STRUCTURES DESIGN GUIDELINES
FOR
LOAD AND RESISTANCE
FACTOR DESIGN

STRUCTURES DESIGN OFFICE

TALLAHASSEE, FLORIDA

JANUARY 2003

Florida Department of Transportation

JEB BUSH
GOVERNOR

605 Suwannee Street
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SECRETARY

January 24, 2003

M E M O R A N D U M

TO: All Users of the Florida Department of Transportation
Structures Design Guidelines

FROM: William N. Nickas, State Structures Design Engineer

COPIES: State Highway Engineer, Freddie Simmons
Director Office of Design, Bob Greer
FHWA, Doug Edwards
Structures Design Engineers (William Domico, Bob Nichols, Jack Evans, Larry Sessions, And Marcus Ansley)
State Geotechnical Engineer, Larry Jones
District Structures Design Engineers (Gerard Moliere, Rod Nelson, Keith Shores, John Danielsen, Neil Kenis, Kim Saing, Jose Rodriguez, and Agnes Spielmann)
District Structures and Facilities Engineers (Pepe Garcia, Bud Rosier, John Locke, Jorge Martos, Ron Meade, Frank Guyamier, and Tom Reynolds)

SUBJECT: FDOT Structures Design Guidelines Topic No. 625-020-154-b
Structures Design Guidelines (LRFD)

The Florida Department of Transportation Structures Design Office has officially released the January 2003 Structures Design Guidelines (LRFD). Copies of the Guidelines, now part of the Structures Manual, can be downloaded from the Structures Office website at www.dot.state.fl.us/structures.

The Structures Manual is compiled and delivered in a Windows Help format including hyperlinks, easy navigation, indexes, and advanced search capabilities. This is a major change in viewing format, but an Adobe Acrobat file (pdf format) is included for printing purposes.

The Structures Manual will be updated every six months (January and July) and serve as the single source for FDOT design criteria for structural designs prepared by the Department and consultant designers. All users of the Manual are encouraged to submit suggestions to improve and further develop the contents thereby providing maximum assistance to the structural engineering community and particularly to those unfamiliar with FDOT policies and design criteria. Suggestions should be submitted to Robert E. Nichols, Florida Department of Transportation, 605 Suwannee Street, MS 33, Tallahassee, Florida 32399-0450, or e-mailed to bob.nichols@dot.state.fl.us.

The design criteria shall be incorporated on all future projects and on those projects currently being developed (i.e., when design and detailing has not already reached final stages). Where incorporation is not feasible, a request for a variance shall be submitted. Specific questions about the Guidelines or their application to a particular project shall be addressed to the appropriate District Structures Design Office.

WNN:ps

Structures Design Guidelines

Table Of Contents

Structures Design Guidelines
INTRODUCTION

I.1 General

These Structures Design Guidelines, (***SDG***) incorporate technical design criteria and include additions, deletions, or modifications to the requirements of the ***AASHTO/LRFD Bridge Design Specifications, Second Edition (1998 with latest interim standards), (LRFD)***.

This manual consists of text, figures, charts, graphs, and tables and provides engineering standards, criteria, and guidelines for developing and designing structures for which the Structures Design Office (SDO) and District Structures Design Offices (DSDO) have overall responsibility. "Structures" include bridges, overhead sign structures, earth retaining structures, and miscellaneous roadway appurtenances.

All general administrative, geometric, shop drawing, and plans processing requirements have been incorporated in the ***Plans Preparation Manual***.

I.2 Format

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

The ***SDG*** are written primarily in the active voice to designers (Professional Engineers, Engineers of Record, Structural Engineers, Geotechnical Engineers, etc.) of bridges and structures for the Florida Department of Transportation and includes article-by-article commentary when appropriate.

I.3 Authority

Section 334.044(2), Florida Statutes.

I.4 Scope

The use of this manual is required of anyone performing structural design or analysis for the Florida Department of Transportation.

I.5 Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
DSDE	District Structures Design Engineer
DSDO	District Structures Design Office
FHWA	Federal Highway Administration
FDOT	Florida Department of Transportation
LRFD	Load and Resistance Factor Design
PPM	Plans Preparation Manual
SDG	Structure Design Guidelines
SDO	Structures Design Office
SSDE	State Structures Design Engineer
TAG	Technical Advisory Group
TDB	Temporary Design Bulletin

I.6 Referenced Documents

Use the ***SDG*** in conjunction with the following documents:

A. AASHTO Load and Resistance Factor Design Bridge Specification (LRFD)

B. FDOT Plans Preparation Manual, Volume 1 (Topic No.: 625-000-007)

C. FDOT Plans Preparation Manual, Volume 2 (Topic No.: 625-000-008)

D. FDOT Detailing Manual (Topic No.: 625-020-200)

E. FDOT Drainage Manual (Topic No.: 625-040-001)

F. FDOT Standard Drawings (Topic No.: 625-020-300)

G. FDOT CADD User's Manual

I.7 Coordination

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

Direct all questions concerning the applicability or requirements of any of these or other referenced documents to the appropriate Structures Design Engineer.

I.8 Distribution

This ***SDG*** is furnished via the SDO web page at no charge. The user must regularly check for additions, modifications and bulletins. Address questions regarding this manual and any modifications to:

Structures Design Office
Mail Station 33
605 Suwannee Street
Tallahassee, Florida 32399-0450
Tel.: (850) 414-4255
http://www11.myflorida.com/structures

I.9 Registration

No registration is required.

I.10 Administrative Management

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

I.10.1 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.10.2 SDO Staff

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

I.11 Modifications

Modifications may be the 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. Manual users are encouraged to suggest modifications and improvements such as design procedures, text clarity, technical data, or commentary. Transmit suggestions in writing to the State Structures Design Engineer, 605 Suwannee St, (ms 33), Tallahassee, FL 32399.

I.11.1 Adoption of Revisions

Revisions to the ***SDG*** are issued by the SDO as Temporary Design Bulletins or Permanent Revisions according to a formal adoption process.

I.11.2 Temporary Design Bulletins

TDB's supersede the current ***SDG,*** are mandatory, and will be issued when the SSDE deems a change to be essential to production or structural integrity issues and in need of immediate implementation. ***TDB's*** may address issues in plans production, safety, structural design methodology, critical code changes, or new specification requirements.

TDB's are effective for up to 360 calendar days unless superceded by subsequent ***TDB's*** or Permanent Revisions to the SDG. ***TDB's*** automatically become proposed Permanent Revisions unless withdrawn from consideration by the SSDE.

TDB's indicate their effective date of issuance, include the reference Topic Number, are numbered sequentially with reference to both the ***SDG*** version number and year of issuance. For example, Temporary Design Bulletin No. C98-2 would be the second Bulletin issued in 1998 for ***SDG***.

I.11.3 Permanent Revisions

Permanent Revisions to the manual are made semi-annually or "as-needed." If the SDO considers an individual revision, or an accumulation of revisions, to be substantive, the manual may be completely rewritten. The following steps are required for adoption of a revision.

A. Revision Assessment

SDO Staff will assess proposed revisions and develop the initial draft 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

Proposed modifications will be transmitted to TAG members and others allowing for no less than two weeks review time before the next scheduled TAG meeting. Other parties include, but are not limited to: State Construction Office, State Maintenance Office, State Materials Office, State Roadway Design Office, Organization, Forms and Procedures, and FHWA.

DSDE members of TAG will coordinate the proposed modifications with all other appropriate offices at the district level.

D. Revision Review by TAG and SDO

Each TAG member will review proposed modifications prior to the meeting where they will be brought forward for discussion. The SDO will review all modification comments received from other FDOT/FHWA offices in preparation for presentation at the meeting.

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 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 weeks after the TAG meeting, the SDO Staff will resolve each recommended modification and the assembled modifications will 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
Unless agreed otherwise, the ***SDG*** version modifications will be distributed within 4 weeks after receipt of the approved modifications from the SSDE. This time frame allows the Organization and Procedures Office to update the Standard Operating System and any electronic media prior to electronic distribution of the ***SDG***.

I.12 Training

No specific training is necessary for the use of this manual. Major revisions are often presented and discussed at the Biennial Structures Design Conference.

CHAPTER 1
GENERAL REQUIREMENTS

1.1 General

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

1.1.1 Design Review

Category 1 structures will be reviewed by the DSDE and Category 2 structures will be reviewed by the SSDE.

1.1.2 Substructure and Superstructure Definitions

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

A. Box culverts, retaining wall systems (including MSE walls), sound barriers on structures and bulkheads are substructures.
B. The substructure will not be classified less severely than the superstructure.

Approach slabs are superstructure; however, Class II Concrete (Bridge Deck) will be used for all environmental classifications.

1.2 Criteria for Deflection and Span-to-Depth Ratios [2.5.2.6]

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

1.3 Environmental Classifications

The District Materials Engineer or the Department's Environmental/Geotechnical Consultant will environmentally classify all new bridge sites. Environmental classification is required for major widenings (see definitions in Chapter 7) and may be required for minor widenings. This determination will be made prior to or during the development of the Bridge Development Report (BDR)/30% Plans Stage, and the results will be included in the documents. The bridge site will be tested, and separate classifications determined for superstructures and substructures.

In the bridge plans “General Notes,” include the environmental classification for both the superstructure and substructure according to the following classifications:
1.) Slightly Aggressive
2.) Moderately Aggressive
3.) Extremely Aggressive

For the substructure, additional descriptive data supplements the environmental classification. After the classification, note in parentheses, the source and magnitude of the environmental classification parameters resulting in the classification.

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

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

Bridge substructure and superstructure environments will be classified as Slightly Aggressive, Moderately Aggressive, or Extremely Aggressive environments according to the following criteria.

1.3.1 Substructure

For Conditions of soil and water:
A. Slightly Aggressive when all of the following conditions exist:
   1.) pH greater than 6.6.
   2.) Resistivity greater than 3,000 ohm-cm.
   3.) Sulfates less than 150 ppm.
   4.) Chlorides less than 500 ppm.
B. Moderately Aggressive: This classification must be used at all sites not meeting requirements for either Slightly Aggressive or Extremely Aggressive Environments.
C. Extremely Aggressive when any one of the following conditions exists:
   1.) For concrete structures: pH less than 5.0.
   2.) For steel structures: pH less than 6.0.
   3.) Resistivity less than 500 ohm-cm.
   4.) Sulfates greater than 1,500 ppm.
   5.) Chlorides greater than 2,000 ppm.

1.3.2 Superstructure

The following environmental classifications apply:

A. Slightly Aggressive:
   Any superstructure situated in an environment that is not classified either as Moderately or Extremely Aggressive (see Figure 1-1).
B. Moderately Aggressive:
   Any superstructure located within 2,500 ft. 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 1-1.
C. 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 1-1.

1.3.3 Chloride Content

A. The District Materials Engineer will determine the chloride content of the water and soil at the proposed bridge site.
B. The chloride content of major bodies of water within 2,500 feet of the proposed bridge site will, in most instances, be available in the Department's Bridge Corrosion Analysis database. If 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 within line-of-sight and within 2,500 feet of the Atlantic Ocean or the Gulf of Mexico are subject to increased chloride intrusion rates on the order of 0.0016 lbs/cy/year at a 2-inch 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 have the option of obtaining representative cores to determine chloride intrusion rates for any superstructure within 2,500 feet of any major body of water containing more than 12,000-ppm chlorides. The District Materials Engineer will take core samples from bridge superstructures in the immediate area of the proposed superstructure. The sampling plan with sufficient samples representing the various deck elevations will be coordinated with the State Corrosion Engineer. The Corrosion Laboratory of the State Materials Office will test core samples for chloride content and intrusion rates.

After representative samples are taken and tested, Table 1.1 will be used to correlate the core results (the chloride intrusion rate in lbs/cy /year at a depth of 2 inch) with the classification.

Table 1.1 Chloride Intrusion Rate/Environmental Classification

CHLORIDE INTRUSION RATE	CLASSIFICATION
$\geq$ 0.0016 lbs/cy/year	Extremely Aggressive
< 0.0016 lbs/cy/year	Moderately Aggressive

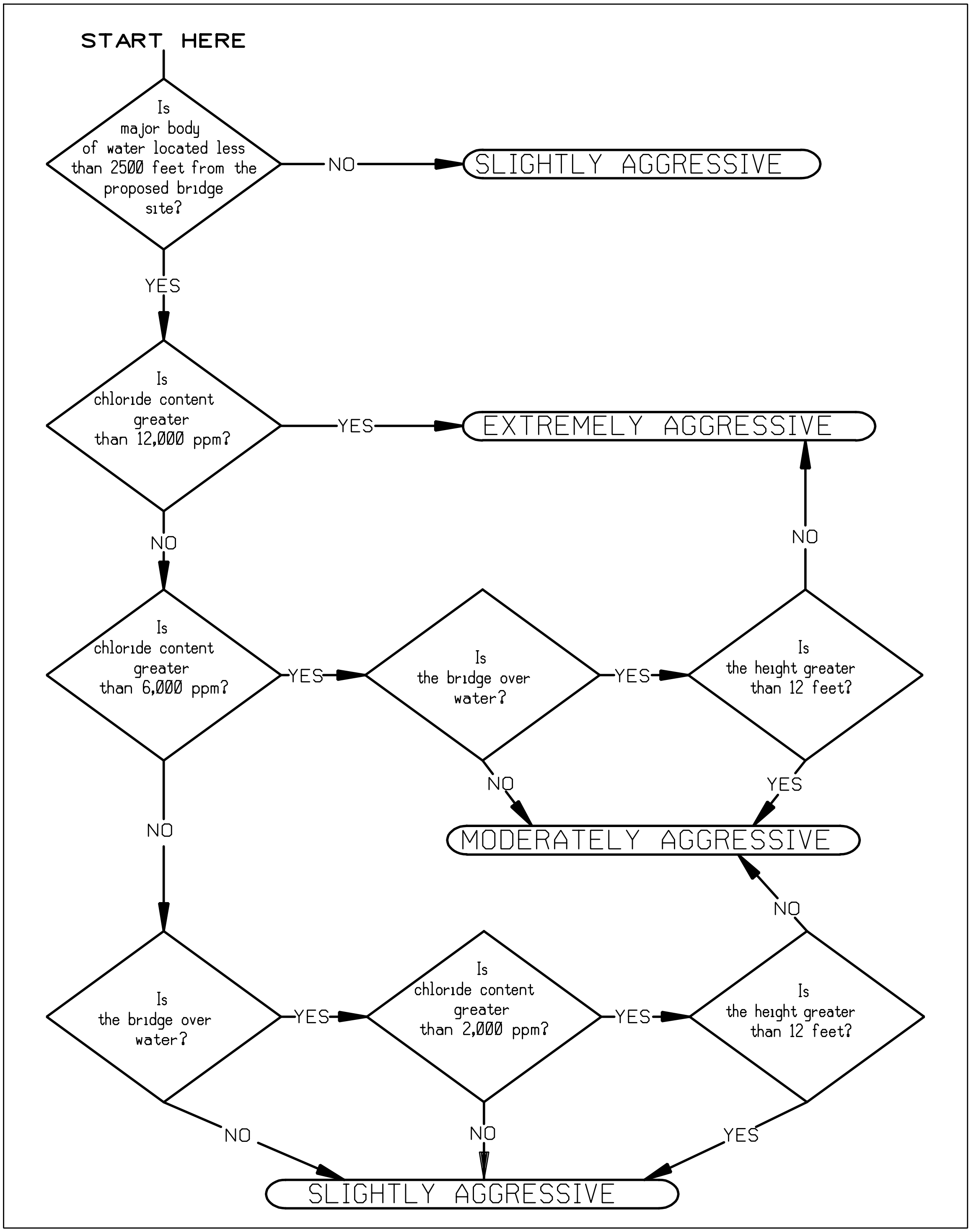


Figure 1-1 Flow Diagram for Determining Environmental Classification

NOTES:
The height of 12 feet 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).

1.4 Concrete and Environment

1.4.1 Cover

Delete AASHTO LRFD Article 5.12.3 and substitute the following requirements:

A. The requirements for concrete cover over reinforcing steel are listed in Table 1.2. Examples of concrete cover are shown in Figures 1-2 through 1-5.

B. 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 1.2.

Table 1.2 Concrete Cover

ITEM DESCRIPTION	CONCRETE COVER (in)	
	S or M*	E*
Superstructure (Precast)		
Internal and external surfaces (except riding surfaces) of segmental concrete boxes, and external surfaces of prestressed beams (except the top surface).	2	2
Top surface of girder top flange.	1	1
Top deck surfaces.		
Short Bridges***	2	2
Long Bridges***	2 ½**	2 ½**
All components and surfaces not included above (including barriers).	2	2
Superstructure (Cast-in-Place)		
All external and internal surfaces (ex. top surfaces)	2	2
Top deck surfaces		
Short Bridges(***)	2	2
Long Bridges(***)	2 ½**	2 ½**
Substructure (Cast-in-Place)		
External surfaces cast against earth and surfaces in contact with water	4	4½
External formed surfaces, columns, and tops of footings not in contact with water	3	4
Internal surfaces	3	3
Top of Girder Pedestals	2	2
Substructure (Precast)	3	4
Prestressed Piling (Including cylinder piling)	3	3
Drilled Shafts	6	6
Retaining Walls (Cast-in-Place or Precast)	2	3
Culverts (Cast-in-Place or Precast)	2	3
Bulkheads (Cast-in-Place)	4	4

NOTES:
*S= Slightly Aggressive; M= Moderately aggressive; E= Extremely Aggressive. **Cover dimension includes a 0.5 inch allowance for milling.
***See Short & Long Bridge Definitions in Chapter 4.

1.4.2 Class and Admixtures

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, those portions of the structure located in each classification will be built with the class of concrete described in Table 1-2 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 must be consistent with the 28-day compressive strength given in the FDOT Standard Specification Section 346.

Commentary:

Example:
Component - submerged piling
Environment - Extremely Aggressive over saltwater
Concrete Class - Table 1.2 Class V (Special)
Quality Control and Design Strength at 28 days - 6,000 psi

B. Admixtures for Corrosion Protection:

Primary components of structures located in Moderately or Extremely Aggressive environments utilize Class IV, V, V (Special), or VI Concrete. These concrete classes use fly ash, slag, silica fume, 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 locations must be clearly identified in the “General Notes”. Technical Special Provisions to the specifications may be required for their implementation.

The use of concrete admixtures to enhance durability must be consistent with these guidelines; however, if deemed necessary the Engineer of Record may request additional measures. The State Corrosion Engineer and the State Structures Design Engineer must approve these additional measures.

When the environmental classification is Extremely Aggressive due to the presence of chloride in the water:

- a.) Specify calcium nitrite in all:
  1. Superstructure components situated less than 12 feet above Mean High Water (MHW).
  2. Retaining walls, including MSE walls situated less than 12 feet above MHW and within 300 feet of the shoreline.
- b.) Specify silica fume in all:
  1. Pile bents.

2. Columns or walls with-in the "splash zone."
3. Sections of post-tensioned cylinder piles within the "splash zone" plus one section above and below the "splash zone" limits.

Do not specify silica fume for drilled shafts.
The splash zone is the vertical distance from 4 feet below MLW to 12 feet above MHW.

Table 1.3 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 V
	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
	Cast In Place Columns Located Directly In Splash Zone	Class II	Class IV	Class V
	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 silica fume may be required to be added to the concrete as an admixture. The use of the admixtures must conform to the requirements of the article of this Chapter entitled “Concrete Class and Admixtures for Corrosion Protection.”

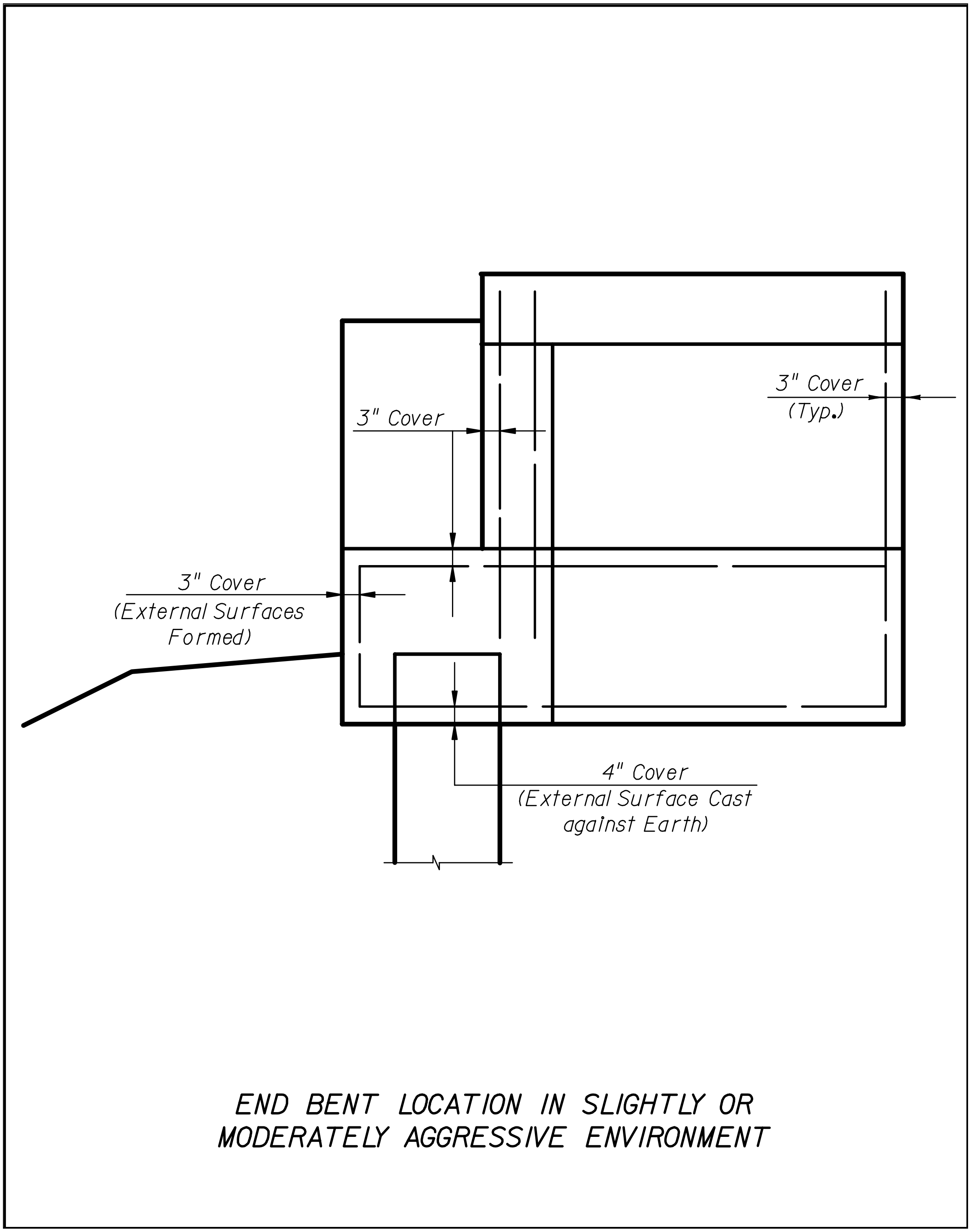
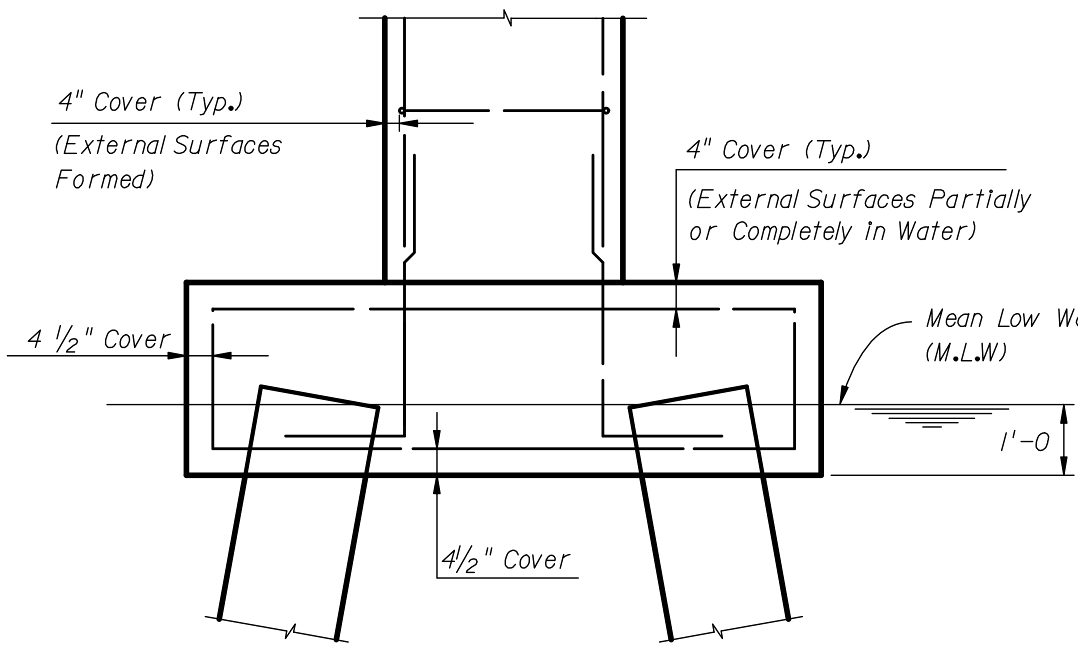

3" Cover
(Typ.)
3" Cover
3" Cover
(External Surfaces
Formed)
4" Cover
(External Surface Cast
against Earth)
END BENT LOCATION IN SLIGHTLY OR
MODERATELY AGGRESSIVE ENVIRONMENT

Figure 1-2



PIER IN WATER
EXTREMELY AGGRESSIVE ENVIRONMENT

Figure 1-3

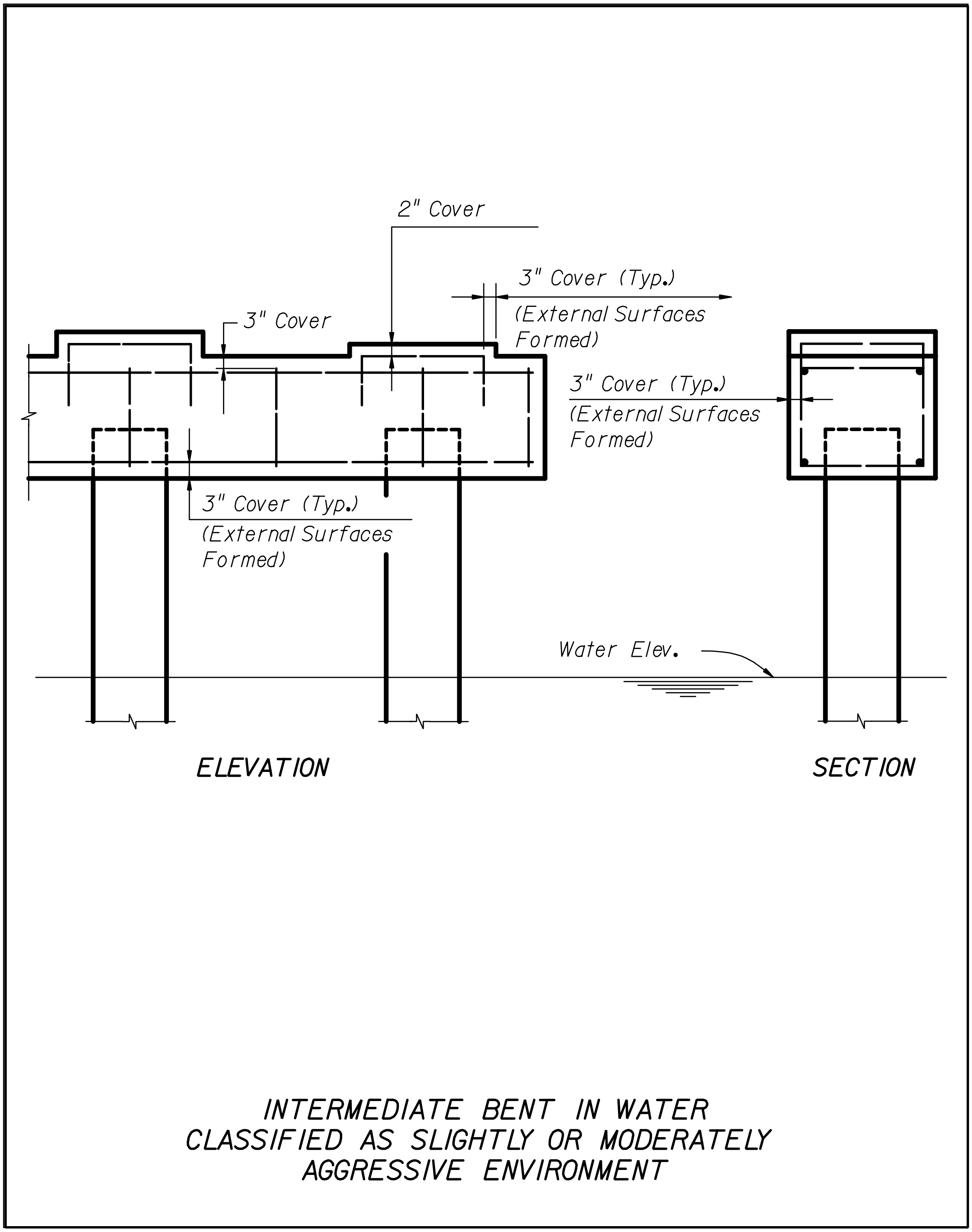

2" Cover
3" Cover (Typ.)
(External Surfaces Formed)
3" Cover
3" Cover (Typ.)
(External Surfaces Formed)
3" Cover (Typ.)
(External Surfaces Formed)
Water Elev.
ELEVATION
SECTION
INTERMEDIATE BENT IN WATER
CLASSIFIED AS SLIGHTLY OR MODERATELY
AGGRESSIVE ENVIRONMENT

Figure 1-4

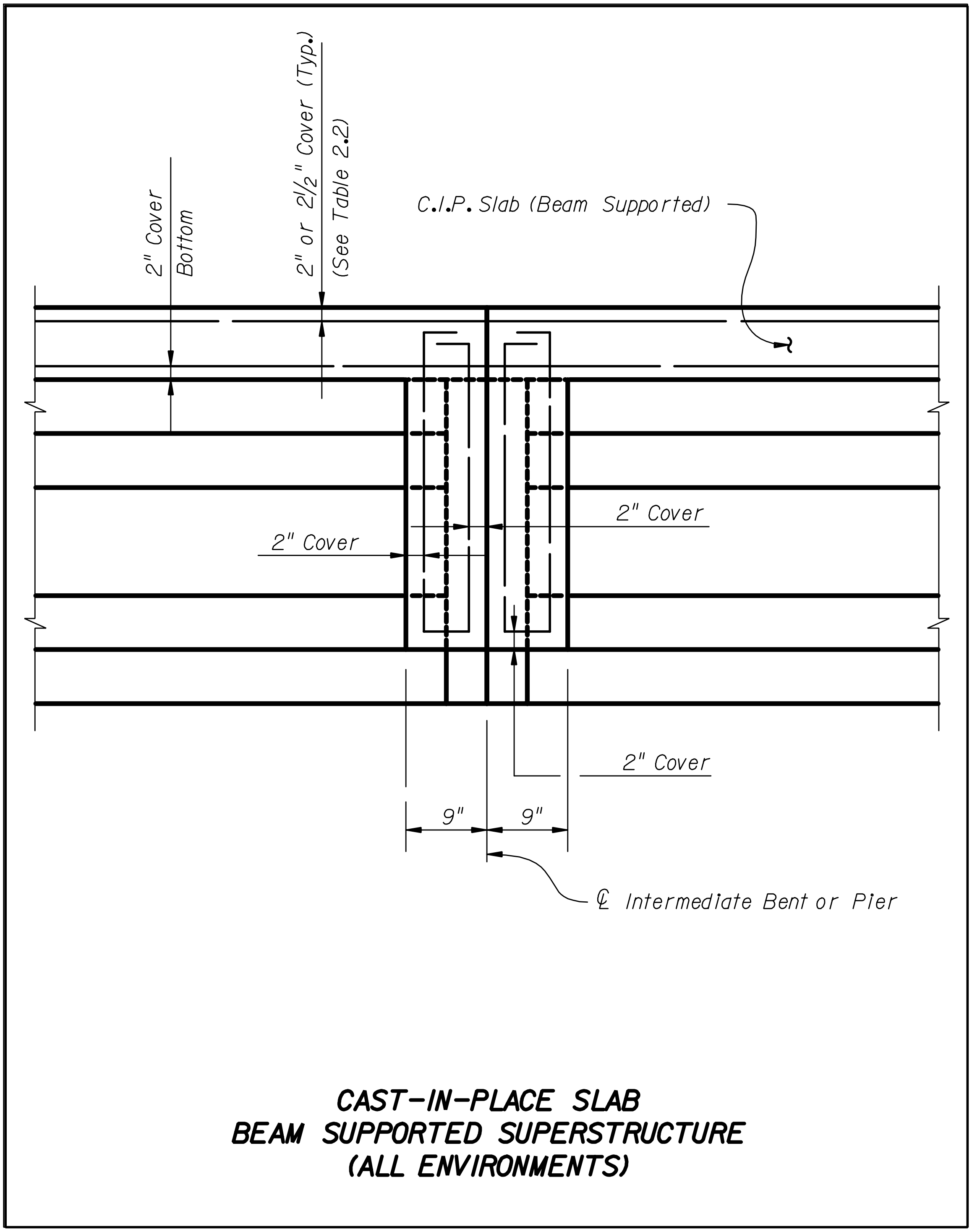

2" Cover
Bottom
2" or 2½" Cover (Typ.)
(See Table 2.2)
C.I.P. Slab (Beam Supported)
2" Cover
2" Cover
2" Cover
9"
9"
℄ Intermediate Bent or Pier
CAST-IN-PLACE SLAB
BEAM SUPPORTED SUPERSTRUCTURE
(ALL ENVIRONMENTS)

Figure 1-5

1.5 Existing Hazardous Material

Survey the project to determine if an existing structure contains hazardous materials such as lead-based paint, asbestos-graphite bearing pads, etc. Information will be provided by the Department or by site testing to make this determination.

When an existing structure has been identified as having hazardous material, no project will be developed without adequate provisions for worker safety, handling, storage, shipping, and disposal of the hazardous material. If proposed work will disturb identified hazardous materials, include in the project documents, protection, handling, and disposal requirements.

When a project involves hazardous materials, the FDOT design project manager will provide assistance in preparing the construction documents and will provide the technical special provisions for handling and disposing of hazardous materials.

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.

1.6 Adhesive Anchor Systems

1.6.1 General

Adhesive Anchor Systems (adhesive bonding material and steel bar anchors installed in clean, dry holes drilled in hardened concrete) are used to attach new construction to existing concrete structures. Anchors may be reinforcing bars or threaded rods depending upon the application.

For pre-approved Adhesive Anchor Systems, refer to the Department's Qualified Products List (QPL), "Adhesive Bonding Material Systems for Structural Applications." Comply with Section 937 of the Specifications. Require that Adhesive Anchors be installed in accordance with manufacturer's recommendations for hole diameter and hole cleaning technique and meet the requirements of Section 416 of the Specifications.

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

Do not use Adhesive Anchor Systems 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.

Unless special circumstances dictate otherwise, design Adhesive Anchor Systems for a ductile failure. A ductile failure requires embedment sufficient to ensure that failure will occur by yielding 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 only for horizontal, vertical, or downwardly inclined installations. Overhead or upwardly inclined installations of Adhesive Anchors are prohibited. Use QPL listed adhesive bonding to splice prestressed piling.

1.6.2 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). [in^2]

A_{n0} = $(16d)^2$, effective area of a single Adhesive Anchor in tension; used in calculating Ψ_{gn} (See Figure 1-6). [in^2]

A_n = effective area of a group of Adhesive Anchors in tension; used in calculating Ψ_{gn}, defined as the rectangular area bounded by a perimeter spaced 8d from the center of the anchors and limited by free edges of concrete (See Figure 1-6). [in^2]

A_{v0} = 4.5(c2), effective breakout area of a single Adhesive Anchor in shear; used in calculating Ψ_{gv} (See Figure 1-7). [in^2]

A_v = effective area of a group of Adhesive Anchors in shear and/or loaded in shear where the member thickness, h, is less than 1.5c; used in calculating Ψ_{gv}, (See Figure 1-7). [in^2]
c = anchor edge distance from free edge to centerline of the anchor). [in]
d = nominal diameter of Adhesive Anchor. [in]
$f'c$ = minimum specified concrete strength. [ksi]
f_y = minimum specified yield strength of Adhesive Anchor steel. [ksi]
f_u = minimum specified ultimate strength of Adhesive Anchor steel. [ksi]
h = concrete member thickness. [in]

h_e = embedment depth of anchor. [in]
N_c = tensile design strength as controlled by bond for Adhesive Anchors. [kips]
N_n = nominal tensile strength of Adhesive Anchor. [kips]
N_o = nominal tensile strength as controlled by concrete embedment for a single Adhesive Anchor. [kips]
N_s = design strength as controlled by Adhesive Anchor steel. [kips]
N_u = factored tension load. [kips]
s = Adhesive Anchor spacing (measured from centerlines of anchors). [in] When using Type HSHV adhesives, the minimum anchor spacing is 12d, unless approved by the State Structures Engineer.

V_c = shear design strength as controlled by the concrete embedment for Adhesive Anchors. [kips]
V_s = design shear strength as controlled by Adhesive Anchor steel. [kips]
V_u = factored shear load. [kips]
T' = 1.08 ksi nominal bond strength for general use products on the QPL (Type V and Type HV). 1.83 ksi nominal bond strength for Type HSHV adhesive products on the QPL for traffic railing barrier retrofits only.

$Ø_c$ = 0.85, capacity reduction factor for adhesive anchor controlled by the concrete embedment, ($Ø_c$ =1.00 for extreme event load case).

$Ø_s$ = 0.90, capacity reduction factor for adhesive anchor controlled by anchor steel.

Ψ_e = modification factor, for strength in tension, to account for anchor edge distance less than 8d (1.0 when c ≥ 8d.

Ψ_{gn} = strength reduction factor for Adhesive Anchor groups in tension (1.0 when s ≥ 16d).

Ψ_{gv} = strength reduction factor for Adhesive Anchor groups in shear and single Adhesive Anchors in shear influenced by member thickness (1.0 when s ≥ 3.0c and h ≥ 1.5c).

1.6.3 Design Requirements for Tensile Loading

Use Equation 1-2 to determine the design tensile strength for Adhesive Anchor steel:

$N_s = Ø_s A_e f_y$ [Eq. 1-2]

Use Equation 1-3 to determine the design tensile strength for Adhesive Anchor bond:

$N_c = Ø_c \Psi_e \Psi_{gn} N_o$ [Eq. 1-3]

where:

$N_o = T'\pi d\ h_e$ [Eq. 1-4]

For anchors with a distance to a free edge of concrete less than 8d, but greater than or equal to 3d, a reduction factor, Ψ_e, as given by Equation 1-5 must be used. For anchors located less than 3d from a free edge of concrete, an appropriate strength reduction factor must be determined by special testing. For anchors with an edge distance greater than 8d, Ψ_e may be taken as 1.0.

$\Psi_e = 0.70 + 0.30\ (c/8d)$ [Eq. 1-5]

For anchors loaded in tension and spaced closer than 16d, a reduction factor, Ψ_{gn}, given by Equation 1-6 must be used. For anchor spacing greater than 16d, Ψ_{gn} must be taken as 1.0.

$\Psi_{gn} = (A_n / A_{no})$ [Eq. 1-6]

1.6.4 Design Requirements for Shear Loading

Adhesive Anchors loaded in shear must 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 1-7:

$Vs = Ø_s\ 0.7\ A_s f_y$ [Eq. 1-7]

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 1-8:

$V_c = Ø_c \Psi_{gv}\ 0.4534\ c1.5 \sqrt{f'_c}$ [Eq. 1-8]

For anchors spaced closer than 3.0c, a reduction factor, Ψ_{gv}, given by Equation 1-9 must be used. For anchor spacing greater than 3.0c, Ψ_{gv} must be taken as 1.0.

$\Psi_{gv} = A_v / A_{v0}$ [Eq. 1-9]

1.6.5 Interaction of Tensile and Shear Loadings

The following linear interaction between tension and shear loadings given by Equation 1-10 must be used unless special testing is performed:

$(N_u / ØN_n) + (V_n / ØV_n) <= 1.0$ [Eq. 1-10]

In Equation 1-10, $Ø_n$ is the smaller of the design tensile strength controlled by the Adhesive Anchor steel (Equation 1-2) or the design tensile strength as controlled by Adhesive Anchor bond (Equation 1-3). $ØV_n$ is the smaller of the design shear strength controlled by the Adhesive Anchor steel (Equation 1-7) or the design shear strength as controlled by concrete breakout (Equation 1-8).

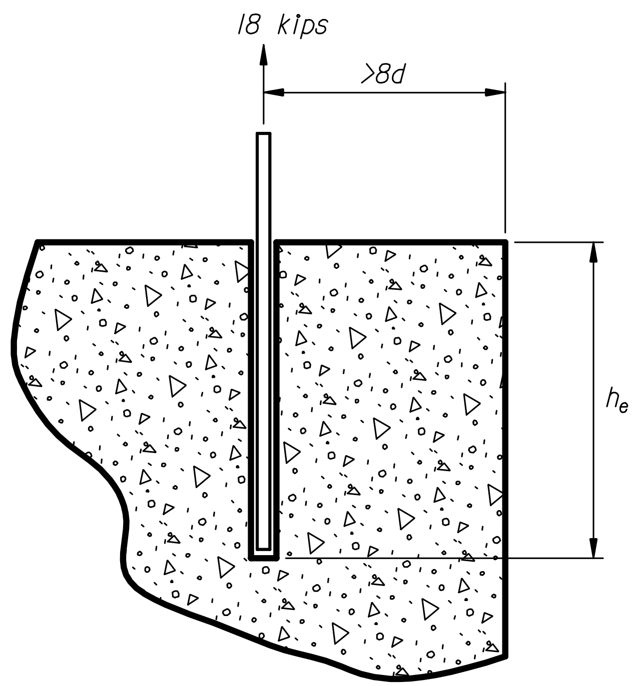
Commentary:
If Adhesive Anchor Systems are required to act as dowels from existing concrete components such that the existing reinforcing steel remains fully effective over its length, then the Adhesive Anchor System must be installed to a depth equal to the development length of the existing reinforcing steel. In this case, the required reinforcing steel spacing, 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.

1.6.6 Example 1 – Single Anchor Away from Edges and Other Anchors

Design an adhesive anchor using threaded rod (ASTM A193, Grade B7) for a factored tension load of 18 kips. The anchor is located more than 8 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 = 18.0 kips
f_u = 100.0 ksi
f_y = 125.0 ksi
T' = 1.08 ksi



Section View

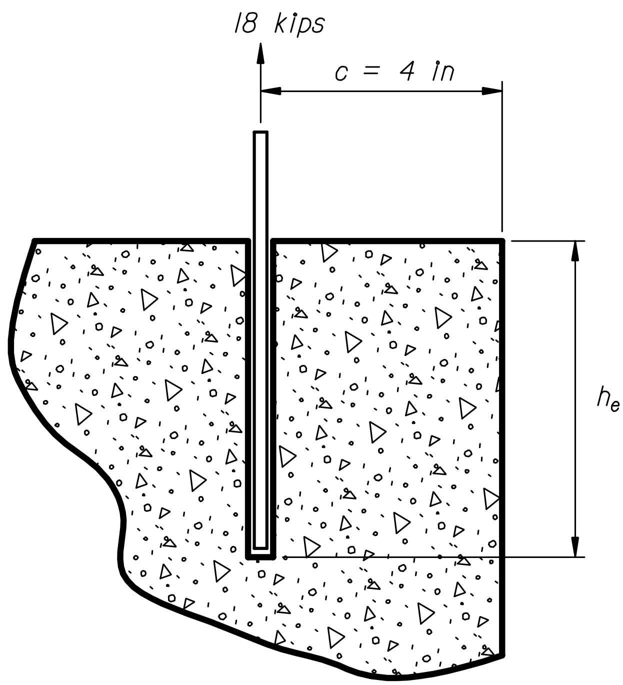
Design Procedure – Example 1	Calculation
Step 1 - Determine required rod diameter	
Determine the required diameter of the threaded rod by setting the factored tension load equal to the design steel strength.	$N_u = N_s$
The effective area for the threaded rod may be taken as 75% of the gross area. As with reinforcing bars, the minimum specified yield strength of the rod is used to determine the required diameter.	$Ns = Ø_s A_e f_y$ where: $Ø_s = 0.9$, $A_e = 0.75 (\pi d^2/4)$, f_y = 100 ksi
Substituting and solving for d:	$18 = (0.9)[0.75 (\pi d^2/4)](100)$ d = 0.583” therefore, use 5/8” 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, Ψ_e and Ψ_{gn} may be taken as unity.	$N_c = Ø_c \Psi_e \Psi_{gn} N_o$ (for embedment) where: $Ø_c = 0.85$ $\Psi_e, \Psi_{gn} = 1.0$ (no edge/spacing concern) $N_o = T' \pi d\, h_e$
For ductile behavior it is necessary to embed the anchor sufficiently to develop 125% of the yield strength or 100% of the ultimate strength, whichever is less.	N_c (req’d) $= 1.25 A_e f_y \leq A_e f_u$
Determine the effective area for a 5/8” threaded rod:	$A_e = 0.75 (\pi 0.625^2/4)$ $A_e = 0.23$ in^2
Determine the required tension force, N_c(req’d), to ensure ductile behavior.	N_c(req’d) $= 1.25 A_e f_y \leq A_e f_u$ N_c(req’d) $= 1.25 (.23)(100) \leq (.23)(125)$ N_c(req’d) = 28.75 kips = 28.75 kips therefore, use N_c(req’d) = 28.75 kips
Substituting and solving for h_e:	$28.75 = 0.85 (1.0) (1.0) (1.08) \pi (.625) h_e$ h_e = 16 in

1.6.7 Example 2 – Single Anchor Away from Other Anchors but Near Edge

Design an adhesive anchor using threaded rod (ASTM A193, Grade B7) for a factored tension load of 18 kips. The anchor is located 4 inches from an edge but is isolated from other anchors. The anchor embedment length is to be sufficient to ensure steel failure.

