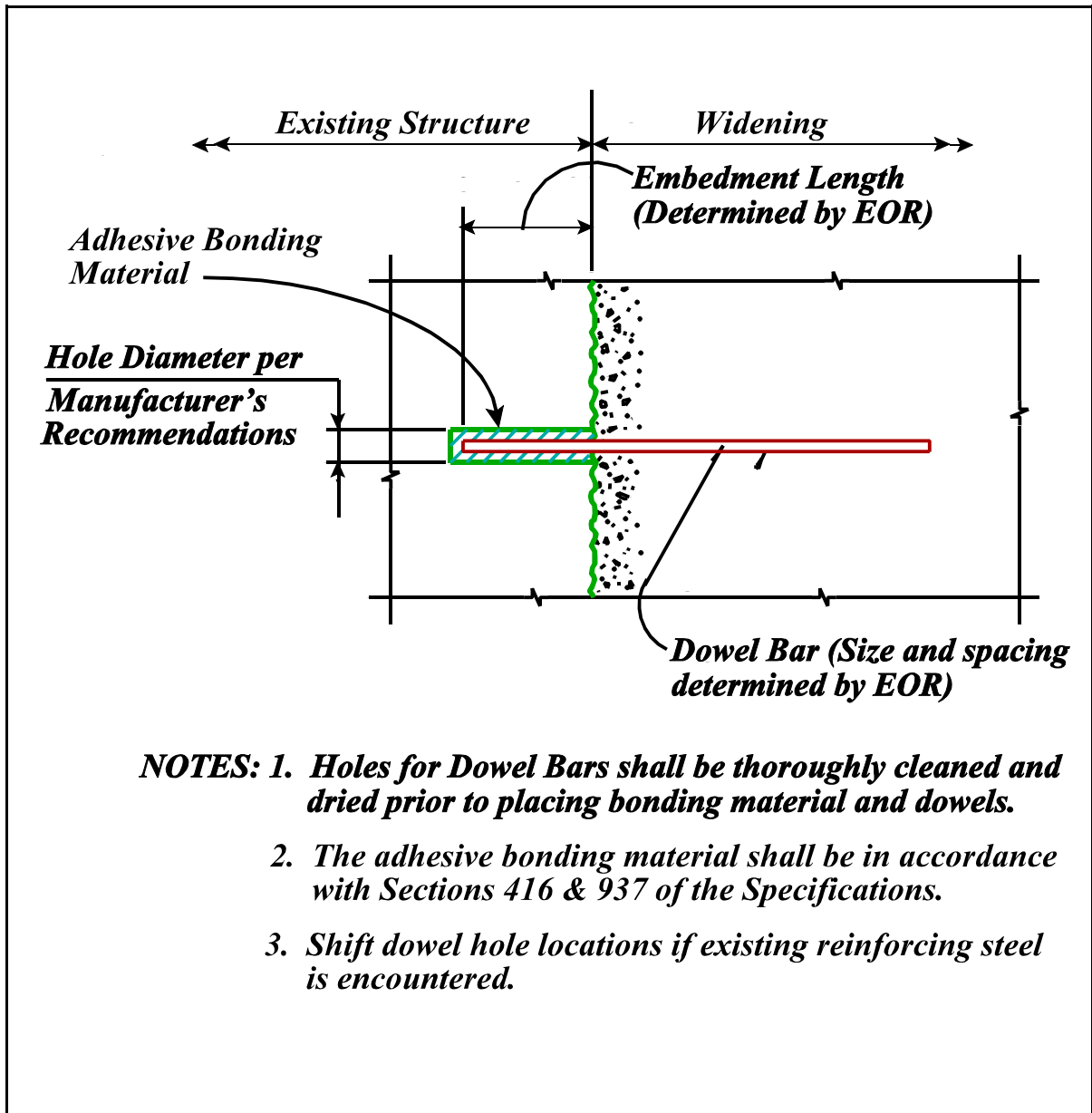
WIDENING DETAIL FOR MONOLITHIC BEAM AND SLAB SUPERSTRUCTURE

Figure 16-2



WIDENING DETAIL FOR AASHTO BEAM SUPERSTRUCTURE

Figure 16-3



WIDENING DETAIL SHOWING DOWEL INSTALLATION

Figure 16-4

Chapter 17

COMPUTER PROGRAMS FOR STRUCTURAL DESIGN, ANALYSIS, AND DRAFTING

NOTE:

The provisions of this chapter have been
relocated to the *Structures Design Office Homepage*
at
<http://www.dot.state.fl.us/structures>

Chapter 18

RETAINING WALLS

18.1 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 LFD criteria for substructure design of the AASHTO ***Standard Specifications for Highway Bridges***.

18.2 Retaining Walls

Retaining wall plans preparation and administrative requirements are included in Chapter 30 of the ***Plans Preparation Manual***, Volume 1 (Topic No. 625-000-005), which must be used in conjunction with the design requirements of this Section. Refer to Chapter 7 for the retaining wall concrete class and reinforcing steel cover requirements. The retaining wall concrete class is required to be shown on the plans.

18.2.1 Wall Types (Site Applications)

There are a number of types of retaining walls presently being used by the Department. Wall site considerations, aesthetics, maintainability and constructability shall all be considered when determining wall types 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 18-2 and 18-3).

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 meters 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 18-4 and 18-5).

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 meters in height (See Figures 18-6 and 18-7).

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 18-8) 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 meters in height. Steel sheet pile walls over 4.5 meters 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 18-9).

The EOR shall design and show on the plans the required sheet pile properties for both cold-rolled and hot-rolled sections. Design of cold-rolled sections shall be based upon 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 cold-rolled and hot-rolled sections.

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 18-10).

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 have been pre-approved. Pre-approved companies have their standard details in the **Roadway and Traffic Design Standards** or on the Structures Design Office's Internet Homepage.

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 18-12).

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 18-13).

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, problems have been encountered 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 main problem is how to remove the existing front slope and maintain the stability of the remaining soil. Several methods have been utilized. One method is to excavate slots or pits in the existing fill to accommodate soldier beams after which the soil is 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 loads and dead and live loads. The temporary wall shall not contribute to the strength of the permanent wall (See Figure 18-14).

M. Hybrid - Gravity/MSE Wall:

There are a few systems that are hybrids. They are basically a gravity system with some MSE characteristics. The particular system that has been approved by FDOT has a concrete stem that extends into the fill a sufficient length to satisfy the external stability requirement for the wall. However, due to its rigid

stem, the system is sensitive to both longitudinal and transverse settlement. This system can be used in cut and/or fill applications (See Figure 18-15).

18.2.2 Design of MSE Walls

A. Corrosion Rates:

The following corrosion rates for metallic reinforcement apply to non-corrosive (Slightly or Moderately Aggressive) Environments only:

- | | | |
|----|--|-----------------|
| 1. | Zinc (first 2 years) | 15 microns/year |
| 2. | Zinc (subsequent years to depletion) | 4 microns/year |
| 3. | Carbon Steel (after depletion of zinc) | 12 microns/year |
| 4. | Carbon Steel (75 to 100 years) | 7 microns/year |

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 is not permitted.** Passive metal soil

reinforcement; i.e., stainless steel, aluminum alloys, etc., is permitted for use only with written approval of the SDO.

7. Geosynthetic Geogrids shall comply with Chapter 31 of the **Plans Preparation Manual**, Volume I (Topic No. 625-000-005).
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
 - (1) $L = 2.5$ meters minimum length
 - (2) $L = 0.7 H_1$, $H_1 =$ mechanical height of wall, in meters (See Figures 18-16 and 18-17)
 - (3) $L =$ length in meters, required for external stability design
 - b. MSE Walls In Front of Abutments on Spread Footings
 - (1) $L = 6.7$ meters
 - (2) $L = 0.6 H_1 + 1.8$ meters
 - (3) $L = 0.7 H_1$

Note: As a rule of thumb, for an MSE structure with reinforcement lengths equal to 70% of mechanical height, the 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 18-16 and 18-17 and can be calculated using the equation provided in Figure 18-16. 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 18-16 and 18-17 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 or stem lengths for the external conditions shown in Figure 18-1 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 meters 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 18-18).

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 18-19.

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 15E.
- (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):

The basic (typical) panel size shall not exceed 3.75 m² in area, and special panels (top out, etc.) shall not exceed 4.65 m² 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 the requirements of Chapter 7.

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} , 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:

Use a soil resistance factor of 0.67 for overturning.

21. Polymeric Reinforcements in Soils:

For geosynthetics, T_a shall not exceed the following values:

- a. $T_a = 19\% T_{ult}$ 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 18-18.

D. Utilities:

No utilities should be allowed in the soil reinforced zone behind MSE or tied-back walls. When utilities are placed in the reinforced zone, they cannot be maintained because excavation in this zone will destroy the structural integrity of these types of walls. Furthermore, pipes carrying fluids, if cracked or broken, could also wash out and destroy the structural integrity of the wall.