Given:

N_u = 18.0 kips
f_u = 100.0 ksi
f_y = 125.0 ksi
T' = 1.08 ksi
c = 4 inches



Section View

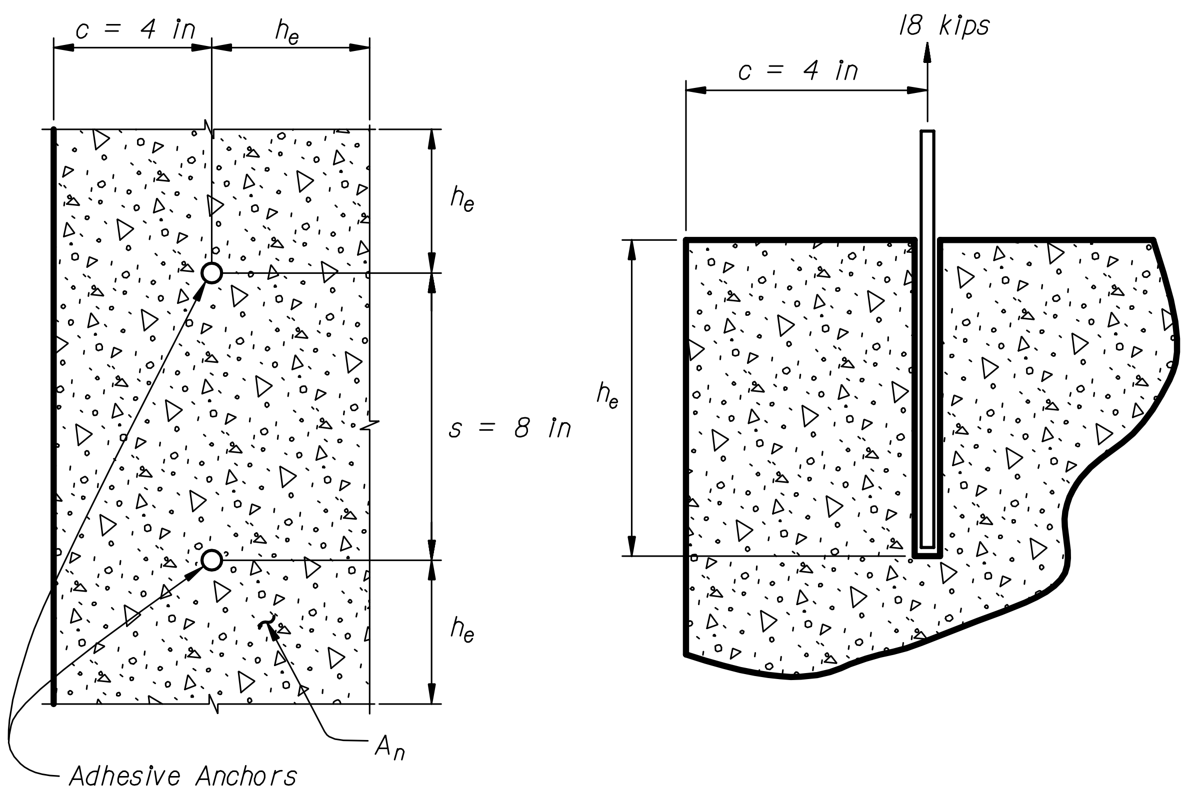
Design Procedure-Example 2	Calculation
Step 1 - Determine required rod diameter	
Determine the required diameter of the threaded rod by setting the factored tension load equal to the design steel strength.	$N_u = N_s$
The effective area for the threaded rod may be taken as 75% of the gross area. As with reinforcing bars, the minimum specified yield strength of the rod is used to determine the required diameter.	$N_s = Ø_s A_e f_y$ where: $Ø_s= 0.9$ $A_e= 0.75 (\pi d^2/4)$ $f_y= 100$ ksi
Substituting and solving for d:	$18 = (0.9)[0.75 (\pi d^2/4)](100)$ d = 0.583 in therefore, use a 5/8" threaded rod
Step 2 - Determine required embedment length to ensure steel failure	
Basic equation for embedment length calculation. Since there are no spacing concerns, Ψ_{gn} may be taken as unity, and, since the edge distance (4 in) is less than 8d (5 in), the edge effect, Ψ_e , will need to be evaluated.	$N_c = Ø_c \Psi_e \Psi_{gn} N_o$ (for embedment) where: $Ø_c = 0.85$ $\Psi_{gn} = 1.0$ (no spacing problem) $N_0 = T' \pi d h_e$
For ductile behavior it is necessary to embed the anchor sufficiently to develop 125% of the yield strength or 100% of the ultimate strength, whichever is less.	$N_c(\text{req'd}) = 1.25 A_e f_y \le A_e f_u$
Determine the effective area for a 5/8" threaded rod:	$A_e = 0.75 (\pi\, 0.625^2/4)$ $A_e = 0.23$ in^2
Determine the required tension force, N_c(req'd), to ensure ductile behavior.	$N_c(\text{req'd}) = 1.25 A_e f_y \le A_e f_u$ $N_c(\text{req'd}) = 1.25 (.23)(100) \le (.23)(125)$ N_c(req'd) = 28.75 kips = 28.75 kips Therefore, use N_c(req'd) = 28.75 kips
Determine edge effect factor, Ψ_e. Note: $c_{cr} = 8d$	$\Psi_e = 0.70 + 0.30 (c /8d)$ $\Psi_e = 0.70 + 0.30 [4/(8)(.625)]$ $\Psi_e = 0.94$
Substituting and solving for h_e:	$28.75 = 0.85 (1.0)(0.94)(1.08) \pi (.625) h_e$ $h_e = 16.98$ inches

1.6.8 Example 3 – Two Anchors Spaced at 8 inches, 4 inches from Edge

Design a group of two adhesive anchors using threaded rod (ASTM A193, Grade B7) for a factored tension load of 18 kips. The anchors are located 4 inches from an edge and are spaced 8 inches apart. Steel failure is not required.

Given:

N_u = 18.0 kips
f_y = 100.0 ksi
f_u = 125.0 ksi
T' = 1.08 ksi
c = 4 inches
s = 8 inches



Plan View

Section View

Design Procedure-Example 3	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.	$N_s = Ø_s A_e f_y$ Where: $Ø_s = 0.9$ $A_e = (2)\ 0.75\ (\pi d^2/4)$ $f_y = 100$ ksi
Substituting and solving for d:	$18 = (0.9)(2)[0.75\ (\pi d^2/4)](100)$ $d = 0.412$ in Although a ½" threaded rod is OK, use a 5/8" threaded rod to minimize embedment length
Design steel strength	$N_s = 0.9(2)(0.75)(\pi 0.625^2/4)(100)$ N_s= 41.4 kips > 18 kips therefore; OK
Step 2 - Determine required embedment length	
Basic equation for embedment length calculation. Since there are edge or spacing concerns, Ψ_e and Ψ_{gn} will need to be determined.	$N_c = Ø_c\ \Psi_e\ \Psi_{gn}\ N_0$ (for embedment) where: $Ø_c = 0.85$ Ψ_e and Ψ_{gn} are calculated below $N_0 = T'\pi d\ h_e$
Determine edge effect factor, Ψ_e.	$\Psi_e = 0.70 + 0.30\ (c/8d)$ $\Psi_e = 0.70 + 0.30\ [4 / (8)(.625)]$ $\Psi_e = 0.94$
Determine group effect factor, Ψ_{gn}.	$\Psi_{gn} = A_n / A_0$ $\Psi_{gn} = (4 + 8d)[8 + 2(8d)]/(16)^2]$ $\Psi_{gn} = (4 + (8)(0.625)[8 +$ $2(8)(0.0625)]/[16(0.625)]^2$ $\Psi_{gn} = 1.62$
Substituting and solving for h_e	$18 = (0.85)(1.62)(0.94)(1.08)\ \pi(.625)h_e$ h_e = 6.55" (say 7") therefore OK
Design adhesive bond strength.	$N_c = (0.85)(1.62)(0.94)(1.08)\pi\ (.625)(7)$ $N_c = 19.21 > 18$. Therefore OK
Step 3 - Final Design Strength	
Strength as controlled by steel.	N_s = 41.4 kips > 18 kips. Therefore OK
Strength as controlled by adhesive bond.	N_c = 19.21 kips > 18 kips. Therefore OK
Final Design.	2 – 5/8" anchors embedded 7 in

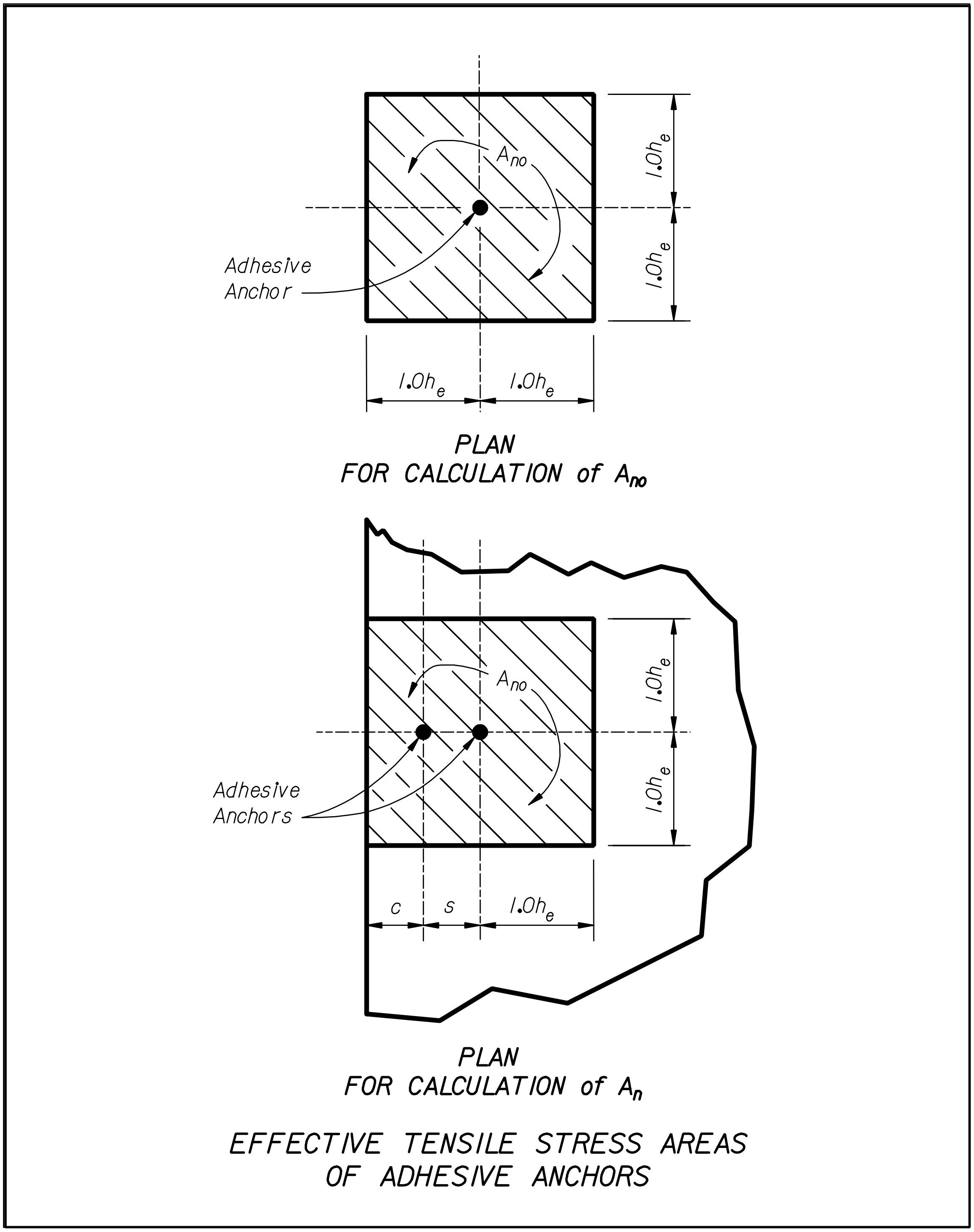


Figure 1-6 Effective Tensile Stress Areas of Adhesive Anchors

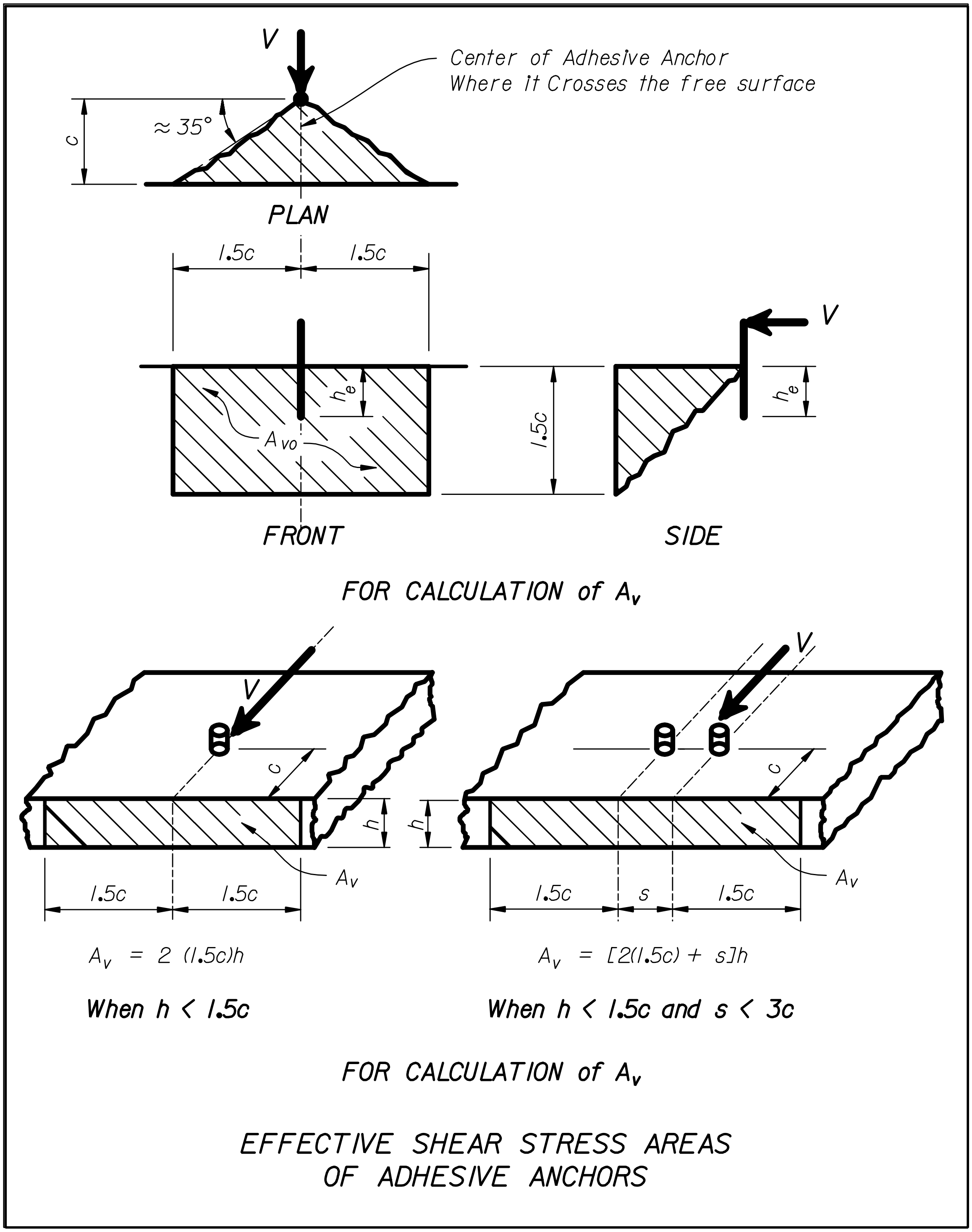


Figure 1-7 Effective Shear Stress Areas of Adhesive Anchors

CHAPTER 2
LOADS AND LOAD FACTORS

2.1 General

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

2.2 Dead loads

A. Future Wearing Surface:
   See Table 2.1 regarding the allowance for a Future Wearing Surface.
B. Sacrificial Concrete:
   Bridge decks subject to the profilograph requirements of Chapter 2 require an added thickness of sacrificial concrete, which must be accounted for as added Dead Load but cannot be utilized for bridge deck section properties. See "SDG_4.2_Concrete_Deck_Slabs"

2.3 Seismic Provisions [3.10.9][3.10.9.2][4.7.4]

2.3.1 General

The majority of Florida bridges will be exempt from seismic or restrainer design requirements. For exempted bridges, only the minimum bearing support dimensions need to be satisfied as required by ***LRFD*** [4.7.4.4]. Exempted bridges 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.

Seismic design is required for unusual designs, continuous steel or concrete girders, steel or concrete box girders, concrete segmental bridges, curved bridges with a radius less than 1,000 feet, those that rely on a system of fixed or sliding bearings for transmitting elongation changes, spans in excess of 165 feet, cable supported bridges, and bridges that do not use conventional elastomeric bearing pads.

Use the "Uniform Load Method" specified in ***LRFD*** [4.7.4.3.1] and [4.7.4.3.2], or any higher-level analysis, for calculating the seismic forces to be resisted by the bearings or other anchorage devices. Only the connections between the superstructure and substructure need to be designed for the seismic forces.

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

2.3.2 Seismic Design for New Construction

When seismic design is required, use the single mode spectral method to determine the seismic design forces to be used for the restraint design between the superstructure and substructure only.

Compare these forces to the values obtained using the simplified seismic Zone 1 method (***LRFD*** 3.10.9.2). Use the lesser of the force values to design the seismic restraints.

The acceleration coefficient in Florida varies from 1% to 3.75%, and must be determined from the Map of Horizontal Acceleration contained in the latest edition of ***LRFD***.

2.3.3 Seismic Design for Widenings

When seismic design is required for a major widening (see definitions in SDG Chapter 7), all bridge elements must comply with the seismic provisions for new construction.

FDOT will consider seismic provisions for minor widenings on an individual basis.

2.3.4 Lateral Restraint

When lateral restraint of the superstructure is required due to seismic loading, comply with the provisions and requirements of Chapter 6, "Lateral Restraint."

2.4 Wind Loads

2.4.1 Wind Loads on Bridges [3.8.1]

Increase the wind pressures of ***LRFD*** by 20% for bridges located in Palm Beach, Broward, Dade, and Monroe counties. Submit wind pressures for bridges over 75 feet high or with unusual structural features, to FDOT for approval.

2.4.2 Wind Loads on Other Structures

Wind speeds for miscellaneous highway related structures are specified in Chapter 29 of the ***Plans Preparation Manual***, Volume 1.

2.4.3 Wind Loads During Construction
During design, analyze the stability and lateral buckling of beams and girders for wind loading and handling/erection during construction and prior to casting the concrete deck slab. Design temporary bracing, shoring, tie-downs, strongbacks, etc. to insure stability and resistance to lateral buckling.

When analyzing the stability of beams and girders, the Factor of Safety against overturning taken about the extreme lateral point of contact bearing, must not be less than 1.5, and the wind pressure during construction will be one-half the calculated horizontal wind pressure used for final design.

2.5 Miscellaneous Loads [3.5.1]

Use the loadings in Table 2.1 unless a more refined analysis is performed.

Table 2.1 Miscellaneous Loads.

ITEM	UNIT	LOAD
Traffic Railing Barrier (32 " F-Shape)	Lb / ft	421
Traffic Railing Median Barrier, (32" F-Shape)	Lb / ft	486
Traffic Railing Barrier (42 " Vertical Shape)	Lb / ft	587
Traffic Railing Barrier (32 " Vertical Shape)	Lb / ft	385
Traffic Railing Barrier (42 " F-Shape)	Lb / ft	624
Traffic Railing Barrier / Sound wall (Bridge)	Lb / ft	1008
Concrete, Counterweight (Plain)	Lb / cf	146
Concrete, Structural	Lb/cf	150
Future Wearing Surface	Lb/sf	15*
Soil; Compacted	Lb/cf	115
Stay-in-Place Metal Forms	Lb/sf	20**

* The Future Wearing Surface allowance applies only to minor widenings or short bridges as defined in SDG_7.2_Widening_Classifications_and_Definitions

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

2.6 Vehicular Collision Force [3.6.5]

Delete ***LRFD*** Article [3.6.5.1] and [3.6.5.2].

2.7 Uniform Temperature [3.12.2]

2.7.1 Segmental Box Girders
Delete ***LRFD*** [3.12.2] and substitute SDG Chapter 4.

2.7.2 Joints and Bearings
Delete ***LRFD*** [3.12.2] and substitute SDG Chapter 6.

2.7.3 All Other Bridge Components

For the design of all other bridge components, use the temperature ranges shown in ***SDG*** Table 6.2.

2.7.4 Temperature Gradient [3.12.3]

Delete the second paragraph of ***LRFD*** [3.12.3] and substitute the following:
"Include the effects of Temperature Gradient in the design of continuous concrete superstructures only. The vertical Temperature Gradient may be taken as shown in ***LRFD*** Figure 3.12.3-2."

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

In lieu of the requirements of ***LRFD*** [4.6.2.2.1] for permanent loads, and in lieu of a more refined analysis, the dead load of barriers and railings applied to exterior beams and stringers (Wext), in lb/ft, may be determined by the following equation when superstructure spans are not less than 40 feet nor more than 150 feet in length:

$$Wext = (W \times C_1 \times C_2) / 100 \quad \text{[Eq. 2-1]}$$
$$C_1 = 0.257\,(S^3(3K-8))^{0.5} + ((10-K)^2 + 39) / 1.4 \quad \text{[Eq. 2-2]}$$
$$C_2 = 2.2 - 0.335\,(L/10) + 0.0279(L/10)^2 - 0.000793(L/10)^3 \quad \text{[Eq. 2-3]}$$

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

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 Wext computed above may be used for the dead load of the barrier applied to the exterior beam adjacent to the barrier, and the balance of the barrier weight distributed equally among the remaining beams.

The distribution methods described above apply to concrete or steel, longitudinal beam or stringer, superstructures on which a concrete slab is cast, compositely, prior to installation of the barrier or railing. For superstructures not conforming to the limitations stated above, the barrier/railing distribution may be determined in accordance with ***LRFD [4.6.2.2]***.

Example Problem 1:
(New Construction, 2-lane bridge)
L = 100 feet
K = 6
S = 8 feet
C1 = 57.68
C2 = 0.847
W = 418 lb/ft

Wext = (418 x 57.68 x 0.847) / 100 = 204 lb/ft
Wint = (2 x (418-204)) / (6-2) = 107 lb/ft

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

Wext = (204)(0.75) = 153 lb/ft (at exterior beam on barrier side)
Wint = (418-153)/(6-1) = 53 lb/ft (all other beams)

2.9 Load Distribution Beam-Slab Bridges [4.6.2.2]

2.9.1 Prestressed Concrete Inverted Tee Girder
The live load distribution for prestressed concrete inverted tee girders can be approximated by methods contained herein as long as the following conditions are met:
A. Span lengths 30 feet to 75 feet;
B. Span to depth ratios of 22 to 38;
C. Design Slab thickness equals 6 inches;
D. No permanent intermediate diaphragms;
E. Distance de = -9 inches [align barrier directly above exterior girder];
F. Girder spacing equals 2 feet.

de = distance from exterior web of exterior girder and the interior edge of curb or traffic barrier.

Either the formulas or the rough approximations can be used. All live load distribution factors **(LLDF)** are in terms of lanes and are based on 2 or more traffic lanes loaded. For the definition of terms used in the formulas see ***LRFD*** **[4.6.2.2]**.

A. Bending:
   1.) Rough approximation LLDF = 0.205
   2.) Formula LLDF = $0.175 + (S/9.5)^{0.01} \times (S/L)^{1.2} \times (Kg/12.0\ Lts^3)^{0.1}$

B. Shear:
   1.) Rough Approximation LLDF = 0.275
   2.) Formula LLDF = $0.4104 + (S/385)\ L + (S^2/100{,}000)\ L^2$

2.9.2 Prestressed Concrete "U" Girder

The live load distribution for prestressed concrete "U" girders can be approximated by methods contained herein as long as the following conditions are met:

A. Span Lengths 70 feet to 160 feet;
B. Span to depth ratios of 18.5 to 26.4;
C. 2 or more girder units;
D. Slab thickness of 8 inches to 9 inches;
E. No permanent intermediate diaphragms;
F. Distance de <= 3.0 feet.

Either the formulas or the rough approximations can be used. All live load distribution factors (LLDF) are in terms of lanes and are based on 2 or more traffic lanes loaded.

A. Bending:

1.) Rough Approximation, Interior and Exterior Girders.
Use interpolation to extract LLDF for the exact girder spacing.

Girder Spacing	LLDF
25 feet	1.46
20 feet	1.18
15 feet	1.02
10 feet	0.96

2.) Formulas

Two girders
$LLDF = 0.96 + 107.2\ (Sd/12.0\ L^2)^{1.31}$

Three or More Girders, Exterior
$LLDF = 0.45 + 15.6\ (Sd/12.0\ L^2)^{0.67}$

Three or More Girders, Interiors
$LLDF = 0.8 + 522.4\ (Sd/12.0\ L^2)^{1.59}$

B. Shear

1.) Rough Approximation, Interior and Exterior Girders.
Use interpolation to extract LLDF for the exact girder spacing.

Girder Spacing	LLDF
25 feet	1.69
20 feet	1.39
15 feet	1.17
10 feet	1.0

2.) Formulas

Two girders
$LLDF = 3.977\ (Sd/12.0\ L^2)^{0.23}$

Three or More Girders; Exterior
$LLDF = 0.1 + 4.266\ (Sd/12.0\ L^2)^{0.26}$

Three or More Girders; Interior
$LLDF = 0.1 + 10.85\ (Sd/12.0\ L^2)^{0.45}$

2.10 Operational Importance [1.3.5]

Unless otherwise approved in writing by the State Structures Design Engineer, the value of the Operational Importance Factor (η 1) in ***LRFD*** **[1.3.5]** for the strength limit state is:

$\eta 1 = 1.0$ for all bridges.

Bridges considered critical to the survival of major communities, or to the security and defense of the United States, should utilize a higher value for $\eta 1$.

2.11 Vessel Collision [3.14]

2.11.1 General [3.14.1]

The design of all bridges over navigable waters must include consideration for possible Vessel Collision (usually from barges or ocean going ships). Conduct a vessel risk analysis to determine the most economical method for protecting the bridge. The marine vessel traffic characteristics are available for bridges located across intracoastal waterways and rivers carrying predominately barges. The vessel traffic is embedded as an integral part of the Department's Vessel Collision Risk Analysis Software. Also, the importance classification is provided for existing bridge sites and will be provided by the Department for any new bridge location. Port facilities and small terminals handling ships are not covered by the catalog of vessel traffic characteristics. In these cases, on-site investigation is required to establish the vessel traffic characteristics. Utilize the ***LRFD*** specification and comply with the procedure described hereinafter.

2.11.2 Research and Information Assembly

(When not provided by the Department)

A. Data Sources:

1. U.S. Army Corps of Engineers, Waterborne Commerce Statistics Center, P.O. Box 61280, New Orleans, LA 70161. Telephone: (504) 862-1472.
2. U.S. Army Corps of Engineers, "Waterborne Commerce of the United States (WCUS), Parts 1 & 2," Water Resources Support Center (WRSC), Fort Belvoir, VA.
3. U.S. Army Corps of Engineers, "Waterborne Transportation Lines of the United States," WRSC, Fort Belvoir, VA.
4. U.S. Army Corps of Engineers (COE), District Offices.
5. U.S. Coast Guard, Marine Safety Office (MSO).
6. Port Authorities and Water Dependent Industries.
7. Pilot Associations and Merchant Marine Organizations.

8. National Oceanic and Atmospheric Administration (NOAA), "Tidal Current Tables; Tidal Current Charts and Nautical Charts," National Ocean Service, Rockville, Maryland.
9. Bridge tender record for bascule bridge at the District Maintenance Office.
10. Local tug and barge companies.

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

2.11.3 Design Vessel [3.14.4][3.14.5.3]
When utilizing the FDOT's Mathcad software for conducting the Vessel Collision risk analysis, a "Design Vessel," which represents 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."

2.11.4 Design Methodology - Damage Permitted [3.14.13]
In addition to utilizing the general design recommendations presented in ***LRFD*** (except as noted herein), the EOR must also use the following design methodology:

A. At least one iteration of secondary effects in columns must be included; i.e., axial load times the initial lateral deflection.
B. The analysis must include the effects of force transfer to the superstructure. Bearings, including neoprene pads, transfer lateral forces to the superstructure. Analysis of force transfer through the mechanisms at the superstructure/ substructure interface must be evaluated by use of generally accepted theory and practice.
C. The ultimate bearing capacity (UBC) of axially loaded piles must be limited to the compressive and/or tensile loads determined in accordance with the requirements of SDG Chapter 3. Load redistribution must not be permitted when the axial pile capacity is reached; rather, axial capacity must be limited to the ultimate limit as established by analysis.

D. Lateral soil-pile response must be determined by concepts utilizing a coefficient of sub-grade modulus provided or approved by the Geotechnical Engineer. Group effects must be considered.
E. For the designer's Vessel Collision risk analysis, the FDOT will determine whether a bridge is critical or non-critical. A list is provided with the Department's software.

Load Combination "Extreme Event II" is:
(Permanent Loads) + WA+FR+CV
With all load factors equal to 1.0. Nonlinear structural effects must be included and can be significant. It is anticipated that the entire substructure (including piles) may have to be replaced if subjected to this design impact load; however, the superstructure must not collapse.

Commentary:
Further refinement or complication of this load case is unwarranted.

2.11.5 Pile Bents
For pile bents utilizing 30 inch or smaller piles subject to minor vessel impacts, design the bent to remain structurally adequate with any one pile removed and live load applied only within the designated, striped traffic lanes. In this event, apply the load combination for Extreme Event II in ***LRFD*** Table 3.4.1-1. and increase the Load Factor for Live Load to 1.0.

Specify that non-sectional, wet cast, cylinder piles used in pile bents subject to vessel impact be filled from 2 feet below MLW to an elevation 15 feet above MHW as a minimum. Sectional spun cast cylinder piles must contain a redundant load path meeting the requirements of Chapter 3.

2.11.6 Widenings
Major widening of bridges spanning navigable waterways must be designed for Vessel Collision. Minor widenings of bridges spanning navigable waterways will be considered on an individual basis for Vessel Collision design requirements.

2.11.7 Movable Bridges
For movable bridges, comply with the requirements of this chapter.

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

2.11.9 Scour with Vessel Collision [3.14.1]
Substructures must be designed for an extreme Vessel Collision load by a ship or barge simultaneous with scour. Design the substructure to withstand the following two Load/Scour (LS) combinations:

A. Load/Scour Combination 1:
$LS_{(1)}$ = Vessel Collision @ ½ Long-Term Scour [Eq. 3-4]
Where:

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

B. Load/Scour Combination 2:
$LS_{(2)}$ = Minimum Impact Vessel @ ½ 100-Year Scour [Eq. 3-5]
Where:
Min. Impact Vessel as defined in ***LRFD*** [3.14.1] with related collision speed.
100-Year Scour as defined in Chapter 4 of the FDOT ***Drainage Manual*** (Topic No. 625-040-001).

When preparing the soil models for computing the substructure strengths, and when otherwise modeling stiffness, analyze and assign soil strength parameters to the soil depth that is subject to Local and Contraction Scour that may have filled back in. The soil model must 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 streambed and that of the soil that is redeposited subsequent to a scour event.

2.11.10 Application of Impact Forces [3.14.14]
When the length to width ratio (L/W) is 2.0 or greater for long narrow footings in the waterway, apply the longitudinal force within the limits of the distance that is equal to the length minus twice the width, (L-2W), in accordance with Figure 2-1.

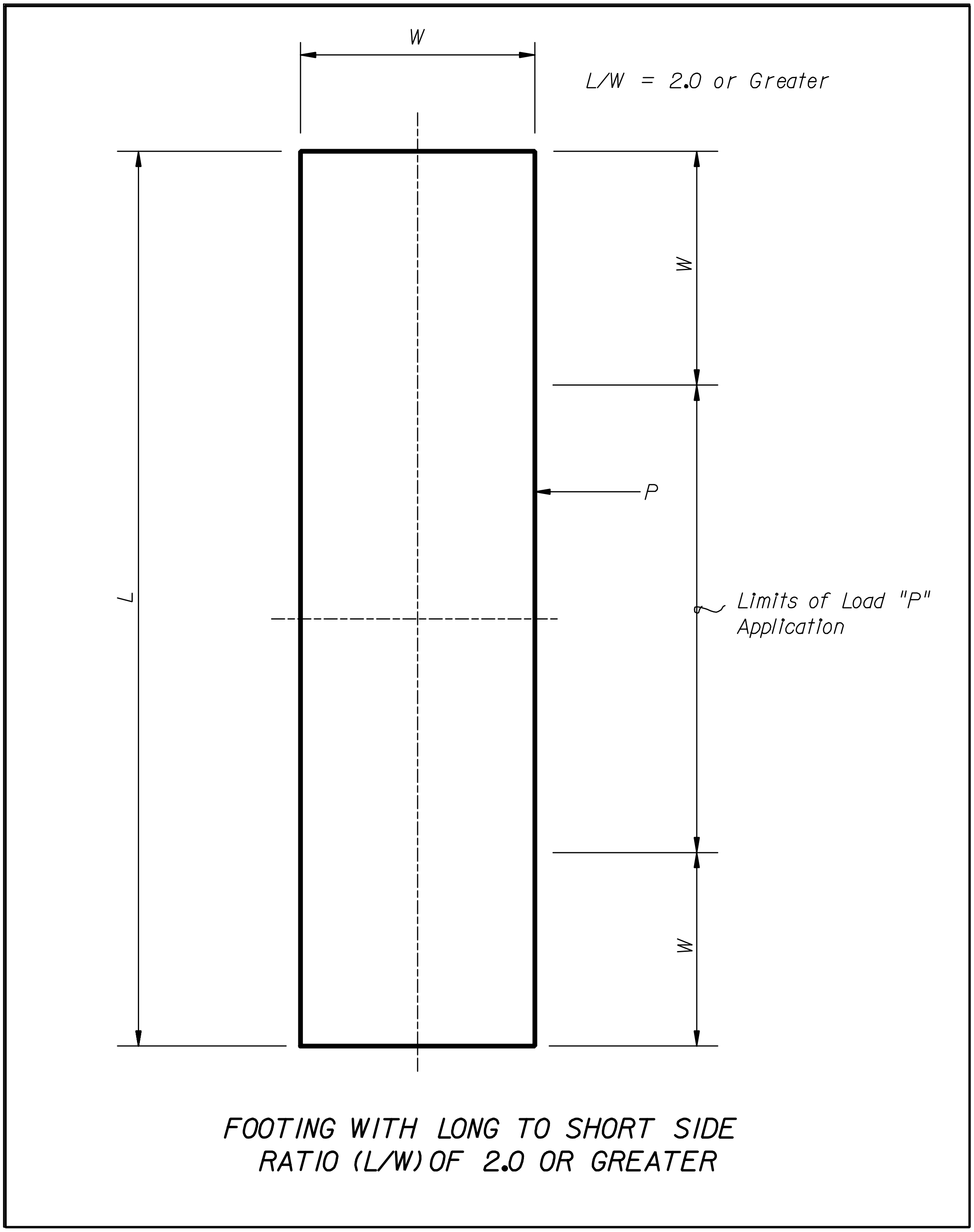

W
L/W = 2.0 or Greater
W
L
P
Limits of Load "P" Application
W
FOOTING WITH LONG TO SHORT SIDE RATIO (L/W) OF 2.0 OR GREATER

Figure 2-1

2.11.11 Impact Forces on Superstructure [3.14.14.2]
Apply Vessel Impact Forces (superstructure) in accordance with ***LRFD*** **[3.14.14.2]**.

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

A. Load Factor for Temperature Gradient ($_{TG}$)
Y_{TG} = Zero (0.0) for all strength limit states.
Y_{TG} = 0.50 for service limit states of continuous concrete superstructures.

B. Load Factor $_{(EQ)}$ for Extreme Event-I Load Combination
Y_{EQ} = 0.0.

C. Load Factor for Foundation Settlement ($_{SE}$)
Y_{SE} = 1.0.

2.12 Substructure Limit States

A. Limit State 1 (Always required - Scour may be "0")
Conventional ***LRFD*** loadings (using load factor combination groups as specified in ***LRFD*** Table 3.4.1-1), but utilizing the most severe case of scour up to and including that from a 100 year flood event.

B. Limit State 2 (Applies only if vessel collision force is specified)
Extreme event of Vessel Impact (using load factor combination groups as specified in the ***LRFD***) utilizing scour depths described in Section 2.11, "Scour with Vessel Collision."

C. Limit State 3 (Applies only if scour is predicted)
Stability check during the super flood (most severe case of scour up to and including that from the 500-year flood) event.

Limit State 3 =

$$Y_p(DC) + Y_p (DW) + Y_p (EH) + 0.5(L) + 0.5(EL) + 1.0(WA) + 1.0(FR) \qquad \text{[Eq. 4-1]}$$

Where, L = LL + IM + CE + BR + PL

(All terms as per ***LRFD***)

2.13 Pedestrian Live Load on Pedestrian Bridges

Design main supporting members for a pedestrian live load of 85 lbs per square foot of bridge walkway area. No reduction in pedestrian live load is permitted.

CHAPTER 3
SUBSTRUCTURE AND RETAINING WALLS

3.1 General

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

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

The Structural Engineer and the Geotechnical Engineer must consider constructability in the selection of the foundation system. Issues such as existing underground and overhead utilities, pile-type availability, use of existing structures for construction equipment, phase construction, conflicts with existing piles and structures, effects on adjacent structures, etc. must be considered in evaluating foundation design.

3.2 Geotechnical Report

The District Geotechnical Engineer or the contracted Geotechnical firm will issue a Geotechnical Report for most projects. This report will include:

A. Detailed Soil conditions.
B. Foundation recommendations.
C. Design parameters.
D. Constructability considerations.
E. Back ground information that may assist the Structural Engineer in determining appropriate pile lengths.
F. Input data for COM624, FB Pier, and other design programs when lateral loads are a major concern.
G. Completed FHWA Report ***Checklist and Guidelines for Review of Geotechnical Reports and Preliminary Plans and Specifications***.
H. Core boring drawings reflecting the foundation data acquired from field investigations.
I. Required Load tests.

The Geotechnical Engineer will contact the District Construction Office and District Geotechnical Engineer, as needed, to obtain local, site-specific foundation construction history.

The Report will be prepared in accordance with the Department's ***Soils and Foundations Handbook***, which is available through the Office of Maps and

Publications Sales in Tallahassee. Geotechnical Reports will 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.

In the event that a contracted geotechnical firm prepares the Geotechnical Report, both the State and District Geotechnical Engineers generally will review it for Category 2 Structures and the District Geotechnical Engineer for Category 1 Structures (See the ***Plans Preparation Manual***, Volume I, Chapter 26 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 will work with the District Geotechnical Engineer throughout the course of the geotechnical activities. Verify the scope of services, as well as the proposed field and laboratory investigations, with the District Geotechnical Engineer prior to commencing any operations.

3.3 Foundation Scour Design [2.6]

This is a multi-discipline effort involving Geotechnical, Structures, and Hydraulics/Coastal Engineers. The process described 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.

A. 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 loadings described in SDG 2.12 Substructure Limit States, (do not impose arbitrary deflection limits 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 configurations. The Structures Engineer must apply sound engineering judgment in comparing results obtained from scour computations with available hydrological, hydraulic, and geotechnical data to achieve a reasonable and prudent design.

B. The Hydraulics Engineer
Utilizing good engineering judgment 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.

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

It is not necessary to consider the scour effects on temporary structures unless otherwise directed by the Department.

3.4 Lateral Load [10.7.3.8][10.8.3.8]

Use a resistance factor of 1.0 for lateral analysis.

3.5 Piles

3.5.1 General

Two sizes of cylinder piles are currently available, 54 inch and 60-inch diameter. For Design Build projects, design cylinder piles, footing and caps so that either of the available sizes are adequate structurally and geometrically interchangeable. Base soil capacity and structural strength designs on the 54-inch pile. Limit the moment capacity for design to 2770 kip-ft. Base details and minimum pile spacing on the 60-inch pile. Include in the plans, an alternate foundation type; prestressed piles or drilled shafts. For Design Build projects, design and detail based on individual pile sizes.

For concrete cover on cylinder pile reinforcement, see Table 1.1 Minimum Concrete Cover Requirements.

3.5.2 Minimum Sizes

Fender Systems.. 14” square piling
Bridges.. 18” square piling
Bridges (Extremely Aggressive Environment due to chlorides) .... 24” square piling
Drilled Shafts .... 36” diameter shafts

Specify minimum 24-inch piles for "Extremely Aggressive" salt water. Smaller piles may be acceptable with the approval 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. If pile bents will be exposed to wet/dry cycles that could necessitate future jacketing, a minimum 24-inch pile must be used. The District Maintenance Engineer may grant exemptions for pedestrian bridges and fishing piers.

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

Delete the first sentence of ***LRFD*** Articles 10.7.1.5 and 10.8.1.6 and substitute the following information:

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

In order to develop the full strength of a voided pile, the upper 8 feet of the pile, as a minimum, must be solid with 4 feet embedded into the pile cap. If the pile void must be filled to achieve a solid section, as after cutting the pile to grade, the void must be filled with structural concrete for an 8-foot minimum length, after all non-metallic form liners are removed from the void.

The strength of a voided pile can be enhanced by grouting a pipe or reinforcing bar cage into the void. With this detail the full composite section capacity of the pile and pipe/cage can be developed. The required length of this composite pile section is a function of the loading but must be no less than 10 feet (6 feet below the bottom of the pile cap).

Pile embedment lengths for larger size piles and their corresponding capacities with or without supplemental reinforcing will require custom designs based upon ***LRFD*** specifications, Department approval, and may require strand development/pile embedment tests.

3.5.4 Downdrag for Pile Foundation Design

For pile or drilled shaft foundations, show the downdrag load on the plans.

For pile foundations, “downdrag” is 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). The dynamic resistance typically equals 0.50 to 1.0 times the ultimate skin friction.

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

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

It is not necessary to consider the scour effects on temporary structures unless otherwise directed by the Department.

3.5.5 Resistance Factors [10.5.5]

Delete ***LRFD*** Tables 10.5.5-2 and 10.5.5-3 and substitute SDG Tables 3.1 for piles and 3.5 for drilled shafts.

Table 3.1 Resistance Factors for Piles

Foundation Type	Application	Design Consideration	Design and/or QC Methodology	Resistance Factor, Φ
Piles	All Structures	Compression	SPT97 (for estimating pile lengths	0.65
			PDA (EOD)	0.65
			Wave Equation Analysis	0.35
			Static Load Testing	0.75
		Uplift	SPT97 (for estimating pile lengths	0.55
			Static Load Testing	0.65
		Lateral	FBPier or similar program	1.00

3.5.6 Battered Piles [10.7.1.11][10.8.1.2]

Plumb piles are preferred; however, if the design requires battered piles, a single batter, either parallel or perpendicular to the centerline of the cap or footing, is preferred. If the design requires a compound batter, orient the pile so that the direction of batter will be perpendicular to the face of the pile. The Structures Engineer must, with input from the Geotechnical Engineer, evaluate the effects of length and batter on the selected pile size. Do not exceed the following maximum batters, measured as the horizontal-to-vertical ratio, "h:v":

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

Commentary:
Longer piles are more susceptible to bending, especially if an obstruction is hit during driving. Also, when driven on batters, the tips of long, slender piles tend to bend downward due to gravity. This creates additional bending stresses. This tendency is especially true in deep water and in very soft/loose soil. These deflections can be severe enough to cause significant, additional stresses due to bending when hit with a pile-driving hammer. These additional stresses can lead to pile failure in bending. Hard subsoil layers can also deflect piles outward in the direction battered causing pile breakage in bending. The ability to install battered piles must be determined during the design phase.

3.5.7 Minimum Pile Tip [10.7.1.11][10.8.1.2]

The minimum pile or drilled shaft tip elevation must be the deepest of the minimum elevations that satisfy lateral stability requirements for the three limit states The minimum tip elevation may be set lower to satisfy any unique soil conditions provided the requirements in the ***FDOT Soil and Foundation Handbook*** are met. The Structures Engineer must establish the minimum tip with the concurrence of the Geotechnical Engineer.

3.5.8 Anticipated Pile Lengths [10.7.1.10]

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.

3.5.9 Test Piles [10.7.1.13]

Test piles include both static and dynamic load test piles which are driven to determine soil capacity, pile driving system, pile driving ability, production pile lengths, and driving criteria. At least one test pile must be located approximately every 200 feet of bridge length with a minimum of two test piles per bridge structure. These requirements 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, specify at least one test pile at each pier. Consider maintenance of traffic requirements, required sequence of construction, geological conditions, and pile spacing when determining the location of test piles. For phased construction, test piles should be located in the first phase of construction. The Geotechnical Engineer must verify the suitability of the test pile locations.

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

Test piles should be at least 15 feet longer than the estimated length of production piles. Additional length may be required by the load frame geometry when static load tests are used. The Structural Engineer must coordinate his recommended test pile lengths and locations with the District Geotechnical Engineer and Geotechnical Consultant, prior to finalization of the plans.

Commentary:
Test piles are exploratory in nature and 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). The Structures Engineer must take these facts into consideration when establishing test pile lengths.

3.5.10 Pile Load Tests [10.7.3.6][10.8.3.6]

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

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

A. Static Load Test [10.7.3.6]

When static load test piles or drilled shafts are required, show on the plans: the number of required tests, the pile or shaft type and size, and test loads.

All piles, which will be statically load-tested, must first be subjected to dynamic load testing. Static load tests must test the pile to failure as required in Section 455 of the Specifications.