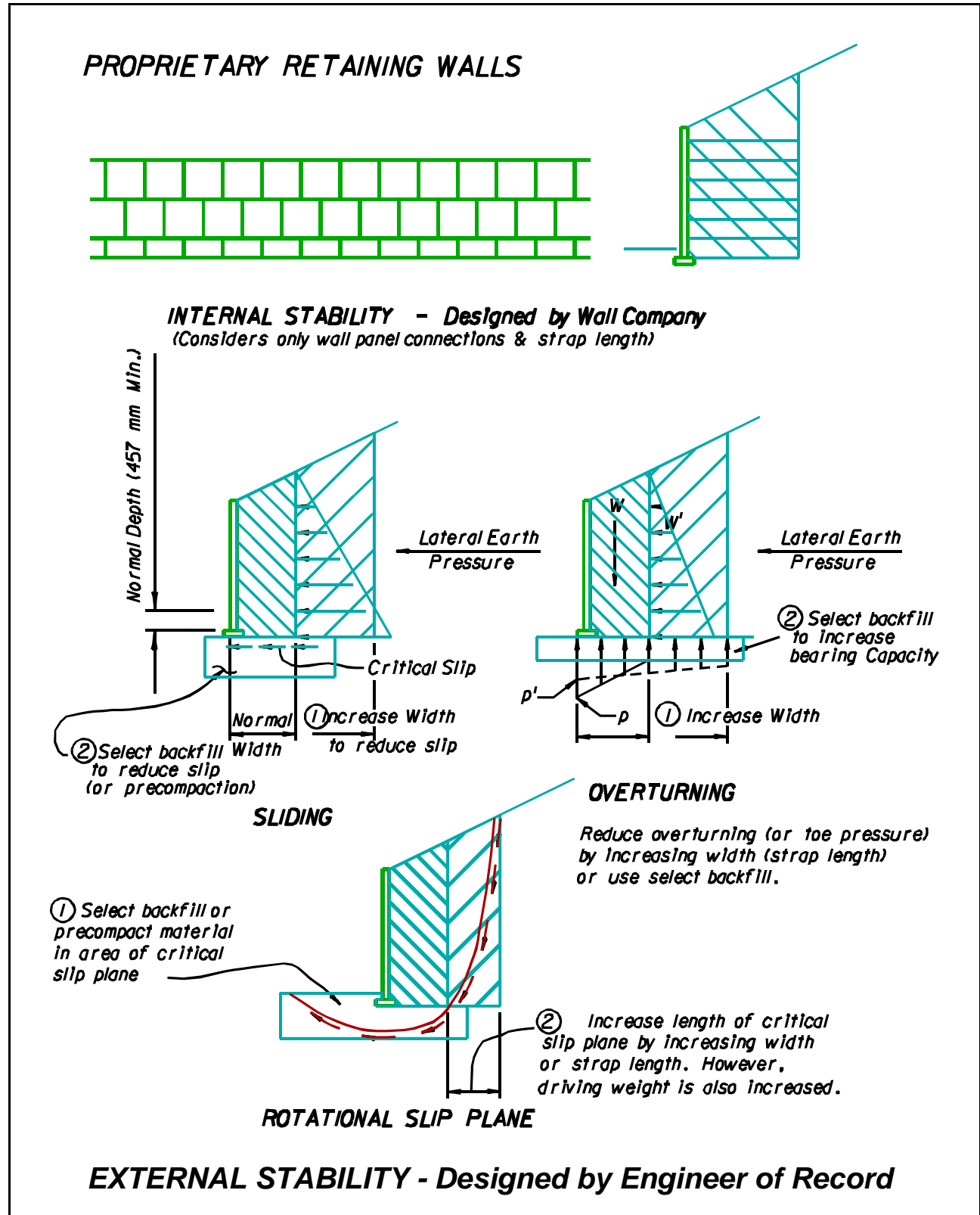


Figure 18-1

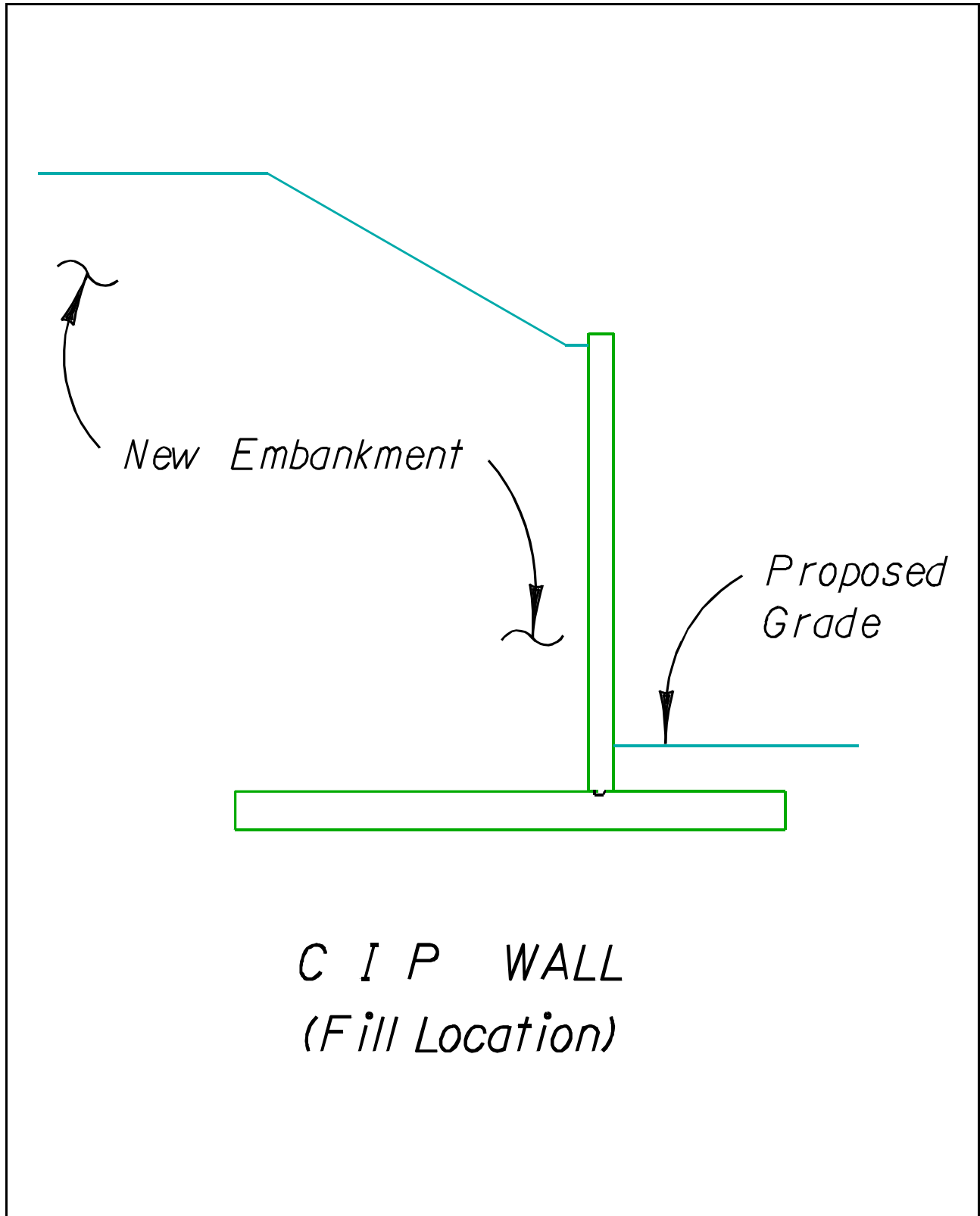


Figure 18-2

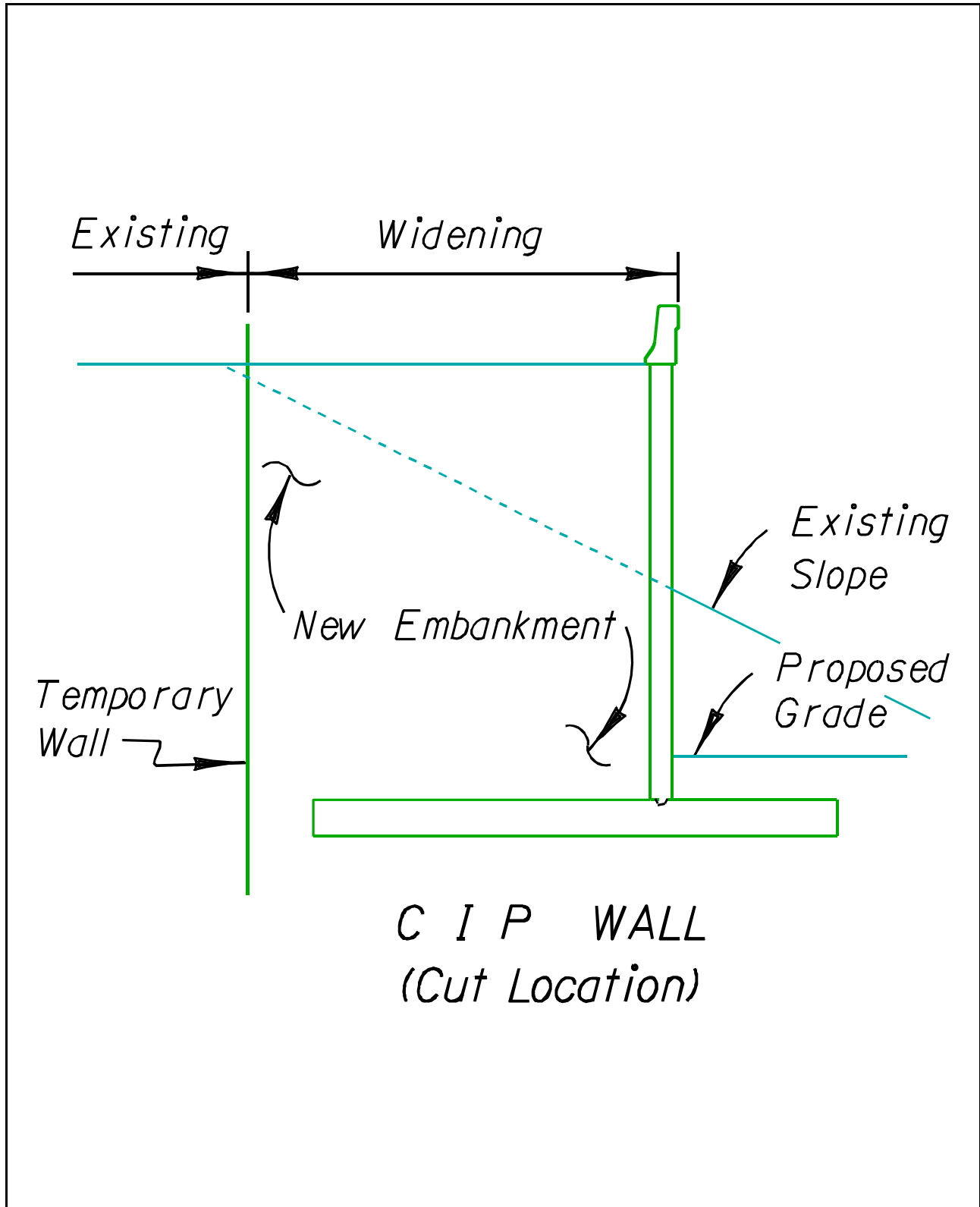


Figure 18-3

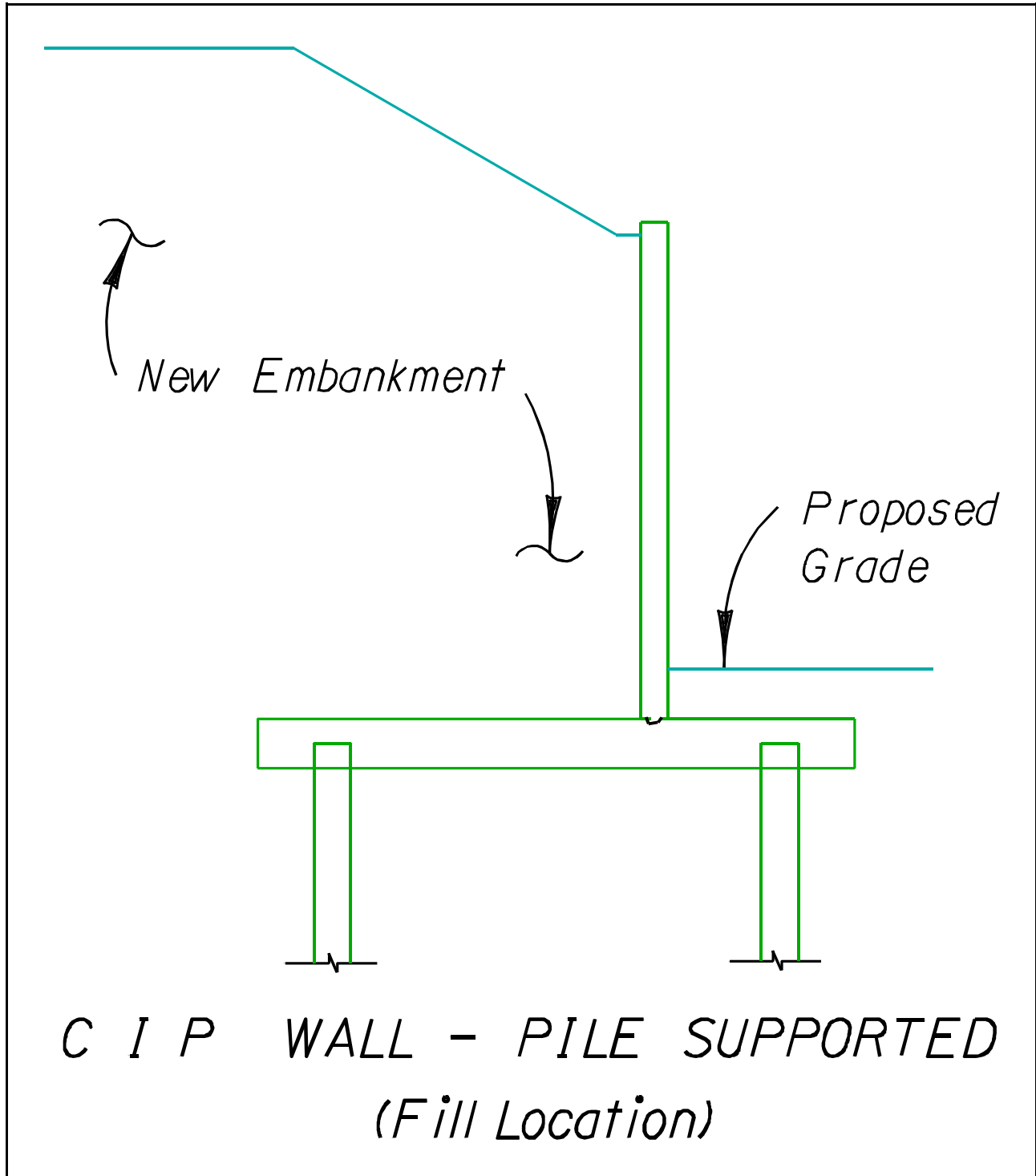


Figure 18-4

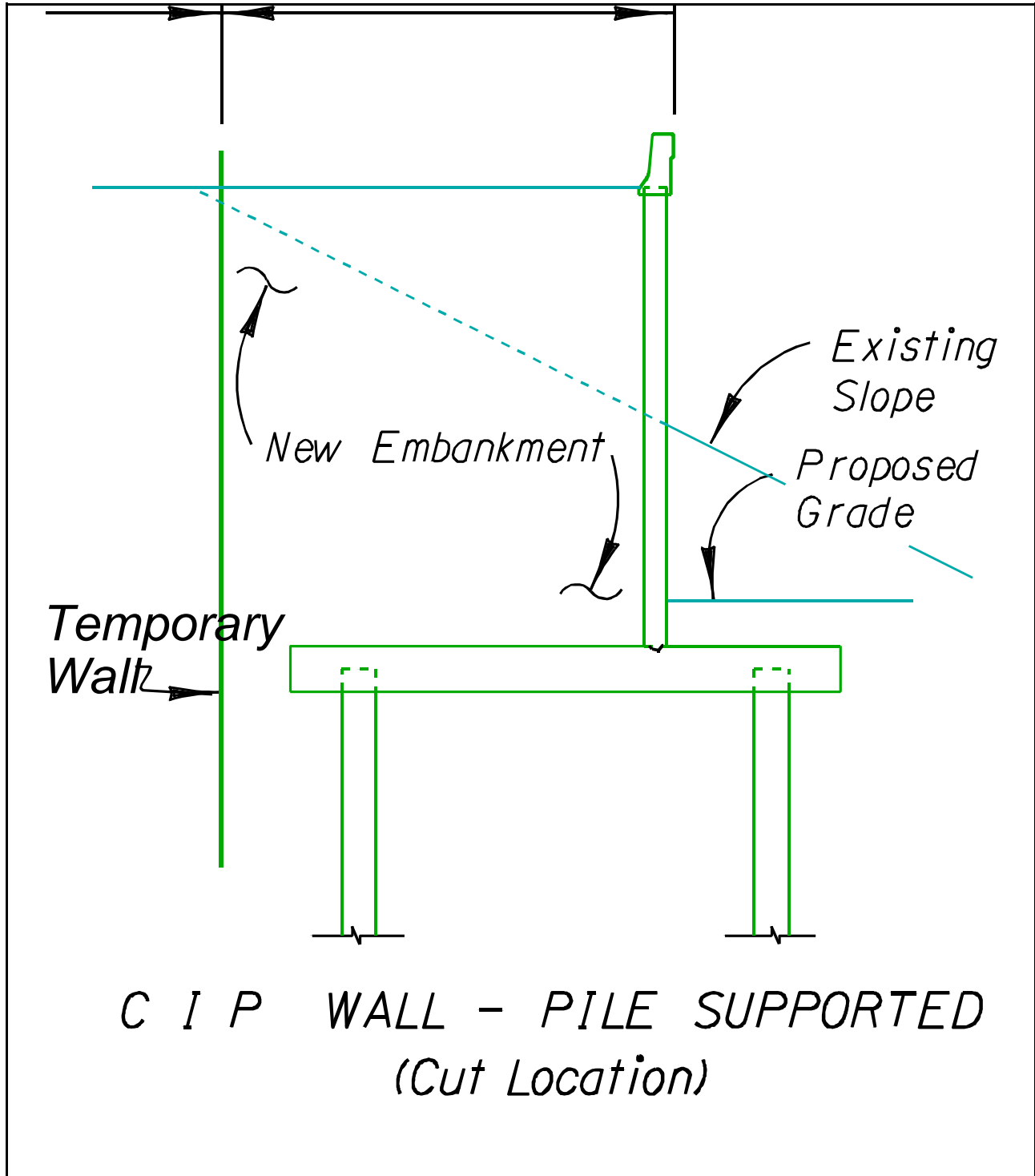


Figure 18-5

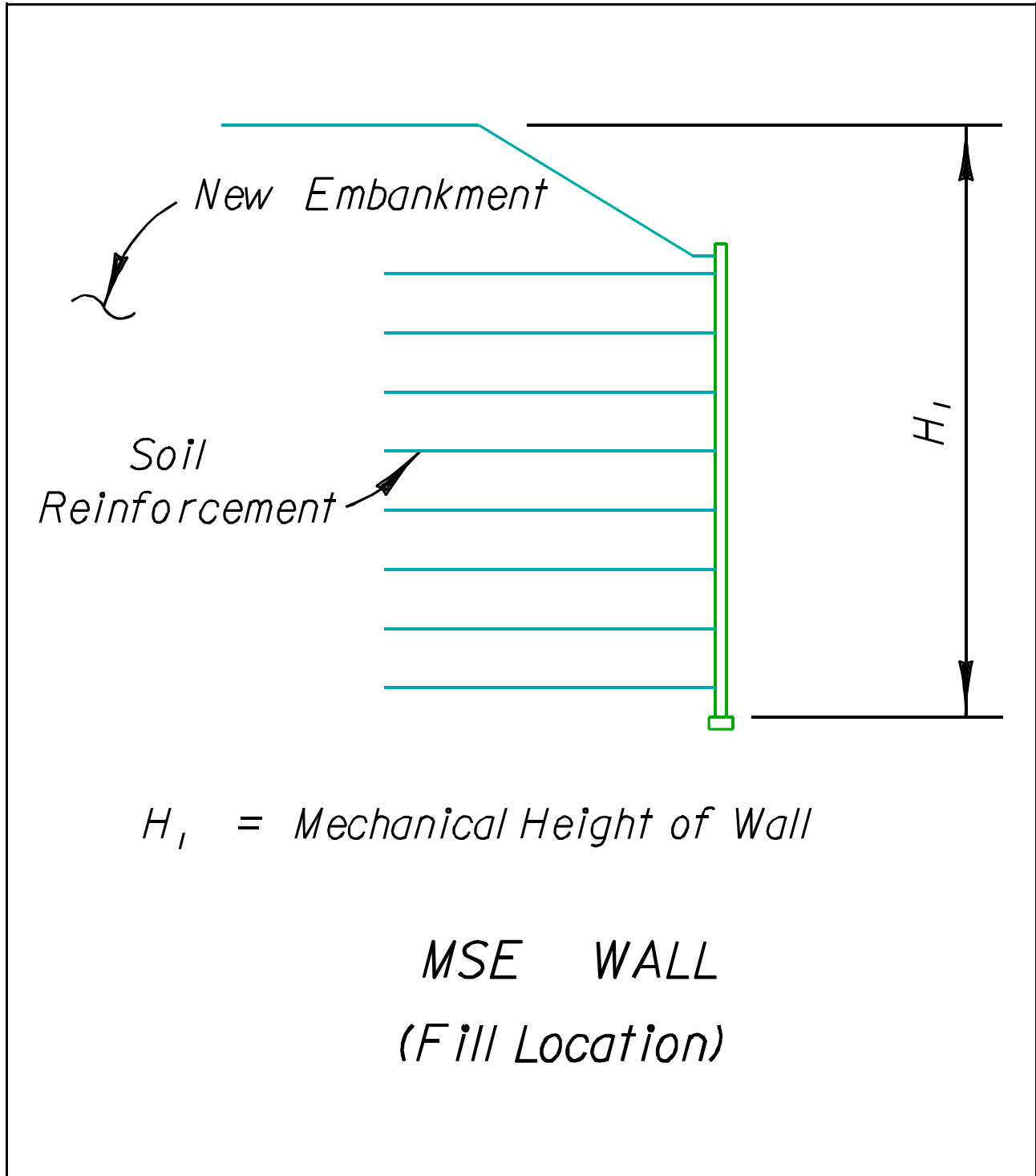


Figure 18-6

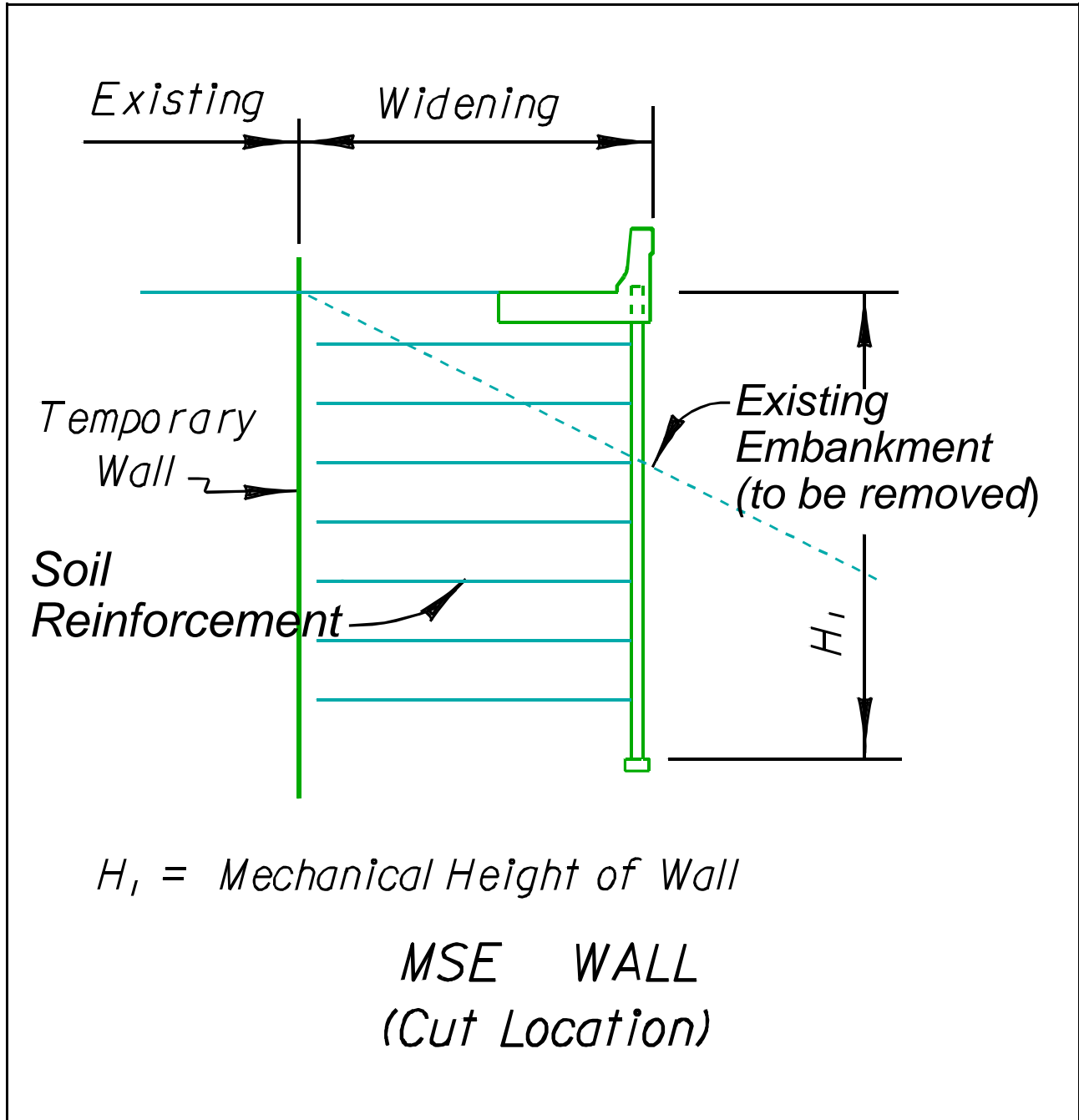


Figure 18-7

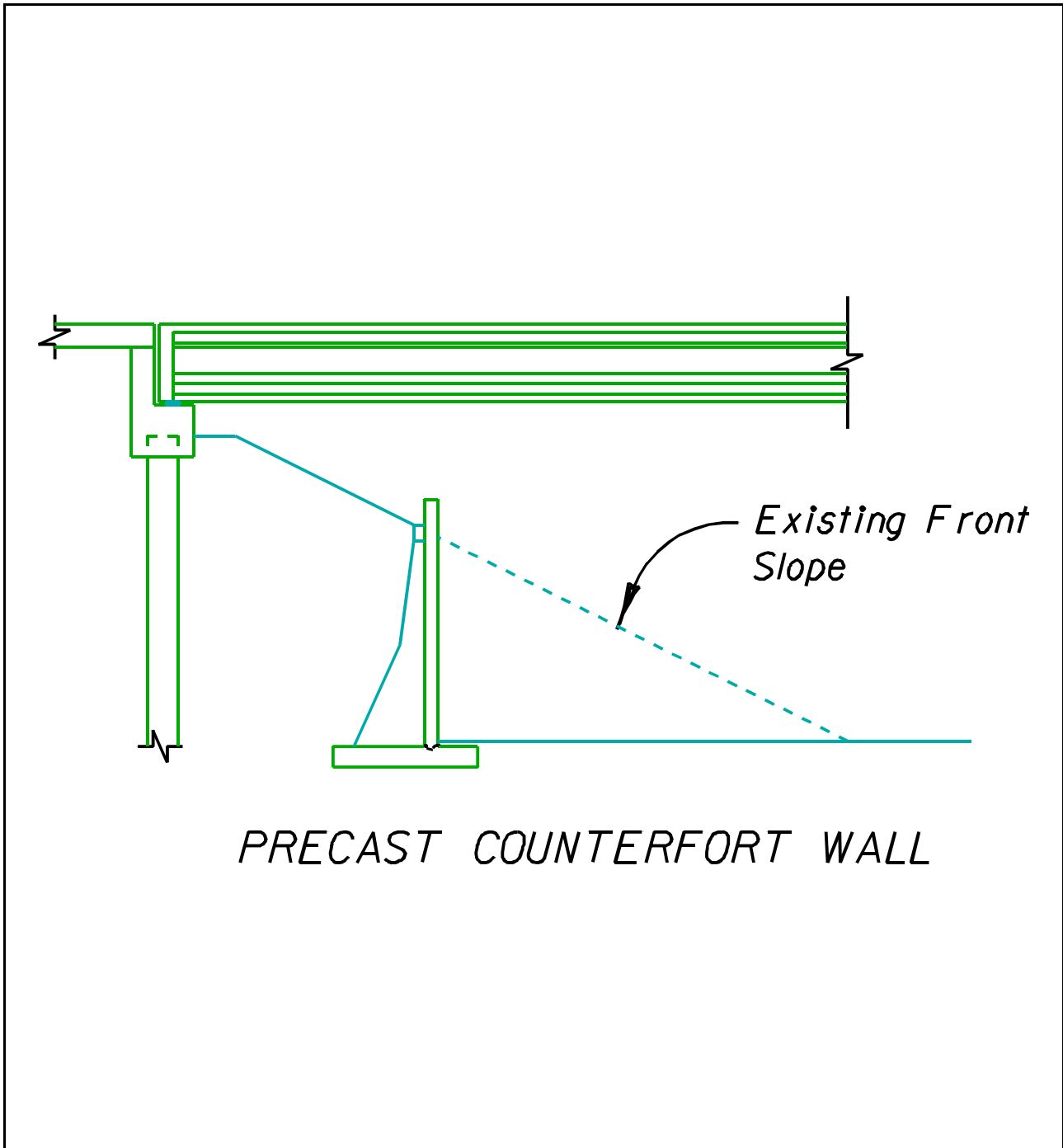


Figure 18-8