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

B. Dynamic Load Test [10.7.1.15]

When dynamic testing is required, (See Geotechnical Report) all test piles must have dynamic load tests. Indicate this requirement with a note on the foundation layout sheet.

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

C. Special Considerations

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

3.5.11 Pile Driving Resistance [10.7.1.3]

The Geotechnical Engineer calculates the Required Driving Resistance (RDR) as:

RDR=UBC= (Factored Design Load + Net Scour + Downdrag)/ Ø [Eq 3-1]

Where: Ø is the resistance factor taken from Table 3.1 and UBC is the Ultimate Bearing Capacity.

The Required Driving Resistance values given in the Pile Installation Table must not exceed the following values unless specific justification is provided and accepted by the Department's District Geotechnical Engineer for Category I structures or the State Geotechnical Engineer for Category II structures:

Table 3.2 Maximum Pile Driving Resistance

Pile Size (inches)	**Capacity (tons)**
18	300
20	360
24	450
30	600
54 inch or 60 inch concrete cylinders	1550

When the minimum tip requirements govern over bearing requirements, construction methods may need to be modified so that pile driving resistance never exceeds the values given above. Construction methods such as preforming or jetting may be required at these locations. See Table 3-3.

The values listed above are based on upper bound driving resistance of typical driving equipment. The maximum driving resistance values listed above should not be considered default values for design. These values may not be achievable in certain areas of Florida based on subsoil conditions. Local experience may dictate designs utilizing substantially reduced ultimate bearing loads. Contact the District Geotechnical Engineer for guidance in the project area.

3.5.12 Pile Jetting and Preforming

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

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

Before specifying in contract documents, verify that jetting will not violate any of the environmental permits.

3.5.13 Foundation Installation Table

For projects utilizing driven piles or drilled shafts include a “Pile Data Table” or a “Drilled Shaft Data Table” and notes as shown below. For items that do not apply, place "N/A" in the column but do not revise or modify the table. Round loads to the nearest ton. Round elevations and pile lengths to the nearest foot.

The “Pile Data Table” (Table 3.3) and “Drilled Shaft Data Table” (Table 3.4) are not required in the Geotechnical Report; however, the Geotechnical Engineer of Record must review the information shown on the plans for these tables.

Table 3.3 Pile Data Table

PILE DATA TABLE														
INSTALLATION CRITERIA								DESIGN CRITERIA						
PILE OR BENT NUMBER	PILE SIZE (IN)	REQUIRED DRIVING RESISTANCE (TONS)	TENSION CAPACITY (TONS)	MINIMUM TIP ELEV. (Ft)	TEST PILE LENGTH (Ft)	REQ'D JET ELEVATION (Ft)	REQ'D PREFORMED ELEVATION (Ft)	FACTORED DESIGN LOAD (Tons)	DOWN DRAG (Tons)	TOTAL SCOUR RESISTANCE (Tons)	NET SCOUR RESISTANCE (Tons)	LONG TERM SOUR ELEVATION (Ft)	100 YEAR SCOUR ELEVATION (Ft)	PILE DIAMETER (IN)

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 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).

Long Term Scour- Specify whether Clear Water Scour or Live Bed Scour as defined in the FDOT Drainage Manual.

100-year Scour- Estimated elevation of scour to the 100 year storm event.

3.5.14 Plan Notes

Use Equation 3.1 to determine the Required Driving Resistance value for Table 3.3, Pile Data Table.

Additional Plan Notes:

A. Minimum Tip Elevation is required ________________________ (reason must be completed by designer, for example: "for lateral stability" , "to minimize post-construction settlements" or " for required tension capacity").

B. When a required jetting or preformed elevation is not shown on the table, do not jet or preform pile locations without prior written approval of the District Geotechnical Engineer. Do not advance jets or preformed pile holes deeper than the jetting or preformed elevations shown on the table without the prior approval of the District Geotechnical Engineer. If actual jetting or preforming elevations differ from those shown on the table, the District Geotechnical Engineer will determine the required driving resistance.

3.5.15 Prestressed Piling [5.13.4.3]

For prestressed piling not subjected to significant flexure under service or impact loading, design strand development in accordance with ***LRFD*** **[5.11.4]** and **[5.8.2.3]**. Bending that produces cracking in the pile, such as that resulting from ship impact loading, is considered significant. A pile embedment of 4 feet into a footing is considered adequate to develop the strength of the pile.

For the standard square FDOT prestressed concrete piles (12 through 30 inch), a pile embedment of 4 feet into a reinforced concrete footing is considered adequate to develop the bending strength of the pile. The pile must be solid, or the pile void filled with structural concrete, within the 4-foot embedment length. A 1 foot embedment is considered a pinned head condition.

For the pinned pile head condition the strand development must be in accordance with ***LRFD*** **[5.11.4]** and **[5.8.2.3]**.

The bending capacity versus pile cap embedment length relationship for pre-stressed piles with widths or diameters larger than 30 inches must be established on a case-by-case basis.

Commentary:
The FDOT Structures Research Center conducted full scale testing of two 30 inch square concrete piles reinforced with prestressing steel and an embedded steel pipe. The piles, which were embedded 4 feet into a reinforced pile cap, developed the calculated theoretical bending strength of the section without strand slip. See FDOT Report No 98-9 "Testing of Pile-to-Pile Cap Moment Connection for 30" Prestressed Concrete Pipe-Pile". It was concluded that the confinement effects of the pile cap serve to improve the bond characteristics of the strand.

3.5.16 Fender Piles

Fender piles generally have a short life expectancy and are considered to be sacrificial, therefore, no corrosion protection is required beyond the use of concrete class as shown in Table 1.2.

There are no specific design requirements for fender piles; however, the length must meet or exceed the minimum pile penetration set forth in Section 455 of the Specifications.

Commentary:
The pile installation constructability must be reviewed by the Geotechnical Engineer to verify that the Contractor can reasonably obtain the pile tips shown in the plans, and the use of any penetration aids (jetting, preforming, etc.) will not jeopardize adjacent structures.

3.5.17 Steel Sheet Piles

Design and detail the sheet pile properties for both cold-rolled and hot-rolled sections. Base design of the cold-rolled section on flexural section properties that are 85% of the full-section values. Detail wall components such as caps and tiebacks to work with both the hot-rolled and cold-rolled sections.

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

3.6 Drilled Shafts

For projects with drilled shafts, include in the plans, a Drilled Shaft Data Table, identical in format and information to Table 3.4 below. Take the value of Φ from Table 3.2.

Table 3.4 Drilled Shaft Data Table

DRILLED SHAFT DATA TABLE									
INSTALLATION CRITERIA					DESIGN CRITERIA				
Pier or Bent No.	Shaft (in)	Tip Elev. (ft)	Min Tip Elev.(ft)	Min. Rock Socket Length (ft)	Factored Design Load (tons)	Downdrag (tons)	Long Term Scour Elev. (ft)	100-yr Scour Elev. (ft)	Φ

Tip Elevation - The elevation to which the shaft must be constructed unless test load data, rock cores, or other geotechnical data obtained during construction allows the Engineer to authorize a different tip elevation.
Min. Tip Elevation -The highest elevation that the shaft tip may be constructed if adjustments are made to the tip elevation specified.

3.6.1 Resistance Factors [10.5.5]

Table 3.5 Resistance Factors for Drilled Shafts

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

*FDOT Research Report D647 kept in the Research Center of the FDOT Central Office.

3.6.2 Construction Joints

For drilled shafts used in bents located in water containing more than 2000 ppm chloride, detail the shaft to extend without a construction joint a minimum of 12 feet above the Mean High Water elevation or bottom of the bent cap, whichever is lower.

Commentary:
It is preferred that taller shafts extend to the bottom of the bent cap without a construction joint.

3.7 Cofferdams and Seals

When showing seal dimensions in the plans, show the maximum water elevation assumed for the seal design. Design the seal concrete thickness using the exceeding pressure obtained from flow net analysis performed by the Geotechnical Engineer. In the absence of a flow net analysis, use the maximum differential water head.

For design of the cofferdam seal, use a Load Factor of 1.0 and assume the maximum service load stresses from Table 3.6, which apply at the time of complete dewatering of the cofferdam.

In the event greater stress values are required, employ mechanical connectors such as weldments or shear connectors for the contact surfaces of the foundation and seal. When connectors are used to increase shear capacity, detail the connections and note the locations on the drawings. Provide substantiating calculations.

Table 3.6 Cofferdam Design Values

Maximum service load stresses at time of complete dewatering of the cofferdam	
Maximum tension in seal concrete from hydrostatic pressure	250 psi*
Adhesive shear stress between seal concrete and concrete piles or shafts	75 psi*
Adhesive shear stress between seal concrete and steel piles or casings	36 psi*

*Values have been adjusted for appropriate Resistance Factors.

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

3.8 Spread Footing [10.5.5][01.6]

In the event that the Geotechnical Report recommends a spread footing foundation, the maximum soil pressures, the minimum footing widths, the minimum footing embedment, and the ***LRFD*** Table 10.5.5-1 Resistance Factors (Φ) will be indicated in the Geotechnical Report. Determine the factored design load and proportion the footings to provide the most cost effective design without exceeding the recommended maximum soil pressures. Communicate with the Geotechnical Engineer to insure that the corresponding settlements do not exceed the tolerable limits.

When dewatering is recommended in the Foundation Report, note it in the plans. Dewatering is required If the ground water elevation is within 2 feet (or higher) of the bottom of the footing.

Verify sliding, overturning, and rotational stability of the footings.

3.9 Mass Concrete [5.10.8.3]

Consider Mass Concrete requirements in selecting member sizes and avoid Mass Concrete if practical; however, when its use is unavoidable, indicate which portions are 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 3 feet and the ratio of volume of concrete to the surface area is greater than 1.0 foot, provisions for mass concrete are required. (The surface area for this ratio includes 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.) Note Volume and surface area calculations in units of feet.

Take 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 in Mass Concrete, consider more and/or better reinforcing steel distribution, analyze the placement of construction joints, refer to other methods as outlined in ACI 207, ACI 244, and ACI 308.

For estimated bridge pay item quantities, include separate pay item numbers for Mass Concrete (Substructure) and Mass Concrete (Superstructure). Do not calculate seal Concrete as Mass Concrete.

3.10 Crack Control

A. Long Walls:

For long walls and other similar construction:

1.) Limit the length of a section to a maximum of 30 feet between vertical construction joints. See the limits of concrete pours in tall piers elsewhere in this Chapter.
2.) Clearly detail required construction joints on the plans.
3.) Specify construction or expansion joints fitted with a water barrier when necessary to prevent water leakage.

B. Footings:

Specify that footings be cast monolithically. Attach struts and other large attachments as secondary castings.

C. Keyways:

Do not place keyways in horizontal construction joints except that a keyway will be used at the junction of a cast-in-place concrete wall and footing. Provide keyways at formed surfaces of vertical construction joints and elsewhere as necessary to transfer applied loads from one cast section to an adjacent, second pour. Specify or detail trapezoidal keyways for ease of forming and stripping. For example, a typical joint must have a keyway about 2 inches deep and about 6 inches wide (or one third the thickness of the member for members less than 18 inches in thickness) running the full length.

3.11 Pier, Column, and Footing Design

For tall piers or columns, detail construction joints to limit concrete lifts to 25 feet. When approved by the Department, a maximum lift of 30 feet may be allowed to avoid successive small lifts (less than approximately 16 feet) which could result in vertical bar splice conflicts or unnecessary splice length penalties.

Detail splices for vertical reinforcing at every horizontal construction joint; except that the splice requirement may be disregarded for any lift of 10 feet or less.

Coordinate the lift heights and construction joint locations with the concrete placement requirements of the specifications.

On structures over water, all vertical post-tensioning must be 12 feet above MHW. On structures over land, all vertical post-tensioning must be 5 feet above land (fill or natural ground). This restriction precludes the use of post-tensioning from the submerged substructure (fresh or salt) to an air-dry condition within the substructure component. This applies to both CIP and/or precast substructures.

Commentary:
Precast pier segments traditionally have horizontal joints every 7' to 10' feet and at least 4 tendons vertically. Even with the highest attention to field inspection we find that many joints are not watertight. Water and chloride intrusion has been documented in superstructure and substructure post-tensioned tendons in Florida. The interstice area within the 7-wire strand and the space between strands offers too much opportunity for water and oxygen intrusion. Tendon geometry and duct placement (wobble) are factors that contribute to how much grout surround the tendon bundle. During the forensic examinations of the corroded tendons it was observed that duct material must offer the primary protection for the strand material.

Precast pier sections with spliced sleeve connections are allowed.

Plant produced, post-tensioned or pretensioned cylinder piles,(horizontally assembled, stressed and grouted) will be allowed due to the small tendon and duct size. The redundancy within the component due to the number of ducts with a maximum of 3 strands per duct has proven effective. When detailing precast, post-tensioned, segmented cylinder piles for use in pile bents, show the void filled with reinforced concrete to produce a redundant load path. Regardless of the environmental classification, the reinforced concrete plug must extend from the cap to 24 feet below the MLW elevation or the ground line for land bents.

The minimum wall thickness for segmental piers must be 12 inches.

Where external tendons exit the bottom of pier caps, provide a drip one half inch by one half inch drip recess around the tendon duct.

For bridges designed for vessel collision, design pier columns solid from the footing to 15 feet above MHW.

Provide solid section to base of pier columns (or fill with concrete)
- Up to 12 feet above mean high water for water piers.
- Up to 1 foot above finished ground level for land piers.

Locate the bottom of all footings, excluding seals, a minimum of 1 foot below MLW.

All hollow substructures must be sealed from possible sources of leaks and contain free-exiting drains or weep-holes to drain away water that may collect from any source - including condensation.

Drains in hollow piers may be formed using 2-inch diameter permanent plastic pipes set flush with the top of the bottom slab or solid section. Slope interior top of solid base toward drains or weep-holes. Provide weep-holes with vermin guards. Show in the Contract Drawings, locations and details for drains taking into account bridge grade and cross-slope.

Table 3.7 Required tendons for post-tensioned substructure elements:

Post-Tensioned Bridge Element	Minimum Number of Elements
Hammerhead Pier Straddle Beams C-Pier Column (Bars Only) C-Pier Cap C-Pier Footings (Bars only)	6
Hollow cast Piers I-Section Piers	8

3.12 Retaining Wall Types

Consider site, economics, aesthetics, maintenance and constructability when determining the appropriate wall type.

Partial height walls are not permitted. Walls founded at or near natural ground level should extend to the top of embankment. The embankment should not slope down to the top of the wall as this condition presents safety and maintenance issues. Walls founded in new embankment are subject to bearing, rotation, settlement, erosion and maintenance issues and should be avoided. The embankment in the outer portion is very difficult if not impossible to compact. Walls subject to large short-term settlements during construction should be carefully analyzed for construction problems. Large short term settlement is defined according to wall type; for C-I-P walls the settlement may be as little as 1” and as much as 4” for MSE walls. Settlements in both longitudinal and transverse directions must be considered. The EOR for the project is responsible for determining the wall type to be used. Walls requiring a two faced wall system should be included in the plans along with any special soil improvement techniques and instrumentation and monitoring program. Any soil improvement techniques to consolidate the foundation soil prior to construction should be shown in the plans.

The following wall types are being used or considered for use by FDOT.

3.12.1 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 bearing pressure and must exhibit very little differential settlement. Verify 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 of 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 soil reinforcement (See Figures 3-2 and 3-3).

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 1000 sq ft and greater than 10 feet in height.

3.12.2 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 3-4 and 3-5).

3.12.3 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. Consider using geosynthetic reinforcement 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 12 feet 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 1000 sq ft, and the wall is greater than 10 feet in height (See Figures 3-6 and 3-7).

3.12.4 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 3-8) and may offer aesthetic and constructability advantages.

3.12.5 Steel Sheet Pile Walls

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 15 feet in height. Steel sheet pile walls over 15 feet are tied back with either prestressed soil anchors, soil nails, or deadmen.

Steel sheet pile 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 3-9).

3.12.6 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 3-10).

3.12.7 Wire Faced Walls

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. Verify that proposed systems 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 (See Figure 3-11).

3.12.8 Soil Nails

The Department is allowing the use of soil nails. The relative cost of this type wall has not been determined because Contractors have yet to bid this alternate (See Figure 3-12).

3.12.9 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 of wall is very competitive with concrete sheet pile walls (See Figure 3-13).

3.12.10 Modular Block Walls
Modular blocks consist of dry cast, 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.

3.12.11 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.

3.12.12 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 must be removed to accommodate a new lane and a retaining wall must be built under the bridge. Several methods have been used to remove the existing front slope and maintain the stability of the remaining soil. One method is to excavate slots or pits in the existing fill to accommodate soldier beams. The soil is then excavated and timber lagging is placed horizontally between the vertical soldier beams. The soldier beams are tied-back by the use of prestressed soil anchors. This procedure will maintain the soil while the permanent wall is built. The permanent wall should be designed to accept all appropriate soil, dead and live loads. The temporary lagging must not contribute to the strength of the permanent wall (See Figure 3-3).

3.12.13 Hybrid - Gravity/MSE Wall
Hybrids are basically gravity systems with some MSE characteristics. The FDOT approved system 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 3-15)

3.13 Retaining Wall Design

Use Chapter 30, ***Plans Preparation Manual*** retaining wall plans preparation and administrative requirements in conjunction with the design requirements of this Section. Refer to Chapter 1 for the retaining wall concrete class and reinforcing steel cover requirements. Show the retaining wall concrete class on the plans.

3.13.1 Earth Loads for Wall Design

The Rankine earth pressure may be used in lieu of lateral earth loads on walls developed from a Coulomb earth pressure. If the Rankine earth pressure is used, the resultant lateral earth load can be assumed to be located at the centroid of the earth pressure diagram.

3.13.2 Corrosion Rates

Delete the corrosion rates specified in LRFD Article [11.9.8.1] and substitute the following requirements:

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

Do not use Metallic reinforcement in Extremely Aggressive Environments without approval from the State Structures Design Engineer.

3.13.3 Soil Reinforcement

A. Design soil reinforcement and connections for permanent walls with a design life of 75 years. Design walls supporting abutments on spread footings with a design life of 100 years.

B. Design soil reinforcement for temporary walls with a design life of not less than the contract time of the project or three years whichever is greater.

C. Verify the stress in the steel soil reinforcement will not exceed 0.55Fy for steel straps, and 0.48Fy for welded wire mats or grids. Specify ASTM A82, Grade 60 steel.

D. Analyze vertical stresses at each reinforcement level for local equilibrium of all forces to that level only. Include the effects of any eccentricity.

E. Specify galvanized steel soil reinforcement for permanent walls. Steel soil reinforcement for temporary walls may be galvanized or uncoated (black) steel.

F. Epoxy coated reinforcement as specified in ***LRFD*** [11.9.8.1] is not permitted. Passive metal soil reinforcement (i.e., stainless steel, aluminum alloys, etc.) is permitted only with written SDO approval.

G. Geosynthetic Geogrids must comply with Chapter 31 of the Plans Preparation Manual, Volume I (Topic No. 625-000-005).

H. Do not design Soil reinforcement skewed more than 15 degrees from a position normal to the wall panel unless necessary and clearly detailed for acute corners.

I. In addition to ***LRFD*** Article [11.9.5.1.4], soil reinforcement lengths, "L", measured from the back of the facing element, must not be less than the greater of the following:

MSE Walls and Walls in Front of Abutments on Piling [11.9.5.1.4]
L = 8 feet minimum length.
L = 0.7 H1, H1 = mechanical height of wall, in feet
(See Figures 3-16 and 3-17)
L = length in feet, required for external stability design.

MSE Walls In Front of Abutments on Spread Footings [11.9.7]
L = 22 feet
L = 0.6 H1 + 6 feet
L = 0.7 H1

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

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

K. The horizontal soil reinforcement connection force of 85% of the maximum calculated force as stated in ***LRFD*** Article [11.5.1.2] must be modified to a minimum of 100% T_{max}, where:

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

L. Creep reduction factors in geosynthetic soil reinforcement must be included in seismic designs.

M. In addition to ***LRFD*** Article [11.9.4.3] for overturning, the following applies:
a.) Use a soil resistance factor of 0.67 for overturning.

N. For Polymeric Reinforcements in Soils, supplement ***LRFD*** Article [11.9.5.1.3] with the following:
For geosynthetics, T_a must 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.

3.13.4 External Stability

A. The required factors of safety for the external stability are as follows:
Overturning....................2.0
Sliding....................1.5
Internal Pullout....................1.5 (Allowable deformation ¾ inch)
Bearing Capacity....................2.5
Overall Stability....................1.5

B. The reinforced backfill soil parameters for the analyses are:
Sand fill (statewide except Dade and Monroe counties)
Moist unit weight.................... 105 pcf min.
Friction angle....................30 degrees.
Limerock fill (Dade and Monroe counties only)
Moist unit weight....................115 pcf
Friction angle....................34 degrees

C. For steel reinforcements, consider the following requirements:
For Bars and Straps
F_y'. ....0.55 Fy at end of design life
F_u ....$0.55F_u$ at net section of bolted connection

D. For Grids and bar mats
F_y' ....0.48 Fy

E. Maximum pullout factors:
f^*_{max}....................1.5 for ribbed strips
N_{pmax} ....30 for grids and bar mats

F. The properties for geogrids will be the same as the geogrids for steepened slopes as shown on Index 501 of the Roadway and Traffic Standards (January 2000, English Units.)

G. H1, the mechanical height, of an MSE structure must 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 3-16 and 3-17 and can be calculated using the equation provided in Figure 3-16. The methodology for this analysis is included in FHWA publication ***FHWA-SA-96-072 Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guideline*** In addition to the two conditions shown in AASHTO, horizontal back-slope and infinite sloping backfill, we are including Figures 3-16 and 3-17 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 must apply. If a traffic surcharge is present and is located within 0.5H of the back of the reinforced soil mass, then it must be included in the analysis.

H. The Geotechnical Engineer of Record for the project is responsible for designing the strap lengths for the external conditions shown in Figure 3-1 and any other conditions that are appropriate for the site.

3.13.5 Acute Corners Less than 75 degrees (MSE Walls)

When two walls intersect forming an internal angle of less than 75 degrees, design the nose section as a bin wall. Submit the calculations for this special design with the plans for review and approval. Design structural connections between the wall elements within the nose section 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 must conform to the requirements for the wall design in general. Design the nose section 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.

Design soil reinforcements 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 10 feet beyond a Rankine failure surface. Design for facing connections, obstructions, pullout and allowable stresses in the reinforcing elements must conform to the requirements of the wall design in general (See Figure 3-18).

3.13.6 Minimum Wall embedment in Soil

Consider scour and bearing capacity when determining wall embedment depth. Consult the District Drainage and Geotechnical Engineers to determine the elevation of the top of leveling pad which must also conform to a minimum embedment to the top of leveling pad of 1’-6” as determined in Figure 3-19. Normal construction practices should be considered.

3.13.7 Apparent Coefficient of Friction (f*)

The coefficient of friction (f*) need not be reduced for saturated back-fill, which are properly placed and compacted.

3.13.8 Connections (MSE Walls)

Design the soil reinforcement to facing panel connection 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 it's connections should be reduced accordingly.

3.13.9 End Bents Behind MSE Walls

A. End Bents on Piling:

All end bent piles must be plumb.

The front face of the end bent cap or footing must be a minimum distance of 2 feet behind the front face of the wall. (2 feet minimum clear distance between face of piling and leveling pad). This dimension is based upon the use of 18-inch piles. For larger piles, the clear distance between the wall and pile must be increased such that no strap is skewed more than 15 degrees.

Soil reinforcement to resist the overturning produced by the earth load, friction, and temperature must be attached to end bents. If the long-term settlement exceeds 4 inches, the reinforcement must not be attached to the end bent. In this event, a special wall behind the backwall must be designed to accommodate the earth load.

B. End Bents on Spread Footings:

The spread footing must be sized so that the bearing pressure must not exceed 3000 psf.

The footing must be a minimum distance of 1 foot behind the facing panel.

The minimum distance between the centerline of bearing on the end bent and the back of the facing panel must be 4 feet.

3.13.10 Facing Panels (MSE Walls) [11.9.5.1.6]

The basic (typical) panel size must not exceed 40 sq ft in area, and special panels (top out, etc.) must not exceed 50 sq ft in area. Full-height facing panels must not exceed 8 feet in height. Use of larger panels will be considered by the SDO on a case-by-case basis. The reinforcing steel concrete cover must comply with Table 1.1.

3.13.11 Utilities

No utilities should be allowed in the soil-reinforced zone behind MSE or tieback walls.

Commentary:
When utilities are placed in the reinforced zone, they cannot be maintained because excavation in this zone will compromise the structural integrity of these wall types. Also, leaking pipes could wash out and destroy the structural integrity.

3.13.12 Minimum Front Face Embedment

The minimum front face embedment must comply with Figure 3-19.

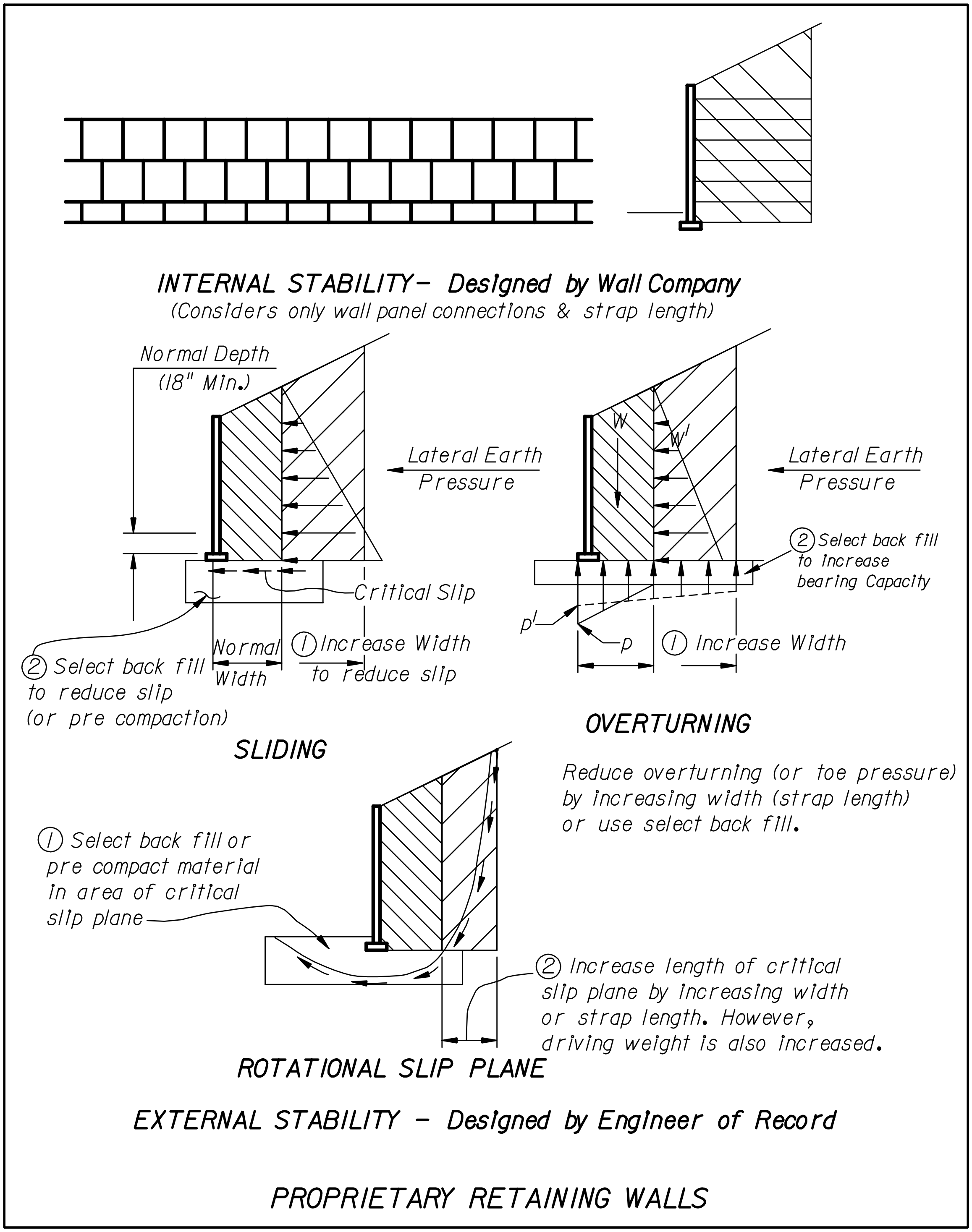
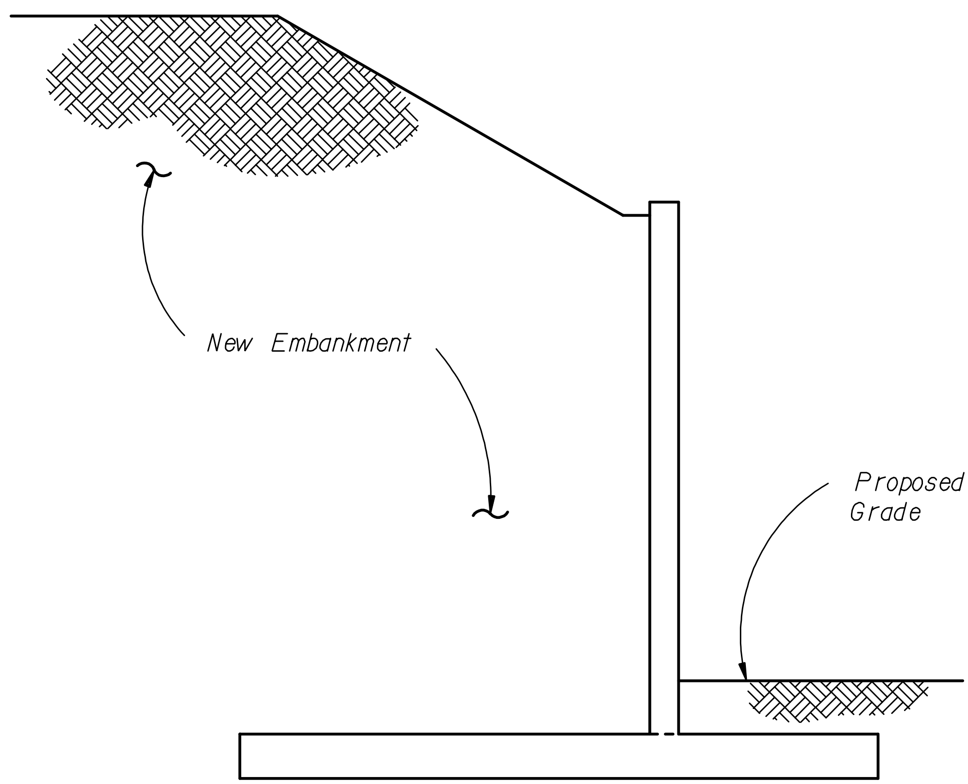

INTERNAL STABILITY- Designed by Wall Company
(Considers only wall panel connections & strap length)
Normal Depth
(18" Min.)
Lateral Earth
Pressure
Critical Slip
2 Select back fill
to reduce slip
(or pre compaction)
Normal
Width
1 Increase Width
to reduce slip
SLIDING
W
W'
Lateral Earth
Pressure
2 Select back fill
to increase
bearing Capacity
p'
p
1 Increase Width
OVERTURNING
Reduce overturning (or toe pressure)
by increasing width (strap length)
or use select back fill.
1 Select back fill or
pre compact material
in area of critical
slip plane
2 Increase length of critical
slip plane by increasing width
or strap length. However,
driving weight is also increased.
ROTATIONAL SLIP PLANE
EXTERNAL STABILITY - Designed by Engineer of Record
PROPRIETARY RETAINING WALLS

Figure 3-1



Cast-In-Place WALL
(Fill Location)

Figure 3-2

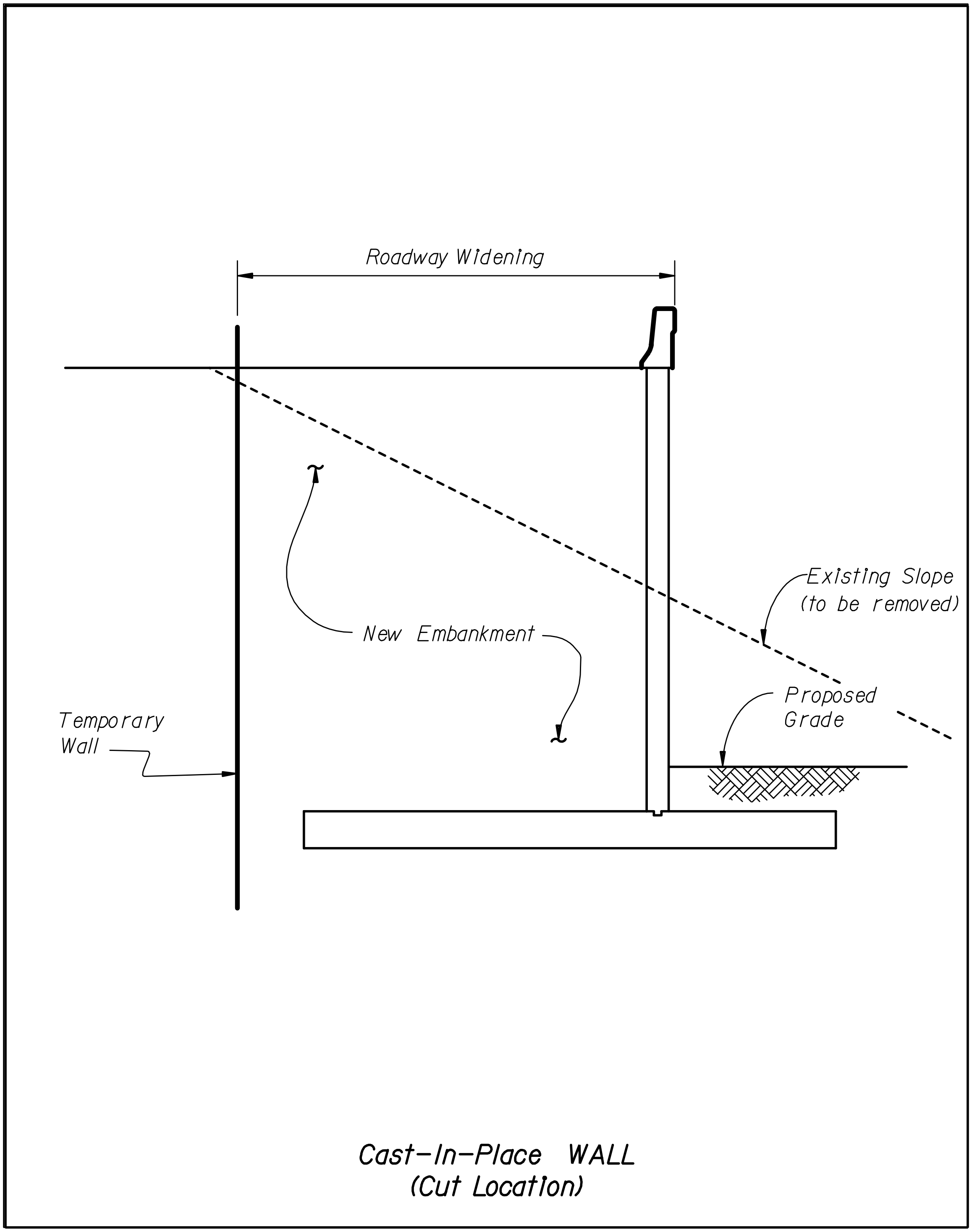
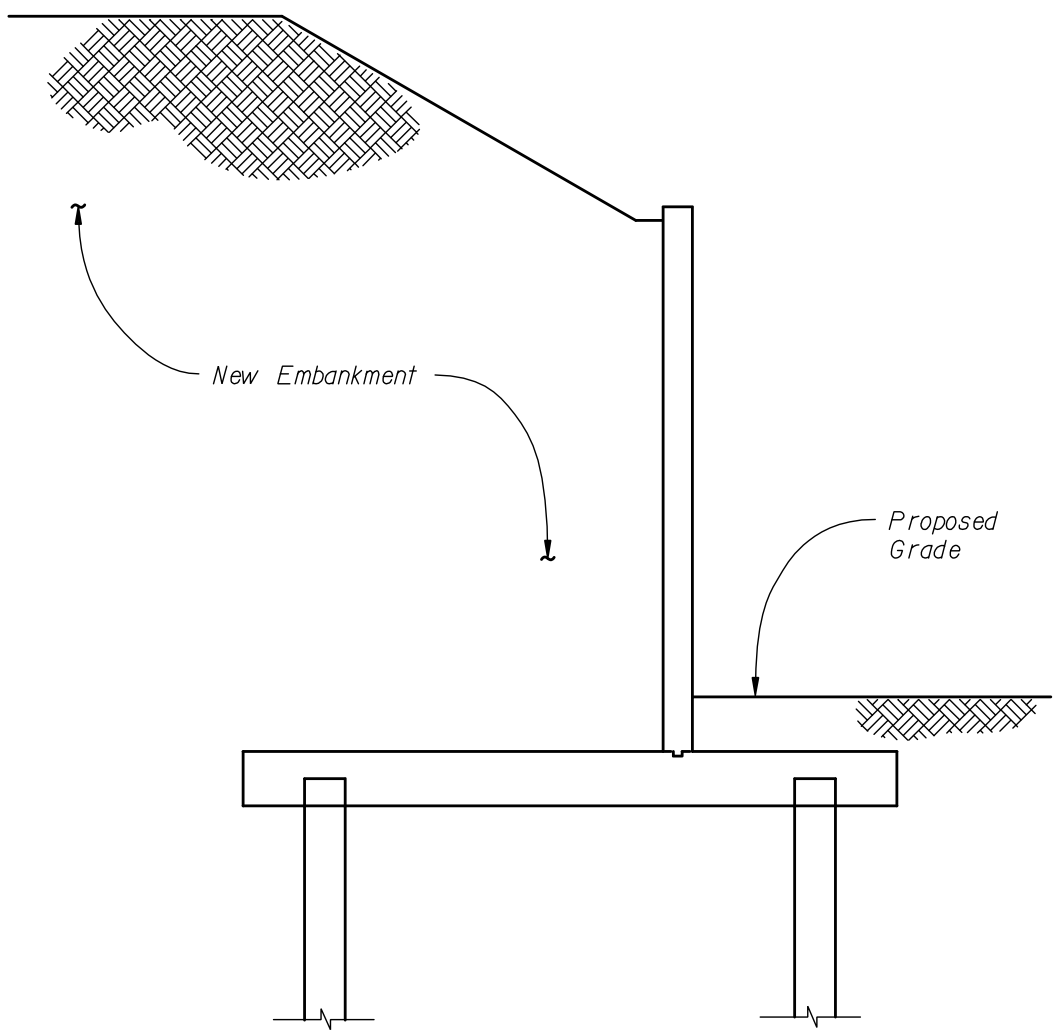
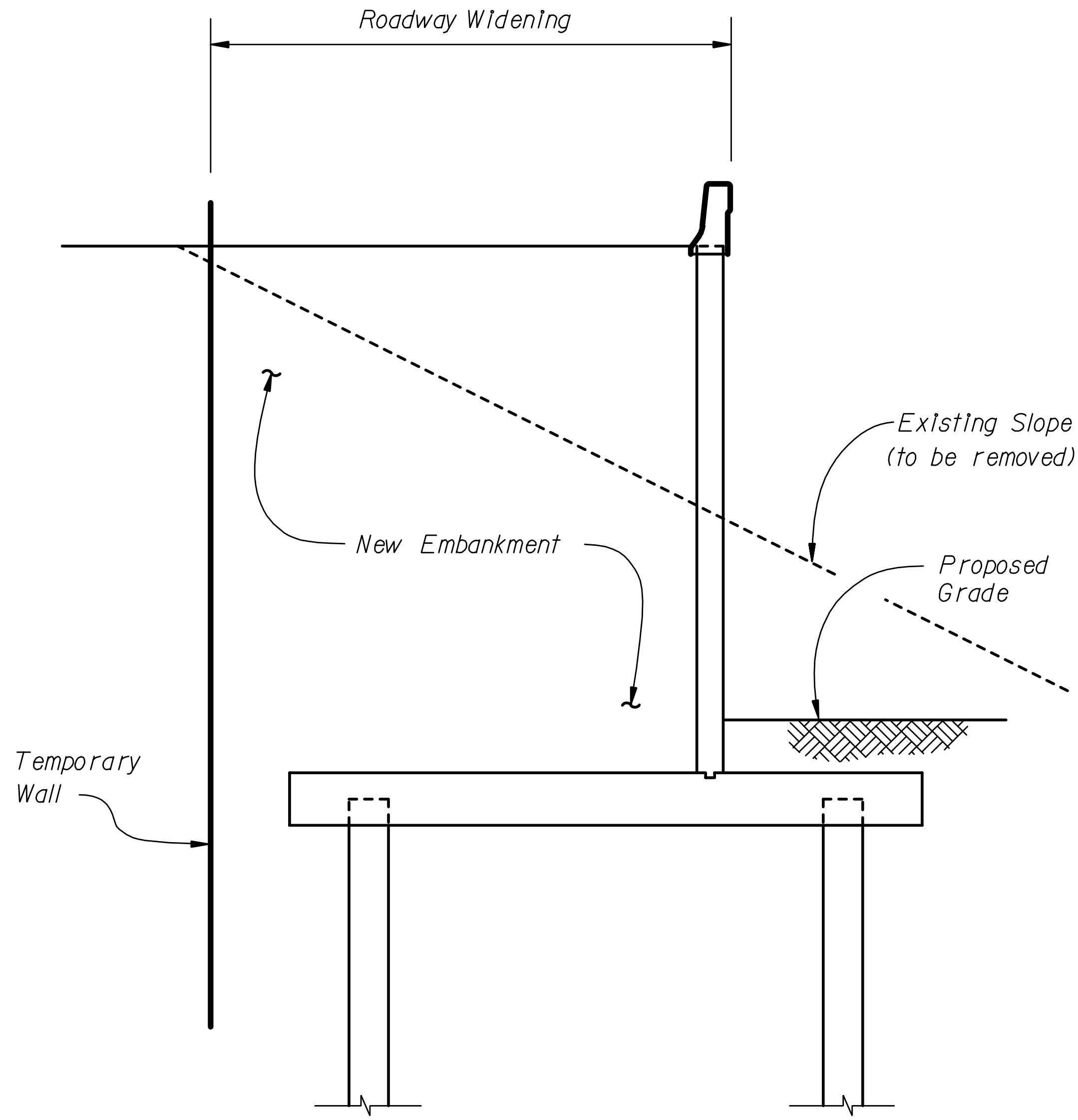

Roadway Widening
Existing Slope
(to be removed)
New Embankment
Proposed
Grade
Temporary
Wall
Cast-In-Place WALL
(Cut Location)

Figure 3-3



Cast-In-Place WALL - PILE SUPPORTED
(Fill Location)

Figure 3-4



Cast-In-Place WALL - PILE SUPPORTED
(Cut Location)

Figure 3-5

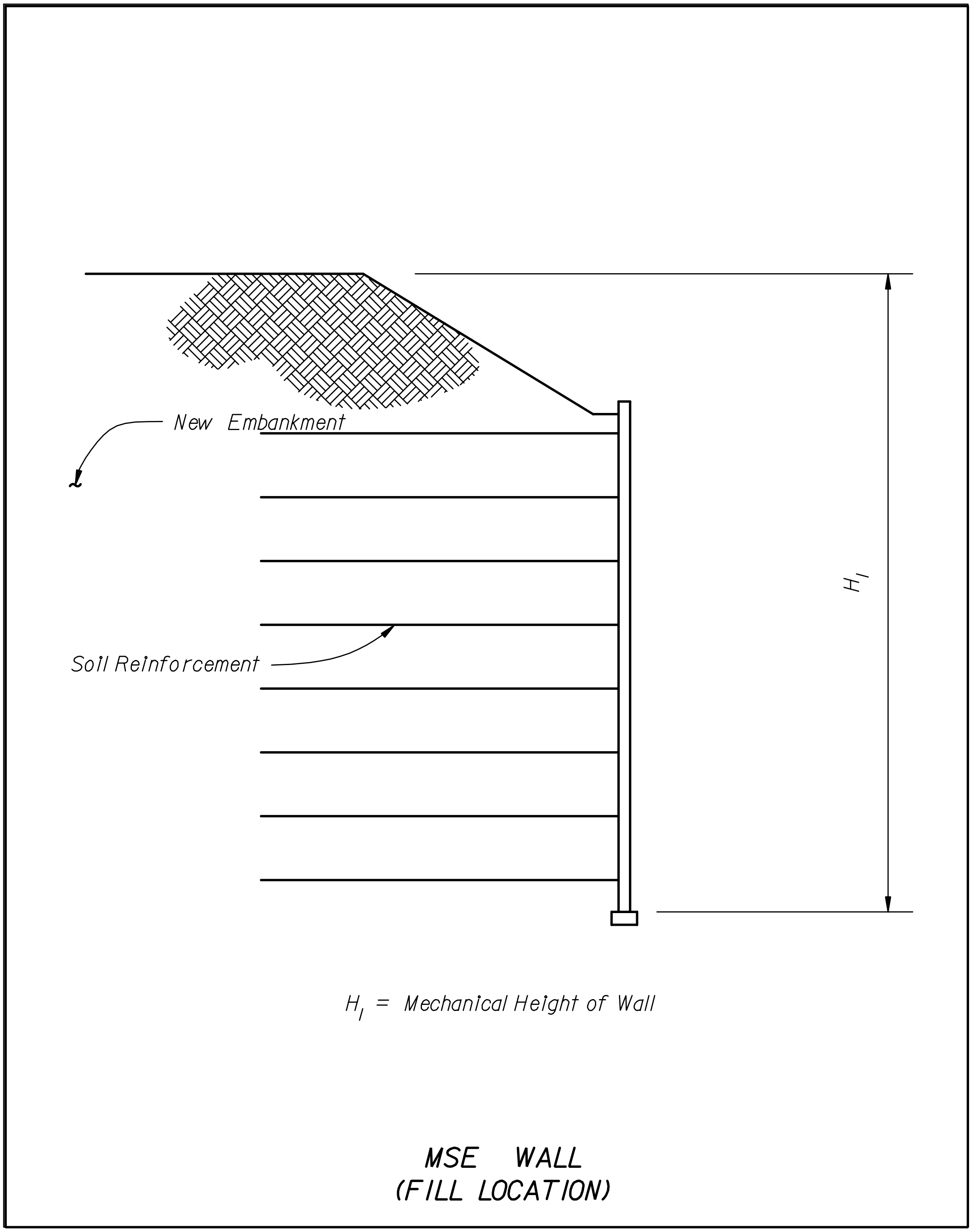

New Embankment
Soil Reinforcement
H_l
H_l = Mechanical Height of Wall
MSE WALL
(FILL LOCATION)