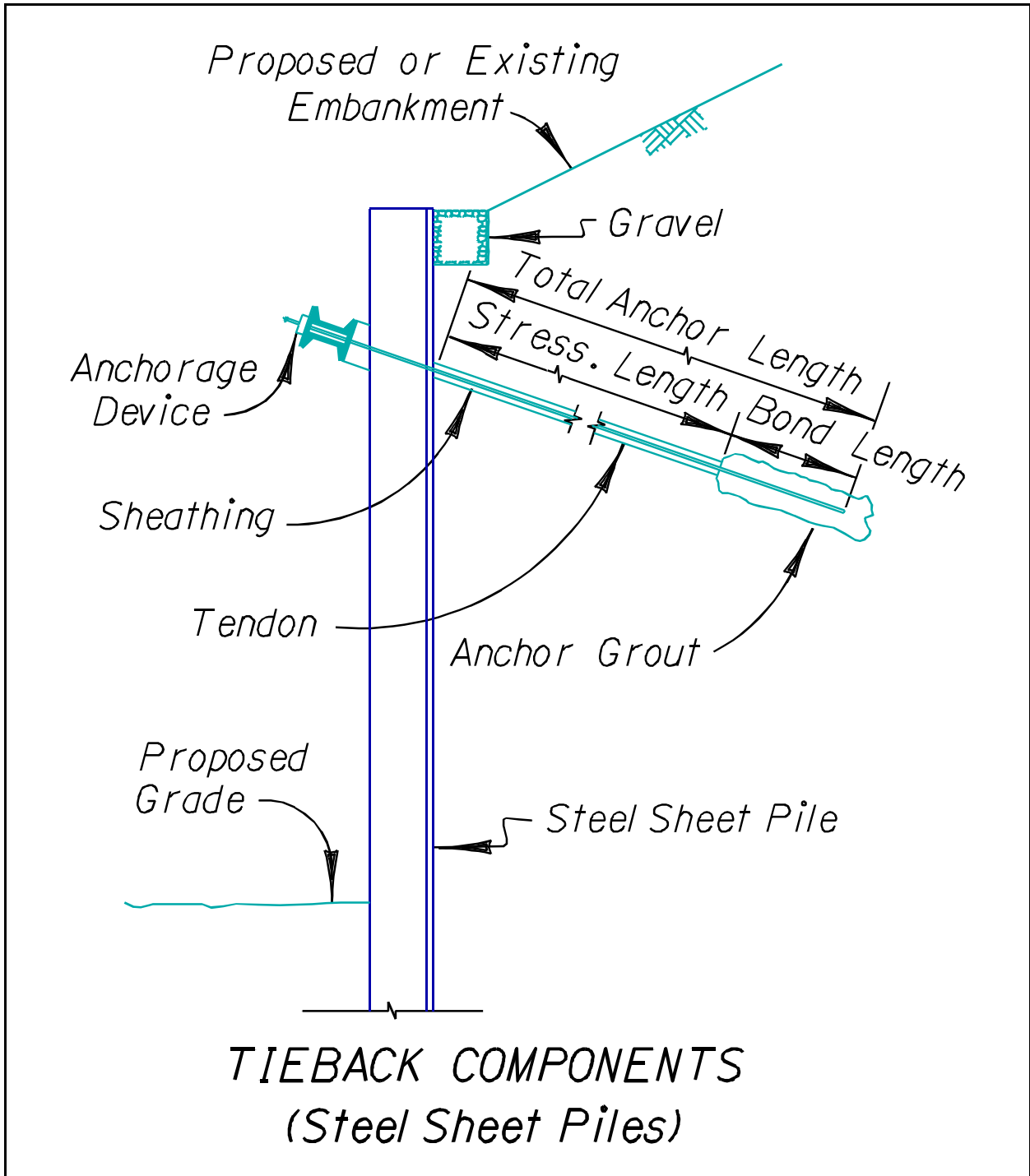


Figure 18-9

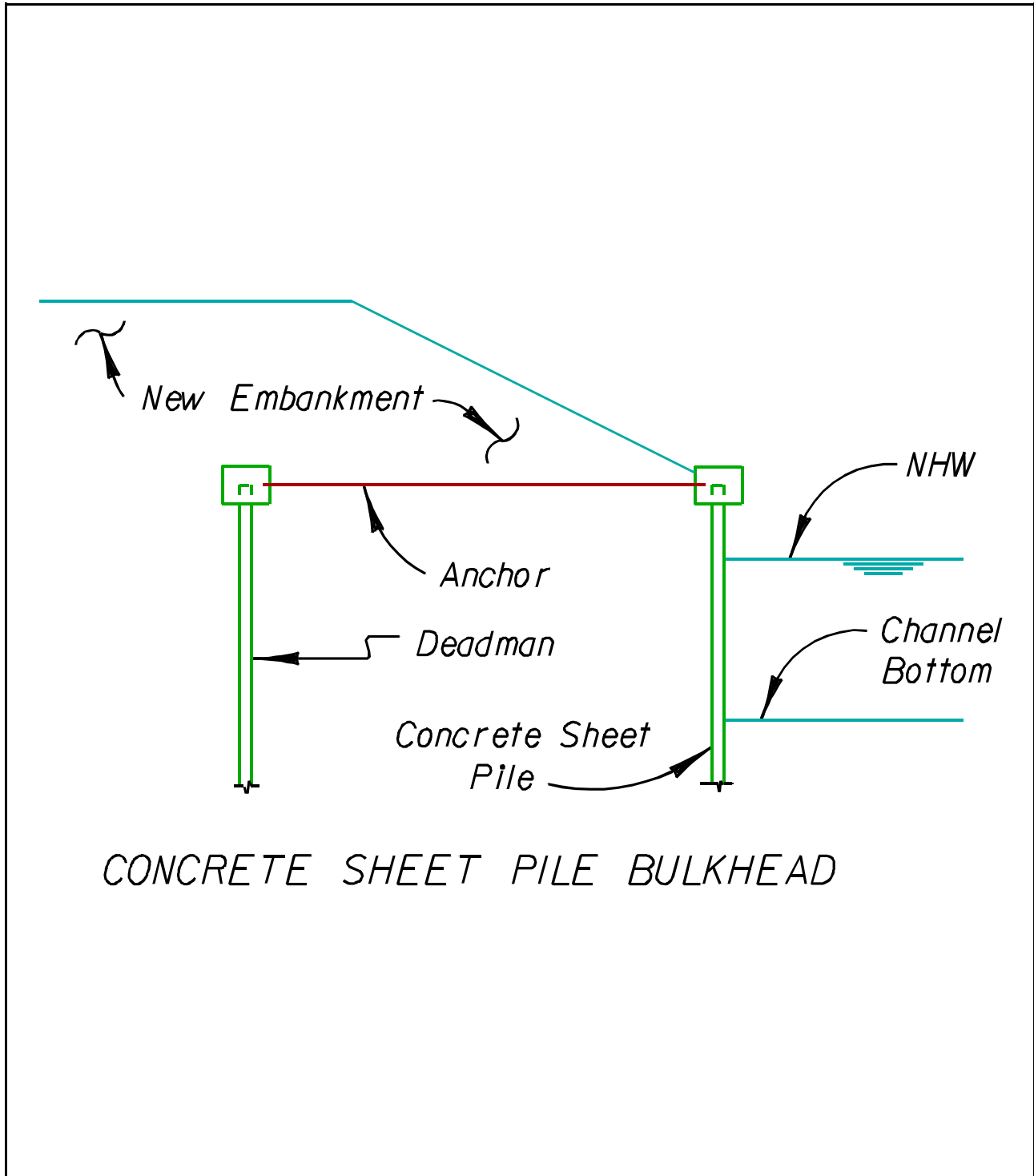


Figure 18-10

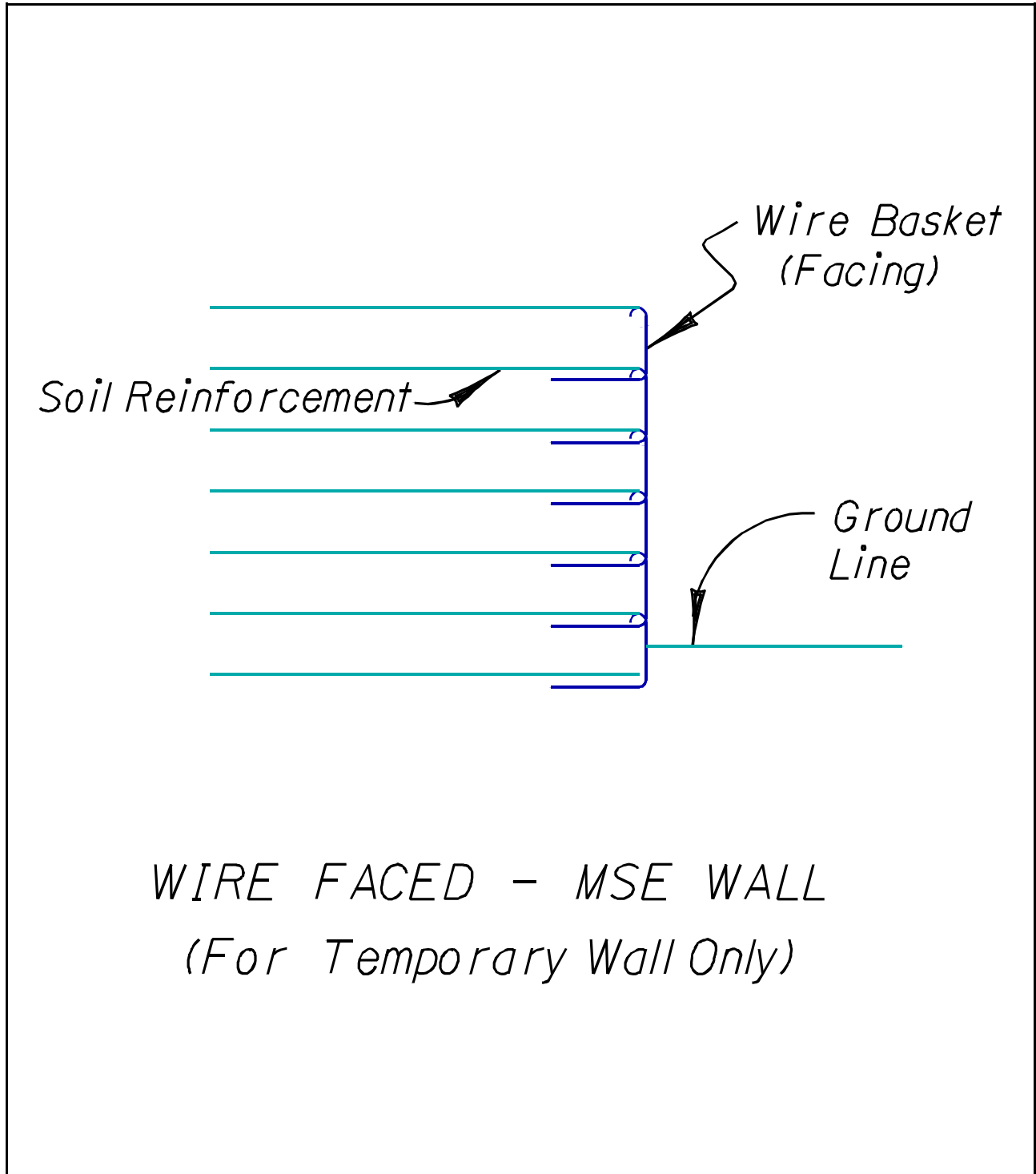


Figure 18-11

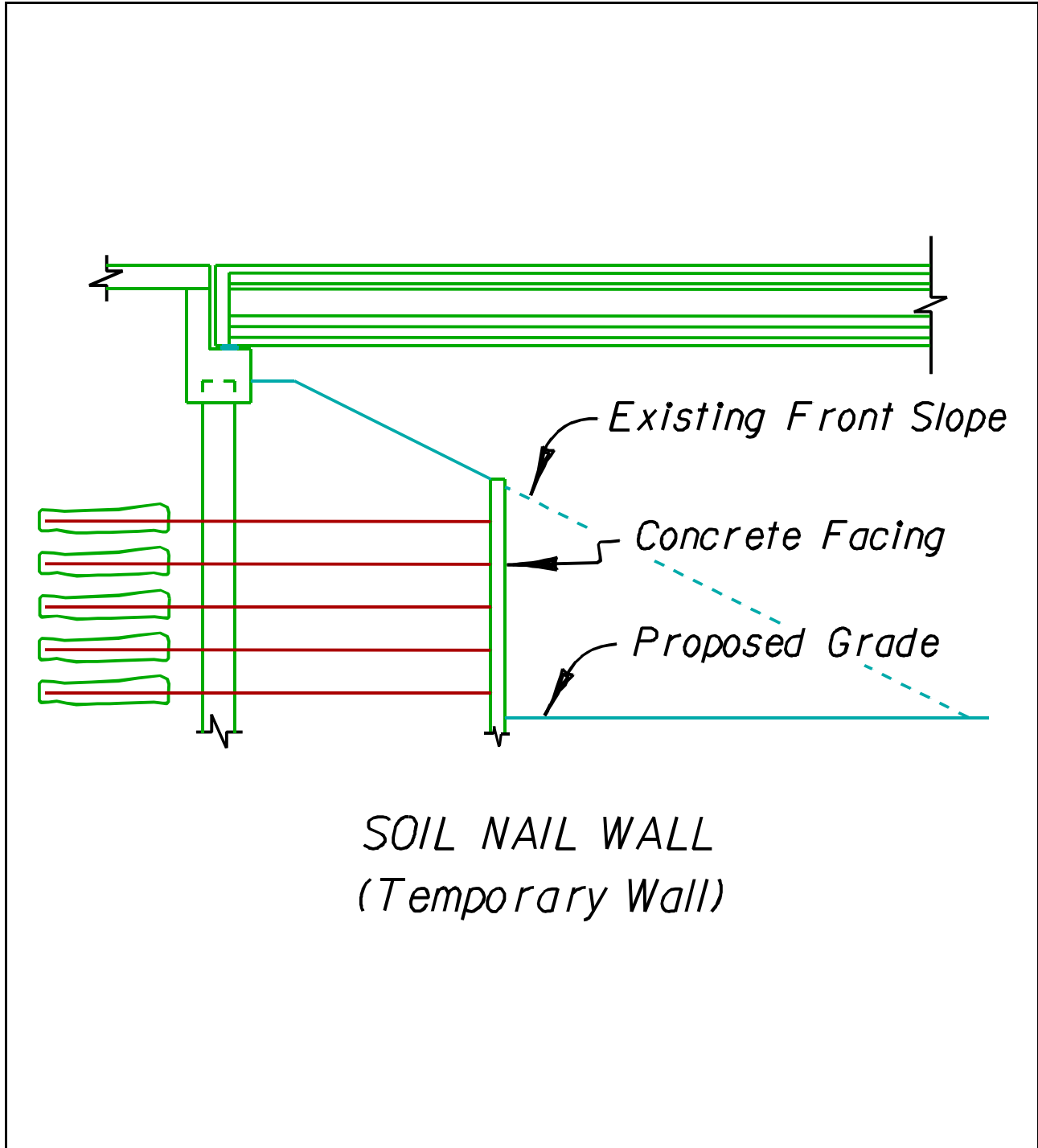


Figure 18-12

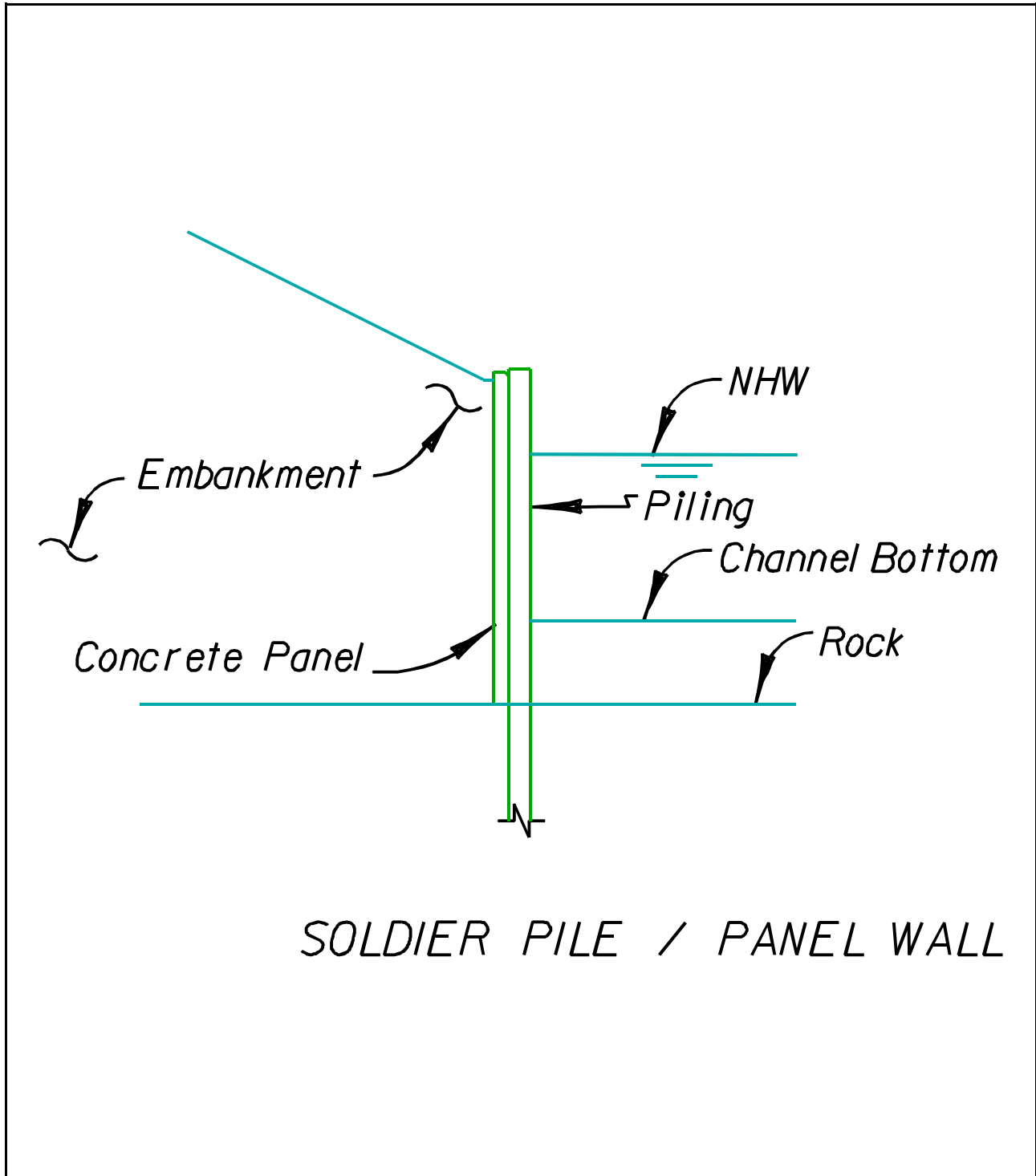