Figure 3-6

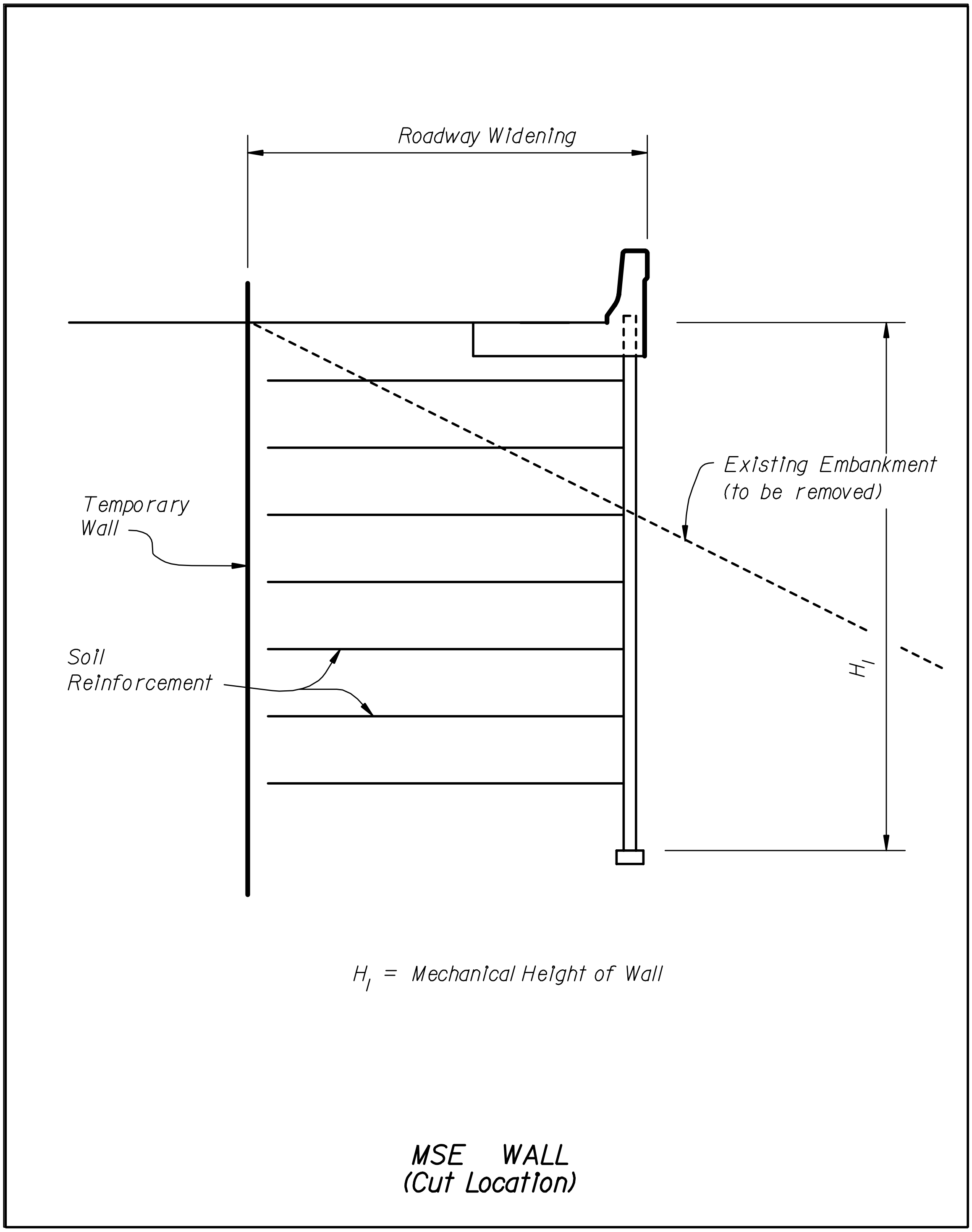

Roadway Widening
Existing Embankment
(to be removed)
Temporary
Wall
Soil
Reinforcement
H_l
H_l = Mechanical Height of Wall
MSE WALL
(Cut Location)

Figure 3-7

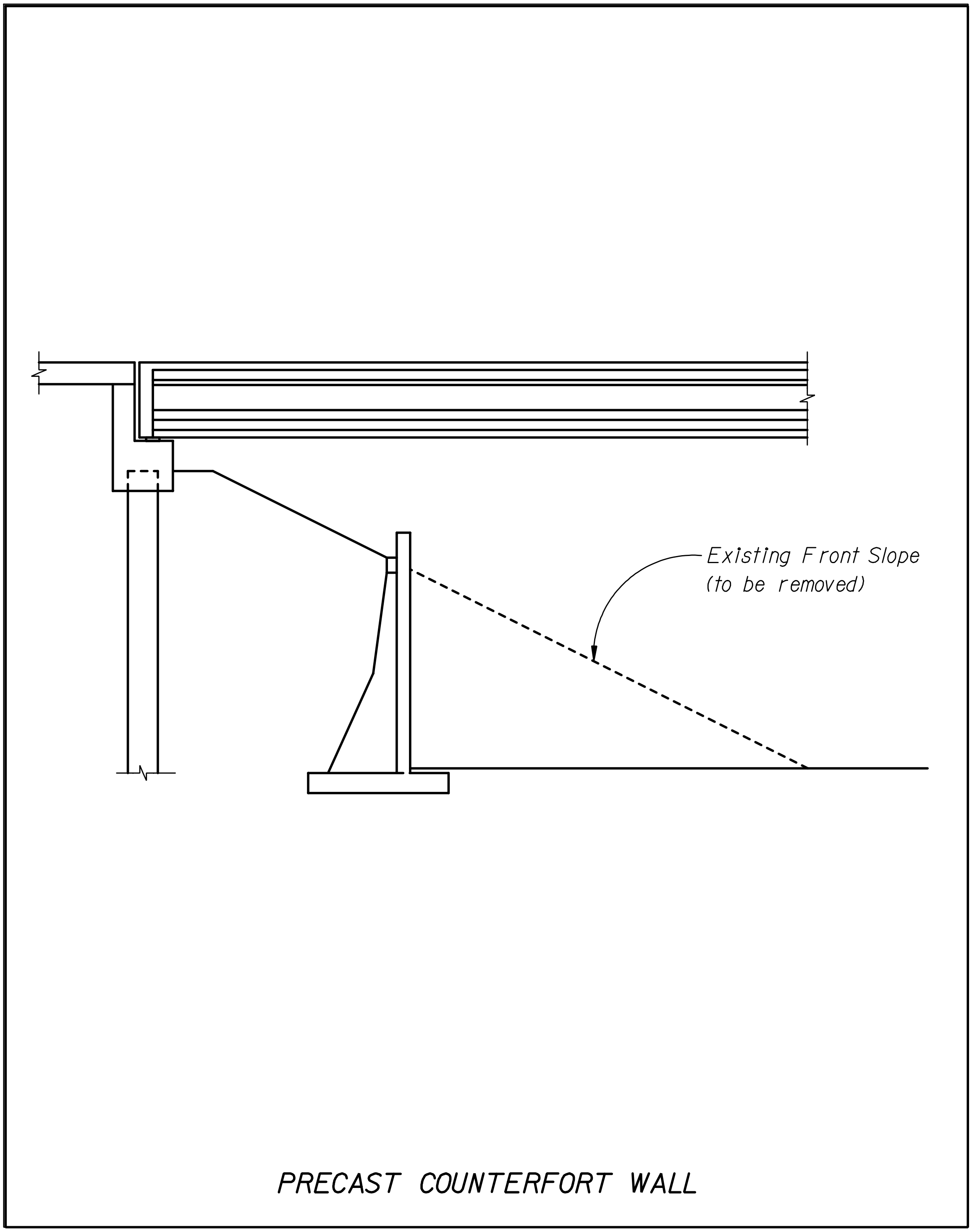
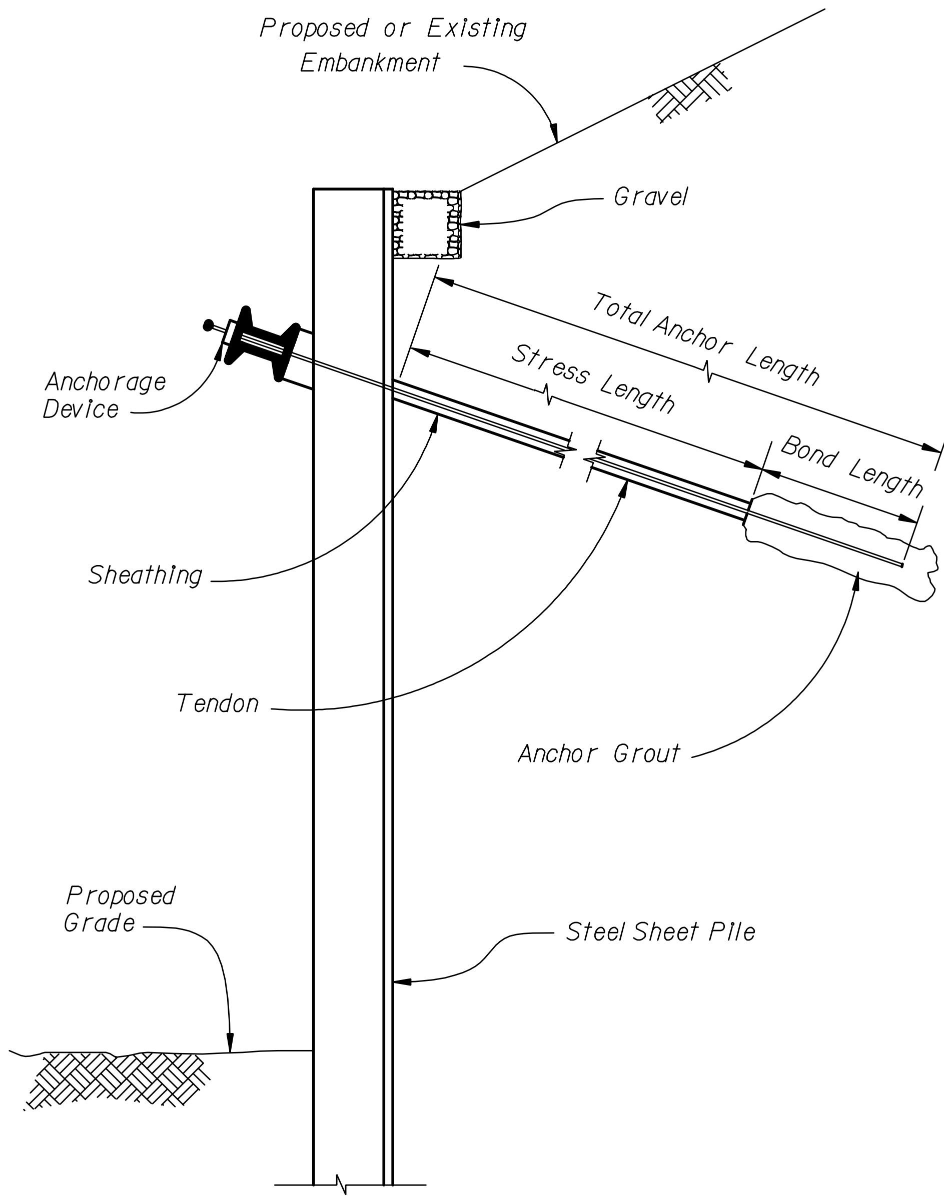

Existing Front Slope
(to be removed)
PRECAST COUNTERFORT WALL

Figure 3-8



TIEBACK COMPONENTS
(Steel Sheet Piles)

Figure 3-9

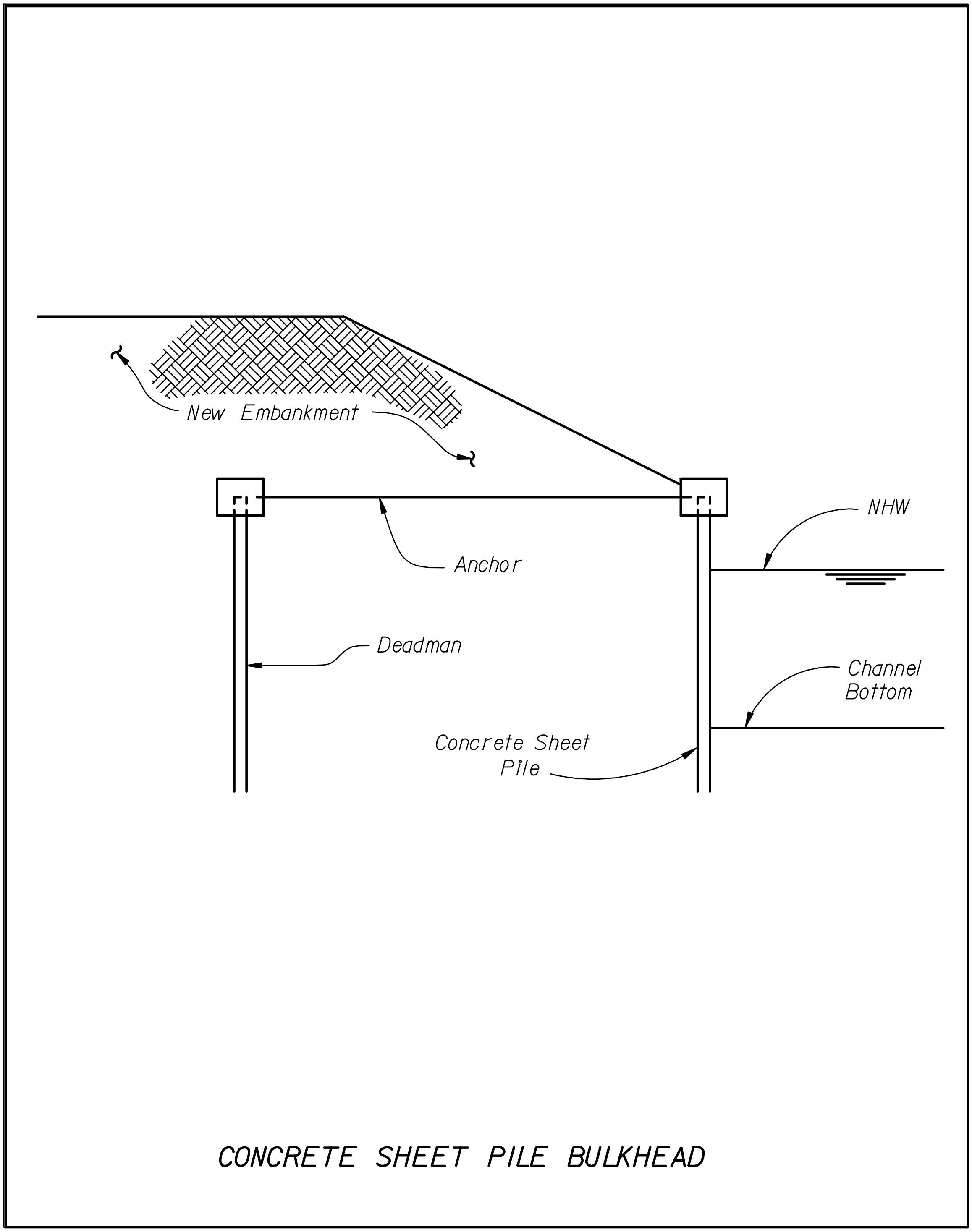

New Embankment
Anchor
NHW
Deadman
Channel Bottom
Concrete Sheet Pile
CONCRETE SHEET PILE BULKHEAD

Figure 3-10

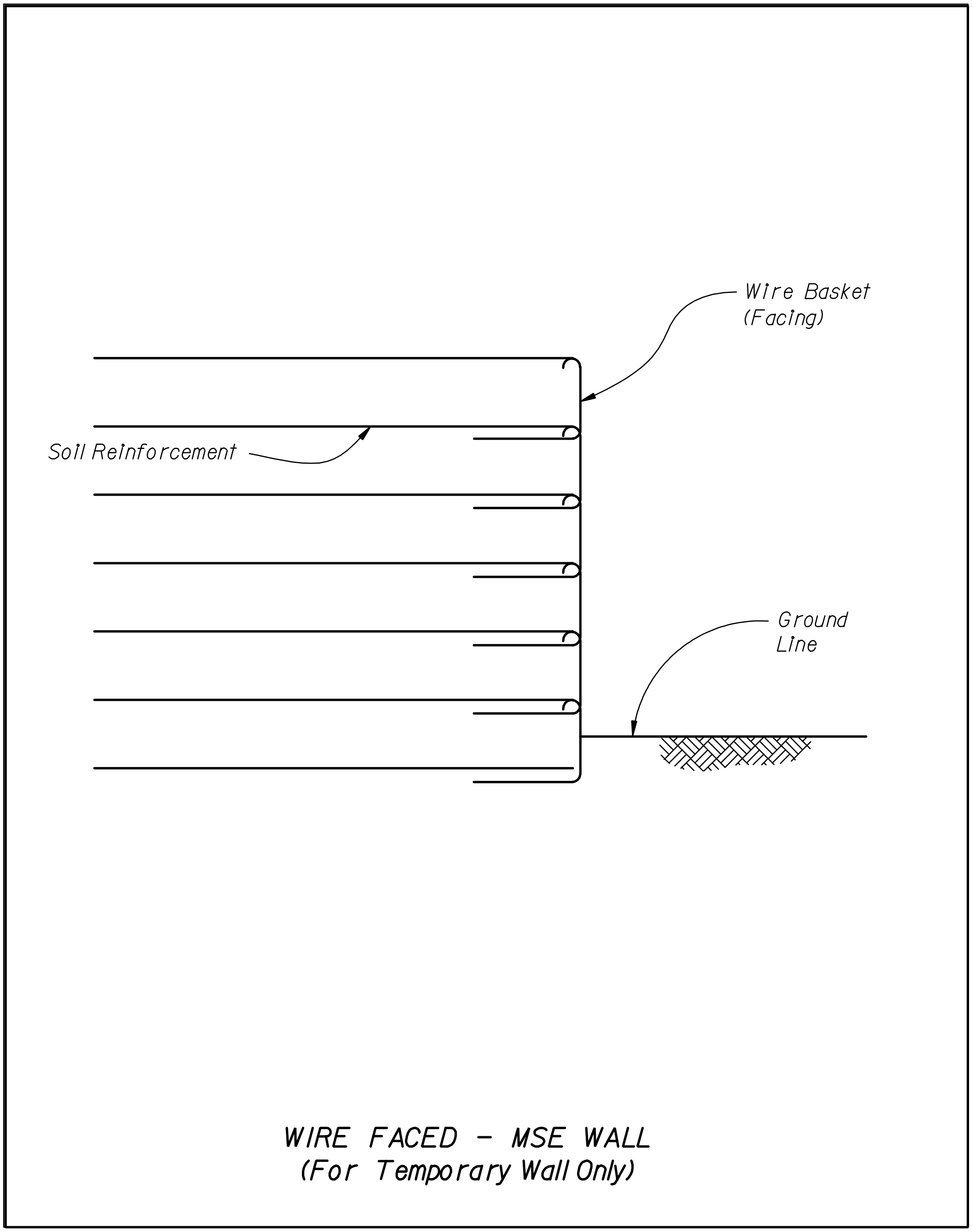

Wire Basket
(Facing)
Soil Reinforcement
Ground
Line
WIRE FACED - MSE WALL
(For Temporary Wall Only)

Figure 3-11

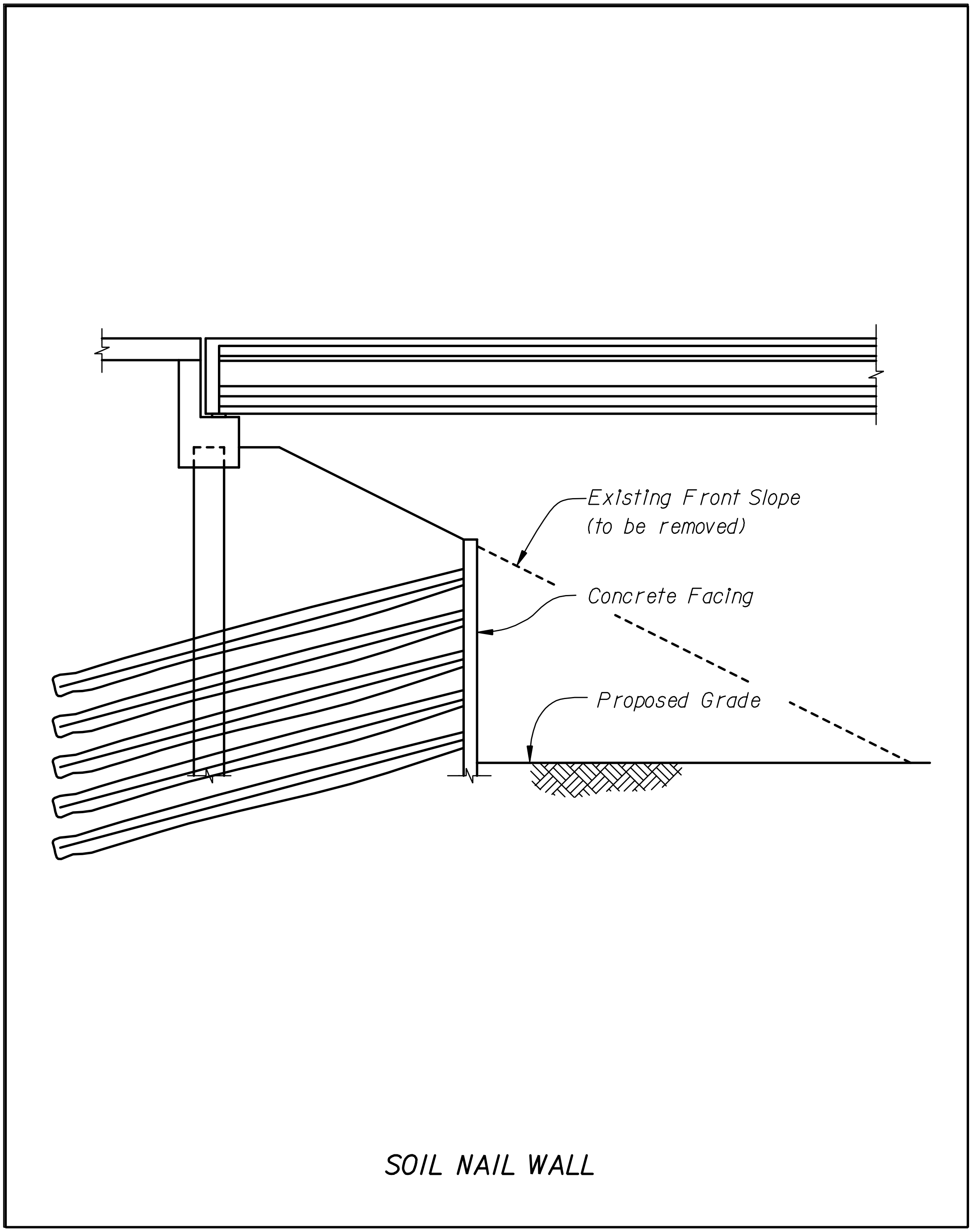

Existing Front Slope
(to be removed)
Concrete Facing
Proposed Grade
SOIL NAIL WALL

Figure 3-12

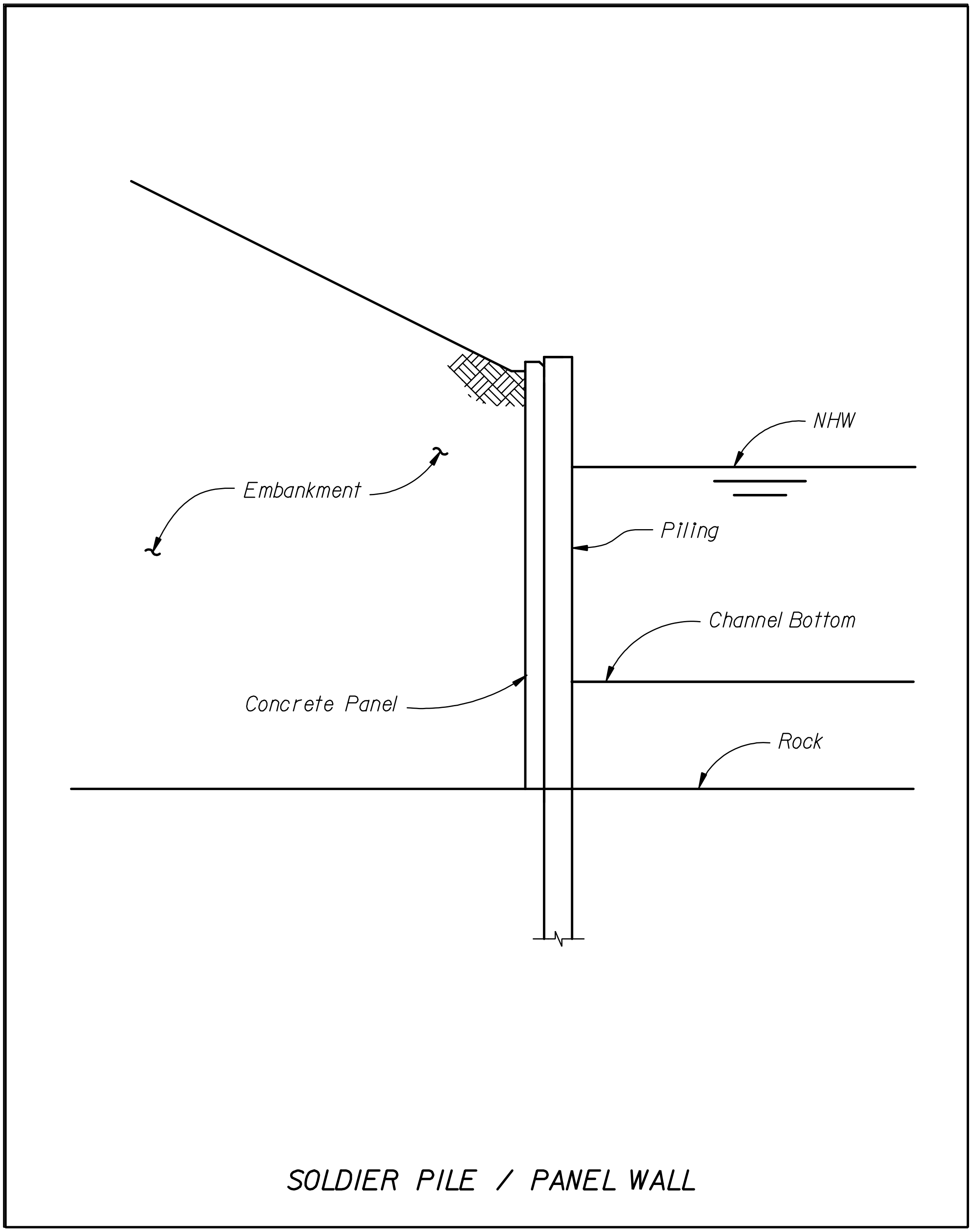

NHW
Embankment
Piling
Channel Bottom
Concrete Panel
Rock
SOLDIER PILE / PANEL WALL

Figure 3-13

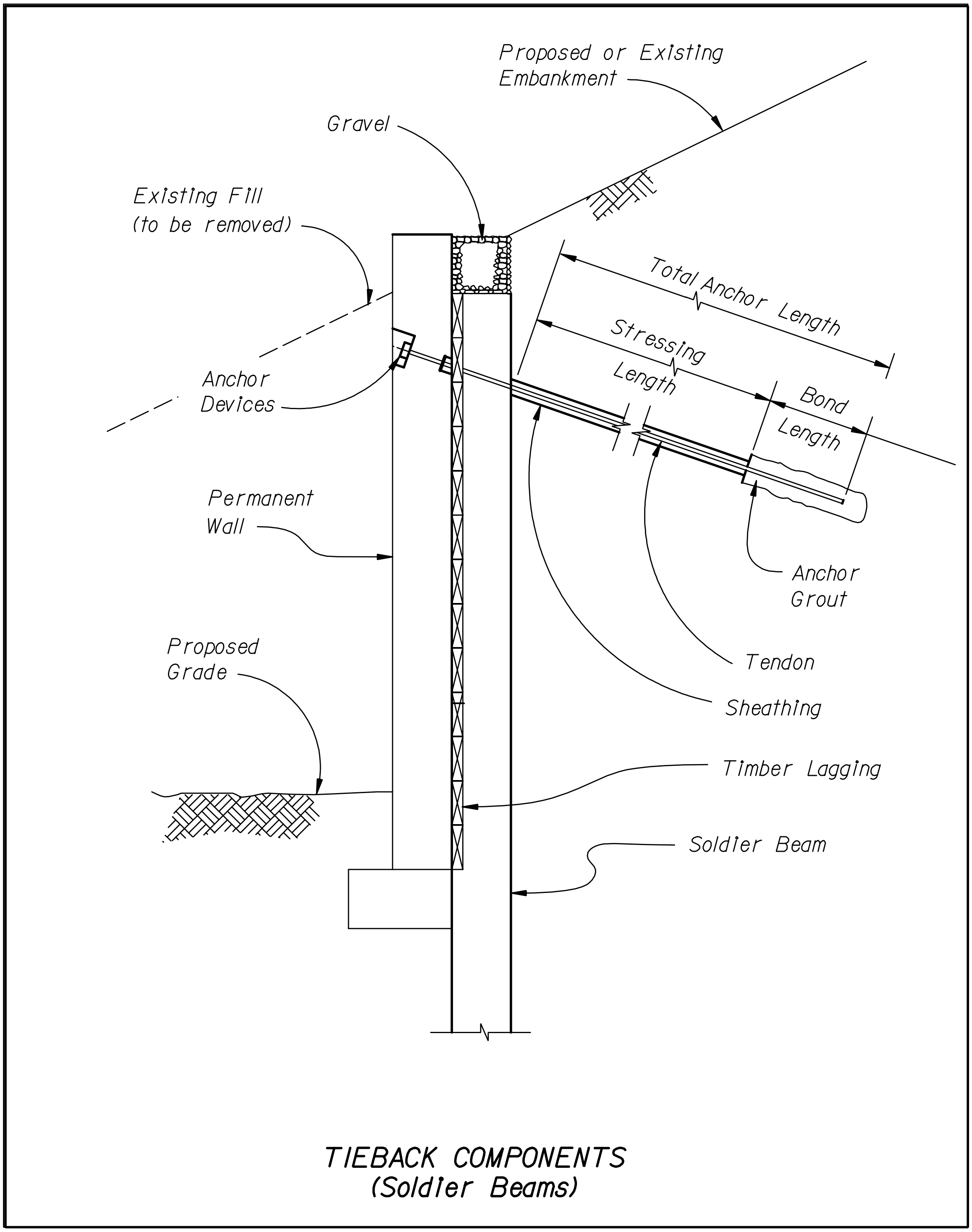

Proposed or Existing
Embankment
Gravel
Existing Fill
(to be removed)
Total Anchor Length
Stressing
Length
Bond
Length
Anchor
Devices
Permanent
Wall
Anchor
Grout
Tendon
Sheathing
Proposed
Grade
Timber Lagging
Soldier Beam
TIEBACK COMPONENTS
(Soldier Beams)

Figure 3-14

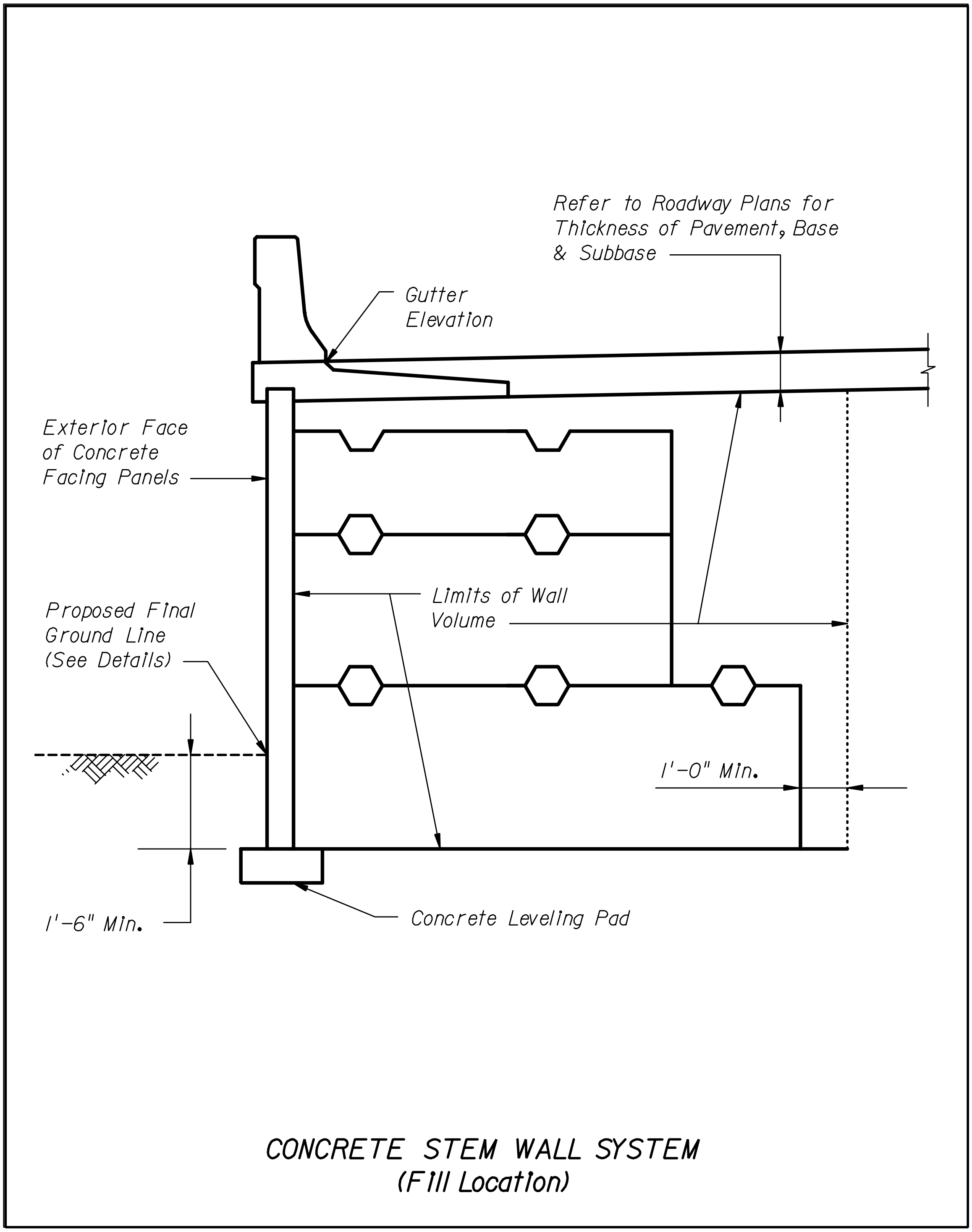
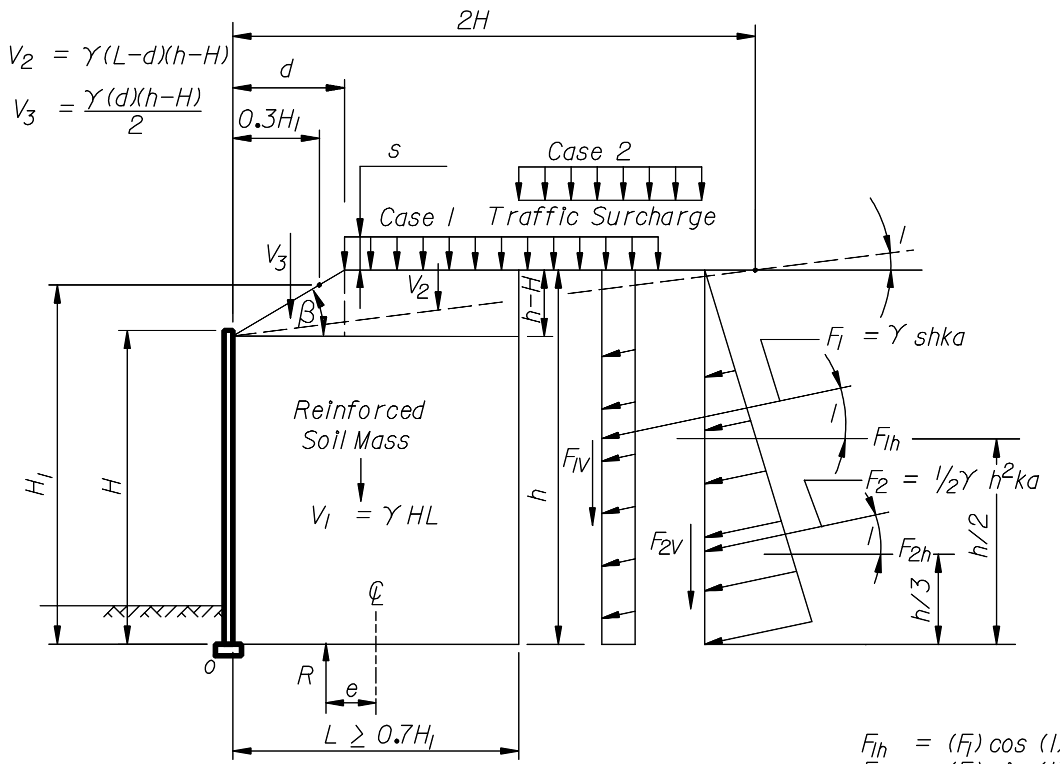

Refer to Roadway Plans for
Thickness of Pavement, Base
& Subbase
Gutter
Elevation
Exterior Face
of Concrete
Facing Panels
Limits of Wall
Volume
Proposed Final
Ground Line
(See Details)
1'-0" Min.
1'-6" Min.
Concrete Leveling Pad
CONCRETE STEM WALL SYSTEM
(Fill Location)

Figure 3-15

$V_2 = \gamma(L-d)(h-H)$

$V_3 = \dfrac{\gamma(d)(h-H)}{2}$



$F_{1h} = (F_1)\cos(I)$
$F_{1V} = (F_1)\sin(I)$
$F_{2h} = (F_2)\cos(I)$
$F_{2V} = (F_2)\sin(I)$

Case 1 used for maximum bearing pressure calculations.
Case 2 used for external stability and bond calculations.

$$H_1 = H + \frac{(\tan\beta)(0.3H)}{(1-0.3\tan\beta)} \quad \text{(Mechanical Height)}$$

K_a For Random Fill: $K_a = \cos(I)\left[\dfrac{\cos(I) - \sqrt{\cos^2(I) - \cos^2\phi}}{\cos(I) + \sqrt{\cos^2(I) - \cos^2\phi}}\right]$

Safety Factor Against Overturning (Moments About Point O):

$$S.F.(O) = \frac{\sum \text{Moments Resisting } (Mr)}{\sum \text{Moments Overturning } (Mo)} = \frac{V_1(L/2) + V_2\left(\frac{L-d}{2} + d\right) + V_3\left(\frac{2d}{3}\right) + F_{1V}(L) + F_{2V}(L)}{F_{1h}\ (h/2) + F_{2h}\ (h/3)} \geq 2.0$$

Safety Factor Against Sliding:

$$S.F.(S) = \frac{\sum \text{Horizontal Resisting Force } (s)}{\sum \text{Horizontal Driving Force } (s)} = \frac{(V_1 + V_2 + V_3 + F_{1V} + F_{2V})\tan\phi}{F_{1h} + F_{2h}} \geq 1.5$$

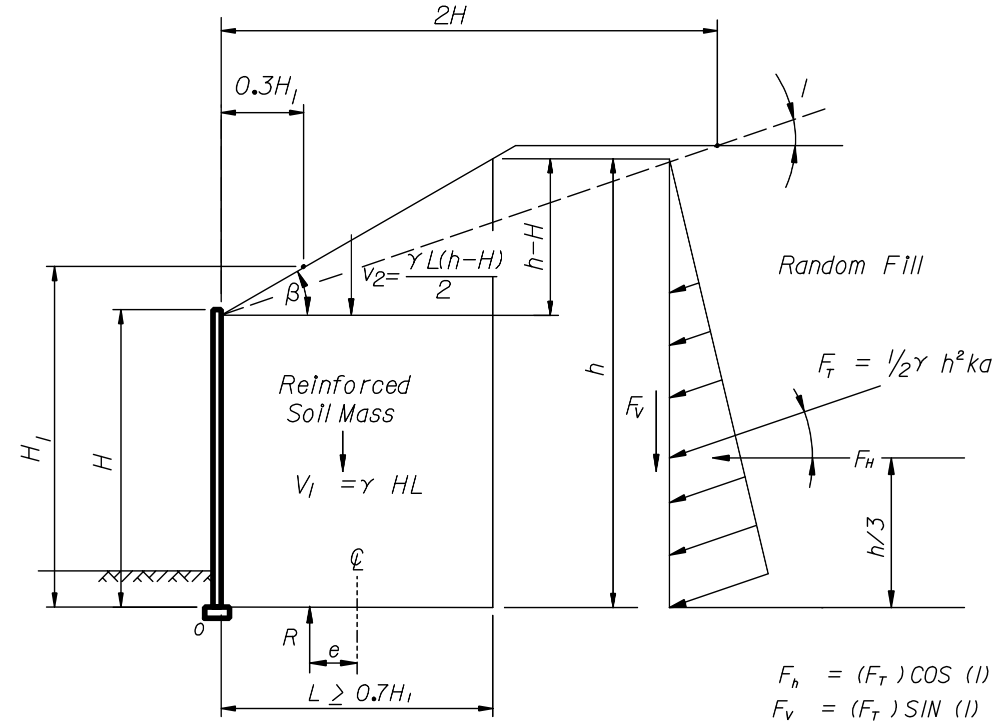
ϕ = Friction Angle of Back fill or Foundation, Whichever is lowest.

$$e = \frac{L}{2} - \frac{Mr - Mo}{R} \leq \frac{L}{6} \qquad \sigma_V = \frac{R}{L-2e}$$

Where: e = Eccentricity R = Resultant of Vertical forces $(V_1 + V_2 + V_3 + F_{1V} + F_{2V})$

BROKEN BACK BACK FILL CASE WITH TRAFFIC SURCHARGE

Figure 3-16



$F_h = (F_T)\,COS\ (I)$

$F_V = (F_T)\,SIN\ (I)$

For Infinite Slope $I = \beta$

$$H_1 = H + \frac{(Tan\beta)\,0.3H}{(1-0.3\ Tan\ \beta)} \quad \text{(Mechanical Height)}$$

K_a For Random Fill: $K_a = COS\ (I)\left[\dfrac{COS\ (I) - \sqrt{COS^2 I - COS^2\phi}}{COS\ (I) + \sqrt{COS^2 I - COS^2\phi}}\right]$

SAFETY FACTOR AGAINST OVERTURNING (MOMENTS ABOUT POINT O):

$$S.F.(O) = \frac{\Sigma \text{ Moments Resisting (Mr)}}{\Sigma \text{ Moments Overturning (Mo)}} = \frac{V_1(L/2) + V\ (2L/3) + F_V\ (L)}{F_H\ (h/3)} \geq 2.0$$

SAFETY FACTOR AGAINST SLIDING:

$$S.F.(S) = \frac{\Sigma \text{ Horizontal Resisting Force (s)}}{\Sigma \text{ Horizontal Driving Force (s)}} = \frac{R\ Tan\ \phi}{F_H} \geq 1.5$$

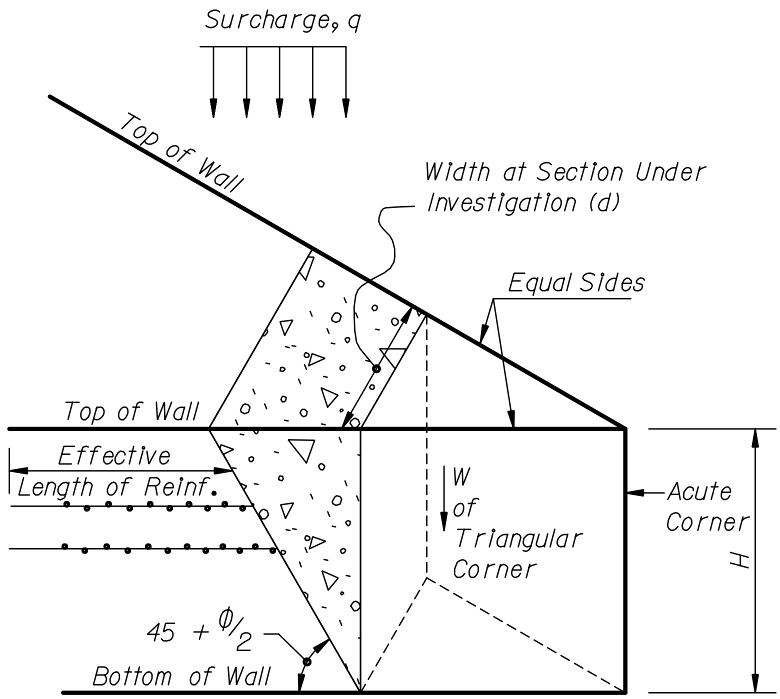
ϕ = Friction Angle of Back fill or Foundation, Whichever is lowest.

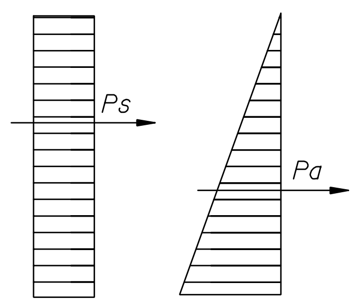
$$e = \frac{L}{2} - \frac{Mr-Mo}{R} \leq \frac{L}{6} \qquad \sigma v = \frac{R}{L-2e}$$

Where: e = Eccentricity   R = Resultant of Vertical forces $V_1 + V_2 + F_V$

BROKEN BACK BACK FILL CASE WITHOUT TRAFFIC SURCHARGE

Figure 3-17





$$Pa = \tfrac{1}{2}\gamma H^2\, Ka \begin{bmatrix}\text{Width at Section} \\ \text{Under Investigation}\end{bmatrix}$$

$$Ps = q\ H\ Ka \begin{bmatrix}\text{Width at Section} \\ \text{Under Investigation}\end{bmatrix}$$

$$\text{F.S. Sliding (Acute Corner)} = \frac{W \times \tan \phi + R}{Pa + Ps} \geq 1.5 \text{ Minimum}$$

R = Sum of the available resistance of reinforcements beyond failure plane.

Note : Must Analyze acute corner at each Column of Panel Joints until no reinforcement beyond Rankine failure surface is required for FS > 1.5 with respect to sliding.

DESIGN CRITERIA FOR ACUTE CORNERS

Figure 3-18

CHAPTER 4
SUPERSTRUCTURE - CONCRETE

4.1 General

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

This Chapter covers the design of prestressed, pretensioned concrete components such as strand transfer and development and shear design.

When using Florida limerock coarse aggregate, use 90% of the value calculated for the Modulus of Elasticity in ***LRFD*** 5.4. For Florida limerock, Wc is typically taken as 0.145 KCF.

4.1.1 Concrete Cover

See Table 1.1 Minimum Concrete Cover in SDG_1.3_Concrete_and_Environment.

4.1.2 Reinforcing Steel [5.4.3]

A. Specify ASTM A615, Grade 60 reinforcing steel for concrete design.

B. Do not specify epoxy coated reinforcing steel for any FDOT project.

4.1.3 Girder Transportation

Coordinate the transportation of heavy and/or long girders with the Department's Permit Office during the design phase of the project.

4.2 Concrete Deck Slabs [5.13.1][9.7]

4.2.1 Bridge Length Definitions for Deck Thickness and Finishing Req's.

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

A. Short Bridges: Bridge structures less than or equal to 300 feet in PGL length.
B. Long Bridges: Bridge structures more than 300 feet in PGL length.

4.2.2 Deck Thickness Determination

For new construction of “Long Bridges” other than inverted-T Beam bridge superstructures, the minimum thickness of bridge decks cast-in-place (CIP) on beams or girders is 8½ inches. The 8½ inch deck thickness includes a one-half inch additional, sacrificial thickness to be included in the dead load of the deck slab but omitted from its section properties.

For new construction of “Short Bridges” other than Inverted-T Beam bridge superstructures, the minimum thickness of bridge decks cast-in-place (CIP) on beams or girders is 8 inches minimum.

The cast-in-place bridge deck thickness for Inverted-T Beam bridge superstructures with Inverted-T Beams spaced on two foot centers is 6 ½ inches and 6 inches for bridges meeting the definition of Long and Short Bridges, respectively.

For “Major Widenings,” (see criteria in Chapter 7) the thickness of CIP bridge decks on beams or girders is 8 inches. However, whenever a Major Widening is selected by the Department to meet profilograph requirements, a minimum deck thickness of 8½ inches to meet the requirements and design methodology for new construction of the preceding paragraph, must be used.

The thickness of CIP bridge decks on beams or girders for minor widenings will be determined on an individual basis but generally will match the thickness of the adjoining existing deck.

The thickness of all other CIP or precast concrete bridge decks is based upon the reinforcing cover requirements of Chapter 1, Table 1.1.

Establish bearing elevations by deducting the determined thickness before milling, from the Finish Grade Elevations required by the Contract Drawings.

4.2.3 Grooving Bridge Decks

For new construction utilizing C-I-P bridge deck (floors) that will not be surfaced with asphaltic concrete, include the following item in the Summary of Pay Items:

Item No. 400-7 - Bridge Floor Grooving… ....……………Sq. Yards

Quantity Determination: Determine the quantity of bridge floor grooving in accordance with the provisions of Article 400-22.4 of the “Specifications.”

4.2.4 Deck Slab Design [9.7.2][9.7.3]

A. Empirical Design Method

For Category 1 structures meeting the criteria in ***LRFD*** **[9.7.2.4]**, design deck slabs by the Empirical Design method of ***LRFD*** **[9.7.2]**.

In lieu of the reinforcing requirements of ***LRFD* [9.7.2.5]**, No. 5 bars at 12” centers must be used in both directions in both the top and bottom layers. Two additional No. 5 bars must be placed between the primary transverse top slab bars (4” nominal spacing) in the slab overhangs to meet the TL-4 loading requirements for the FDOT standard barriers. One of the additional bars must be extended to the mid-point between the exterior beam and the first interior beam; the second additional bars must be extended 3 feet beyond this mid-point. The maximum deck overhang must not exceed 6 feet.

B. Traditional Design Method

For all Category 2 Structures and for Category 1 Structures that do not meet the requirements of ***LRFD*** [**9.7.2.4]**, design deck slabs in accordance with the Traditional Design method of ***LRFD*** [9.7.3].

For the deck overhang design and median barriers, the following minimum transverse top slab reinforcing may be provided (without further analysis) where the indicated minimum slab depths are provided and the total deck overhang is 6 feet or less. However, for 8" thick decks with eight foot sound wall traffic railings the deck overhang is limited to 1’-6". The extra slab depth for deck grinding is not included.

Traffic Railing Barrier (Test Level)	**Slab Depth**	**As/ft (sq in)**
32” F-Shape (TL-4)	8"	0.80
32" Vertical Face (TL-4)	8"	0.80
32" F-Shape Median (TL-4)	8"	0.40 **
8'-0" Sound wall (TL-4)	8"	0.93***
8'-0" Sound wall (TL-4)	10"	0.66***
42" F-Shape (TL-5)	10"	0.75
42” Vertical Face (TL-5)	8” (with 6” sidewalk)	0.40*(0.40)

* Minimum reinforcing based on the 42” vertical face traffic railing mounted on a 6” thick sidewalk above an 8” deck with 2” cover to the top reinforcing in both the deck and sidewalk. Specify No. 4 Bars at 6” spacing placed transversely in the top of the raised sidewalk.

** Minimum reinforcing required in both top and bottom of slab. Less reinforcing may be provided in the bottom, provided the sum of the top and bottom reinforcing is not less than 0.80 sq.in/ft.

*** For the eight foot sound wall, the area of top slab reinforcing 6 feet each side of deck expansion joints must be increased by 30% to provide a minimum 1.21 sq in/ft for an 8" thick slab and 0.86 sq in/ft for a 10” thick slab. Evaluate the development length of this additional reinforcing and detail hooked ends for all bars when necessary.

For traffic railings located inside the exterior beam (other than median barriers), the minimum transverse reinforcing in the top of the slab may be reduced by 40% provided the bottom reinforcing is not less than the top reinforcing.

If the above reinforcing is less than or equal to twice the nominal slab reinforcing, the extra reinforcing must be cut-off 12 inches beyond the midpoint between the two exterior beams. If the above reinforcing is greater than twice the nominal slab reinforcing, then half of the extra reinforcing or up to 1/3 the total reinforcing must be cut-off midway between the two exterior beams. The remaining extra reinforcing must be cut-off at 3/4 of the two exterior beam spacing, but not closer than 2 feet from the first cut-off.

4.2.5 Traffic Railing Design Requirements

In lieu of the Traditional Design Method shown above, the following design values may be used to design the top transverse slab reinforcing, for the types listed:

Traffic Railing Type (Test Level)	Mc	Tu	Ld
32” F-Shape (TL-4)	15.7	7.1	7.67
32" Vertical Face (TL-4)	16.9	7.1	7.67
32" Median (TL-4)	15.3	3.5	7.67
8'-0" Sound wall (TL-4)	20.1 ***	***5.9	21.00
42" F-Shape (TL-5)	20.6	9.0	13.75
42” Vertical Face (TL-5)	25.8	10.6	12.50

*** For the 8’-0” Sound wall, increase the ultimate slab moment and tensile force by 30% for a distance of 6 feet each side of all deck expansion joints, except on approach slabs.

Mc (kip-ft/ft) - Ultimate slab moment from traffic railing impact. (The ultimate traffic railing and slab dead load moment at the traffic railing face (gutter line) must be added to Mc.)

Tu (kip/ft) - Ultimate tensile force to be resisted.

Ld (ft) - Distribution length along the base of the traffic railing at the gutter line near a traffic railing open joint (Lc + traffic railing height).

The following relationship must be satisfied:

$$\frac{T_u}{\varphi P_n} + \frac{M_u}{\varphi M_n} \leq 1.0$$

Where φ = 1.0 and Pn = As fy (tension steel only)

For locations inside the gutter line, these forces may be distributed over a longer length of Ld + 2D(tan 45º) feet. Where "D" equals the distance from the gutter line to the critical slab section. At open transverse deck joints, use half of the increased distribution length D(tan 45º).

For flat slab bridges the transverse moment due to the traffic railing dead load may be neglected. The area of transverse top slab reinforcing determined by analysis, for flat slab bridges with edge traffic railings must not be less than 0.30 sq in/ft within 4 feet of the gutter line for any TL-4 traffic railing or 0.40 sq in/ft within 10 feet of the gutter line for any TL-5 railing.

When more than 50% of the total transverse reinforcing must be cut off, a minimum of 2 feet must separate the cut-off locations.

For traffic railings located inside the exterior beam, or greater than 5'-0" from the edge of flat bridges, the designer may assume that only 60% of the ultimate slab moment and tensile force are transferred to the deck slab on either side of the traffic railing.

4.2.6 Reinforcing Steel over Intermediate Piers or Bents

When CIP slabs are made composite with simple span concrete beams, and are cast continuous over intermediate piers or bents, design supplemental longitudinal reinforcing in the tops of slabs. Size, space, and place reinforcing in accordance with the following criteria:

A. No. 5 Bars placed between the continuous, longitudinal reinforcing bars.
B. A minimum of 35 feet in length or 2/3 of the average span length whichever is less.
C. Placed symmetrically about the centerline of the pier or bent, with alternating bars staggered 5 feet.

4.2.7 Minimum Negative Flexure Slab Reinforcement [6.10.3.7]

Any location where the top of the slab is in tension under any combination of dead load and live load is considered a negative flexural region.

Commentary:
See Chapter 7 for additional slab reinforcing requirements.

4.2.8 Crack Control in Continuous Superstructures [5.10.8]

To minimize shrinkage cracking, develop a designated casting sequence for continuous beam or girder superstructures. The sequence should result in construction joints spaced at not more than 80 feet. Develop camber diagrams taking into consideration the casting sequence and the impact on the changing cross section characteristics of the superstructure. State on the plans that a minimum of 72 hours is required between pours and that the casting sequence may not be changed unless the Contractor's Specialty Engineer performs a new structural analysis, and new camber diagrams are calculated.

4.2.9 Structures Continuous for Live Load
In structures designed continuous for live load, design supplemental longitudinal reinforcing extended beyond the point where the slab is in tension under any load combination.

4.2.10 Concrete Decks on Continuous Steel Girders
For longitudinal reinforcing steel within the negative moment regions of continuous, composite steel girder superstructures, comply with the requirements of ***LRFD*** **[6.10.3.7]**.

In addition, design the remainder of the deck with No. 5 longitudinal steel at 12 inch spacing in the top of the slab and No. 4 Bars at 12 inch spacing in the bottom of the slab.

4.2.11 Skewed Decks [9.7.1.3]
A. Reinforcing Placement when the Slab Skew is 15 Degrees or less:
Place the transverse reinforcement parallel to the skew for the entire length of the slab.

B. Reinforcing Placement when the Slab Skew is more than 15 Degrees:
Place the required transverse reinforcement perpendicular to the centerline of span. In this case, because the typical required transverse reinforcement cannot be placed full-width in the triangular shaped portions of the ends of the slab, the required amount of longitudinal reinforcing must be doubled for a distance along the span equal to the beam spacing. In addition, three No. 5 Bars at 6” spacing, full-width, must be placed parallel to the end skew in the top of each end of the slab.

Regardless of the angle of skew, the traffic railing reinforcement cast into the slab need not be skewed.

4.2.12 Temperature and Shrinkage Reinforcement
For all cast in place decks, design temperature and shrinkage reinforcement per ***LRFD*** **[5.10.8]** except do not exceed 12” spacing and the minimum bar size is No 4.

4.2.13 Stay-in-Place Forms
For prestressed beam and steel girder superstructures, design and detail stay-in-place metal forms. State in the General Notes whether or not stay-in-place forms are permitted on the bridge.

Include in the "General Notes" for each bridge project, 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. Composite stay-in-place forms are not permitted.

A. Metal Stay-in-Place Forms:
Do not use metal stay-in-place forms in moderately aggressive or extremely aggressive environments; 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.

4.3 Prestressed, Pretensioned Components

4.3.1 Prestressed Beams

The use of ASTM A416, Grade 270, low-relaxation, straight, prestressing strands is preferred for the design of prestressed beams. However, the following requirements apply to simply supported, fully pretensioned beams, whether of straight or depressed (draped) strand profile, except where specifically noted otherwise.

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

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

Commentary:
Grouping beam designs in accordance with the priority list maximizes casting bed usage and minimizes variations in materials and stranding.

B. In analyzing stresses and/or determining the required length of debonding, stresses must be limited to the following values:
   1.) Tension at top of beam at release (straight strand only):

      Outer 15 percent of design span: $ft = 12\sqrt{f'ci}$ (psi)

      Center 70 percent of design span: $ft = 6\sqrt{f'ci}$ (psi)

   2.) Tension at top of beam at release

      (depressed strands only): $ft = 6\sqrt{f'ci}$ (psi)

C. In order to achieve uniformity and consistency in designing strand patterns, the following parameters 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 3 inches 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 must be utilized.
2.) Use “L-shaped” longitudinal bars in the webs and flanges in end zone areas.
3.) The minimum compressive concrete strength at release will be the greater of 4.0 ksi or 0.6 f’c. Higher release strengths may be used and specified when required but generally should not exceed 0.8 f’c.
4.) Design and specify prestressed beams to conform to classes and related strengths of concrete as shown in Table 4.1.
5.) The use of time-dependent, inelastic creep and shrinkage is not allowed in the design of simple span, pretensioned components either during design or construction. Prestress loss must be calculated in accordance with ***LRFD*** [5.9.5].

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

6.) Stress and camber calculations for the design of simple span, pretensioned components must be based upon the use of transformed section properties.
7.) 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, evaluate the edges of flanges of those beams to safely and adequately support the self-weight of the forms, concrete, and construction load specified in Section 400 of the ***FDOT Standard Specifications for Road and Bridge Construction***.
8.) The design thickness of the composite slab must be provided from the top of the stay-in-place metal form to the finished slab surface, and the superstructure concrete quantity will not include the concrete required to fill the form flutes.

Table 4.1 Concrete Classes and Strengths

Class of Concrete	28-Day Compressive Strength (f'c) KSI
Class III*	5.0
Class IV	5.5
Class V (special)	6.0
Class V	6.5
Class VI	8.5

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

4.3.2 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 must be based on 120-day-old beam concrete. On the build-up detail, show the age of beam concrete used for camber calculations as well as the value of camber due to prestressing minus the dead load deflection of the beam.

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

4.4 Precast, Prestressed Slab Units [5.14.4.3]

To control the maximum camber expected in the field for precast prestressed slab units built without a topping, the design camber must not exceed ¼ inch. For precast prestressed slab units built with a topping, the design camber must not exceed 1 inch.

Unless otherwise specified on the plans, the design camber must be computed for 120-day-old slab concrete. The design camber shown on the plans is the value of camber due to prestressing minus the dead load deflection of the slab unit after all prestress losses.

In order to accommodate the enhanced post-tensioning system requirement of three levels of protection for strand, transverse post-tensioned pre-stressed slab units must incorporate a double duct system. The outer duct must be cast into the slab and sized to accommodate a differential camber of 1". The inner duct must be continuous across all joints and sized based upon the number of strands or the diameter of the bar coupler. Specify that both the inner duct and the annulus between the ducts be grouted.

4.5 Florida Bulb-Tee Beams [5.14.1.2.2]

The minimum web thicknesses for Florida Bulb-Tee beams are:
A. Pretensioned Beams .... 6 ½ inches
B. Post-Tensioned Beams .... See Table 4.5

4.6 Precast Prestressed Double-T Beams

All bridge structures with precast, prestressed double-tee beams must comply with the design criteria and details provided in the ***Standard Drawings***.

4.7 Box Girders

4.7.1 General

During preliminary engineering and when determining structure configuration, give utmost consideration to accessibility and to the safety of bridge inspectors and maintenance.

Precast, pretensioned Florida U-Beams are exempt from special requirements for inspection and access.

4.7.2 Access and Maintenance

A. Height [2.5.2.6.3]
For maintenance and inspection, the minimum interior, clear height of box girders is 6 feet. Proposed heights less than 6 feet will require approval from the SDO. If structural analysis requires less than 5 feet box depth, consult the SDO and the District Structures and Facilities Engineer for a decision on the box height and access details.

B. Electrical Service
Design box girder bridges with interior lighting and electrical outlets in all boxes and detail in accordance with SDO's Standard Drawings.

C. Access
Design box girders with exterior, in-swinging, hinged, doors (min. 32"x 42" or 36" dia). Equip doors with a lock and hasp. Require that all locks be keyed alike. Indicate on plans that access openings are not to be used for utilities or other attachments. Analyze access opening sizes and bottom flange locations for structural effects on the girder. Avoid access locations over traffic lanes and locations that will require extensive maintenance of traffic operations or that would otherwise impact the safety of inspectors or the traveling public.

D. Ventilation and Drain Holes
Design each box girder with minimum 2" diameter ventilation or drain holes spaced about every 50 feet or as needed to provide proper drainage. Require .25" screen on all exterior holes not covered by a door. This includes holes in webs through which drain pipes pass, ventilation holes, drain holes, and openings in diaphragms at abutments and at expansion joints.

Provide drains to prevent water (including condensation) from ponding in the vicinity of post-tensioning components. Show details on Contract Drawings. Include the following:

1.) Specify that drains may be formed using 2-inch diameter permanent plastic pipes (PVC with UV inhibitor) set flush with the top of the bottom slab.
2.) A small drip recess, ½ inch by ½ inch around bottom of pipe insert.
3.) Drains at all low points against internal barriers, blisters, etc.
4.) Drains on both sides of box, regardless of cross slope (to avoid confusion.)
5.) Show locations and details for drains taking into account bridge grade and cross-slope.
6.) Vermin guards for all drains.
7.) A note stating, "Install similar drains at all low spots made by barriers introduced to accommodate means and methods of construction, including additional blocks or blisters."

E. Other Box Sections

Provide accessibility to box sections such as precast hollow pier segments in a manner similar to that for box girders, particularly concerning the safety of bridge inspectors and maintenance personnel. During preliminary engineering and when determining structure configuration, give utmost consideration to box girder accessibility and the safety of bridge inspectors and maintenance personnel.

Due to the wide variety of shapes and sizes of hollow sections such as precast concrete pier segments, numerous site constraints and environmental conditions, each application will be considered on an individual, project-by-project basis. In all cases, contact the SDO for guidance in designing adequate inspection access and safety measures.

Precast, pretensioned Florida U-beams are exempted from special requirements for inspection and access.

4.7.3 Thermal Effects [5.14.2.3.5]

A. For detailing purposes, take the normal mean temperature from Table 6.2.
B. For Seasonal Variation (expansion/contraction), refer to Table 6.2 for temperature ranges.
C. Show temperature-setting variations for bearing and expansion joints on the bridge plans.

4.7.4 Creep and Shrinkage [5.14.2.3.6]

Calculate creep and shrinkage strains and effects in accordance with LRFD using Relative Humidity of 75%.

4.7.5 Post-tensioning Anchorages

When temporary or permanent post-tensioning anchorages are required in the top or bottom slab of box girders, design and detail interior blisters, face anchors or other SDO approved means. Block-outs that extend either to the interior or exterior surfaces of the slabs are not permitted.

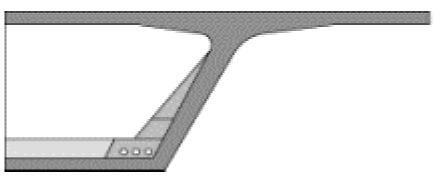
Provide continuous typical longitudinal mild reinforcing through all segment joints for Cast-in-place segmental construction.

Design and detail so that all future post-tensioning utilizes external tendons (bars or strands) and so that any one span can be strengthened independently of adjacent spans.

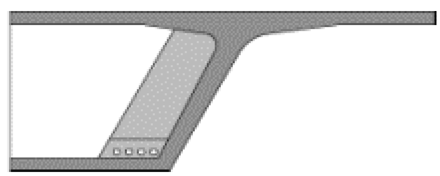
Detail anchor blisters so that tendons terminate no closer than 12 inches to a joint between segments.

Detail all interior blisters set back a minimum of 12 inches from the joint. Provide a "V"-groove around the top slab blisters to isolate the anchorage from any free water.

Transverse bottom slab ribs are not allowed. Design full height diaphragms directing the deviation forces directly into the web and slab.

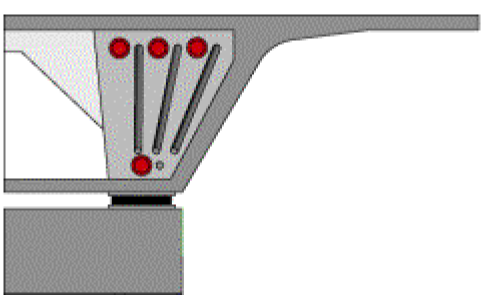


Previous Practice

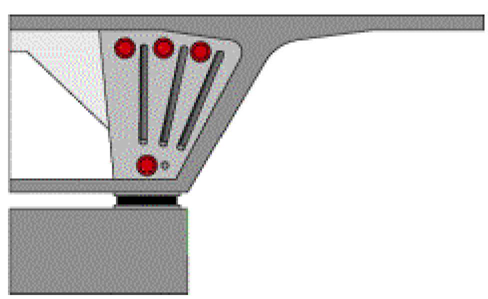


New Practice

Raised corner recesses in the top corner of pier segments at closure joints are not allowed. The typical cross section must be continued to the face of the diaphragm. Locate tendon anchorages to permit jack placement.



Previous Practice



New Practice

4.8 Post-Tensioned Bridges [5.14.2]

This section applies to both concrete boxes and post-tensioned I-girders unless otherwise noted.

4.8.1 Erection Schedule and Construction System

Include in the design documents, in outlined, schematic form, a typical erection schedule and anticipated construction system. Clearly state in the plans, the assumed erection loads, along with times of application and removal of each of the erection loads.

4.8.2 Construction Data Elevation and Camber Curve for Box Girders

A. General:

Base Construction Data Elevations on the vertical and horizontal highway geometry. Calculate the Camber Curve based on the assumed erection loads used in the design and the assumed construction sequence.

B. Construction Data Elevations:

Show construction data elevations in 3D space with “x”, “y”, and “z” coordinates. Locate the data points at the centerline of the box and over each web of the box.

C. Camber Curve:

Provide Camber Curve data 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.

4.8.3 Final Computer Run for Box Girder Structures

Prove the final design by a full longitudinal analysis taking into account the assumed construction process and final long-term service condition, including all time related effects. Perform the analysis using a SDO approved computer program.

4.8.4 Integrated Drawings

Show congested areas of post-tensioned concrete structures on integrated drawings with an assumed post-tensioning system. Such areas include 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.

For curved structures, evaluate and accommodate possible conflicts between webs and external tendons. Check for conflicts between future post-tensioning tendons and permanent tendons.

Select the assumed post-tensioning system, embedded items, etc. in a manner that will accommodate competitive systems using standard anchorage sizes of 4-0.6” dia, 7-0.6” dia, 12-0.6” dia, 19-0.6 dia and 27-0.6” dia. Integrated drawings utilizing the assumed system must be detailed 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.

4.8.5 Prestress

Design the structure for initial and final prestress forces.

A. Secondary Effects:

1.) Prestress forces in continuous structures, which can result in substantial secondary effects

2.) In curved structures, draped web tendons will produce transverse bending stresses in the box cross-section due to the lateral component of forces arising from plan curvature.

B. Deck Slab:

Detail all box girder deck slabs to be transversely post-tensioned. Where draped post-tensioning is used in deck slabs, consideration must be given to the final location of the center of gravity of the prestressing steel within the duct. Reduce critical eccentricities over the webs and at the centerline of box by ¼ inch from theoretical to account for construction tolerances.

C. Tendon Geometry:

When coordinating design calculations with detail drawings, account for the fact that the center of gravity of the duct and the center of gravity of the prestressing steel are not necessarily coincidental.

D. Required Prestress:

On the drawings, show prestress force values for tendon ends at anchorages.

E. Internal/External Tendons:

External tendons must remain external to the section without entering the top or bottom slab.

F. Strand Couplers:

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

G. Post-tensioned Beams:

For pretensioned beams made continuous by field-applied post-tensioning, design the pretensioning to meet the following minimum criteria:

1.) The pretensioning acting alone must comply with the minimum steel requirements of ***LRFD*** [5.7.3.3.2].

2.) The pretensioning must be capable of resisting all loads applied prior to post-tensioning, including a superimposed dead load equal to 50% of the uniform weight of the beam, without exceeding the stress limitation for pretensioned concrete construction.

3.) The initial mid span camber at release, including the effect of the dead load of the beam, is at least ½ inch. In computing the initial camber, the value of the modulus of elasticity of the concrete, Ec, must be in accordance with Section 4.1 for the minimum required strength of concrete at release of the pretensioning force. The pretensioning force in the strands must be reduced by losses due to elastic shortening and steel relaxation.

4.) For post-tensioning tendon anchorage zones as well as beam segments in which ducts deviate both horizontally and vertically, show all post-tensioning hardware and reinforcing steel on integrated drawings.

5.) For pretensioned beams made continuous by field-applied post-tensioning, design the pretensioning to satisfy the following minimum criteria:

a.) The pretensioning acting alone must comply with the minimum steel requirements of ***LRFD*** [5.7.3.3.2].

b.) The pretensioning must be capable of resisting all loads applied prior to post-tensioning, including a superimposed dead load equal to 50% of the uniform weight of the beam, without exceeding the stress limitation for pretensioned concrete construction.

4.8.6 Material

A. Concrete:

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

1.) Precast superstructure (including CIP joints).. 5.5 ksi
2.) Precast pier stems.. 5.5 ksi
3.) Post-tensioned I-girders .... 5.5 ksi

B. Post-Tensioning Steel:

1.) Strand..ASTM A416, Grade 270, low relaxation.
2.) Parallel wires .... ASTM A421, Grade 240.
3.) Bars .... .ASTM A722, Grade 150.

C. Post-Tensioning Anchor set (to be verified during construction):

1.) Strand.. ½ inch
2.) Parallel wires .... ½ inch
3.) Bars 0

4.8.7 Expansion Joints

A. Design:

Design expansion joints for the full range of movement anticipated due to creep, shrinkage, elastic shortening, and temperature effects and the requirements of Chapter 6. Do not design superstructures utilizing expansion joints within the span (i.e. ¼ point hinges).

B. Settings:

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

1.) Allowance for opening movements based on the total anticipated movement resulting from the combined effects of creep, shrinkage, and temperature rise and fall. For box girder structures, compute creep and shrinkage from the time the expansion joints are installed through day 4000. Base temperature rise and fall on 120% of the maximum value given in Table 6.2. Place a note on the plans stating that all expansion joints be installed after superstructure segment erection and deck profiling is completed for the entire bridge.

2.) To account for the larger amount of opening movement, expansion devices should be set precompressed to the maximum extent possible. In calculations, allow for an assumed setting temperature of 85 degrees F. Provide a table on the plans giving precompression settings according to the prevailing conditions. Size expansion devices and set to remain in compression through the full range of design temperature from their initial installation until a time of 4,000 days.

3.) Provide a table of setting adjustments to account for temperature variation at installation. Indicate the ambient air temperature at time of installation, and note that adjustments must be calculated for the difference between the ambient air temperature and the mean temperature given in Table 6.2.

C. Armoring:

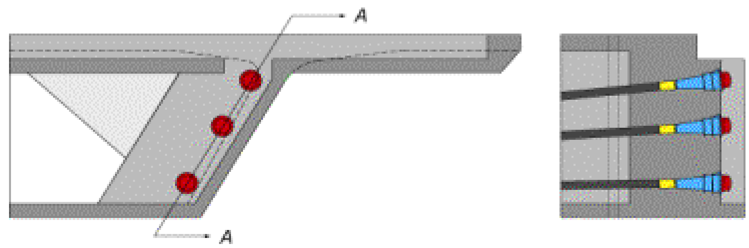
Design and detail concrete corners under expansion joint devices with adequate steel armoring to prevent spalling or other damage under traffic. The armor should be minimum 4 x 4 x 5/8 galvanized angles anchored to the concrete with welded studs or similar devices. Specify that horizontal concrete surfaces supporting the expansion joint device and running flush with the armoring have a finish acceptable for the device. Detail armor with adequate vent holes to assure proper filling and compaction of the concrete under the armor.

D. Details For Post-Tensioned Bridges:

At expansion joints, provide a recess and continuous expansion joint device seat to receive the assembly, anchor bolts, and frames of the expansion joint, i.e. a finger or modular type joint. In the past, block-outs have been made in such seats to provide access for stressing jacks to the upper longitudinal tendon anchors set as high as possible in the anchor block. Lower the upper tendon anchors and re-arrange the anchor layout as necessary to provide access for the stressing jacks.

At all expansion joints, protect anchors from dripping water by means of skirts, baffles, v-grooves, or drip flanges. Ensure that drip flanges are of adequate size and shape to maintain structural integrity during form removal and erection.

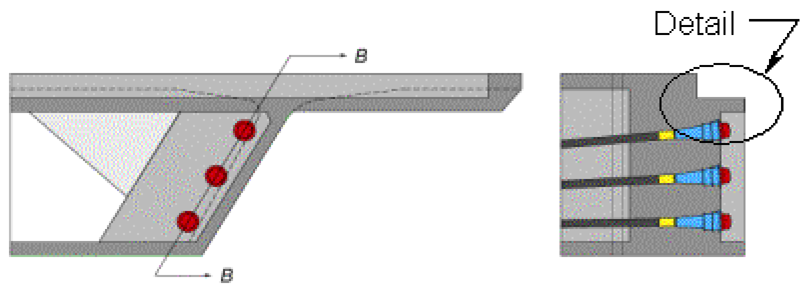
Previous Practice



EJ Pier Segment End View

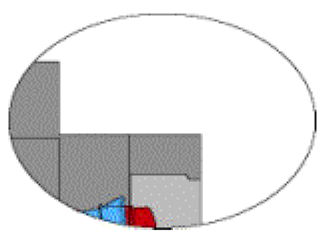
Section A-A

New Practice

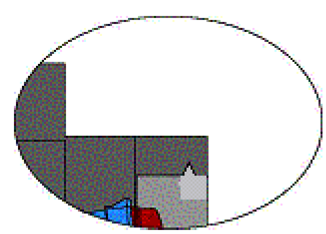


EJ Pier Segment End View

Section B-B



Detail A – Drip



Detail A – "V"

4.8.8 Ducts and Tendons

Show offset dimensions to post-tensioning duct trajectories from fixed surfaces or clearly defined reference lines at intervals not exceeding 5 feet. When the radius of curvature of a duct exceeds one-half degree per foot, show offsets at intervals not exceeding 30 inches. In regions of tight reverse curvature of short tendon sections, show offsets 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 must 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, consider the arrangement, installation, stressing sequence, and grouting in order to avoid potential problems with cross grouting of ducts.

Detail post-tensioned bulb-tee beams to utilize round ducts only.

Table 4.2 Minimum Center-to-Center Duct Spacing:

Post Tensioned Bridge Type	*Minimum Center To Center Longitudinal Duct Spacing
Precast Segmental Balanced Cantilever Cast-In-Place Balanced Cantilever	8”, 2 times outer duct diameter, or outer duct diameter plus 4 ½ inches whichever is greater.
Spliced I-Girder Bridges	4 inches, outer duct diameter plus 1.5 times maximum aggregate size, or outer duct diameter plus 2 inches whichever is greater.
C.I.P. Voided Slab Bridges C.I.P. Multi-Cell Bridges	When all ducts are in a vertical plane – 4 inches, outer duct diameter plus 1.5 times maximum aggregate size, or outer duct diameter plus 2 inches whichever is greater. **For two or more ducts set side-by-side - outer duct diameter plus 3 inches.

* - Bundled tendons are not allowed.
** - The 3-inch measurement must be measured in a horizontal plane.

Size ducts for all post-tensioning bars ½ inch larger than the diameter of the bar coupler.

Internal post-tensioning ducts must be positively sealed with duct couplers at all segment joints to eliminate the possibility of contamination. Design and detail internal tendons to be perpendicular to all segment joints and all closure pours to be a minimum of 18 inches.

To allow room for the installation of duct couplers, detail all external tendons to provide a 1 ½ inch clearance between the duct surface and the face of the concrete.

Diablos are not allowed.

Detail both internal and external tendons to be placed in steel pipes when the radius is less than shown in the table below. Detail all deviation saddles to contain steel pipe regardless of the tendon radius.

Table 4.3 Minimum Tendon Radiuses

Tendon Size	Minimum Radius (ft)
19-0.5” dia, 12-0.6” dia	8’
31-0.5” dia, 19-0.6” dia	10’
55-0.5” dia, 37-0.6” dia	13’

All balanced cantilever bridges must utilize a minimum of 4 positive moment external draped continuity tendons (2 per web) that extend to adjacent pier diaphragms.

Table 4.4 Min. Tendons Required for Critical Post-tensioned Sections:

Post Tensioned Bridge Element	Minimum Number of Tendons
Mid Span Closure Pour C.I.P. and Precast Balanced Cantilever Bridges	Bottom slab – 2 per web Top slab – 1 per web (4- 0.6 inch diameter minimum)
Span by Span Segmental Bridges	4 tendons per web
C.I.P. Multi-Cell Bridges	3 tendons per web
Spliced I-Girder Bridges	*3 per girder
Unit End Spans C.I.P. and Precast Balanced Cantilever Bridges	3 tendons per web
Diaphragms - Transverse Post-Tensioning (When Vertical Post-Tensioning is Required)	6 if strength is provided by P.T. only 4 if strength is provided by combination of P.T. and mild reinforcing
Diaphragms – When Vertical Post-Tensioning is Required	4 **
Segments – (When Vertical Post-Tensioning is Required)	2 per web

* 3 girders minimum per bridge.
** 2 per additional cell

4.8.9 Minimum Dimensions

Table 4.5 Dimensions for sections containing post-tensioning tendons:

Post Tensioned Bridge Element	Minimum Thickness
Webs; I-Girder Bridges	8 inches or outer duct plus 5 inches whichever is greater.
Regions of Slabs without longitudinal tendons	8 inches, or as required to accommodate grinding, concrete covers, transverse and longitudinal P.T. ducts and top and bottom mild reinforcing mats, with allowance for construction tolerances whichever is greater.
Regions of slabs containing longitudinal internal tendons	9 inches, or as required to accommodate grinding, concrete covers, transverse and longitudinal P.T. ducts and top and bottom mild reinforcing mats, with allowance for construction tolerances whichever is greater.
Clear Distance Between Circular Voids - C.I.P. Voided Slab Bridges	Outer duct diameter plus 5 inches, or outer duct diameter plus vertical reinforcing plus concrete cover whichever is greater.
Segment Pier Diaphragms containing external post-tensioning	*6'-0"
Webs of C.I.P. boxes with internal tendons	For single column of ducts –12 inches, or 3 times outer duct diameter whichever is greater. **For two or more ducts set side by side – Web thickness must be sufficient to accommodate concrete covers, longitudinal P.T. ducts, 3 inch min. spacing between ducts, vertical reinforcing, with allowance for construction tolerances.

*Pier segment halves with C.I.P. closure joint are acceptable.
** The 3 inch measurement must be measured in a horizontal plane.

4.8.10 Corrosion Protection

Detail all post-tensioned bridges consistent with the Specifications and Standards and include the following corrosion protection strategies:

A. Enhanced Post-tensioned systems.
B. Fully grouted tendons.
C. Multi-level anchor protection.
D. Watertight bridges.
E. Multiple tendon paths.

Enhanced post-tensioning systems require three levels of protection for strand and four levels for anchorages. Note: Deck overlays are not considered a level of protection for strands or anchorages.

A. Three levels of Strand Protection

1.) Within the Segment or Concrete Element
  a.) Internal Tendons.
    i.) Concrete cover.
    ii.) Plastic duct.
    iii.) Complete filling of the duct with approved grout.
  b.) External Tendons
    i.) Hollow box structure itself.
    ii.) Plastic duct.
    iii.) Complete filling of the duct with approved grout.
2.) At the segment face or construction joint (Internal and External Tendons)
  a.) Epoxy seal (pre-cast construction) or wet cast joint (cast-in-place construction.)
  b.) Continuity of the plastic duct.
  c.) Complete filling of the duct with approved grout.

B. Four Levels of Anchorage Protection
1.) Anchorages located on interior surfaces (interior diaphragms, etc.)
  a.) Grout.
  b.) Permanent grout cap.
  c.) Elastomeric seal coat.
  d.) Concrete box structure.
2.) Anchorages located on exterior surfaces (Pier Caps, expansion joints, diaphragms etc.)
  a.) Grout.
  b.) Permanent grout cap.
  c.) Encapsulating pour-back.
  Seal coat (Elastomeric/Methyl Methacrylate on riding surface.)

4.8.11 Epoxy Joining of Segments

All joints between precast segmental bridge segments must contain epoxy on both faces. This requirement applies to substructure and superstructure precast units. Dry segment joints are not allowed.

4.8.12 Post-Tensioned I-Girders

Individual deck block-outs for second stage post-tensioning access for I-Girder bridges are not allowed. Detail the deck in this region to be poured full width and extending one foot beyond the last anchorage upon completion of post-tensioning operations.

4.8.13 Creep and Shrinkage for Post-tensioned I-Girder Structures

A. Calculate creep and shrinkage strains and effects in accordance with ***LRFD*** using Relative Humidity of 75%.

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

1.) Ultimate Creep Coefficient..2.0
2.) Ultimate Shrinkage Strain...0.0004
3.) Beam Age when Deck is Cast ....120 Days

C. These creep and shrinkage values include corrections for slump, humidity, and volume/surface ratio; and must be used for both the beam and the deck slab.

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

CHAPTER 5
SUPERSTRUCTURE - STEEL

5.1 General

This Chapter includes guidance for the design and detailing of steel bridges. Design steel bridge components in accordance with ***LRFD*** and the requirements of this Chapter.

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

See Chapter 7 for concrete decks of steel bridges.

For a bridge with steel members curved for part or all of its length, design in accordance with the current AASHTO ***Guide Specifications for Horizontally Curved Highway Bridges***, the AASHTO ***Standard Specifications for Highway Bridges, and Interim Specifications.***

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

5.1.1 Girder Transportation
Coordinate the transportation of heavy and/or long girders with the Department's Permit Office during the design phase of the project.

5.2 Dead Load Camber [6.7.2]

Design the structure, including the slab, with a sequence for placing the concrete deck. Show the placement sequence on the plans. Develop camber diagrams to account for the deck placing sequence. Analyze the superstructure geometry and properties and use the appropriate level of analysis to determine deflections and camber.

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

5.3 Structural Steel [6.4.1]

Specify that all structural steel conform to AASHTO M270 (ASTM A709), Grade 36, 50, or 50W or ASTM A709, Grade HPS 70W. The Department for use in special cases may approve grade 100 or 100W. Show the AASHTO M270 or ASTM A709 designation on the contract documents. Do not specify painting of weathering steel unless approved by the SDO.

Miscellaneous hardware, including shapes, plates, and threaded bar stock, may conform to ASTM A709, Grade 36.

Commentary:
AASHTO M270 and ASTM A709 are bridge steel specifications that include notch toughness and weldability requirements. These requirements may not be present in other ASTM Specification such as A36 or A588. Steels meeting bridge steel specifications are pre-qualified for use in welded bridges.

5.4 Bolts [6.4.3.1]

Design structural bolted connections as “slip-critical.” Use ASTM A325, Type 1, high-strength bolts for painted connections, and Type 3 bolts for unpainted weathering steel connections. Do not use ASTM A490 bolts unless approved by the SDO or DSDO. Non-high-strength bolts may conform to ASTM A307.

Bolt diameters of 3/4 , 7/8, 1, or 1 1/8 inch typically should be used. Larger bolts may be used with prior approval by the Department. One size bolt should be used for all structural connections on any given structure.

5.5 Minimum Steel Dimensions [6.7.3]

The minimum thickness of plate girder and box girder webs is 7/16 inch.

The minimum flange size for plate girders and top flanges of box girders is 3/4 x 12 inches.

The minimum box girder bottom flange thickness is 1/2 inches.

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

Specify flange plate widths and web plate depths in 1-inch increments.

Specify plates in accordance with the commonly available thicknesses of Table 5.1.

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

Table 5.1 Thickness Increments for Common Steel Plates

THICKNESS INCREMENT (in)	PLATE THICKNESS (in)
1/16”	3/16” to 3/8”
1/8”	3/8” to 1”
1/4”	> 1”

5.6 Box Sections

5.6.1 General

During preliminary engineering and when determining structure configuration, give utmost consideration to accessibility and to the safety of bridge inspectors and maintenance.

Precast, pretensioned Florida U-Beams are exempt from special requirements for inspection and access.

5.6.2 Height

For maintenance and inspection, the minimum interior, clear height of box girders is 6 feet. Proposed heights less than 6 feet will require approval from the SDO. If structural analysis requires less than 5 feet box depth, consult the SDO and the District Structures and Facilities Engineer for a decision on the box height and access details.

5.6.3 Electrical Service

Design box girder bridges with interior lighting and electrical outlets in all boxes and detail in accordance with SDO Standard Drawings.

5.6.4 Access

Design box girders with exterior, in-swinging, hinged, doors (min. 32”x42” or 36” dia). Equip doors with a lock and hasp. Require that all locks be keyed alike. Indicate on plans that access openings are not to be used for utilities or other attachments. Analyze access opening sizes and bottom flange locations for structural effects on the girder. Avoid access locations over traffic lanes and locations that will require extensive maintenance of traffic operations or that would otherwise impact the safety of inspectors or the traveling public.

5.6.5 Ventilation and Drain Holes

Design each box girder with minimum 2” diameter ventilation or drain holes spaced about every 50 feet or as needed to provide proper drainage. Require .25” screen on all exterior holes not covered by a door. This includes holes in webs through which drain pipes pass, ventilation holes, drain holes, and openings in diaphragms at abutments and at expansion joints.

Provide drains to prevent water that enters from any source, including condensation, from ponding in the vicinity of post-tensioning components. Show details on the Contract Drawings. Include the following:

A. Specify that drains may be formed using 2-inch diameter permanent plastic pipes (PVC with UV inhibitor) set flush with the top of the bottom slab.
B. A small drip recess, ½ inch by ½ inch around bottom of pipe insert.
C. Drains at all low points against internal barriers, blisters, etc.
D. Drains on both sides of box, regardless of cross slope (to avoid confusion.)
E. Show locations and details for drains taking into account bridge grade and cross-slope.
F. Vermin guards for all drains.
G. A note stating, “The Contractor shall install similar drains at all low spots made by barriers introduced to accommodate his means and methods of construction, including additional blocks or blisters.”

5.6.6 Cross Frames [6.7.4]

Design external diaphragms as an “X-frame” or a “K-frame” as noted for “I-girders.” Detail “X-frames” or “K-frames” bolted to girders at stiffener locations. Internal diaphragms may be connected by welding or bolting to stiffeners in the fabrication shop. For box girders, use a “K-frame” internal diaphragm.

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

5.6.7 Lateral Bracing [6.7.5]

For box girders, design an internal lateral bracing system in the plane of the top flange.

Detail the plans to avoid interferences between the lateral bracing and stay-in-place metal forms.

Commentary:
A single diagonal member is preferred over an “X-diagonal” configuration for ease of fabrication and erection.

5.7 Diaphragms and Cross Frames for "I-Girders" [6.7.4]

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

5.8 Transverse Intermediate Stiffeners [6.10.8.1]

Specify that intermediate stiffeners providing diaphragm connections are fillet welded to the compression flange and fillet welded or bolted to the tension flange or flanges subject to stress reversal.

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

Specify that intermediate stiffeners on plate girders without diaphragm connections have a “tight fit” at the tension flange and be fillet welded to the compression flange.

Specify that intermediate stiffeners on box girders be fillet welded to the compression flange and cut back at the tension flange end or on flange subject to stress reversal.

5.9 Bearing Stiffeners [6.10.8.2]

For bearing stiffeners that provide diaphragm connections, specify a “mill-to-bear” finish on the bottom flange and fillet welded connections to both the top and bottom flanges. In negative moment regions only, stiffeners with attached diaphragms may be bolted to the top flange.

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

5.10 Longitudinal Stiffeners [6.10.8.3]

Avoid the use of longitudinal stiffeners. If they must be used, the stiffener should be made continuous on one side of the web with transverse stiffeners located on the other side of the web. For aesthetic reasons, avoid placing transverse stiffeners on the exterior face of exterior girders.

Commentary:
If longitudinal stiffeners are being considered, an analysis of material and labor costs should be performed to justify their use. Their use may be justified on deep, haunched girders but normally cannot be justified on constant depth girders. When longitudinal stiffeners are used on the same side of the web as the transverse stiffeners, the intersection of the stiffeners must be carefully designed with respect to fatigue.

5.11 Connections and Splices [6.13]

Specify and detail bolted (not welded) field connections. Only prior written approval by the SDO or the appropriate DSDO allows field welding and then, only when bolting is impractical or impossible.