Figure 18-13

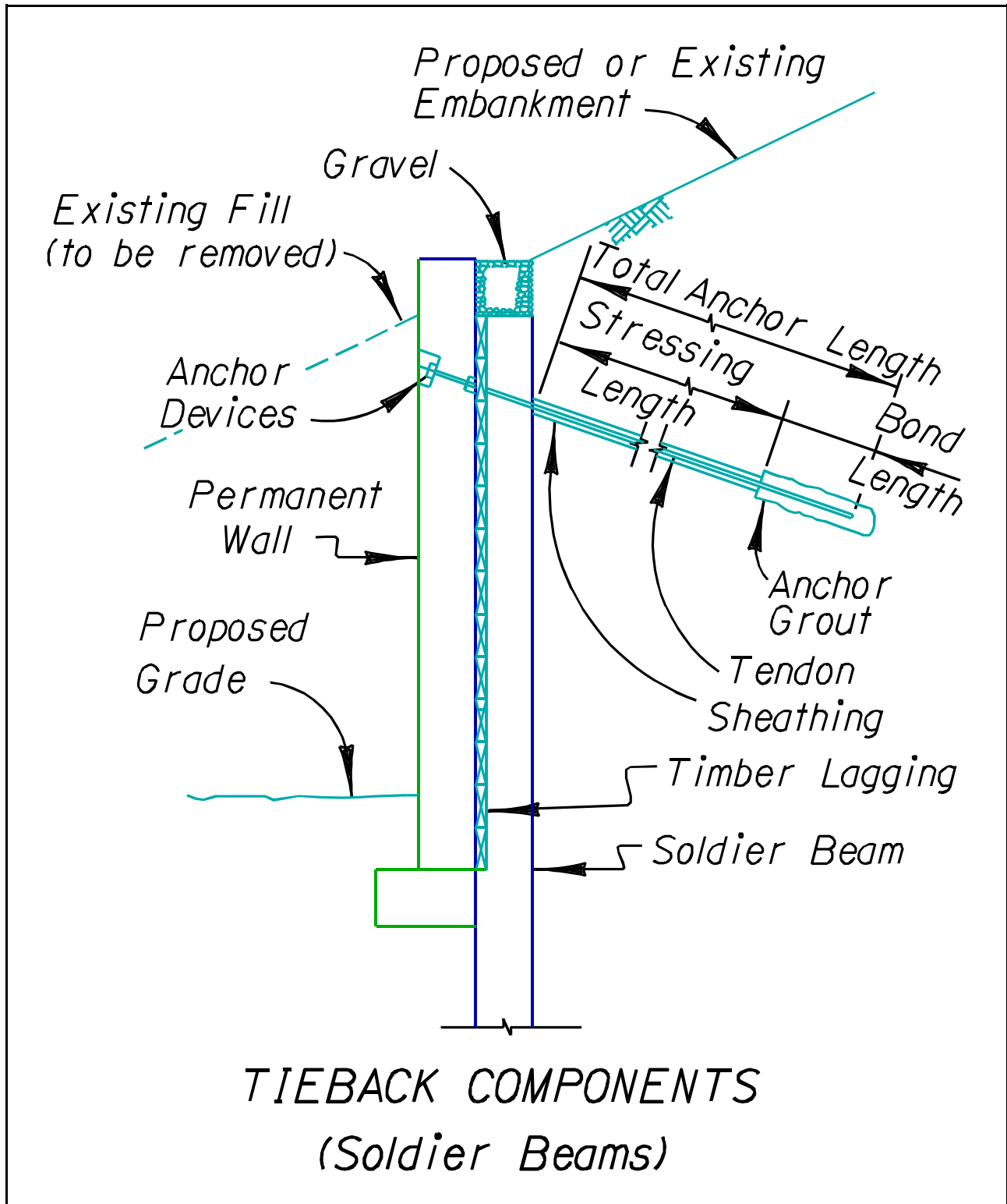


Figure 18-14

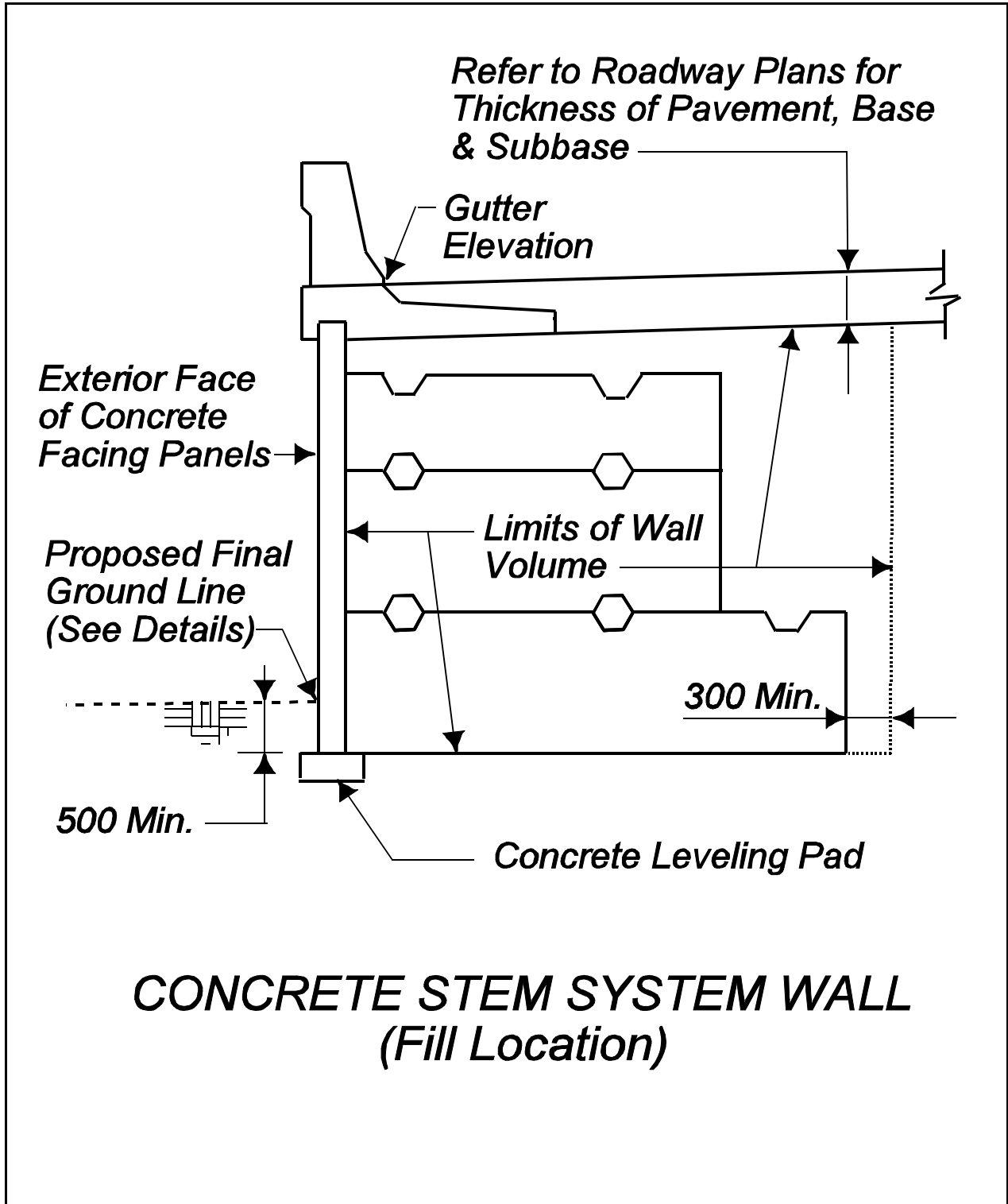


Figure 18-15

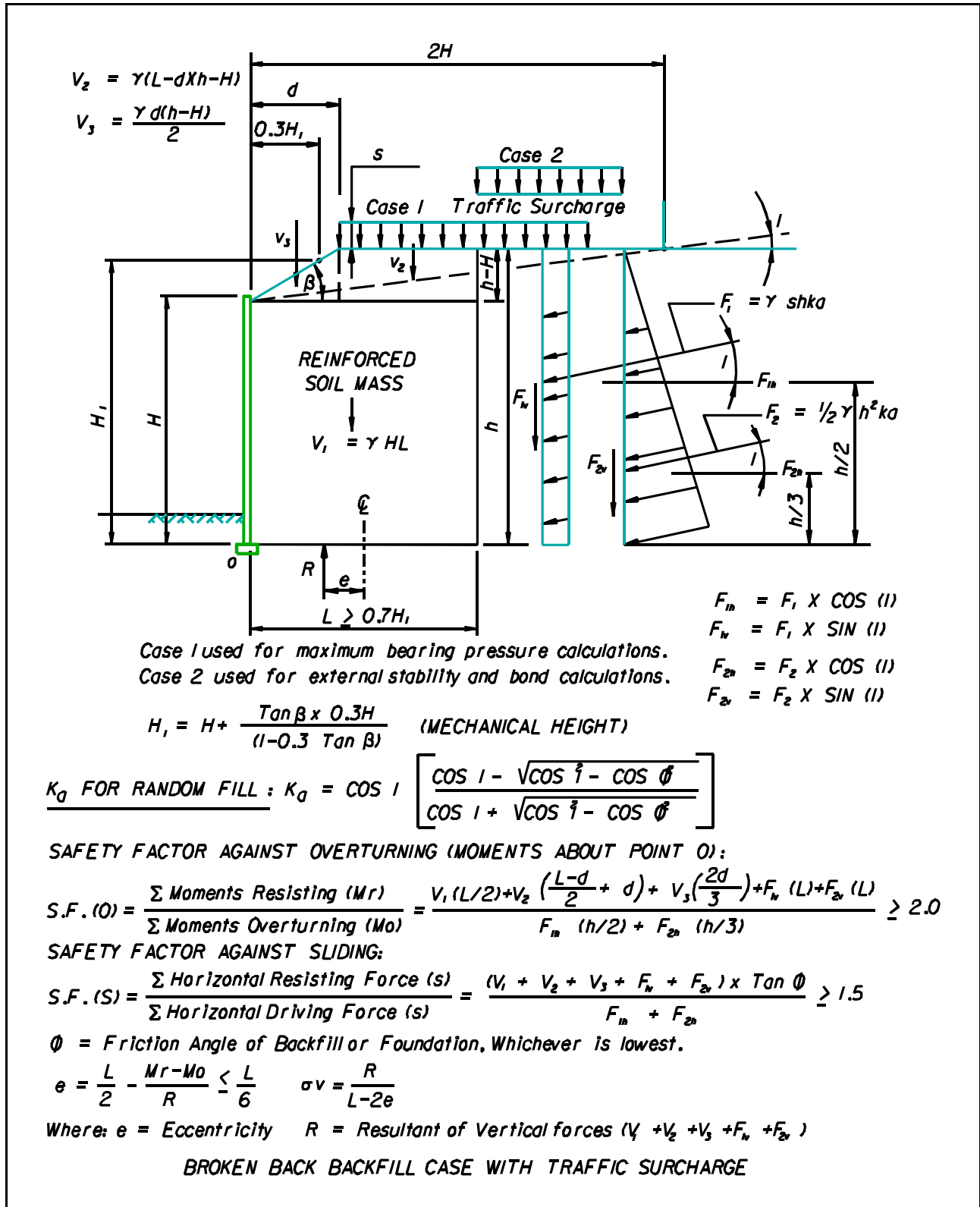


Figure 18-16

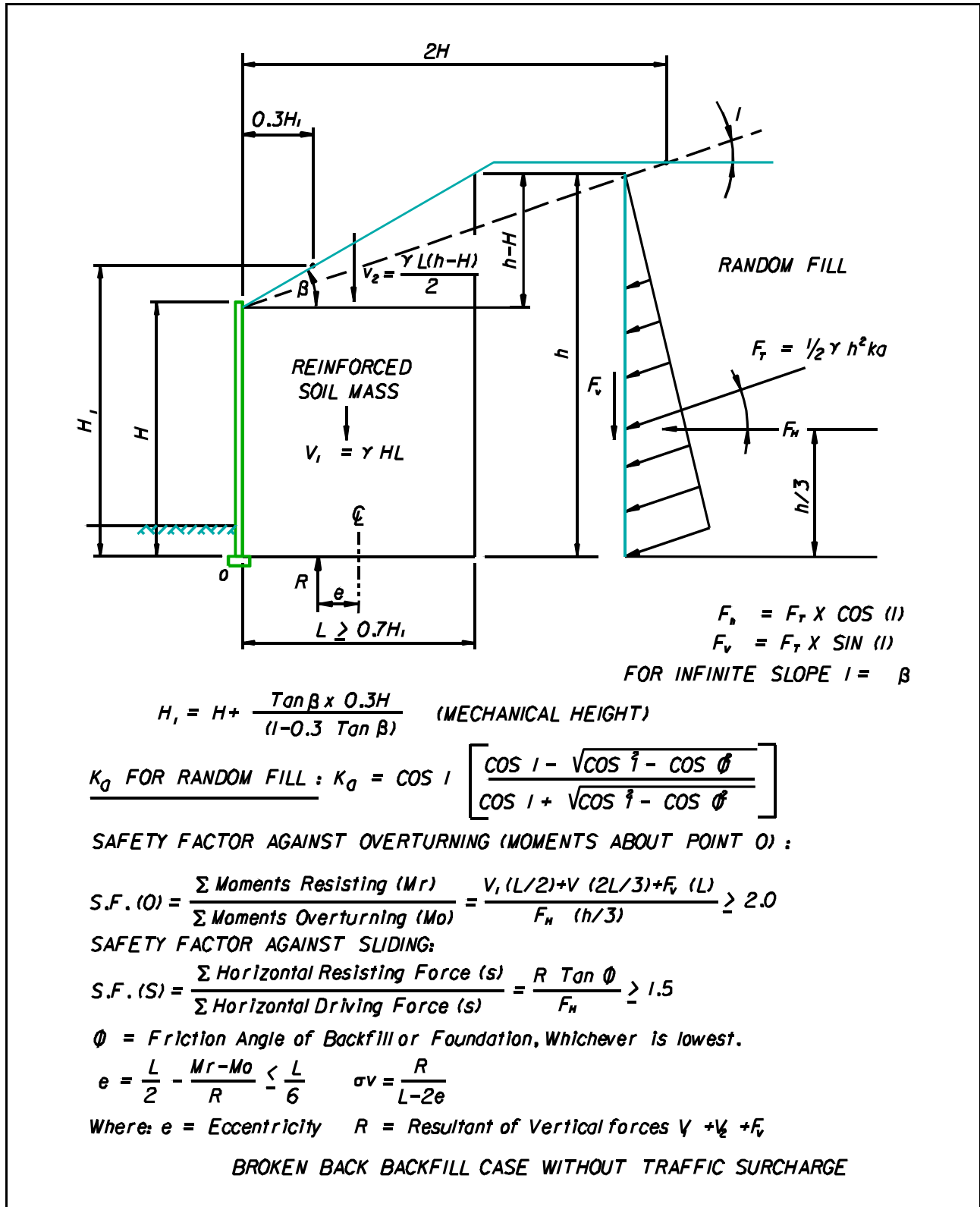