Where cantilever brackets are connected to exterior girders and tie plates are used to connect the top flange of the bracket to the top flange of the floor beam, do not show the tie plates connected to the girder top flange. Detail short, slotted holes in the top flange of the cantilever brackets (perpendicular to the bracket web), to account for alignment tolerances. Reduce the allowable bolt stress accordingly.

5.11.1 Slip Resistance [6.13.2.8]

Design bolted connections for Class B surface condition. When the thickness of the plate adjacent to the nut is greater than or equal to ¾ inch, base the strength of the connection on the bolt shear strength with threads excluded from the shear plane.

Commentary:
This surface condition agrees with Florida fabrication practice.

5.11.2 Welded Connections [6.13.3]

Do not show a specific, pre-qualified, complete-joint penetration weld designation on the plans unless a certain type of weld; i.e., “V,” ”J,” ”U,” etc., is required.

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

On the plans, identify areas that are subject to tension and areas subject to stress reversal.

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

When welding is required during rehabilitation or widening of an existing structure, show on the plans, the type of existing base metal. Advise the District Structures Design Office if the base metal type cannot be determined, or if the type is not an approved base metal included in the ANSI/AASHTO/AWS Bridge Welding Code. The State Materials Office and the Department’s independent inspection agency will then determine the welding procedures and welding inspection requirements for the project.

5.11.3 Welded Splices [6.13.6.2]

At flange transitions, do not reduce the cross-sectional area by more than one-half the area of the larger flange plate.

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

Maintain constant flange widths within each field-bolted section.

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

The following criteria may be used to make a determination of the number of pounds, Δw, of material that must be saved to justify the cost of introducing a flange transition:
A. For 36 ksi material: $\Delta w = 300 + (25.0) \times$ (area of the smaller flange plate, in^2),
B. For 50 ksi material : $\Delta w = 250 + (21.3) \times$ (area of the smaller flange plate, in^2),
C. For 70 ksi material : $\Delta w = 220 + (18.8) \times$ (area of the smaller flange plate, in^2).

5.12 Corrosion Protection

Specify uncoated weathering steel or a three-coat inorganic zinc coating system on exterior surfaces. For box interiors, specify a single coat of inorganic zinc. Other systems must be approved by the SDO or the DSDO.

Specify white or gray for box interior colors. The final exterior finish color is an aesthetic treatment and must be approved by the DSDO.

5.12.1 Uncoated Weathering Steel

Design and specify weathering steel per the FHWA Technical Advisory T5140.22.

Sites classified as Slightly Aggressive for Superstructures (See Figure 1-1.) are suitable for the use of weathering steel without long-term, on-site panel testing.

Specify ASTM A325 Type 3 bolts for weathering steel.

Painting of the exterior girder/fascia may be required for aesthetic appearance.

5.12.2 Single Coat Inorganic Zinc

Suitable locations for the use of single coat inorganic zinc coating without on site testing must comply with all of the following:
A. General Criteria: Environments classified as Slightly Aggressive for Superstructure.
B. Site Specific Criteria:
 1.) Locations where the pH of the rainfall or condensation is between 4-10.
 2.) Locations not subjected to salt spray or salt laden run-off.

3.) Locations not subjected to concentrated acid pollution caused by the following sources: coal burning power plant; phosphate plant; acid manufacturing plant; any site yielding high levels of sulfur compounds.

Commentary:
Inorganic zinc coatings (either gray or green) quickly fade in approximately 3 months to a uniform gray color similar to Federal Color Standard Number 37866. For color, the fascia of the exterior girders can be coated with the three-coat inorganic zinc system.

5.12.3 Galvanizing

A. 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 must be hot-dip galvanized, A325 (fine thread) bolts must be mechanically galvanized when utilized with galvanized steel components. Other applications not requiring full tensioning of the bolts may use hot-dip galvanized A325 bolts.

B. Galvanizing of Bolts for Miscellaneous Structures
Specify hot-dipped galvanized bolts for connecting structural steel members of miscellaneous structures such as overhead sign structures, traffic mast arms, ground-mounted signs, etc.

CHAPTER 6
SUPERSTRUCTURE COMPONENTS

6.1 General

This Chapter contains information and criteria related to the design, reinforcing, detailing, and construction of superstructure elements and components for bridge structures and includes deviations from ***LRFD***. The Chapter covers deck slabs, beams, barriers, curbs, pilasters, joints, bearings, and deck drains, etc.

6.2 Curbs and Medians [13.11]

For bridge projects that utilize curbs, match the curb height and batter on the roadway approaches. Bridges with sidewalks are usually encountered in an urban environment, and the curb height will normally be 6 inches. When the roadway approaches have a raised median, design the bridge median to match that on the roadway.

6.3 Temperature Movement [3.12.2]

For all bridges other than longitudinally post-tensioned, segmental concrete bridges, calculate movement due to temperature variation (range) with an assumed mean temperature of 70 degrees Fahrenheit at the time of construction. Base joint and bearing design on the expansion and contraction for temperature ranges of Table 6.2.

6.4 Expansion Joints

Minimize the use of joints. Bridge deck joints are a continuous minor maintenance problem and may be a source of structural deterioration resulting in a major bridge maintenance problem.

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 "Long Bridges," (see Chapter 4) detail expansion joints for installation as an independent operation performed after deck planing requirements have been met.

For new construction use only joint types listed in Table 6.1. The values in Table 6.1 are maximum design values for each joint type.

Specify silicone seals for all cast-in-place, flat slab bridges. Limit the continuous unit lengths to accommodate the allowable movement listed in Table 6-1.

Table 6.1 Joint Width Limitations by Joint Type

Joint Type	Maximum Joint Width*
Poured Rubber	¾”
Silicone Seal	2”
Strip Seal	3”
Modular Joint	Unlimited
Finger Joint	Unlimited

*Joints in sidewalks must meet all requirements of Americans with Disabilities Act.

Table 6.2 Temperature Range by Superstructure Type

Structural Material of Superstructure	Temperature (Degrees Fahrenheit)			
	Mean	High	Low	Range
Concrete Only	70	95	45	50
Concrete Deck on Steel Girder	70	110	30	80
Steel Only	70	120	30	90

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

6.4.1 Expansion Joint Provisions [14.5.1]

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

1.) Proprietary joint device must be designed in accordance with ***LRFD*** and installed in strict conformance with the manufacturer's specifications unless directed otherwise by the Department.
2.) Ensure that joint components will accommodate the full range of structure movement without exceeding the manufacturer's limitations and/or design values, particularly at maximum joint opening.
3.) The joint must provide a good riding surface, relatively free from vibration and noise.
4.) Design frame rails to resist all anticipated loads, including impact
5.) The joint will not transfer undue stresses to the structure.
6.) Specify joint seal elastomers with a minimum five year service life.
7.) Design the joint for minimum maintenance and ease of access and parts replacement.
8.) Design a reasonably leak-proof joint with a continuous sealing element for the entire joint length.
9.) Specify that the joint material be resistant to corrosion and ultraviolet rays and is not a catalyst or vehicle for electrolytic action.

B. In some instances, open joints may be acceptable if they 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.

6.4.2 Movement [14.4] [14.5.3]

When designing expansion joints, analyze movement due to temperature variation, the effects of creep, shrinkage, skew, rotation, lateral shear, and vertical shear.

Design conventional, reinforced and/or prestressed concrete superstructure joints for the calculated movement due to temperature change plus creep and shrinkage, or for 115% of the calculated movement due to temperature change alone, however, the joint opening must not be less than 5/8 inch at 70 degrees Fahrenheit.

Design longitudinally post-tensioned, superstructure joints for creep and shrinkage plus 120% of the calculated movement due to temperature change.

The width, "W", of the joint must meet the requirements of ***LRFD*** **[14.5.3.2]**, except that "W" for the different joint types must not exceed the appropriate value from Table 6.1. When setting the joint, either the design width (W) must be decreased by the amount of anticipated movement due to creep and shrinkage, or the joint must be set to the minimum width for installing the joint, whichever results in the wider opening.

6.4.3 Joints for Bridge Widening

A. Contact the District Structures and Facilities Office to determine the type and condition of all existing joints to be widened. Existing joint types by group are:
   1.) Group 1: Strip seal, compression seal, poured rubber, open joint, silicone seal, copper water-stop, and "Jeene."
   2.) Group 2: Sliding Plate, finger joint, and modular.

B. When existing joints are to be widened, determine the extent of existing concrete deck to be removed. Limit removal of existing concrete to what is 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 the Contractor not damage the existing deck reinforcing steel when installing the new joint.

6.4.4 Lengthening 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 an existing armored joint 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 1 inch at 70 degrees Fahrenheit, 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 1 inch at 70 degrees Fahrenheit, provide a blockout on the entire bridge deck for the new joint system.
F. If the existing joint is a Jeene Joint and is performing satisfactorily, extend the joint gap into the widening, remove the existing Jeene Joint and provide a new Jeene Joint. If it is not practicable to install a new Jeene Joint, provide a blockout for the new joint system.

Commentary:
Concrete spalls adjacent to existing joints must be repaired prior to joint lengthening. All neoprene seals must be one piece and should extend to the face of the barrier at the high side of deck surface and past the gutter line with a 4 inch upturn at the low side of the deck surface. 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.

6.4.5 Widenings - Group 2 Joints

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

6.4.6 Post Tension Bridges.........See 4.8.7 Expansion Joints

6.5 Bearings

Bridge bearings must accommodate the movements of the superstructure and transmit loads to the substructure supports. The type of bearing 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). Larger longitudinal movements can be accommodated by using PTFE polytetrafluoroethylene [Teflon] bearing surfaces on external steel load plates.

Structures with large bearing loads and/or multi-directional movement might require other bearing devices such as pot, spherical, or disc bearings.

6.5.1 Design

Specify composite neoprene bearing pads and other bearing devices that have been designed in accordance with ***LRFD*** Method B, the Department's ***Standard Specifications for Road and Bridge Construction***, and this document.

Whenever possible, and after confirming their adequacy, standard designs 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. If other bearing systems or components are considered, the bearing types must be selected based on a suitability analysis . Comply with ***LRFD*** Table 14.6.2-1, Bearing Suitability, to select an appropriate bearing type. The special design requirements of ***LRFD*** covers specific material properties, mating surfaces, and design requirements such as coefficient of friction, load resistance, compressive stress, compressive deflection, and shear deformation, as applicable to the various bearing systems.

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

6.5.2 Maintainability

The following provisions apply to all bridges with the exception of flat slab superstructures (cast-in-place or precast) resting on thin bearing pads.

A. Design and detail superstructure using bridge bearings that are reasonably accessible for inspection and maintenance.
B. On all new designs make provisions 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.
C. Design and detail provisions for the removal of bearings, such as jacking locations, jacking sequence, jack load, etc. Verify that the substructure width is sized to accommodate the jacks and any other required provisions.
D. When widening a bridge that does not already include provisions for replacing bearings, consult the District Maintenance Engineer who will decide if bearing replacement provisions must be made on the plans.

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. Certain non-conventional structures, such as steel or segmental concrete box girders, require separate details and notes describing the procedures. Always include a plan note stating that the jacking equipment is not part of the bridge contract.

6.5.3 Lateral Restraint

Determine if 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 include considerations for the effects of geometry, creep, shrinkage, temperature, and/or seismic on the structure. When lateral restraint of the superstructure is required, develop the appropriate method of restraint as described hereinafter.

A. Elastomeric Bearings
   When the required restraint exceeds the capacity of the bearing pad, the following appropriate restraint must be provided:
   1.) For concrete girder superstructures, provide concrete blocks cast on the substructure and positioned to not interfere with bearing pad replacement.
   2.) For steel girder superstructures, provide extended sole plates and anchor bolts.

B. Mechanically Restrained Bearings
   Bearings that provide restraint through guide bars or pintles (e.g., pot bearings), must be designed to provide the required lateral restraint.

6.6 Deck Drainage [2.6.6]

A. Minimize the use of scuppers for deck drainage. Design scuppers or deck drains to pipe drainage to the ground or use extended downspouts. Ensure that run-off will be carried away from substructure as well as underlying infrastructure.

B. In order to facilitate run-off and avoid corrosion, minimize the use of stiffeners and bracing and avoid crevices.

6.6.1 Deck Drains

For simple deck drains (i.e. scuppers) encased in concrete, specify schedule 80, UV-resistant Polyvinyl-Chloride (PVC), gray cast iron, or ductile cast iron. For other deck drains encased in concrete, such as grates and inlets, specify Gray Cast Iron, Ductile Cast Iron, or Galvanized Steel. Do not specify Gray Cast Iron or Ductile Cast Iron for use in Extremely Aggressive Environments. The maximum width of a grate slot is 1 1/2 inches.

6.6.2 Drain Conveyances

For drain conveyances encased in concrete, specify Polyethylene (PE), or Ductile Cast Iron. For drain conveyances not encased in concrete, specify machine-made or filament-wound “Fiberglass” (Glass-Fiber-Reinforced-Thermosetting-Resin), or Ductile Cast Iron. Do not specify ductile cast iron in Extremely Aggressive Environments. Do not specify pipe diameters less than 4 inches, or bends greater than 45 degrees.

6.7 Traffic Railings [13.7]

6.7.1 General

A. Unless otherwise approved, all new traffic railings including barrier/sound wall combinations must:

1.) Have passed crash testing performed in accordance with the ***National Cooperative Highway Research Program (NCHRP) Report No. 350*** and ***LRFD***.
2.) Have been structurally evaluated to be equivalent to or greater in strength to other safety shape railings that have been crash tested to the TL-4 criteria of ***NCHRP Report No. 350***.
3.) Meet the strength and geometric requirements of ***LRFD*** Chapter 13 in accordance with the test levels and crash test criteria.
4.) Be upgraded on both sides of a structure 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.
5.) Be constructed on decks reinforced in accordance with Chapter 7.

6.7.2 Non-Standard or New Railing Designs

A. The traffic railings shown on the Structures Standard Drawings, Indices 700 through 780 meet the applicable crash testing requirements and should be used on structures in Florida. The applicability of each of these standard traffic railings is addressed in the Plans Preparation Manual, Volume I. Variations from these Structures Standards such as the use of non-standard railings, new railing designs, or modifications to standard railings require the prior approval of the SDO. Variations that require approval include but are not limited to reinforcement details, surface treatments, material substitutions, geometric discontinuities, end transition details, or traffic face geometry.

A non-standard or new traffic railing design may be approved by the SDO if it:

1.) Has passed crash testing performed in accordance with the NCHRP Report 350 criteria to minimum Test Level 4.
2.) Has been approved by FHWA for specific uses after evaluation of results from successful crash testing based on criteria that predate NCHRP Report 350 Test Level 4.
3.) Has been evaluated by FDOT and identified as similar in strength and geometry to another railing that has been successfully crash tested in accordance with NCHRP Report 350 Test Level 4 criteria.

Commentary:
The background for this policy is based on the Test Level Selection Criteria as defined in Section 13 of the AASHTO LRFD Bridge Design Specifications and historical construction costs and in-service performance of standard FDOT TL-4 traffic railings. This background can be summarized as follows:

1. *In general, a greater potential exists for overtopping or penetrating a shorter height, lower test level traffic railing versus a similarly shaped TL-4 rail. This potential is further aggravated on tall bridges and on bridges over intersecting roadways or*

water deep enough to submerge an errant vehicle. Vehicle performance during higher speed impacts is also more critical on lower test level traffic railings.

2. *Little construction cost savings can be realized by using a lower test level traffic railing. In some cases, particularly with the more elaborate or ornate traffic railing designs, initial construction costs and long term repair and maintenance costs could actually be greater than those for a standard FDOT TL-4 design.*
3. *Aesthetically pleasing and open TL-4 designs are available for use on bridges where appropriate.*
4. *On bridges with sidewalks where special aesthetic treatments are desired or required, the use of an aesthetic pedestrian railing located behind a TL-4 railing is a more appropriate solution. The aesthetics of the traffic railing should complement the pedestrian railing.*

B. For more detailed information on Florida bridge traffic railings, refer to the Structures Standard Drawings. For additional information about crash-tested bridge traffic railing currently available or about bridge traffic railing currently under design, contact the Structures Design Office.

6.7.3 FHWA Policy

A. FHWA policy is that all new or replacement traffic railing on National Highway System or Interstate Highway System bridges must meet Test Level 3 (TL-3) crash-test criteria at a minimum. Since September 1,1986, the Federal Highway Administration (FHWA) has required highway bridges on the National Highway System (NHS) and the Interstate Highway System to have crash-tested railing. Current policy is stated in the following documents:

1.) Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA).
http://www.fhwa.gov/legregs/legislat.html
Requires that measures to enhance the crashworthiness of roadside features accommodate vans, minivans, pickup trucks, and 4-wheel drive vehicles, as well as cars.

2.) National Cooperative Highway Research Program (NCHRP) Report 350, Recommended Procedures for the Safety Performance Evaluation of Highway Features.
http://safety.fhwa.dot.gov/programs/roadside_hardware.htm
Provides guidance for testing highway features to assess safety performance of those features. Guidance includes definitions of crash-test levels with specified vehicle, speed, and impact angle for each level.

3.) May 30, 1997, memorandum from Dwight Horne on the subject of “Crash Testing of Bridge Railings”.
http://safety.fhwa.dot.gov/fourthlevel/hardware/pdf/bridge.pdf
Identifies 68 crash-tested bridge rails, consolidating earlier listings and establishing tentative equivalency ratings that relate previous testing to NCHRP Report 350 test levels.

4.) July 25, 1997 memorandum from Donald Steinke on the subject of “Identifying Acceptable Highway Safety Features.”
http://fhwa.dot.gov/legsregs/directives/policy/ra.htm

Clarifies and summarizes policies on bridge traffic railing, points to authorities for requiring testing of bridge traffic railing, and identifies methods for submitting new rails for testing. This document also identifies exceptions, one of which is the replacement or retrofitting of existing bridge traffic railing unless improvements are being made on a stretch of highway that includes a bridge with obsolete railing.

B. FHWA provides current information on three general categories of roadside hardware that has been tested and evaluated using NCHRP Report 350 criteria.
See Bridge Railings at
http://safety.fhwa.dot.gov/fourthlevel/hardware/bridgerailings.htm

6.7.4 Existing Traffic Railings

A. General

1.) FDOT promotes highway planning that replaces or upgrades non-crash tested traffic railing on existing bridges to current standards, or that at least increases the strength or expected crash performance of these traffic railings. FDOT has developed two sets of Structures Standards, the Index 770 and 780 Series, for retrofitting existing bridges with traffic railing types that have performed well in crash tests and are reasonably economic to install. Detailed instructions and procedures for retrofitting obsolete traffic railings on existing bridges are included in the Structures Standard Drawings.

2.) When rehabilitation or renovation work is proposed on an existing structure with traffic railings that do not meet the criteria for new railings as provided above, replace or retrofit them to meet the crash-worthy criteria unless an exception is approved.

B. FHWA Policy on Existing Traffic Railings

The FHWA requires that bridge railing on the National Highway System (NHS) meet requirements of NCHRP Report 350.

"…all new or replacement safety features on the NHS covered by the guidelines in the NCHRP Report 350 that are included in projects advertised for bids or are included in work done by force-account or by State forces on or after October 1, 1998, are to have been tested and evaluated and found acceptable in accordance with the guidelines in the NCHRP Report 350" (See Section 6.4.2, Number 4).

However, FHWA softens this requirement somewhat by allowing exceptions:

"Bridge railings tested and found acceptable under other guidelines may be acceptable for use on the NHS." This is a specific reference to the Horne memo titled "Crash Testing of Bridge Railings" (See Section 6.4.2, Number 2).

"The FHWA does not intend that this requirement (that new safety features installed on the NHS be proven crashworthy in accordance with the guidelines in the NCHRP Report 350) result in the replacement or upgrading of any existing installed features beyond what would normally occur with planned highway improvements."

This statement is qualified by a requirement that states have a "rational, documented policy for determining when an existing non-standard feature should be upgraded."

C. Traffic Railing Retrofit Concepts and Standards
Existing non-crash tested traffic railings designed in accordance with past editions of the AASHO and AASHTO Standard Specifications for Highway Bridges will likely not meet current crash test requirements or strength and height requirements of the AASHTO LRFD Bridge Design Specifications. The retrofitting of these existing non-crash tested traffic railings reduces the separate but related potentials for vehicle snagging, vaulting and or penetration that can be associated with many obsolete, non-crash tested designs.

The Thrie Beam Guardrail Retrofit and Vertical Face Retrofit Structures Standards, Index 770 and 780 Series respectively, are suitable for retrofitting specific types of obsolete bridge traffic railings. These retrofits provide a more economical solution for upgrading obsolete traffic railings when compared with replacing the obsolete traffic railings and portions of the bridge decks that support them. As these retrofits do not provide for any increase in clear width of bridge deck, and in a few cases decrease clear width by approximately 2 inches, they should only be considered for use on existing bridges where adequate lane and shoulder widths are present. Detailed guidance and instructions on the use of these retrofits is included in the individual Structures Standards.

When selecting a railing for a widening or rehabilitating a bridge project, evaluate the following aspects of the project:

1.) Elements of the bridge structure.
  - a.) Bridge width, alignment and grade.
  - b.) Type, aesthetics, and strength of existing railing.
  - c.) Bridge length and its potential for posting speed limits.
  - d.) Approach and trailing end treatments (guardrail, crash cushion or rigid shoulder barrier).
  - e.) Strength of supporting deck configuration.
  - f.) Load rating of existing bridge.

2.) Characteristics of the bridge location.
  - a.) Position of adjacent streets and their average daily traffic.
  - b.) Structure height above lower terrain or waterway.
  - c.) Approach roadway's width, alignment and grade.
  - d.) Bridge design speed, posted speed, average daily traffic and percentage of truck traffic.
  - e.) Accident history on the bridge.

3.) Features of the retrofit designs.
  - a.) Placement or spacing of anchor bolts or dowels.
  - b.) Reinforcement anchorage and potential conflicts with existing reinforcement, voids, etc.
  - c.) Self weight of retrofit railing.

d.) Effect of retrofit on shoulder width and sight distances to ensure no increase in accident rate.
e.) MOT required for initial construction of retrofit and for potential future repairs.

D. Evaluation of Existing Supporting Structure Strength for Traffic Railing Retrofits
The Thrie Beam Guardrail and Vertical Face traffic railing retrofits are based on designs that have been successfully crash tested in accordance with NCHRP Report 350 to Test Level 4. The original designs have been modified for use with some of the wide variety of traffic railings and supporting deck and wing wall configurations that were historically constructed on Florida bridges. In recognition of the fact that the traffic railings and supporting decks and wing walls of these existing bridges were designed to meet the less demanding requirements of past AASHO and AASHTO Bridge Codes, modifications have been made to the original retrofit designs in order to provide for better distribution of vehicle impact force through the traffic railing retrofit and into the supporting bridge deck or wing wall. For Thrie Beam Guardrail Retrofit installations on narrow curbs and or lightly reinforced decks, a smaller post spacing is used. In addition, through-bolted anchors are used for some Thrie Beam Guardrail Retrofit installations. For the Vertical Face Retrofit, additional longitudinal reinforcing steel and dowel bars at the open joints are used within the new railing.

Existing bridge decks, wing walls and retaining walls that will support a traffic railing retrofit must be evaluated to determine if sufficient strength is available to ensure that the retrofit will perform in an equivalent manner to which it was crash tested. Existing bridges may contain Grade 33 reinforcing steel if constructed prior to 1952 or Grade 40 reinforcing steel if constructed prior to 1972. Use 90% of the ultimate tensile strength of these materials when determining the existing capacity for combined tension and moment from traffic railing impacts (f_s = 49.5 ksi for Grade 33, f_s = 63 ksi for Grade 40). For bridges with varying spacings and sizes of transverse reinforcing steel in the deck or curb, the average area of transverse steel for the span may be used.

The following design values will be used to analyze existing cast-in-place reinforced concrete bridge decks at the gutter line for the FDOT traffic railing retrofit standard indices:

Traffic Railing Type	**Standard Index No.**	**M_g**	**T_u**
Thrie-Beam Retrofit	Nos. 772, 776 & 777	5.8	4.7
Thrie-Beam Retrofit	Nos. 773 & 774	8.3	6.7
Thrie-Beam Retrofit	No. 775	9.7	7.9
Vertical-Face Retrofit	Nos. 782-785	12.9	7.5

M_g (kip-ft/ft) - Ultimate deck moment at the gutter line from the traffic railing impact.
T_u (kip/ft) - Total ultimate tensile force to be resisted.

The following relationship must be satisfied at the gutter line:

$$\frac{T_u}{\varphi P_n} + \frac{M_u}{\varphi M_n} \leq 1.0 \qquad \text{(Eq. 6-1)}$$

Where:
Ø = 1.0

Pn = $A_s f_s$ (kips/ft) - Nominal tensile capacity based on the areas of transverse reinforcing steel in both the top and bottom layers of the deck (A_s). This reinforcing steel must be fully developed at the critical section.

Mu (kip-ft/ft) - Total ultimate deck moment from traffic railing impact and factored dead load at the gutter line (Mg + 1.25*$M_{Dead\ Load}$).

Mn (kip-ft/ft) - Nominal moment capacity at the gutter line determined by traditional rational methods for reinforced concrete. The bottom layer of steel must not be included unless a strain compatibility analysis is performed to determine the steel stress in this layer with the compressive strain in the concrete limited to 0.003.

Decks constructed of longitudinally prestressed, transversely post-tensioned voided or solid slab units generally only contain minimal transverse reinforcing ties. Retrofitting bridges with this type of deck will not be permitted after January 1, 2010. For these type bridges, the strength checks of the deck at the gutter line will not be required. Only Index No. 776 or Index 780 Series retrofits can be used to retrofit these bridges.

In addition to checking the existing deck capacity at the gutter line, the following minimum areas of reinforcing steel must also be satisfied unless a more refined analysis is performed to justify a lesser area of steel at these locations:

Minimum Steel Area (in2/ft) for Standard Index				
Location of Reinforcing Steel	**772,776 & 777**	**773 & 774**	**775**	**780 Series**
Transverse in top of curb beneath post: Grade 33 Grade 40 & 60	 0.320 .25	 0.40 0.31	 0.40 3.21	 NA NA
Vertical in face of curb for thickness “D” Grade 33 Grade 40 & 60	 0.20 0.20*	 2.25/(D-2**) 1.80/(D-2)**	 2.65/(D-2)** 2.10/(D-2)**	 3.30/(D-2)** 2.60/(D-2)**

* 0.16 in2/ft is acceptable for D > 15 inches.
** Minimum area of reinforcing steel must not be less than 0.16 in2/ft.

Where:
D (inches) = Horizontal thickness of the curb at the gutter line.

If the minimum areas of reinforcing in the curb given above are not satisfied, the following design values may be used for a refined analysis of the existing curb beneath the post for the Index 770 Series retrofits:

Traffic Railing Type	Standard Index No.	M_p	T_u
Thrie-Beam Retrofit	Nos. 772, 776 & 777	9.7	7.9
Thrie-Beam Retrofit	Nos. 773, 774 & 77	12.0	9.9

Mp (kip-ft/ft) - Ultimate deck moment in the curb at centerline of post from the traffic railing impact.
Tu (kip/ft) - Total ultimate tensile force to be resisted.

The following relationship must be satisfied in the curb at centerline of post:

$$\frac{T_u}{\varphi P_n} + \frac{M_u}{\varphi M_n} \leq 1.0 \qquad \text{(Eq 6-2)}$$

Where:
Ø = 1.0

Pn = $A_s f_s$ (kips/ft) - Nominal tensile capacity based on the areas of transverse reinforcing steel in both the top and bottom of the curb (A_s). This reinforcing steel must be fully developed at the critical section.

M_u (kip-ft/ft) - Total ultimate moment in the curb from traffic railing impact and factored dead load at centerline of post (M_p + 1.25*$M_{Dead\ Load}$).

Mn (kip-ft/ft) - Nominal moment capacity of the curb at centerline of post determined by traditional rational methods for reinforced concrete. The bottom layer of steel in the curb must not be included unless a strain compatibility analysis is performed to determine the steel stress in this layer with the compressive strain in the concrete limited to 0.003.

The ultimate moment capacity of existing wing walls and retaining walls supporting the traffic railing retrofits must not be less than 9.7 kip-ft/ft for Index 770 Series retrofits and 12.0 kip-ft/ft for Index 780 Series retrofits. Wing walls for Index 780 Series retrofits must also be a minimum of 5 feet in length and pile supported. For Index 780 Series retrofits only, wing walls that do not meet these criteria must not be used to anchor the ends of guardrail transitions and must be shielded by continuous guardrail as shown on the Structures Standard Drawings. For both 770 and 780 Series retrofits, retaining walls must be continuous without joints for a minimum length of 10 feet and adequately supported to resist overturning.

An exception will be required for bridges or components of bridges that do not meet the preceding strength requirements. The potential for damage to the existing bridge deck or wing walls due to a very severe crash, such as that modeled by full scale crash testing, may be acceptable in specific cases. Contact the Structures Design Office for additional guidance and assistance in these cases.

6.7.5 Historic Bridges

Federal Law protects Historic Bridges and special attention is required for any rehabilitation or improvement. The Director of the Division of Historical Resources of the Florida Department of State serves as Florida's State Historic Preservation Officer (SHPO). The SHPO and FDOT are responsible for determining what effect any proposed project will have on a historic bridge.

Bridges that are listed or eligible to be listed in the National Register of Historic Places present a special railing challenge because the appearance of the bridge may be protected even though the historic railing may not meet current standards. When a project is determined to involve a historically significant bridge, the District should contact the Structures Design Office for assistance with evaluating the existing bridge railings.

Original railing on a historic bridge is not likely to meet:

A. Current crash test requirements.
B. Current standards for railing height (a minimum of 32 inches for Test Level 4) and for combination traffic and pedestrian railings,
C. The limits on the size of openings in the railing (small enough that a 6 inch diameter sphere cannot pass through them).

Options for upgrading the railing on historic bridges usually include the following:

A. Place an approved traffic railing inboard of the existing railing, leaving the existing railing in place. This is sometimes appropriate when a pedestrian walkway exists on or is planned for the bridge.
B. Replace the existing railing with an approved, acceptable railing of similar appearance.
C. Remove the current railing and incorporate it into a new acceptable railing. This may be appropriate in rare instances where an existing railing is especially decorative.
D. Design a special railing to match the appearance of the existing railing. It may not be necessary to crash test the new railing if the geometry and calculated strength equal or exceed a crash tested traffic railing.

6.7.6 Requirements for Test Levels 5 and 6 [13.7.2]

Provide a traffic railing 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. A vehicle penetrating or overtopping the traffic railing would cause high risk to the public or surrounding facilities.
C. Sharply curved ramp structures with moderate to heavy truck traffic.

Contact the SDO for guidance if a TL-5 or TL-6 traffic railing is being considered.

6.7.7 Exceptions

In the event that an upgrade to the traffic railing on an existing bridge could degrade rather than improve bridge safety, during the early phases of a project consult the Structures Design Office about a possible design exception.

Factors to consider include the following:
A. Remaining time until scheduled replacement or major rehabilitation of bridge.
B. Design speed and operating speed of traffic in the bridge location, preferably no greater than 45 mph.
C. Resistance to impact of the existing railing.
B. Whether the bridge ends are intersections protected by stop signs or traffic signals.
C. Whether the geometry is straight into, along and out of the bridge.
D. Overall length of the bridge.
E. Whether traffic on the bridge is one-way or two-way.
F. Accident history on the bridge, including damages to and repairs of the existing railing.
G. Risk of fall over the side of the bridge.
H. Whether the bridge has an intersecting roadway or railroad track below.
I. Whether a railing upgrade will further narrow an already narrow lane, shoulder or sidewalk.
J. Load rating of the existing bridge.
K. Special historic or aesthetic concerns

Exceptions to the requirements of this Article must be approved in accordance with Chapter 23 of the ***Plans Preparation Manual***, Volume I with concurrence of the DSDE or SSDE as appropriate.

6.7.8 Sound Barriers

Do not attach sound barriers to the top of traffic railings unless the system has been crash tested and meets TL-4 acceptability requirements of ***NCHRP Report 350***. Non-crash tested sound barriers may be attached to structures if located behind an approved traffic railing and mounted at least five feet from the face of the traffic railing at deck level. The Traffic Railing Barrier/Sound wall, Structures Standard Index No. 1550, is crash tested and approved for TL-4 use on Florida bridges.

CHAPTER 7
WIDENING AND REHABILITATION

7.1 General

Prior to widening any structure, review the inspection report and load rating analysis of the structure. If the inventory load rating does not meet or exceed HL-93 loading, perform a more refined analysis to accurately load-rate the structure. If the rating is still below the required capacity, the bridge should be strengthened or replaced.

7.2 Classifications and Definitions

7.2.1 Major Widening

A "Major Widening" is new construction work to an existing bridge facility which doubles the total number of traffic lanes or bridge deck area of the existing bridge. The area to be calculated is the transverse coping-to-coping dimension.

7.2.2 Minor Widening

A "Minor Widening" is new construction work to an existing bridge facility that does not meet the criteria of a major widening.

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

7.3 Analysis and Design

7.3.1 Aesthetics
Design widenings to match the aesthetic level of the existing bridge. Additions to existing bridges should not be obvious "add-ons." Consider specifying a Class 5 Finish for the existing bridge.

7.3.2 Materials
Materials used in the construction of the widening should have the same thermal and elastic properties as those of the existing structure.

7.3.3 Load Distribution
Proportion the main load carrying members of the widening to provide longitudinal and transverse load distribution characteristics similar to the existing structure.

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

When determining the distribution of the dead load for the design of the widening, and when performing stress checks of the existing structure, consider the construction sequence and degree of interaction between the widening and the existing structure after completion. Use ***LRFD*** criteria for the distribution of live load.

7.3.4 Specifications
Design all widenings in accordance with ***LRFD***. Review stresses in the main exterior member of the existing structure for construction conditions and the final condition; i.e., after attachment of the widened portion of the structure. When computations indicate overstresses in the exterior member of the existing structure, request a variation from the appropriate FDOT Structures Design Office.

7.3.5 Overlays
Asphalt overlays on bridge decks should be removed.

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

7.3.7 Other Special Considerations
A. Design all widenings for HL-93 loading regardless of the loading used in the original design.
B. When detailing connections and selecting or permitting construction methods, consider the amount of differential camber present prior to placing the new deck.
C. Avoid open or sealed longitudinal joints in the riding surface (safety hazards.)
D. Specify that live load vibrations from the existing structure be minimized or eliminated during deck pour and curing.
E. Refer to Chapter 6 for bearing replacement capabilities of widened structures.
F. Provide ample clearance between proposed driven piles and existing piles, utilities, or other obstructions. This is especially critical for battered piles.
G. Bearing fixity and expansion devices should be the same in both the widened and existing bridges.

7.4 Attachment to Existing Structure

7.4.1 Drilling
When drilling into heavily reinforced areas, specify exposure of the main reinforcing bars by chipping. Specify that drilled holes have a minimum edge distance of three

times the metal anchor diameter (3d) from free edges of concrete and 1 inch minimum clearance between the edges of the drilled holes and existing reinforcing bars. Specify core drilling for holes larger than 1½ inches in diameter. If it is necessary to drill through reinforcing bars, specify core drilling.

Adhesive Anchor Systems must be SDO approved and comply with the criteria and requirements of Chapter 1.

7.4.2 Dowel Embedments

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

A. Reduce the allowable stresses in the reinforcing steel by the ratio of the actual embedment divided by the required embedment.

B. If embedded anchors used to develop the reinforcing steel, use Adhesive Anchor Systems designed in accordance with Chapter 1.

7.4.3 Surface Preparation

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

7.4.4 Connection Details

Figures 7-1 through 7-4 are details that have been used successfully for bridge widenings for the following types of bridge superstructures.

A. Flat Slab Bridges (Figure 7-1):
   A portion of the existing slab should be removed in order to expose the existing transverse reinforcing for splicing. If the existing reinforcing steel cannot be exposed, the transverse slab reinforcing steel for the widening may be doweled directly into the existing bridge without meeting the normal splice requirement. When splicing to the existing steel is not practical, Adhesive Anchor Systems, designed in accordance with Chapter 1, must be utilized for the slab connection details as shown in Figures 7-1 and 7-4

B. T-Beam Bridges (Figure 7-2):
   The connection shown in Figure 7-2 for the slab connection is recommended. Limits of slab removal are at the discretion of the EOR but subject to the Department's approval.

C. Steel and Concrete Girder Bridges (Figure 7-3):
   The detail shown in Figure 7-3 for the slab connection is recommended for either prestressed concrete or steel beam superstructures.

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

7.5 Expansion Joints

See Chapter 6.

7.6 Traffic Railing

See Chapter 6.

7.7 Construction Sequence

Show on the preliminary plans, a construction sequence which takes into account the Traffic Control requirements. Submit Traffic Control Plans for traffic needs during construction activities on the existing structure such as installation of new joints, deck grooving, etc.

Include in the final plans, a complete outline of the order of construction along with the approved Traffic Control Plans. Include details for performing any necessary repairs to the existing bridge.

7.8 Widening Rules

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

A. The widened portion of a minor widening must match the existing structure. Refer to Chapter 4 for deck slab design.
B. Avoid mixing concrete and steel beams in the same span.
C. The use of beams of the same type as those in the existing structure is preferred; however, if the existing beams are cast-in-place concrete, detail the widened deck supported on prestressed beams.
D. Provide the required vertical clearance on Interstate structures unless an exception is granted. To assist in meeting the minimum vertical clearance requirements on a widening, the standard beam depth may be decreased. Where the existing bridge does not satisfy current vertical clearance requirements and where the economics of doing so are justified, the superstructure must be elevated and/or the under passing roadway must be lowered as part of the widening project.

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

E. The transverse reinforcement in the new deck should be spaced to match the existing spacing. Different bar sizes may be used if necessary.
F. Voided-slab bridges require special attention. Contact the DSDE for guidance. The DSDE will coordinate with the SDO to establish recommendations and criteria for the widening of the particular structure.

G. For all widenings, confirm that the available existing bridge plans depict the actual field conditions. Notify FDOT's Project Manager of any discrepancies, which are critical to the continuation of the widening design.

Commentary:
In general, confirming the agreement of existing plans with actual field conditions should be included as part of any new survey. A structural engineer must be involved in checking that the existing plans agree with actual field conditions for items such as:
Bridge location, pier location, skew angle, stationing.
Span lengths.
Number and type of beams.
Wingwall, pier, and abutment details.
Utilities supported on the bridge.
Finished grade elevations.
Vertical and horizontal clearances.
Other features critical to the widening.

7.9 Deck Grooving

For widened superstructures where at least one traffic lane is to be added, specify grooving for the entire deck area.

For projects with shoulder widening only, add a note to the plans specifying that the bridge floor finish match that of the existing bridge deck surface. However, if the existing bridge deck surface is in poor condition, contact the DSDO 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, specify that the existing bridge deck be sealed after grooving. Include a note in the "General Notes" and detail a sketch showing the surfaces to be coated. If the existing deck is not to be grooved, do not specify a penetrant sealer.

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

For all new construction utilizing C-I-P bridge deck (floors) that will not be surfaced with asphaltic concrete, include the following item in the Summary of Pay Items:

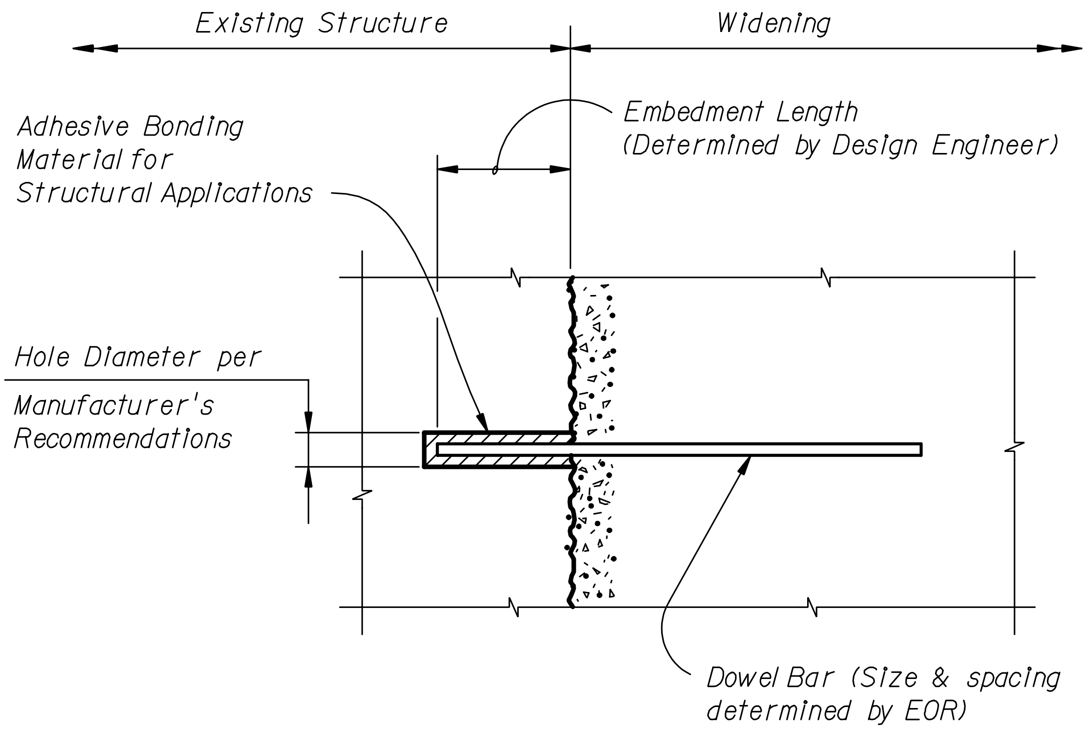
Item No. 400-7 - Bridge Floor Grooving .... Sq. Yards

Quantity Determination: Determine the quantity of bridge floor grooving in accordance with the provisions of Article 400-22.4 of the "Specifications."

7.9.1 Penetrant Sealers

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, and the existing deck is to be grooved, specify penetrant sealers for the existing bridge deck after grooving. Do not specify penetrant sealers for new bridge structures or if the existing deck is not to be grooved,

Include in the bridge plans, an item in the “General Notes” and a sketch describing the concrete surfaces to be coated with penetrant sealer.



NOTES:

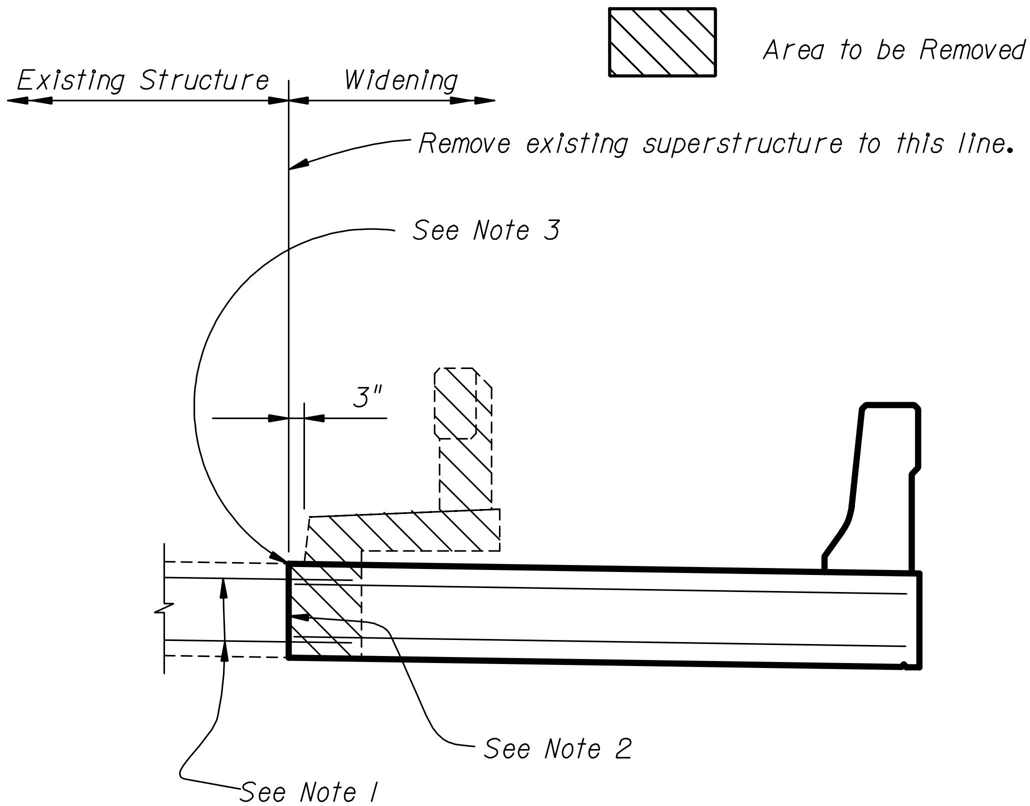
1. Verify that holes for Dowel Bars are thoroughly clean and dry prior to placing bonding material.

2. Comply with Sections 416 & 937 of the Specifications.

3. Shift dowel hole locations if existing reinforcing steel is encountered.

WIDENING DETAIL SHOWING DOWEL INSTALLATION

Figure 7-1



NOTES:

1. Existing transverse reinforcing to remain in place.
   Clean bars, straighten and embed into the slab widening. If bars are broken or otherwise determined to be unsatisfactory by the Engineer, replace with dowel bars as shown in Figure 7-1.
2. Clean all contacting surfaces between the old and new concrete immediately before casting concrete.
3. Score concrete for full length of span by sawing to top of reinforcing
   Avoid damaging reinforcing steel during sawing operation and slab removal.

WIDENING DETAIL FOR FLAT SLAB SUPERSTRUCTURE

Figure 7-2 Flat Slabs

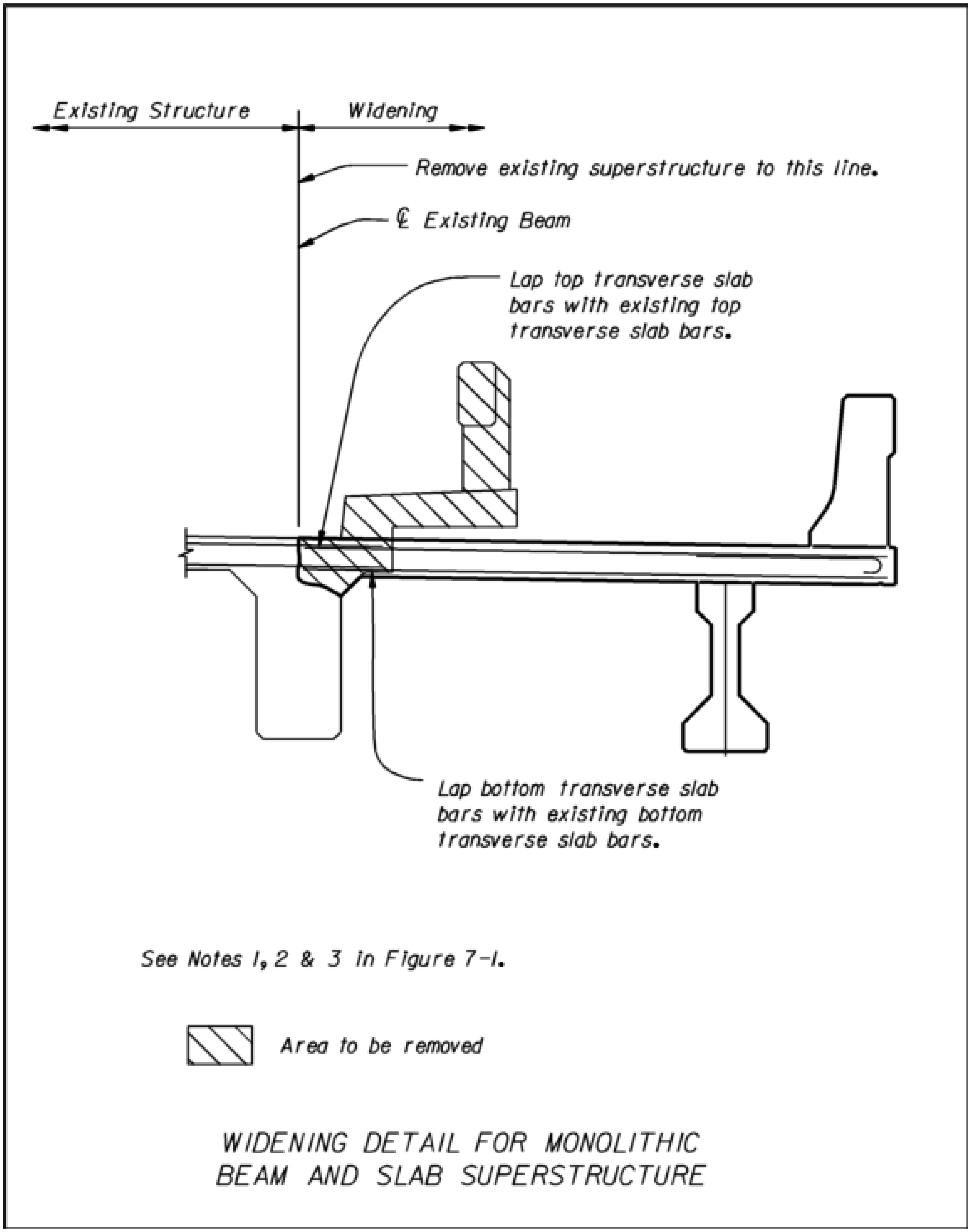

Existing Structure
Widening
Remove existing superstructure to this line.
℄ Existing Beam
Lap top transverse slab bars with existing top transverse slab bars.
Lap bottom transverse slab bars with existing bottom transverse slab bars.
See Notes 1, 2 & 3 in Figure 7-1.
Area to be removed
WIDENING DETAIL FOR MONOLITHIC BEAM AND SLAB SUPERSTRUCTURE

Figure 7-3

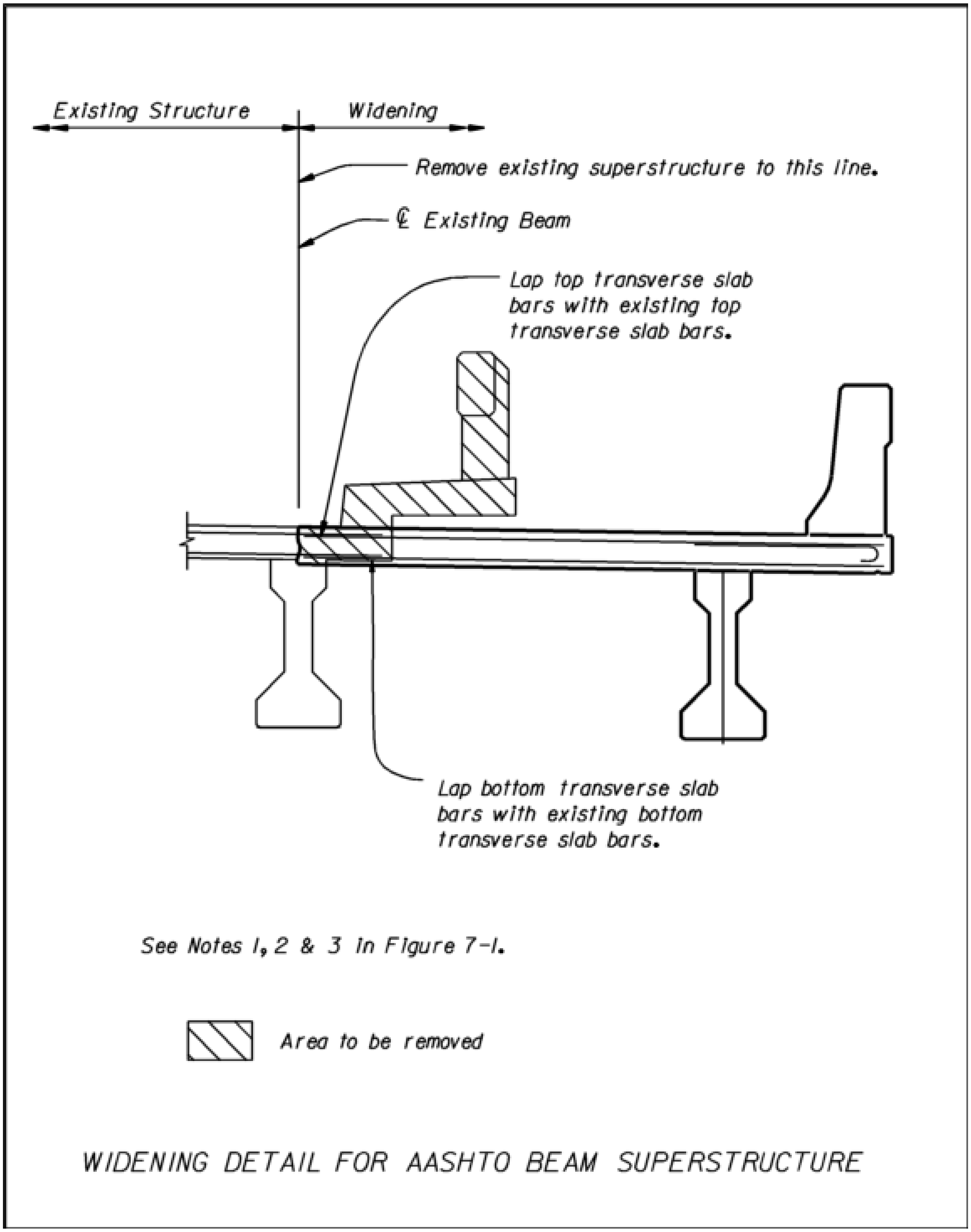

Existing Structure
Widening
Remove existing superstructure to this line.
℄ Existing Beam
Lap top transverse slab bars with existing top transverse slab bars.
Lap bottom transverse slab bars with existing bottom transverse slab bars.
See Notes 1, 2 & 3 in Figure 7-1.
Area to be removed
WIDENING DETAIL FOR AASHTO BEAM SUPERSTRUCTURE

Figure 7-4

CHAPTER 8
MOVABLE BRIDGES

8.1 General

This chapter contains information and criteria related to the design of movable bridge projects. It sets forth the basic Florida Department of Transportation (FDOT) design criteria that are exceptions or additions to those specified in the AASHTO ***LRFD-Movable Highway Bridge Design Specifications***, First Edition, 2000 and any interim releases thereafter and herein referred to as ***LRFD-MHB*** Specifications, except English (use soft conversions to convert to English units). Where applicable, other sections of this **SDG** also apply to the design of movable bridges.

8.1.1 Applicability

A. The design criteria of this chapter are applicable for new bridges and the electrical/machinery design for rehabilitation of existing bridges. The requirements for structural rehabilitation will be determined on a bridge-by-bridge basis, based on evaluations during the BDR phase and approval by the Structures Design Office (SDO). Projects for which the criteria are applicable will result in designs that
   1.) Preferably, provide new bascule bridges with a "two leafs per span" configuration.

   Commentary:
   Single leaf bascules are discouraged, but may be considered for small channel openings where navigational and vehicular traffic is low. Assure reliable operation of movable bridges through redundancy features in drive and control systems, for both new and rehabilitation projects.

B. Specific practices required by the Department in considering the applicability and requirements of bascule bridges:
   1.) Examine and evaluate alternative bridge configurations offering favorable life cycle cost benefits.

C. Consider improved design or operational characteristics providing advantage to the traveling public.
   1.) Incorporate design and operational features which are constructible and which can be operated and maintained by the Department's forces.
   2.) Maintain consistency of configuration, when feasible, for movable bridges throughout the State.

8.1.2 Redundancy

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

Commentary:
Redundant drive configurations include:

A. Hydraulic drive systems operated by multiple hydraulic cylinders. In these systems, a pump drive motor or its hydraulic pump can be isolated and bridge operations can continue while repairs are accomplished.

B. Gear driven systems that may be driven through one gear train into a single rack of a two-rack bridge.

Provide 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 one drive/motor system. Provide an alternator to alternate drives/motors for each opening.

Design hydraulic cylinder actuated movable bridges to function in spite of loss of a main pump motor, hydraulic pump, or drive cylinder.

Design the system to include all necessary valves, 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.

When operating with either a single rack drive functioning or unsymmetrical hydraulic cylinder forces applied to the leaf, design the structure for Movable Bridge - Specific load combinations, strength BV-I and BV-II. Reduce the load factors for strength BV-I to 1.35 [Table 2.4.2.3-1].

8.1.3 Trunnion Support Systems

Provide trunnion support systems as follows:

A. Simple, rotating trunnion configuration, with bearing supports, on towers, on both inboard and outboard sides of the trunnion girder.

B. Sleeve bearings should be considered for only small bascule bridges and must be approved by the SDO at the BDR phase. Design constraints and cost justification must be provided.

C. Design trunnion supports on each side of the main girder with similar stiffness vertically and horizontally.

8.1.4 Vertical Clearance Requirements
Design bascule leaf for unlimited vertical clearance between the fenders in the full open position. Any encroachment of the leaf into the horizontal clearance zone must receive favorable Coast Guard review prior to approval of the BDR.

8.1.5 Tender Parking
Provide two parking spaces for bridge tenders in all new bridge designs, preferably on the tender house side of the bridge.

8.1.6 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. Creep Speed: Not more than 10% of full speed, final creep speed will be determined by bridge conditions.
C. Emergency Stop: Leaf stops 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.
D. End-of-Travel Function: Contact connection where a closed contact allows operation and an open contact stops operation (i.e., leaf limit switches).
E. Fully Seated: Leaf is at rest on live load shoes, interlock OK to drive span locks.
F. Fully Open: Tip of leaf clears fender of a vertical line as defined by Coast Guard.
G. Hard Open: Leaf opening such that counterweight bumper blocks come in contact with pier bumper blocks.
H. Indicating Function: Contact connection where a closed contact indicates operation and an open contact indicates no operation (i.e., indicating lights.)
I. Interlocks or Safety Interlocks: Ensure events occur in sequence and no out-of-sequence events can occur.
J. Mid-Cycle Stop: Leaf(s) stop following normal ramping after depressing the STOP push-button when in the middle of an opening or closing cycle.
K. Near Closed: A point 8 to 10 degrees (approximately, final position to be field determined) before FULL CLOSED, drive to creep speed.
L. Near Open: A point 8 to 10 degrees (approximately, final position to be field determined) before FULL OPEN, drive to creep speed.
M. Ramp: Rate of acceleration or deceleration of leaf drive.

8.1.7 Movable Bridge Terminology
See Figure 8-2 for standard bridge terminology.

8.2 Construction Specifications and Design Calculations

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

Provide detailed calculations to justify all equipment and systems proposed. Submit calculations, for review, for all aspects of the systems and components. Substantiate recommended equipment and sizes. Submit calculations in an 8 ½ x 11-inch binder.

8.3 Special Strength Load Combination [2.4.2.2]

Evaluate the following owner-specified special design vehicle for Strength II Load Combination of the LRFD Specification as follows: Apply the full HL-93 loading to the design of the cantilever girders for bascule bridges, assuming the span locks are not engaged to transmit live load to the opposite leaf.

8.4 Speed Control for Leaf-driving Motors

Design a drive system that is capable of operating the leaf in no more than 70 seconds (See Figure 10-1) under normal conditions.
Clearly indicate on the plans the following required torques:

A. T_A - the maximum torque required to accelerate the leaf to meet the required time of operation.
B. T_S - the maximum torque required for starting the leaf.
C. T_{CV} - the maximum torque required for constant velocity.

8.4.1 Requirements for Mechanical Drive Systems

Specify a drive capable of developing the torques stated above and operating the leaf (at full speed) in the 70 seconds time limit stated above. Compute the acceleration torque for the inertia and the loading specified for the maximum constant velocity torque [5.4.2]. In addition the drive must be capable of holding the leaf (zero speed) for the required maximum staring torque [5.4.2].

8.4.2 Requirements for Hydraulic Drive Systems

Design a drive capable of developing the acceleration torque required for the inertia and the loading specified for the maximum constant velocity torque [5.4.2] and operating the leaf at full speed in the 70 seconds time limit stated above. Longer operating times are allowed for operation under abnormal conditions. Do not exceed 130 seconds under any condition.

8.5 Machinery Systems

8.5.1 Trunnions and Trunnion Bearings

A. Trunnions:

1.) Provide all shoulders with fillets of appropriate radius. Provide suitable provisions for expansion between shoulders and bearings.
2.) For keys and keyways see 6.7.8 and 6.7.10 of the AASHTO Specification.
3.) Do not use keyways between the trunnion and the trunnion hub/collar.
4.) For trunnions over 8 inch diameter, provide a hole 1/5 the trunnion diameter lengthwise through the center of the trunnion. Extend the trunnion at least 5/8 inch beyond the end of the trunnion bearings. Provide a 2-inch long counterbore concentric with the trunnion journals at each of the hollow trunnion ends.
5.) In addition to the shrink fit, drill dowels of appropriate size through the hub into the trunnion after the trunnion is in place.
6.) When anti-friction roller bearings are used, the surface finish must be compatible with 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.

7.) For rehabilitation of existing Hopkins trunnions, verify that trunnion eccentrics have capability for adjustment to accommodate required changes in trunnion alignment. If not, provide repair recommendations.

8.) If Hopkins trunnions are used, design three-piece eccentric assembly.

B. Hubs and Rings:

1.) Provide hubs and Rings with a mechanical shrink fit.

2.) See Figure 8-3, for minimum requirements.

8.5.2 Racks and Girders

Detail a mechanical, bolted connection between the rack and girder. Specify a machined finish for the connecting surfaces.

8.5.3 Span Balance [1.5]

A. New Construction:

1.) Design new bascule bridges such that the center of gravity may be adjusted vertically and horizontally.

2.) Design mechanical drive system bridges such that the center of gravity is forward (leaf heavy) of the trunnion or located at an angle (α) 30° to 50° above a horizontal line passing through the center of trunnion with the leaf in the down position. This will result in the leaf being tail (counterweight) heavy in the fully open position and leaf heavy in the fully closed position.

3.) Design hydraulic drive systems bridges such that the center of gravity is forward (leaf heavy) of the trunnion throughout the operating (opening) angle.

4.) Design both single and double leaf bascule for a leaf heavy out of balance condition which will produce an equivalent leaf reaction of 2 kips minimum) when the leaf is down. Design the live load shoe to resist this equivalent leaf reaction.

B. Rehabilitation Projects:

Apply the above provisions for 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 "W", "L" and "α".

Where: α = angle of inclination of the center of gravity above a horizontal line through the trunnion when the leaf is closed.

W = total weight of the leaf.

L = distance from the trunnion axis to the leaf center of gravity.

8.5.4 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:

A. Service Factor of 1.0 or higher indicating “Actual Input” and “Output Torque” requirements.
B. Cm and Km = per the current AGMA 6010 Standard.
C. Ca and Ka = 1.0
D. Cl and Kl = 1.0
E. Cr and Kr = 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," must conform to the current AGMA 6010 Standard for through hardened and for casehardened gears.

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

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

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

Provide a reducer capable of withstanding an overload torque of 300% of full-load motor torque. This torque must be greater than the maximum holding torque for the span under the maximum brake-loading conditions.

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

Specify units with means for filling and completely draining the case, that 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 must be of rugged construction and protected against breakage. Specify drains with shutoff valves to minimize spillage. Furnish each unit with a moisture trap breather of the desiccant type with color indicator to show desiccant moisture state.

Specify splash lubrication on the gears and bearings, unless otherwise required by the gear manufacturer. Do not specify pressurized lubrication systems on speed reducer boxes unless approved in writing by the Mechanical/Electrical Section of the SDO. Provide remote indication of pressurized lubrication system malfunction.

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

8.5.5 Open Gearing

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

8.5.6 Span Locks [6.8.1.5.1]

A. General:

1. Design span locks attached to the main bascule girders, if possible, giving consideration to maintenance access.
2. Specify a 4 x 6 inch minimum rectangular lock bars, unless analysis shows need for a larger size. Submit design calculations and the selection criteria for review and approval.
3. Install the bar in the guides and receivers with bronze wear fittings top and bottom, properly guided and shimmed. Provide lubrication 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 must be 0.010 to 0.025 inches. Specify the total horizontal clearance on the guides to be 1/16 ±1/32 inches.
4. Provide adequate stiffening behind the web for support of guides and receivers.
5. Mount guides and receivers with ½ inch minimum shims for adjusting. Slot wear-plate shims for insertion and removal.
6. Provide lubrication fittings at that are convenient for routine maintenance.
7. Mount actuation elements on the lock to activate limit switches to control each end of the stroke. Incorporate a means to adjust the limit switch actuation. Taper the receiver end of the lock-bar to facilitate insertion into the receivers of the opposite span.
8. Connection of the lock-bar to the hydraulic cylinder must 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 pinholes on male clevises.
9. 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. Provide a hydraulic hand pump and quick-disconnect fittings on the piping to allow pulling or driving of the lock-bar on loss of power. Specify the time of driving or pulling the bar at 5 to 9 seconds.
10. Design and specify access platforms with access hatches located out of the travel lanes.

B. Lock Design Standards:

1. The empirical formula, Equation 8-1 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.

   a. Where:

$$S = P/4 \left(A/L\right)^2 \left(3 - A/L\right)$$

   S = Shear in lock in kips for a given load on the span, “P.”
   A = Distance in feet from the support to the given load, “P.”
   L = Distance in feet from the support to the center lock.

   b. See Figure 8-4 for diagrammatic sketch of “S,” “A,” and “L.”

   c. Double the Dynamic Load Allowance (IM) to 66% for lock design.

2. Use a Dynamic Load Allowance of 100% for Lock Design on a double-leaf bascule span expected to carry traffic with ADTT (Average Daily Truck Traffic) > 2500.

8.5.7 Brakes

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

8.5.8 Couplings

Submit calculations and manufacturer’s literature for coupling sizes specified. Provide coupling schedule on plans.

8.5.9 Clutches

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

8.5.10 Bearings (Sleeve and Anti-Friction)

A. Sleeve Bearings must be grease-lubricated bronze bushings 8 inches in diameter or less and must 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 must be adaptor mounting, self-aligning, expansion and/or non-expansion types. Housings must be cast steel and capable of withstanding the design radial load in any direction, including uplift. The same supplier as the bearing must furnish housing. Specify bases cast without mounting holes. Mounting holes must be “drilled-to-fit” at the site at the time of assembly with the supporting steel work. Seals must retain the lubricant and exclude water and debris. Cap bolts on pillow blocks must be high-strength steel. The cap and cap bolts must be capable of resisting the rated bearing load as an uplift force. Where clearance or slotted holes are used, the clearance space must be filled with a non-shrink grout after alignment to insure satisfactory side load performance.

8.5.11 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 must be capable of developing the strength of reinforcement without damage to the concrete. All concrete anchors must be undercut bearing, expansion-type anchors. The anchorage must 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 must meet the ductile failure criteria of American Concrete Institute (ACI) Standard 349, Appendix B. Design an expansion anchoring system that can develop the tensile capacity of the bolt without slip or concrete failure. The bolt must 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 must 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 must be sufficient to assure steel failure, with concrete cone shear strength greater than the strength of the bolting material.

A. Anchor Bolt Design:

1.) Design anchor bolts subject to tension at 200% of the allowable basic stress and shown, by tests, to be capable of developing the strength of the bolt material without damage to concrete.

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

a. Tension, fs_{max} = 0.9fy
b. Compression and Bending, fs_{max} = 0.9fy
c. Shear, fs_{max} = 0.55fy (shear-friction provisions of ACI, Section 11.7, must apply)
d. The permissible design strength for the expansion anchor steel is reduced to 90% of the values for embedment steel.
e. For bolts and studs, the area of steel required for tension and shear based on the embedment criteria must be considered additive.
f. Calculate the design pullout strength of concrete, P_c, in pounds, as:

$$P_c = 3.96\phi\sqrt{f'_c}A$$

g. Where:

Ø = Capacity reduction factor, 0.65
A = Projected effective area of the failure cone, in²
f'_c = Specified compressive strength of concrete, psi

Steel strength controls when the design pullout strength of the concrete, Pc, exceeds the minimum ultimate tensile strength of the bolt material.
The effective stress area is 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 must 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 must 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.

8.5.12 Fasteners

A. All bolts for connecting machinery parts to each other and to supporting members must be as shown on the plans or specified otherwise and conform to one of the following types:
   1.) High-strength bolts.
   2.) Turned bolts, turned cap screws, and turned studs.
   3.) High-strength turned bolts, turned cap screws, and turned studs.

B. High strength bolts must meet the requirements of ASTM A325. High strength bolts must have regular hexagonal heads. Holes for high-strength bolts must be no more than 0.010 inch [0.25 mm] larger than the actual diameter of individual bolts and drilling holes to match the tolerances for each bolt. The clearance must be checked with 0.011-inch [0.28 mm] wire. The hole must be considered too large if the wire can be inserted in the hole together with the bolt. Install, wherever possible, high-strength bolts connecting machinery components to structural elements or to other machinery components comprised of different thicknesses such that the bolt head is adjacent to the connected element with the least thickness.

C. Turned bolts, turned cap screws, and turned studs must have turned shanks and cut threads. Turned bolts must have semi-finished, washer-faced, hexagonal heads and nuts. All finished shanks of turned fasteners must be 0.06 inches [1.6 mm] larger in diameter than the diameter of the thread, which must determine the head and nut dimensions.

D. Drill or ream-assemble all elements connected by bolts to assure accurate alignment of the hole and accurate clearance over the entire length of the bolt within the specified limits.

E. Provide bolt heads, nuts, castle nuts, and hexagonal head cap screws dimensioned in accordance with ANSI Standard B18.2.3, Hexagon Bolts and ANSI Standard B18.2.4.6 Nuts.

F. Provide heavy series heads and nuts for turned bolts, screws, and studs.

G. ASTM A325 bolts must have regular hex heads.

H. The dimensions of socket-head cap screws, socket flathead cap screws, and socket-set screws must conform to ANSI Standard B18.3.1. Provide screws made of heat-treated alloy steel, cadmium-plated, and furnished with a self-locking nylon pellet embedded in the threaded section. Unless otherwise called for on the plans or specified herein, provide setscrews of the headless safety type with threads of coarse thread series and cup points. Do not use setscrews to transmit torsion nor as the fastening or stop for any equipment that contributes to the stability or operation of the bridge.
I. Threads for cap screws must conform to the coarse thread series and must have a Class 6g tolerance. For bolts and nuts, the bolt must conform to the coarse thread series and must have a Class 6g tolerance. The nut must have a Class 6H in accordance with the ANSI Standard B18.2.6 Screw Threads [ANSI Standard B18.2.6M Metric Screw Threads].
J. Spotface bolt holes through unfinished surfaces. Insure the head and nut are square with the axis of the hole.
K. Unless otherwise called for, sub drill all bolt holes in machinery parts or connecting these parts to the supporting steel work at least 0.03 in [0.80 mm] smaller in diameter than the bolt diameter. Ream for the proper fit at assembly or at erection with the steel work after the parts are correctly assembled and aligned.
L. Ream or drill holes in shims and fills for machinery parts to the same tolerances as the connected parts at final assembly.
M. Furnish positive locks of an approved type for all nuts except those on ASTM A325 Bolts. If double nuts are used, they must be used for all connections requiring occasional opening or adjustment. Provide lock washers made of tempered steel if used for securing.
N. Install high-Strength bolts with a hardened plain washer meeting ASTM F436 at each end.
O. Wherever possible, insert high strength bolts connecting machinery parts to structural parts or other machinery parts through the thinner element into the thicker element.
P. Provide cotters that conform to SAE standard dimensions and are made of half-round stainless steel wire, ASTM A276, Type 316.
Q. Anchor bolts connecting machinery parts to masonry must be ASTM A325 material, hot-dipped galvanized per ASTM A153 unless otherwise approved by the Engineer. Show bolts on the masonry drawings. Anchor bolts for new construction must be cast in place and not drilled. The material and loading requirements must be as shown on the plans. The filler material may be a non-shrink grout, Babbitt metal, or zinc.
R. All fasteners must be of United States manufacture and must be clearly marked with the manufacturer’s designation.

8.6 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. Calculate pressure drops for all components of their circuits including valves, filters, hoses, piping, manifolds, flow meters, fittings, etc. Power requirements must 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 2500 psi. During short periods of time in emergency operations, pressure can increase to 3000 psi, maximum. Correlate hydraulic system strength calculations 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 must 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 170° F during the most adverse ambient temperature conditions anticipated.

8.6.1 Hydraulic Pumps

Specify minimum pressure rating of pumps to be 1.5 times the maximum operating pressure. Specify pumps of the Pressure Compensated type. Variation of the pressure setting, including ±50 cst viscosity change must be ±2.5% maximum. Overall minimum efficiency must be 0.86. Boost pumps of any power, and auxiliary or secondary pumps less than 5 hp [3.7 kW], need not be pressure compensated.

8.6.2 Cylinders

Design the hydraulic cylinder drive systems to prevent sudden closure of valves, and subsequent sudden locking of cylinders, in the event of a power failure or emergency stop. Specify cylinders designed according to ASME Pressure Vessel Code, Section 8. Specify cylinders with a minimum theoretical static failure pressure rating of 11 (10,000 psi) as defined by NFPA Standards; and designed to operate on biodegradable-based hydraulic fluid unless otherwise approved by the Mechanical/Electrical Section of the SDO.

Specify stainless steel rods with chrome plated finish 0.005 to 0.012 inches thick per QQ-C-320C, Class 2a or others as approved by the Mechanical/Electrical Section of the SDO.

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

8.6.3 Control Components

A. Flow Control Valves: Use of non-compensated flow control valves must 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 is 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.

8.6.4Hydraulic 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 must be 4. Provide calculations indicating that the velocity of fluid is at or below 4.3 ft/s in suction lines, 6.5 ft/s in return lines, and 21.5 ft/s in pressure lines.

B. Manifolds: Specify the use of manifolded components.

C. Flexible Hose: Specify flexible hose only in cases where motion or vibration makes the use of rigid piping undesirable. Insure that the minimum ratio of burst pressure rating divided by design pressure in the line is 4.

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

8.6.5 Miscellaneous Hydraulic Components

A. Receivers (Reservoirs): Tanks in open-loop systems must 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 170° F. Tanks in closed-loop hydrostatic systems must circulate, filter, and cool enough oil to maintain a maximum oil temperature of 170° F. Specify suction port strainers with oil shut-off valves. Specify tanks with easy drainage and provided with adequate openings that allow easy cleaning of all surfaces from the inside. Specify sumps with magnetic traps to capture metal particles. Specify Stainless Steel ASTM A316L tank material. Specify the use of air bladders to avoid water contamination from air moisture condensation due to the breathing effect of the tank.

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

C. Hydraulic Fluids: Insure that the manufacturers of the major hydraulic components used in the bridge approve the hydraulic fluid specified for use.

8.7 Electrical [AASHTO 8]

8.7.1 Electrical Service [AASHTO 8.1]

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

8.7.2 Alternating Current Motors [AASHTO 8.5]

Size and select motors per AASHTO requirements. On hydraulic systems provide 25% spare motor capacity. Specify motors that 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. Testing Procedure: ANSI/IEEE 112, Test Method B. Load test motors to determine freedom from electrical or mechanical defects and compliance with performance data.
D. 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 10 Hp [7.5 kW] and larger must be TEFC.
E. Thermistor System (Motor Sizes 25 Hp [19 kW] and Larger): Three PTC thermistors embedded in motor windings and epoxy encapsulated solid-state control relay for wiring into motor starter.
F. 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 centerline at end of NEMA standard shaft extension. Stamp bearing sizes on nameplate.
G. Nominal Efficiency: Meet or exceed values in ANSI Schedules at full load and rated voltage when tested in accordance with ANSI/IEEE 112.
H. Nominal Power Factor: Meet or exceed values in ANSI Schedules at full load and rated voltage when tested in accordance with ANSI/IEEE 112.
I. Insulation System: NEMA Class F or better.
J. Service Factor: 1.15 (except when calculating motor size, use 1.0).

8.7.3 Engine Generators [AASTHO 8.3.9]

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 50 gallons). Specify day tank with a minimum 10-gallon capacity. Do not use the day tank capacity as part of the main tank capacity. Submit calculations justifying generator size recommended.

8.7.4 Automatic Transfer Switch [AASHTO 8.3.8]

Design switch in conformance with the requirements of the latest edition of NFPA 110. Specify Automatic Transfer Switch with engine generator. Specify an Automatic Transfer Switch that is fully rated to protect all types of loads, inductive and resistive, from loss of continuity of power, without de-rating, either open or enclosed.
Specify withstand, closing, and interrupting ratings sufficient for voltage of the system and the available short circuit at the point of application on the drawings. Provide short circuit calculations to justify ATS proposed.

8.7.5 Electrical Control [AASHTO 8.4]

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 that will comply with the Operation Sequence furnished by the Mechanical/Electrical Section of the SDO.
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 seconds main drives only. In an emergency, hit this button to stop machinery and prevent damage or injury. Specify a button that is reset by twisting clockwise (or counterclockwise) to release to normal up position.
At a minimum, provide alarms for the following events:

A. All bridge control failures.
B. All generator/Automatic Transfer Switch failures.
C. All traffic signal failures.
D. All navigation light failures.
E. All traffic gate failures.
F. All span-lock failures.
G. All brake failures (if applicable).
H. All leaf limit switch failures.
I. All drive failures; including motor high temperature (motors larger than 25 Hp) and all hydraulic system failures.
J. Near and far-leaf total openings (not an alarm but part of the monitoring function).
K. All uses of bypass functions, type and time (not an alarm but part of the monitoring function).

8.7.6 Motor Controls [AASHTO 8.6]

Specify full-size NEMA rated starters. Do not use IEC starters unless space constraints require their use, and then, only by obtaining prior approval from the Mechanical/Electrical Section of the SDO.
Do not include panelboards and transformers in the Motor Control Center (MCC) unless space constraints require it, and then, only by obtaining prior approval from the Mechanical/Electrical Section of the SDO.

8.7.7 Programmable Logic Controllers [AASHTO 8.4.2.3]

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

8.7.8 Limit and Seating Switches [AASHTO 8.4.1]

Label limit switches as follows:

LEAF LIMIT SWITCHES

LS-FOFS	**F**ar **O**pposite bascule leaf **F**ULLY **S**EATED. Normally open (NO), closes when leaf is on live load shoes.
LS-FOFC	Far Opposite bascule leaf FULLY CLOSED. Normally open (NO) held closed by leaf until drive needs to go to 0 speed (Closing Cycle).
LS-FONCA	Far Opposite bascule leaf NEAR CLOSED switch "A". Normally open (NO) held closed by leaf until drive needs to go to creep speed (Closing Cycle).
LS-FONCB	Far Opposite bascule leaf NEAR CLOSED switch "B". Normally open (NO) held closed by leaf until drive needs to go to creep speed (Closing Cycle).
LS-FONOA	Far Opposite bascule leaf NEAR OPEN switch "A". Normally open (NO) held closed by leaf until drive needs to go to creep speed (Opening Cycle).
LS-FONOB	Far Opposite bascule leaf NEAR OPEN switch "B". Normally open (NO) held closed by leaf until drive needs to go to creep speed (Opening Cycle).
LS-FOFO	Far Opposite bascule leaf FULLY OPEN. Normally open (NO) held closed by leaf until drive needs to go to 0 speed (Opening Cycle).
LS-FAFS	Far Adjacent bascule leaf FULLY SEATED. Normally open (NO), closes when leaf is on live load shoes.
LS-FAFC	Far Adjacent bascule leaf FULLY CLOSED. Normally open (NO) held closed by leaf until drive needs to go to 0 speed.
LS-FANCA	Far Adjacent bascule leaf NEAR CLOSED switch "A". Normally open (NO) held closed by leaf until drive needs to go to creep speed (Closing Cycle).
LS-FANCB	Far Adjacent bascule leaf NEAR CLOSED switch "B". Normally open (NO) held closed by leaf until drive needs to go to creep speed (Closing Cycle).
LS-FANOA	Far Adjacent bascule leaf NEAR OPEN switch "A". Normally open (NO) held closed by leaf until drive needs to go to creep speed (Opening

	Cycle).
LS-FANOB	Far Adjacent bascule leaf NEAR OPEN switch "B". Normally open (NO) held closed by leaf until drive needs to go to creep speed (Opening Cycle).
LS-FAFO	Far Adjacent bascule leaf FULL OPEN. Normally open (NO) held closed by leaf until drive needs to go to 0 speed (Opening Cycle).
LS-NOFS	Near Opposite bascule leaf FULLY SEATED. Normally open (NO), closes when leaf is on live load shoes.
LS-NOFC	Near Opposite bascule leaf FULLY CLOSED. Normally open (NO) held closed by leaf until drive needs to go to 0 speed.
LS-NONCA	Near Opposite bascule leaf NEAR CLOSED switch "A". Normally open (NO) held closed by leaf until drive needs to go to creep speed (Closing Cycle).
LS-NONCB	Near Opposite bascule leaf NEAR CLOSED switch "B". Normally open (NO) held closed by leaf until drive needs to go to creep speed (Closing Cycle).
LS-NONOA	Near Opposite bascule leaf NEAR OPEN switch "A". Normally open (NO) held closed by leaf until drive needs to go to creep speed (Opening Cycle).
LS-NONOB	Near Opposite bascule leaf NEAR OPEN switch "B". Normally open (NO) held closed by leaf until drive needs to go to creep speed (Opening Cycle).
LS-NOFO	Near Opposite bascule leaf FULLY OPEN. Normally open (NO) held closed by leaf until drive needs to go to 0 speed (Opening Cycle).
LS-NAFS	Near Adjacent bascule leaf FULLY SEATED. Normally open (NO), closes when leaf is on live load shoes.
LS-NAFC	Near Adjacent bascule leaf FULLY CLOSED. Normally open (NO) held closed by leaf until drive needs to go to 0 speed.
LS-NANCA	Near Adjacent bascule leaf NEAR CLOSED switch "A". Normally open (NO) held closed by leaf until drive needs to go to creep speed (Closing Cycle).
LS-NANCB	Near Adjacent bascule leaf NEAR CLOSED switch "B". Normally open (NO) held closed by leaf until drive needs to go to creep speed (Closing Cycle).
LS-NANOA	Near Adjacent bascule leaf NEAR OPEN switch "A". Normally open (NO) held closed by leaf until drive needs to go to creep speed (Opening Cycle).
LS-NANOB	Near Adjacent bascule leaf NEAR OPEN switch "B". Normally open (NO) held closed by leaf until drive needs to go to creep speed (Opening Cycle).
LS-NAFO	Near Adjacent bascule leaf FULL OPEN. Normally open (NO) held closed by leaf until drive needs to go to 0 speed (Opening Cycle).

TRAFFIC GATE LIMIT SWITCHES

LS-FOCTGD	**F**ar **O**n-**C**oming **T**raffic **G**ate **D**own.
LS-FOCTGDI	Far On-Coming Traffic Gate Down Interlock.
LS-FOCTGU	Far On-Coming Traffic Gate Up.
LS-FOCTGUI	Far On-Coming Traffic Gate Up Interlock.
LS-FOCTGH	Far On-Coming Traffic Gate Handcrank (Normally Closed, opens when hand crank is inserted in traffic gate motor).
LS-FOCTGOD	Far On-Coming Traffic Gate Operator Door Switch
LS-FOGTGD	Far Off-Going Traffic Gate Down.
LS-FOGTGDI	Far Off-Going Traffic Gate Down Interlock.
LS-FOGTGU	Far Off-Going Traffic Gate Up.
LS-FOGTGUI	Far Off-Going Traffic Gate Up Interlock.
LS-FOGTGH	Far Off-Going Traffic Gate Handcrank (Normally Closed, opens when hand crank is inserted in traffic gate motor).
LS-FOGTGOD	Far Off-Going Traffic Gate Operator Door Switch
LS-NOCTGD	Near On-Coming Traffic Gate Down.
LS-NOCTGDI	Near On-Coming Traffic Gate Down Interlock.
LS-NOCTGU	Near On-Coming Traffic Gate Up.
LS-NOCTGUI	Near On-Coming Traffic Gate Up Interlock.
LS-NOCTGH	Near On-Coming Traffic Gate Handcrank (Normally Closed, opens when hand crank is inserted in traffic gate motor).
LS-NOCTGOD	Near On-Coming Traffic Gate Operator Door Switch
LS-NOGTGD	Near Off-Going Traffic Gate Down.
LS-NOGTGDI	Near Off-Going Traffic Gate Down Interlock.
LS-NOGTGU	Near Off-Going Traffic Gate Up.
LS-NOGTGUI	Near Off-Going Traffic Gate Up Interlock.
LS-NOGTGH	Near Off-Going Traffic Gate Handcrank (Normally Closed, opens when hand crank is inserted in traffic gate motor).
LS-NOGTGOD	Near Off-Going Traffic Gate Operator Door Switch.

BARRIER GATES LIMIT SWITCHES (If Present)

LS-FOCTBU	**F**ar **O**n-**C**oming **T**raffic **B**arrier Gate **U**p Limit Switch
LS-FOCTBUI	Far On-Coming Traffic Barrier Gate Up Interlock Limit Switch
LS-FOCTBD	Far On-Coming Traffic Barrier Gate Down Limit Switch
LS-FOCTBDI	Far On-Coming Traffic Barrier Gate Down Interlock Limit Switch
LS-FOCTBH	Far On-Coming Traffic Barrier Gate Handcrank Inserted Limit Switch
LS-FOCTBOD	Far On-Coming Traffic Barrier Gate Operator Door Switch
LS-NOCTBU	Near On-Coming Traffic Barrier Gate Up Limit Switch

LS-NOCTBUI	Near On-Coming Traffic Barrier Gate Up Interlock Limit Switch
LS-NOCTBD	Near On-Coming Traffic Barrier Gate Down Limit Switch
LS-NOCTBDI	Near On-Coming Traffic Barrier Gate Down Interlock Limit Switch
LS-NOCTBH	Near On-Coming Traffic Barrier Gate Handcrank Inserted Limit Switch
LS-NOCTBOD	Near On-Coming Traffic Barrier Gate Operator Door Switch
LS-FOGTBU	Far Off-Going Traffic Barrier Gate Up Limit Switch
LS-FOGTBUI	Far Off-Going Traffic Barrier Gate Up Interlock Limit Switch
LS-FOGTBD	Far Off-Going Traffic Barrier Gate Down Limit Switch
LS-FOGTBDI	Far Off-Going Traffic Barrier Gate Down Interlock Limit Switch
LS-FOGTBH	Far Off-Going Traffic Barrier Gate Handcrank Inserted Limit Switch
LS-FOGTBOD	Far Off-Going Traffic Barrier Gate Operator Door Switch
LS-NOGTBU	Near Off-Going Traffic Barrier Gate Up Limit Switch
LS-NOGTBUI	Near Off-Going Traffic Barrier Gate Up Interlock Limit Switch
LS-NOGTBD	Near Off-Going Traffic Barrier Gate Down Limit Switch
LS-NOGTBDI	Near Off-Going Traffic Barrier Gate Down Interlock Limit Switch
LS-NOGTBH	Near Off-Going Traffic Barrier Gate Handcrank Inserted Limit Switch
LS-NOGTBOD	Near Off-Going Traffic Barrier Gate Operator Door Switch

SIDEWALK GATES LIMIT SWITCHES (If Present)

LS-FOCSGO	**F**ar **O**n-**C**oming **S**idewalk **G**ate **O**pen Limit Switch
LS-FOCSGOI	Far On-Coming Sidewalk Gate Open Interlock Limit Switch
LS-FOCSGC	Far On-Coming Sidewalk Gate Closed Limit Switch
LS-FOCSGCI	Far On-Coming Sidewalk Gate Closed Interlock Limit Switch
LS-FOCSGH	Far On-Coming Sidewalk Gate Handcrank Inserted Limit Switch
LS-FOCSGOD	Far On-Coming Sidewalk Gate Operator Door Switch
LS-NOCSGO	Near On-Coming Sidewalk Gate Open Limit Switch
LS-NOCSGOI	Near On-Coming Sidewalk Gate Open Interlock Limit Switch
LS-NOCSGC	Near On-Coming Sidewalk Gate Closed Limit Switch

LS-NOCSGCI	Near On-Coming Sidewalk Gate Closed Interlock Limit Switch
LS-NOCSGH	Near On-Coming Sidewalk Gate Handcrank Inserted Limit Switch
LS-NOCSGOD	Near On-Coming Sidewalk Gate Operator Door Switch
LS-FOGSGO	Far Off-Going Sidewalk Gate Open Limit Switch
LS-FOGSGOI	Far Off-Going Sidewalk Gate Open Interlock Limit Switch
LS-FOGSGC	Far Off-Going Sidewalk Gate Closed Limit Switch
LS-FOGSGCI	Far Off-Going Sidewalk Gate Closed Interlock Limit Switch
LS-FOGSGH	Far Off-Going Sidewalk Gate Handcrank Inserted Limit Switch
LS-FOGSGOD	Far Off-Going Sidewalk Gate Operator Door Switch
LS-NOGSGO	Near Off-Going Sidewalk Gate Open Limit Switch
LS-NOGSGOI	Near Off-Going Sidewalk Gate Open Interlock Limit Switch
LS-NOGSGC	Near Off-Going Sidewalk Gate Closed Limit Switch
LS-NOGSGCI	Near Off-Going Sidewalk Gate Closed Interlock Limit Switch
LS-NOGSGH	Near Off-Going Sidewalk Gate Handcrank Inserted Limit Switch
LS-NOGSGOD	**N**ear **O**ff-**G**oing **S**idewalk **G**ate **O**perator **D**oor Switch

SPAN LOCK LIMIT SWITCHES

LS-SLAD	**S**pan **L**ock **A** **D**riven Limit Switch
LS-SLADI	Span Lock A Driven Interlock Limit Switch
LS-SLAP	Span Lock A Pulled Limit Switch
LS-SLAPI	Span Lock A Pulled Interlock Limit Switch
LS-SLAH or LS-SLAOP	Span Lock A Handcrank Inserted Limit Switch or Over Pressure Switch
LS-SLBD	Span Lock B Driven Limit Switch
LS-SLBDI	Span Lock B Driven Interlock Limit Switch
LS-SLBP	Span Lock B Pulled Limit Switch
LS-SLBPI	Span Lock B Pulled Interlock Limit Switch
LS-SLBH or LS-SLBOP	Span Lock B Handcrank Inserted Limit Switch or Over Pressure Switch
LS-SLCD	Span Lock C Driven Limit Switch
LS-SLCDI	Span Lock C Driven Interlock Limit Switch
LS-SLCP	Span Lock C Pulled Limit Switch
LS-SLCPI	Span Lock C Pulled Interlock Limit Switch
LS-SLCH or LS-SLCOP	Span Lock C Handcrank Inserted Limit Switch or Over Pressure Switch
LS-SLDD	Span Lock D Driven Limit Switch
LS-SLDDI	Span Lock D Driven Interlock Limit Switch

LS-SLDP	Span Lock D Pulled Limit Switch
LS-SLDPI	Span Lock D Pulled Interlock Limit Switch
LS-SLDH or LS-SLDOP	Span Lock D Handcrank Inserted Limit Switch or Over Pressure Switch

TAIL LOCK LIMIT SWITCHES (If Present)

LS-TLAD	**T**ail **L**ock "**A**" bar **D**riven Limit Switch.
LS-TLADI	Tail Lock "A" bar Driven Interlock Limit Switch.
LS-TLAP	Tail Lock "A" bar Pulled Limit Switch.
LS-TLAPI	Tail Lock "A" bar Pulled Interlock Limit Switch.
LS-TLAHC	Tail Lock "A" Hand crank Coupled (Normally Closed, opens when hand crank is inserted in lock motor).
LS-TLBD	Tail Lock "B" bar Driven Limit Switch.
LS-TLBDI	Tail Lock "B" bar Driven Interlock Limit Switch.
LS-TLBP	Tail Lock "B" bar Pulled Limit Switch.
LS-TLBPI	Tail Lock "B" bar Pulled Interlock Limit Switch.
LS-TLBHC	Tail Lock "B" Hand crank Coupled (Normally Closed, opens when hand crank is inserted in lock motor).
LS-TLCD	Tail Lock "C" bar Driven Limit Switch.
LS-TLCDI	Tail Lock "C" bar Driven Interlock Limit Switch.
LS-TLCP	Tail Lock "C" bar Pulled Limit Switch.
LS-TLCPI	Tail Lock "C" bar Pulled Interlock Limit Switch.
LS-TLCHC	Tail Lock "C" Hand crank Coupled (Normally Closed, opens when hand crank is inserted in lock motor).
LS-TLDD	Tail Lock "D" bar Driven Limit Switch.
LS-TLDDI	Tail Lock "D" bar Driven Interlock Limit Switch.
LS-TLDP	Tail Lock "D" bar Pulled Limit Switch.
LS-TLDPI	Tail Lock "D" bar Pulled Interlock Limit Switch.
LS-TLDHC	Tail Lock "D" Hand crank Coupled (Normally Closed, opens when hand crank is inserted in lock motor).