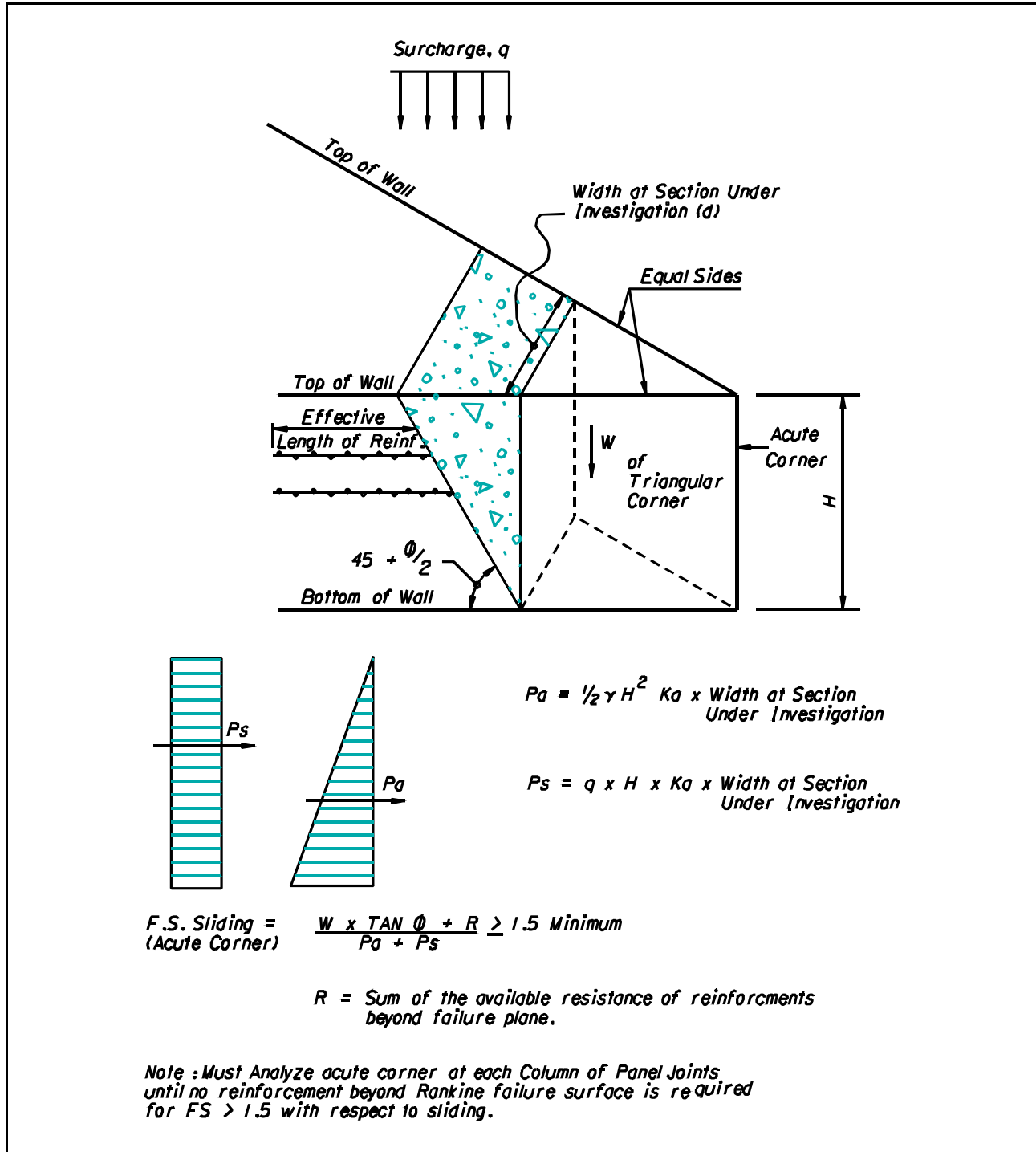


Figure 18-17



ACUTE CORNER PRESSURE DIAGRAM

Figure 18-18

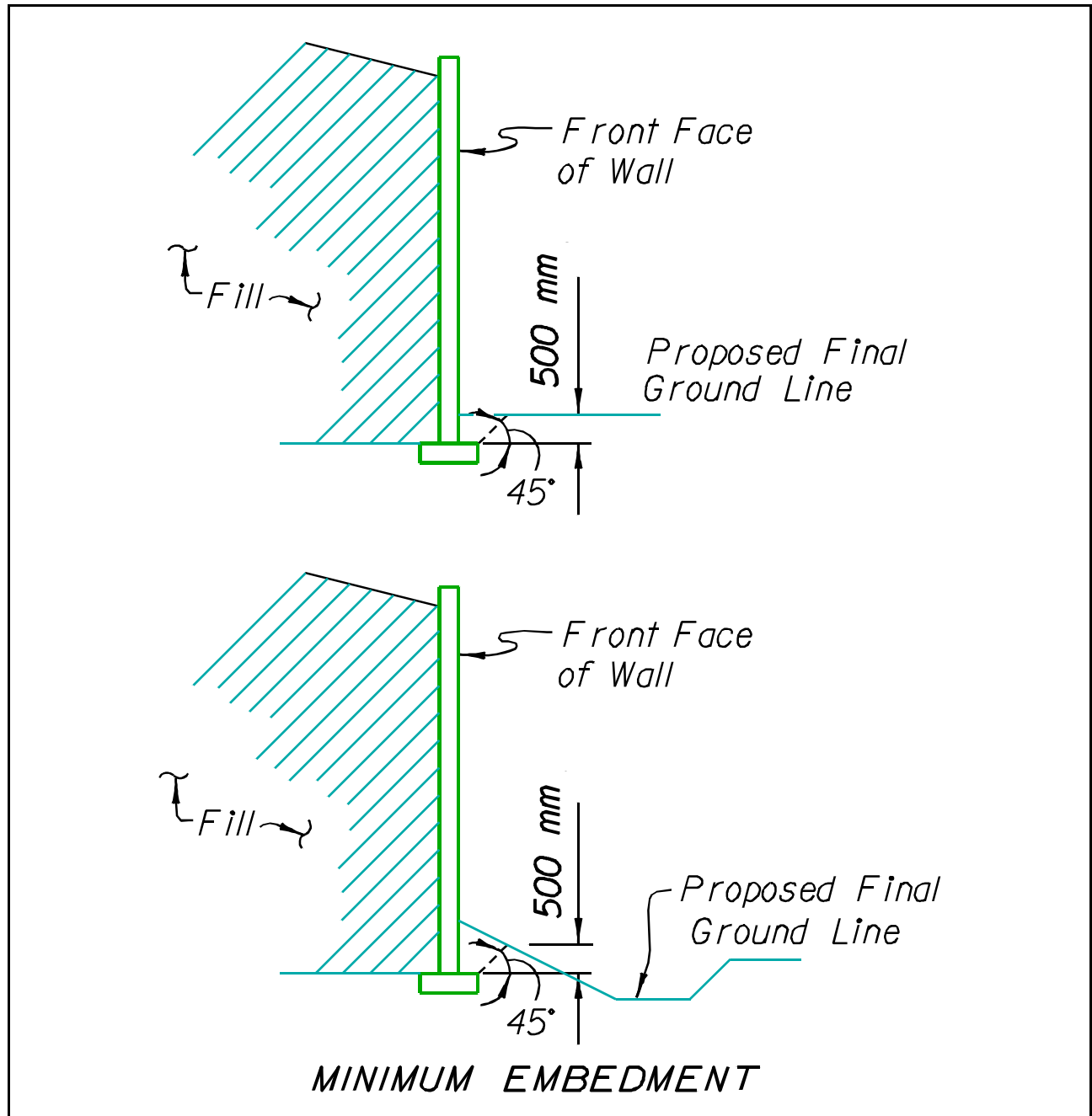


Figure 18-19

Chapter 19

SHOP DRAWINGS AND ERECTION DRAWINGS

NOTE:

The provisions of this chapter have been relocated to the *Plans Preparation Manual* (Topic No.: 625-000-005), Chapter 28.

Chapter 20

SIGNING AND SEALING PLANS

NOTE:

The provisions of this chapter have been relocated to the *Plans Preparation Manual* (Topic No.: 625-000-005), Chapter 19.

Chapter 21

MISCELLANEOUS HIGHWAY RELATED STRUCTURES

NOTE:

The provisions of this chapter have been relocated to the *Plans Preparation Manual* (Topic No.: 625-000-005), Chapter 29.