BRAKE LIMIT SWITCHES

LS-FAOABR	**F**ar **A**djacent Motor **A** **B**rake **R**eleased Limit Switch
LS-FAOABMR	Far Adjacent Motor A Brake Manually Released Limit Switch
LS-FAOABS	Far Adjacent Motor A Brake Set Limit Switch
LS-FAOBBR	Far Adjacent Motor B Brake Released Limit Switch
LS-FAOBBMR	Far Adjacent Motor B Brake Manually Released Limit Switch
LS-FAOBBS	Far Adjacent Motor B Brake Set Limit Switch
LS-FAABR	Far Adjacent Machinery Brake Released Limit Switch
LS-FAABMR	Far Adjacent Machinery Brake Manually Released Limit Switch

LS-FAABS	Far Adjacent Machinery Brake Set Limit Switch
LS-NAOABR	Near Adjacent Motor A Brake Released Limit Switch
LS-NAOABMR	Near Adjacent Motor A Brake Manually Released Limit Switch
LS-NAOABS	Near Adjacent Motor A Brake Set Limit Switch
LS-NAOBBR	Near Adjacent Motor B Brake Released Limit Switch
LS-NAOBBMR	Near Adjacent Motor B Brake Manually Released Limit Switch
LS-NAOBBS	Near Adjacent Motor B Brake Set Limit Switch
LS-NAABR	Near Adjacent Machinery Brake Released Limit Switch
LS-NAABMR	Near Adjacent Machinery Brake Manually Released Limit Switch
LS-NAABS	Near Adjacent Machinery Brake Set Limit Switch
LS-FOOABR	Far Opposite Motor A Brake Released Limit Switch
LS-FOOABMR	Far Opposite Motor A Brake Manually Released Limit Switch
LS-FOOABS	Far Opposite Motor A Brake Set Limit Switch
LS-FOOBBR	Far Opposite Motor B Brake Released Limit Switch
LS-FOOBBMR	Far Opposite Motor B Brake Manually Released Limit Switch
LS-FOOBBS	Far Opposite Motor B Brake Set Limit Switch
LS-FOABR	Far Opposite Machinery Brake Released Limit Switch
LS-FOABMR	Far Opposite Machinery Brake Manually Released Limit Switch
LS-FOABS	Far Opposite Machinery Brake Set Limit Switch
LS-NOOABR	Near Opposite Motor A Brake Released Limit Switch
LS-NOOABR	Near Opposite Motor A Brake Manually Released Limit Switch
LS-NOOABS	Near Opposite Motor A Brake Set Limit Switch
LS-NOOBBR	Near Opposite Motor B Brake Released Limit Switch
LS-NOOBBMR	Near Opposite Motor B Brake Manually Released Limit Switch
LS-NOOBBS	Near Opposite Motor B Brake Set Limit Switch
LS-NOABR	Near Opposite Machinery Brake Released Limit Switch
LS-NOABMR	Near Opposite Machinery Brake Manually Released Limit Switch
LS-NOABS	**N**ear **O**pposite Machinery **B**rake **S**et **L**imit **S**witch

AUXILIARY DRIVE LIMIT SWITCHES (If Present)

LS-FOADC	**F**ar **O**pposite **A**uxiliary **D**rive Coupled (Normally Closed, opens when drive is coupled).
LS-FAADC	**F**ar **A**djacent **A**uxiliary **D**rive Coupled (Normally Closed, opens when

drive is coupled).

LS-NOADC **N**ear **O**pposite **A**uxiliary **D**rive **C**oupled (Normally Closed, opens when drive is coupled).

LS-NAADC **N**ear **A**djacent **A**uxiliary **D**rive **C**oupled (Normally Closed, opens when drive is coupled).

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

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

Do not connect limit switches in series. Connect each limit switch 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.
Do not use electronic limit switches. Plunger type switches are optional.

8.7.9 Safety Interlocking [AASHTO 8.4.1]

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.

8.7.10 Instruments [AASHTO 8.4.5]
Provide wattmeter for each drive (pump) motor and provide leaf position indication for each leaf.

8.7.11 Control Console [AASHTO 8.4.6]
Provide a Control Console that contains 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 must be broken across approved terminals.

No components other than push buttons, selector switches, indicating lights, terminal blocks, etc., must be allowed in the Control Console. Refer to the Technical Special Provisions issued by the Mechanical/Electrical Section of the SDO.

8.7.12 Electrical Connections between Fixed and Moving Parts
AASHTO 8.9.5] Use extra flexible wire or cable.

8.7.13 Electrical Connections across the Navigable Channel
Specify a submarine cable assembly consisting of the following three separate cables:

A. Power Cable: Jacketed and armored with twelve (12) #4 AWG copper and twelve (12) #10 AWG copper conductors.
B. Signal and Control Cable: Jacketed and armored with fifty (50) #12 AWG copper and five (5) pairs of twisted shielded #14 AWG copper conductors.
C. 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 5 feet below the mean low tide to a point 5 feet above mean high tide. Specify a watertight seal at both ends of the sleeve.

8.7.14 Service Lights [AASHTO 8.11]
Provide minimum of 30-foot candles in all areas of the machinery platform.

8.7.15 Navigation Lights [AASHTO 1.4.4.6.2]

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.

8.7.16 Grounding and Lightning Protection [AASHTO 8.12 and 8.13]

Provide the following systems:

A. Lightning Protection System: Design per the requirements of NFPA 780 Lightning Protection Code. Protect the bridge with Class II materials.
B. Surge Suppression System: Design Transient Voltage Surge Suppressor (TVSS) system to protect all power, control, signaling, and communication circuits and all submarine conductors that enter or leave the control house. It is imperative to maintain proper segregation of protected and non-protected wiring within the Bridge Tender House. Refer to the Technical Special Provisions issued by the Mechanical/Electrical Section of the SDO.
C. Grounding and Bonding System: All equipment installed on the bridge/project must 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 must 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. Test all driven grounds to a maximum of 5 ohms to ground.

All main connections to the copper-bonding conductor must be cad-welded.

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 5 feet.

8.7.17 Movable Bridge Traffic Signals and Safety Gates [AASHTO 1.4.4]

Refer to **Roadway Standard Index** No. 17890 for Traffic Control Devices for Movable Span Bridge Signals.

8.7.18 Communications Systems [N/A]

Design and specify a Public Address System, an Intercom System, and a Marine Radio System for each movable bridge. The three systems must work independent of each other and meet the following criteria:

A. Public Address System: One-way handset communication from the operators console to multiple zones (marine channel, roadway, machinery platforms, and other rooms). Specify an all call feature so that the operator may call all 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 that 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 must have a hands free capability. A call initiated from one station to another must open a channel and give a tone at the receiving end. The receiving party must 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 must be capable of operation in a high noise, salt air environment. A handset must be mounted adjacent to the control 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 120-volt charger located adjacent to the operator's console.

8.7.19 Functional Checkout [N/A]

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 consists of two parts. Perform the first part before delivery, and the second part after installation on the bridge. Both tests must be comprehensive. Perform the off-site functional testing to verify that all equipment is functioning as intended.

All repairs or adjustments must be made before installation on Department property. All major electrical controls must be assembled and tested in one place, at one time. The test must include as a minimum: control console, PLC, relay back-up system, Motor Control Center, and all other equipment required, in the opinion of the Electrical Engineer of Record, to complete the testing to the satisfaction of the Mechanical/Electrical Section of the SDO.

If not satisfactory, repeat the testing at no cost to the Department. All equipment must be assembled and inter-connected (as they would be on the bridge) to simulate bridge operation. No inputs or outputs must be forced. Indication lights must be provided to show operation and hand operated toggle switches may be used to simulate field limit switches.

After the off-site testing is completed to the satisfaction of the Mechanical/Electrical Section of the SDO, the equipment may be delivered and installed. The entire bridge control system must be re-tested before the bridge is put into service. The field functional testing must include, but is not necessarily limited to, the off-site testing procedure.

As a minimum, perform the following tests of Control Functions for both manual and semi-automatic operations:

Commentary:
The Electrical Engineer of Record is encouraged to include tests for other equipment not included in the minimum tests listed below.

A. Demonstrate the correct operation of the bridge sequence as described in the Technical Special Provisions and in the drawings.
B. Demonstrate EMERGENCY STOP of each span (leaf) at, or during, each phase of opening and closing the bridge (phases include ramping up or down, full-speed, and creep-speed).
C. Demonstrate EMERGENCY STOP does prevent energization of all rotating machinery in any mode of operation.
D. Demonstrate that the leaf (s) do not come to a sudden stop on a power failure.
E. Interlocks:
   1. Simulate the operation of each limit switch to demonstrate correct operation and interlocking of systems.
   2. Demonstrate BYPASS operation for each failure for each required bypass.
   3. Simulate each failure for which there is an alarm message to demonstrate correct message displays.
   4. Testing of interlocks must be sufficient to demonstrate that unsafe or out of sequence operations are prevented.
   5. Observe Position Indicator readings with bridge closed and full open to assure correct readings.
F. 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.
G. 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.
H. 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.
I. Emergency Power:
   1. The complete installation must 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 must 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 must notify the Engineer in advance and perform a field test to demonstrate load-carrying capability, stability, voltage, and frequency.
3. Specify a dielectric absorption test on generator winding with respect to ground. A polarization index must be determined and recorded. Submit copies of test results to the Engineer.
4. Phase rotation test must be made to determine compatibility with load requirements.
5. Engine shutdown features such as low oil pressure, over-temperature, over-speed, over-crank, and any other feature as applicable must be function-tested.
6. In the presence of the Engineer, perform resistive load bank tests at one hundred percent (100%) nameplate rating. Loading must be 25%-rated for 30 minutes, 50%-rated for 30 minutes, 75%-rated for 30 minutes, and 100%-rated for 2 hours. Maintain records throughout this period and record water temperature, oil pressure, ambient air temperature, voltage, current, frequency, kilowatts, and power factor. The above data must be recorded at 15-minute intervals throughout the test.

J. 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.

K. Programmable Logic Controller (PLC) Program:

1. Demonstrate the completed program's capability prior to installation or connection of the system to the bridge. Arrange and schedule the demonstration with the Engineer and the Electrical Engineer of Record.
2. A detailed field test procedure must be written and provided to the Electrical Engineer of Record for approval. Provide for testing as listed below:

a. Exercise all remote limit switches to simulate faults including locks, gates, traffic lights, etc. Readouts must appear on the alphanumeric display.
b. 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.
c. Initiate a PLC sequence of operation interweaving the by-pass functions with the semi-automatic functions for all remote equipment.
d. 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.

e. 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.

8.8 Control House Architectural Design

A control house is the 'building' designed as part of a movable bridge (Bascule, 'drawbridge', etc.) which is occupied by the bridge operator. This facility houses the business functions, and mechanical & electrical systems used to operate the bridge. This includes equipment such as pumps, motors, generators, etc. and systems such as controls, lighting, plumbing, and HVAC.

The design of new control houses and renovation of existing control houses must comply with the requirements of the FLORIDA BUILDING CODE.

A. Operation areas contain business functions.
B. Equipment areas contain mechanical and electrical equipment.

8.8.1 General

A. These guidelines are intended primarily for use in the design of new control houses but many items apply to renovations of existing houses.
B. The operator should be able to hear all traffic (vehicular and marine) as well as see traffic from the primary workstation in the operation area.
C. Heat gain can be a problem. Where sight considerations permit, insulated walls should be used as a buffer against heat gain. Provide 4 to 5 foot roof overhangs.
D. The preferred wall construction is reinforced concrete; minimum six inches thick with architectural treatments such as fluted corner pilasters, arches, frieze ornamentation, horizontal banding or other relief to blend with local design considerations.
E. Finish exterior of house with stucco, Class V coating or spray-on granite or cast stone.
F. Design the Bridge Tenders Room with a minimum of 200 square feet of usable floor space. This allows enough room for a toilet, kitchenette, and coat/mop closet as well as wall-hung desk and control console. Add additional square footage for stairwells, or place stairs on exterior of structure.
G. Windowsills should be no more than 34 inches from the floor. This allows for operator vision when seated in a standard task chair.
H. Consideration should be given to lines of sight form control station during column sizing and spacing.
I. For operator standing at control console, verify site lines to:
   1. Traffic gates for both directions of automobile traffic.
   2. Marine traffic for both directions of the navigable channel.
   3. Locations where pedestrians normally will stop.
   4. Under side of bridge, at channel, via video cameras (4) with recording capabilities for accident investigation.

8.8.2 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 potable water distribution system and all water lines per the requirements of American Water Works Association (AWWA).

8.8.3 Site Sanitary Sewage System

Gravity lines to manholes are preferred. Avoid the use of lift stations. However if lift stations are required, special consideration must be given to daily flows as well as pump cycle times.
Low daily flows result in long cycle times and associated odor problems. Include pump and flow calculations and assumptions in 60% submittal.
For bridges which are not served by a local utility company, or where connection is prohibitively expensive, and where septic tanks are not permitted or practical, coast guard approved marine sanitation devices are acceptable.

8.8.4 Stairs, Steps and Ladders

Stair treads must be at least 3 feet wide and comply with codes in regard to riser and tread dimensions. Comply with OSHA requirements. The preferred tread is skid-resistant open grating. Avoid the use of ladders or stair ladders.

Commentary:
Ships ladders may be used, only as last resort for very limited height (4 feet vertical maximum).

Stairs and landings may be on the exterior of the house.

Commentary:
This reduces heating and cooling requirements as well as providing more usable floor space.

For interior stairwells, spiral stairs are acceptable although not preferred Minimum 6 foot diameter.
Special attention must be paid to clearances for moving equipment into or out of a control house.
Design stair assembly to support live load of 100-lbs/sq ft with deflection of stringer not to exceed 1/180 of span. Include calculations in 60% submittal.
In situations where stairs cannot be accommodated, stair ladders may be used. Vertical ladders should not be used.

8.8.5 Handrails and Railing

Specify standard I.P.S. size schedule 40 1½ inch O.D. 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 load of 200 pounds at any point and in any direction, plus a continuous load of 50 psf in any direction without damage or permanent set. Include calculations in 60% submittal.

Grating and Floor Plates. Specify skid resistant open grating, except at control level.

8.8.6 Framing and Sheathing

A. include a specification section for the following items if used:
   1. Structural floor, wall, and roof framing.
   2. Built-up structural beams and columns.
   3. Diaphragm trusses fabricated on site.
   4. Floor, wall, and roof sheathing.
   5. Sill gaskets and flashing.
   6. Preservative treatment of wood.
   7. Fire retardant treatment of wood.
   8. Telephone and electrical panel back boards. Concealed wood blocking for support of toilet and bath accessories, wall cabinets, and wood trim.
   9. All other sections applicable to tender house design and construction.

8.8.7 Desktop and Cabinet

A. Specify and detail a wall-hung desktop with drawer. Show desktop.
B. Specify and detail a minimum 7 feet of 36-inch base cabinets and 7 feet of 24 or 36-inch wall cabinets.
C. Specify cabinet hardware and solid-surfacing material counter-tops and desktop.

8.8.8 Insulation

A. Design the tender house so that insulation meets the following requirements: Walls - RI9, Roof assembly - R30.
B. Board Insulation may be used at underside of floor slabs, exterior walls, and between floors separating conditioned and unconditioned spaces.
C. Batt Insulation may be used in ceiling construction and for filling perimeter and door shim spaces, crevices in exterior wall and roof.

8.8.9 Fire-Stopping

A. Specify, design, and detail fire-stopping for wall and floor penetrations.
   1. Main Floor Walls: 1 Hour
   2. Stair Walls (Interior): 2 Hours
   3. Interior Partitions: ¾ Hour

8.8.10 Roof

A. Do not use flat roofs, “built-up” roofs, etc.
B. Design: Hip roof with minimum 6:12 pitch and 4 to 5 foot overhang.
C. Roof Material: Specify and detail either standing seam 18 gauge metal or glazed clay tiles. Note: many of the coastal environments will void the manufacturer’s warranty for metal. Before specifying a metal roof determine if the manufacturer will warrant the roof in the proposed environment, if not, use tiles meeting or exceeding the Grade I requirements of ASTM C 1167.
D. Soffit: Specify ventilated aluminum or vinyl.
E. Fascia: Specify aluminum, vinyl or stucco.
F. Design for hurricane force uplift on roof, roof framing, decking, anchors and other components. Include roof load and uplift calculations in 60% submittal.
G. During design, consider underlayment, eave, and ridge protection, nailers and associated metal flashing.

8.8.11 Doors And Hardware

A. Specify and detail armored aluminum entry doors; ballistics rated for.357 Magnum fired at point blank range.

B. Interior Doors:
   1. Passage - Solid core or solid wood
   2. Closets – Louvered

3. Hardware:
   1. Specify corrosion resistant, heavy-duty, commercial ball-bearing hinges and levered locksets and dead bolts for entry doors.
   2. Specify adjustable thresholds, weather-stripping, seals and door gaskets.
   3. Specify interior locksets.
   4. Call for all locks keyed alike and spare keys.

8.8.12 Windows

A. Specify windows complying with the American Architectural Manufacturers Association standards (AAMA) for heavy commercial windows.

B. Specify double-hung, marine glazed heavy commercial (DHHC) type extruded aluminum windows. Glazing must be ballistics-rated for .357 Magnum fired at point blank range.

C. Specify windows to be counter balanced to provide 60% lift assistance.

D. Specify operating hardware and insect screens.

E. Specify perimeter sealant.

F. When ballistic glazing is required, specify glazing that is rated for .357 Magnum round fired at point blank range.

G. Structural Loads: ASTM E330-70. With 60-lb/sq ft [2.87 kPa] exterior uniform load and 60-lb/sq ft [2.87 kPa] interior load applied for I0 seconds with no glass breakage, permanent damage to fasteners, hardware parts, actuating mechanisms or any other damage.

H. Air Leakage: No more than 0.35-cfm/min/sq ft [0.002cu m/s/sq in] of wall area, measured at a reference differential pressure across assembly of 1.57 psf [75 Pa] as measured in accordance with ASTM E283.

I. Water Leakage: None, when measured in accordance with ASTM E331 with a test pressure of 6 psf applied at 5 gallons per hour per square ft.

8.8.13 Veneer Plaster (Interior Walls)

Specify ¼ inch plaster veneer over ½ inch moisture-resistant gypsum wallboard (blueboard), masonry and concrete surfaces.

8.8.14 Gypsum Board (Interior Walls)

Specify ½ inch blueboard for plaster veneer.
Specify ½ inch fiberglass reinforced cement backer board for tile.
Specify taped and sanded joint treatment and smooth texture finish.

8.8.15 Floor Tile

Specify non-skid quarry tile on tender's level.
Do not use vinyl floor tiles or sheet goods.

8.8.16 Epoxy Flooring

Specify fluid applied non-slip epoxy flooring for electrical rooms, machinery rooms and machinery platforms. Ensure that the manufacturer of the product warrants the product used and that it is installed per their instructions. Do not specify painted floors.

8.8.17 Painting
Specify paint for woodwork and walls.

8.8.18 Wall Louvers
Use rainproof intake and exhaust louvers and size to provide required free area. Design with minimum 40% free area to permit passage of air at a velocity of 335 ft/min [160 L/sec] without blade vibration or noise with maximum static pressure loss of 0.25 inches [6 mm] measured at 375 ft/m in [175 L/sec].

8.8.19 Toilet and Bath Accessories
Specify a mirror, soap dispenser, tissue holder, paper towel dispenser, and a waste paper basket for each bathroom.
Specify a bathroom exhaust fan.
Porcelain water closet and lavatory.

8.8.20 Equipment and Appliances
Specify a shelf mounted or built-in 1.5 cubic foot microwave with digital keypad and users manual.
Specify an under counter refrigerator with users manual.
Specify a fire extinguisher for each room. Type ABC 10.

8.8.21 Furnishings
Specify two gas lift front-tilt task chairs.
Provide one R5 cork bulletin board.
Specify window treatment (blinds or shades).

8.8.22 Pipe and Fittings
Specify pipe fittings, valves, and corporation stops, etc.
Provide hose bib outside house.
Provide wall-mounted, corrosion resistant (fiberglass or plastic) hose hanger and 50-foot nylon reinforced ¾ inch garden hose in a secure area.
Provide stops at all plumbing fixtures.
Provide primed floor drains.
Provide air traps to eliminate/reduce water hammer.
Provide ice maker supply line.

8.8.23 Plumbing Fixtures
Specify a single bowl, stainless steel, self-rimming kitchen counter sink, a sink faucet, a lavatory, a lavatory faucet, and an elongated toilet.
Do not specify ultra-low flow fixtures unless the bridge has a marine digester system.
Specify all trim, stops, drains, tail pieces, etc. for each fixture.
Provide 10-gallon water heater for kitchen sink and lavatory.

8.8.24 Heating, Ventilating and Air Conditioning
A central split unit is preferred but multiple, packaged units may be acceptable for rehabs. 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 60% or 75% submittal. For highly corrosive environments use corrosive resistant equipment.
Specify packaged terminal air conditioning units.
Specify packaged terminal heat pump units.
Specify and detail wall sleeves and louvers.
Specify controls.
Specify and show ceiling fans on floor plan.
Specify ventilation equipment for machinery levels and attic.

8.8.25 Interior Luminaires

Specify fixtures under section 508 of FDOT Standard Specifications for Road and Bridge Construction, current adopted edition.
Avoid the use of heat producing fixtures.
Do not use PCV Ballasted lighting fixtures

8.8.26 Video Equipment

Cameras (4): One camera each channel direction to provide full view of channel and fender system and one camera each direction of traffic to provide view of all travel lanes and sidewalks.
Monitors (4): One for each camera.
Provide recording capabilities for each camera.

8.9 Maintainability

8.9.1 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.

8.9.2 Trunnion Bearings

Design trunnion bearings so that replacement of bushings can be accomplished with the span jacked ½ inch [12 mm] and in a horizontal position. Provide suitable jacking holes or puller grooves in bushings to permit extraction. Jacking holes must utilize standard bolts pushing against the housing that supports the bushing.

Specify trunnion bushings and housings of a split configuration. The bearing cap and upper-half bushing (if an upper-half bushing is required) must be removable without span jacking or removal of other components.

8.9.3 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. Locate one set of span-jacking surfaces under the trunnions (normally, this will be on the bottom surface of the bascule girder). Locate a second set on the lower surface at the rear end of the counterweight.

Locate span stabilizing connector points on the bascule girder forward and back of the span jacking surfaces. The stationary stabilizing connector point (forward) must be in the region of the Live Load Shoe. Locate stationary stabilizing connector points (rear) on the cross girder support at the rear of the bascule pier. Provide connector points to attach stabilizing structural steel components.

Commentary:
Position the stationary jacking surface 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:

Span-jacking Surface: An area located under the trunnion on the bottom surface of the bascule girder.
Span Stabilizing Connector Point (forward): An area adjacent to the live load shoe point of impact on the bottom surface of the bascule girder.
Span Stabilizing Connector Point (rear): An area at the rear end of the counterweight on the lower surface of the counterweight girder. (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.)
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.

8.9.4 Trunnion Alignment Features

Provide center holes in trunnions to allow measurement and inspection of trunnion alignment. Span structural components must not interfere with complete visibility through the trunnion center holes. Provide individual adjustment for alignment of trunnions.
Provide a permanent walkway or ladder with work platform to permit inspection of trunnion alignment.

8.9.5 Lock Systems

Span locks are to be accessible from the bridge sidewalk through a suitable hatch or access door. Provide a work platform suitable for servicing of the lockbars under the deck and in the region around the span locks.

Design lock systems to allow disabling an individual lock, for maintenance or replacement, without interfering with the operation of any of the other lockbars on the bascule leaf.

Design tail locks, when required, so that the lockbar mechanism is accessible for repair without raising the leaf. The lockbar drive mechanism must be accessible from a permanently installed platform within the bridge structure.

Provide adjustable lockbar clearances for wear compensation.

8.9.6 Machinery Drive Systems

Design machinery drive assemblies so that components are individually removable from the drive system without removal of other major components of the drive system.

Commentary:

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

8.9.7 Lubrication Provisions

Bridge system components requiring lubrication must be accessible without use of temporary ladders or platforms. Provide permanent walkways and stairwells to permit free access to regions requiring lubrication. Lubrication fittings must be visible, clearly marked and easily reached by maintenance personnel.

Designs for automatic lubrication systems must provide for storage of not less than three (3) months supply of lubricant without refilling. Refill must 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.

8.9.8 Drive System Bushings

All bearing housings and bushings in open machinery drive and lock systems must utilize split-bearing housings and bushings and must be individually removable and replaceable without affecting adjacent assemblies.

8.9.9 Local Switching

Provide "Hand-Off-Automatic" switching capability for maintenance operations on traffic gate controllers and brakes and motors for center and tail-lock systems.

Provide "On-Off" switching capability for maintenance operations on span motor and machinery brakes, motor controller panels, and span motors.
Remote switches must be lockable for security against vandalism.

8.9.10 Service Accessibility

Provide a service area not less than 30 inches wide around system drive components.

8.9.11 Service Lighting and Receptacles

Provide lighting of machinery and electrical rooms as necessary to assure adequate lighting for maintenance of equipment, but with a minimum lighting level of 30 fc [300 Lux]. Provide switching so that personnel may obtain adequate lighting without leaving the work area for switching. Provide master switching from the control tower.

Provide each work area with receptacles for supplementary lighting and power tools such as drills, soldering and welding equipment.

8.9.12 Communications

Provide permanent communications equipment between the control tower and areas requiring routine maintenance (machinery drive areas, power and control panels locations traffic gates and waterway).

8.9.13 Wiring Diagrams

Provide wiring diagrams for each electrical panel inside the panel door. Enclose diagrams in glass or plastic of optical quality.

8.9.14 Diagnostic Reference Guide for Maintenance

Specify diagnostic instrumentation and system fault displays for mechanical and electrical systems. Malfunction information must be presented on a control system monitor located in the bridge control house. Data must be automatically recorded. System descriptive information, such as ladder diagrams and wiring data, must be available on the system memory to enable corrective actions on system malfunctions and to identify areas requiring preventative maintenance.

8.9.15 Navigation Lights

Specify dual lamps and transfer relays with two long-life, 130 volt, brass base, 100W lamps; or LED array of at least 12 particularly bright RED or GREEN LEDs symmetrically arranged around the lens focal point and with a minimum of 50,000 hour life on fenders and center of channel positions to reduce effort required for maintenance of navigation lights.

8.9.16 Working Conditions for Improved Maintainability

When specified by the Department, for either new or rehabilitated bascule bridge design, use enclosed machinery and electrical equipment areas. Air-condition areas containing electronic equipment to protect the equipment as required by the equipment manufacturer and the Mechanical/Electrical Section of the SDO.

8.9.17 Weatherproofing

New and rehabilitated bascule bridge designs must 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.

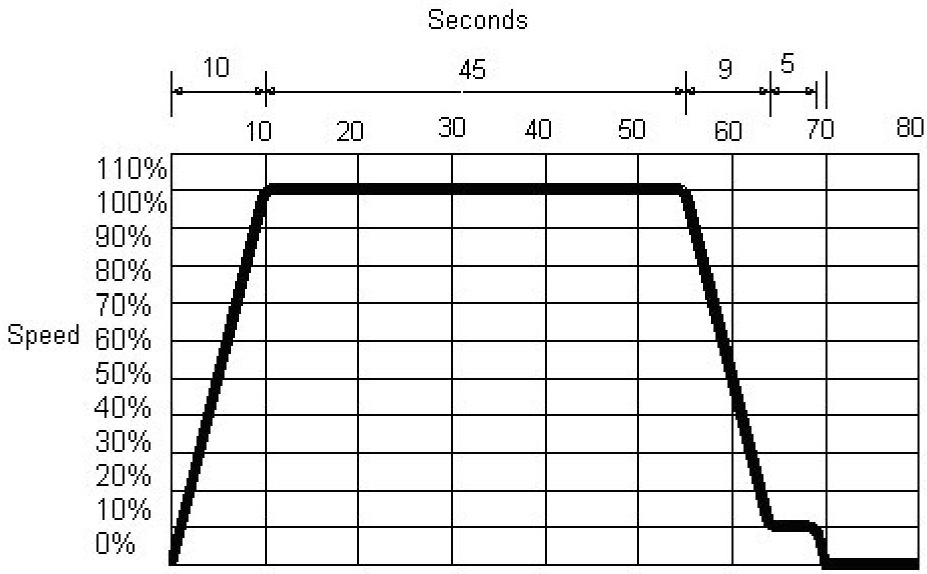


Figure 8-1
Speed Ramp

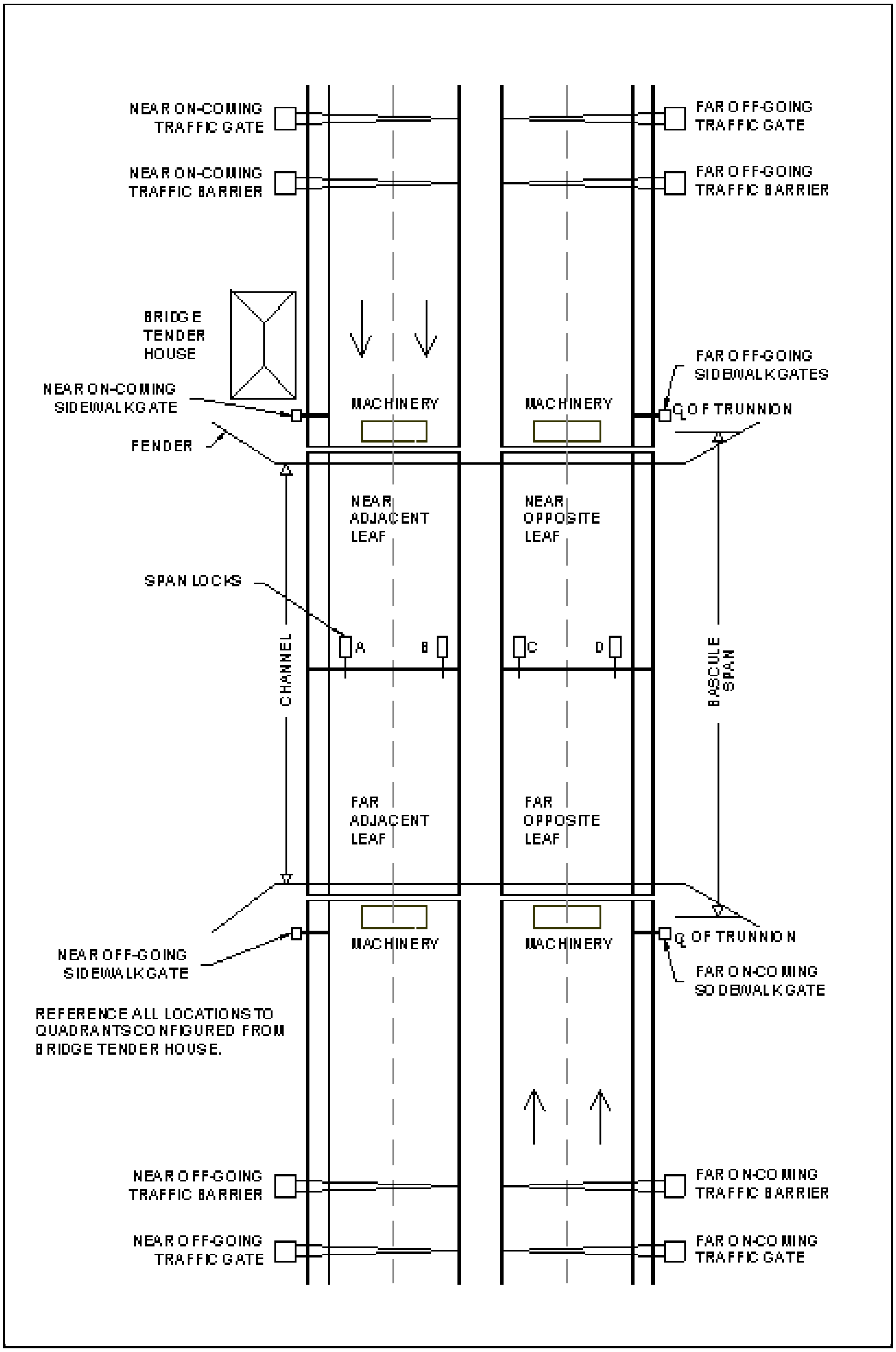


Figure 8-2
Movable Bridge Terminology

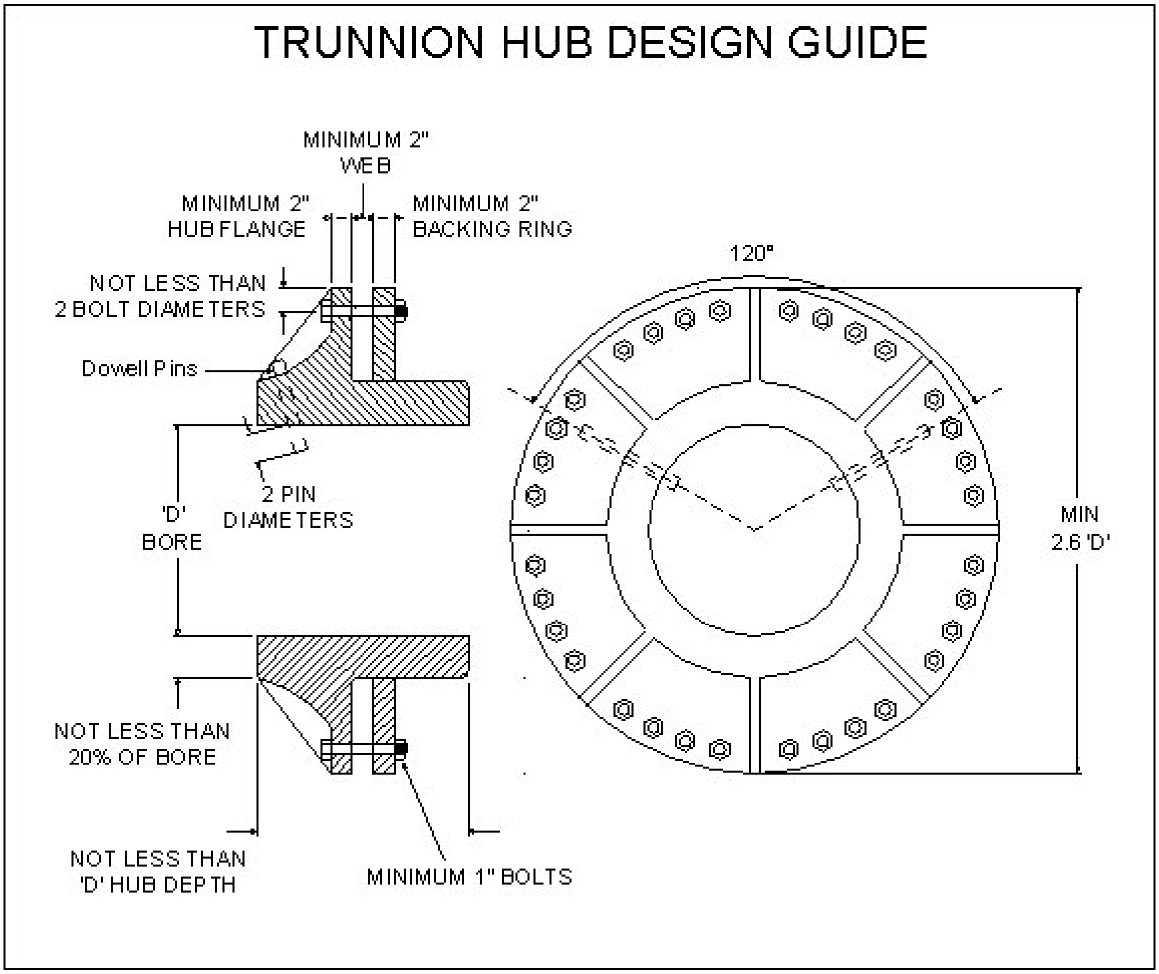


Figure 8-3
Trunnion Hubs

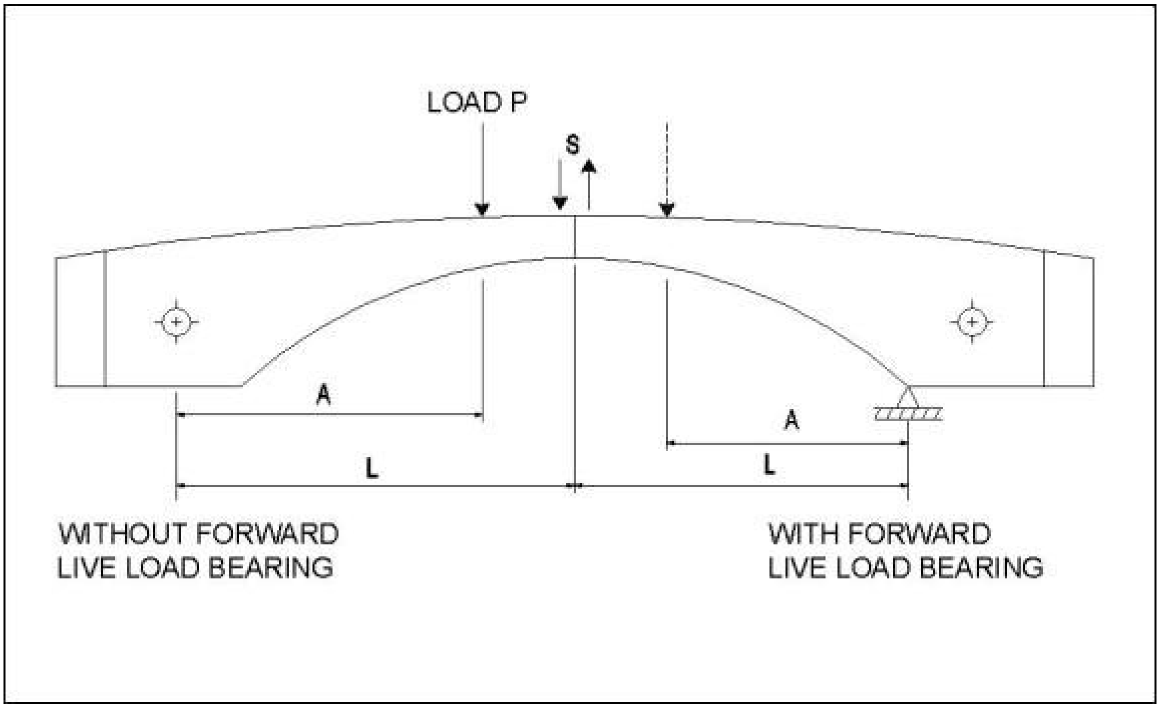


Figure 8-4
Lock Design Criteria

CHAPTER 9
BRIDGE DEVELOP REPORT COST ESTIMATING

9.1 General

The purpose of the Bridge Development Report is to select the most appropriate structure type for the site under consideration. One of the most important considerations is to select the most cost efficient bridge to fit the unique circumstances at the site. The purpose of the procedure established in this chapter is to bring uniformity to the cost estimating portion of the decision making process. Project cost data is provided for information purposes.

Use this procedure for all bridge structures with the exception of the structure types stated below. This process is not suitable for cost estimating structure types without repeatable bid history including the following bridge types: movable; cable stayed; cast-in-place on form travelers; arches and tunnels. These very unique structures should be cost estimated by the use of fundamental process of developing cost based on labor, materials, and equipment and construction time.

This concept for cost estimating is a three-step process. The first step is to utilize the average unit material costs provided herein to develop a cost estimate based on the completed preliminary design. The second step is to adjust the total bridge cost for the unique site conditions by use of the site adjustment factors. The third and final step is to review the computed total bridge cost on a cost per square foot basis and compare this cost against the historical cost range for similar structure types. This three-step process should produce a reasonably accurate cost estimate for structure type selection. However, if a site has a set of odd circumstances, which will affect the bridge cost, account for these unique site conditions in the estimate. If the estimated cost is outside the cost range in step three, provide documentation supporting the variance in cost.

9.2 BDR Bridge Cost Estimating

The applicability of this three-step process is explained in the general section. The process stated below is developed for estimating the bridge cost after the completion of the preliminary design, which includes member selection, member size and member reinforcing. This process will develop costs for the bridge superstructure and substructure from beginning to end bridge. Costs for all other items including but not limited to the following are excluded from the costs provided in this chapter: mobilization, operation costs for existing bridge(s); removal of existing bridge or bridge fenders; lighting; walls; deck drainage systems; embankment; fenders; approach slabs; maintenance of traffic; load tests; bank stabilization.

Step One:
Utilizing the cost provided herein, develop the cost estimate for each bridge type under consideration.

9.2.1 Substructure

A. Prestressed Concrete Piling; cost per linear foot (furnished and installed)

**Size of Piling	Driven Plumb or 1" Batter	Driven Battered
18 in.	$38	$47
24 in.	$53	$67
30 in.	$63*	$80

*When heavy mild steel reinforcing is used in the pile head, add $250.
** When silica fume is used, add $6 per LF to the piling cost.

B. Steel Piling; cost per linear foot (furnished and installed)
14” x 73 H Section $35
14” x 89 H Section $38
20” Pipe Pile $84
24” Pipe Pile $90
30” Pipe Pile $152

C. Drilled Shaft; cost per linear foot
1.) On Land with casing salvaged. (Total in-place cost)
3 ft $239
4 ft $277
5 ft $340
6 ft $441
7 ft $542
2.) In water with casing salvaged. (Total in-place cost)
3 ft $277
4 ft $302
5 ft $353
6 ft $479
7 ft $605
8 ft $806
3.) In water with permanent casing. (Total in-place cost)
3 ft $428
4 ft $466
5 ft $554
6 ft $643
7 ft $781
8 ft $970
9 ft $1184

D. Sheet Piling Walls
1.) Prestressed concrete; cost per linear foot.

10” x 30” $71
12” x 30” $86
2.) Steel; cost per square foot
Permanent Cantilever Wall..$20
Temporary Cantilever Wall $6

E. Cofferdam Footing (cofferdam and seal concrete)
Prorate the cost provided herein based on area and depth of water. A cofferdam footing having the following attributes will cost $328,000.
Area; 63 ft x 37.25 ft. Depth of seal; 5 ft. Depth of water over the footing; 16 ft.

F. Substructure Concrete; cost per cubic yard.
Concrete $550
Mass concrete $315
Seal concrete..$344
Shell fill $5
Admixtures
For calcium nitrite, add $32 per cubic yard. (@4.5 gal per cubic yard).
For silica fume, add $25 per cubic yard. (@60 lbs. per cubic yard.)

G. Reinforcing Steel; cost per pound $0.46

9.2.2 Superstructure

A. Bearing Material
1.) Neoprene Bearing Pads; Cost per Cubic Foot $500
2.) Multirotational Bearings, Cost per Each
Capacity in Kips Cost
1-251 ....$3,465
251-500 ....$3,780
501-750 ....$4,410
751-1000 ....$5,040
1001-1250 ....$5,670
1251-1500 ....$6,300
1501-1750 ....$6,930
1751-2000 ....$7,875
>2000 ....$10,080

B. Bridge Girders
1.) Structural Steel; cost per pound (includes coating costs).
Rolled wide flange sections..$0.90
Plate girders; straight $1.04
Plate girders; curved $1.21
Box girders; straight $1.32
Box girders; curved $1.54

When uncoated weathering steel is used, reduce the price by $0.04 per pound. Inorganic zinc coating systems have an expected life cycle of 20 years.

2.) Prestressed Concrete Girders; cost per linear foot.

AASHTO Type II $80
AASHTO Type III $86
AASHTO Type IV $100
AASHTO Type V $120
AASHTO Type VI $130
Fl Bulb Tee; 54” $90
Fl Bulb Tee; 63” $98
Fl Bulb Tee; 72” $120
Fl Bulb Tee (M); 78” $135
78” Haunched units (CJ to CJ) $380
Fl Double Tee; 18” $185
Fl Double Tee; 24” $200
Fl Double Tee; 30” $270
Fl Inverted Tee; 16” $50*
Fl Inverted Tee; 20” $56*
Fl Inverted Tee; 24” $62*
Fl Inverted Tee; 16” $50*
Fl Tub (U-Beam); 48” $300*
Fl Tub (U-Beam); 54” $330*
Fl Tub (U-Beam); 63” $370*
Fl Tub (U-Beam); 72” $400*
Solid Flat Slab (36’x15”) $110
Solid Flat Slab (36’x18”) $125

* Price is based on ability to furnish products without any conversions of casting beds and without purchasing of forms. If these conditions do not exist, add the following costs:

Inverted Tee - $202,000
Fl Tub - $403,000

3.) Cast-in-Place Superstructure Concrete; cost per cubic yard.

Box Girder Concrete; straight $650
Box Girder Concrete; curved $675
Deck Concrete $425

4.) Concrete for Pre-cast Segmental Box Girders; cantilever construction; price per cubic yard. For deck area between 300,000 and 500,000 interpolate between the stated cost per cubic yard.

<=300,000 SF $693
<=500,000 SF $567
>500,000 SF $567

5.) Reinforcing Steel; cost per pound $0.46

6.) Post-tensioning Steel; cost per pound.

Strand; longitudinal $1.53
Strand; transverse $1.82
Bars $3.90

7.) Railings and Barriers, cost per linear foot.
- Traffic Barrier $44
- Pedestrian Railing $57
- Bicycle Railing $69*

*For metal railing add $38 per linear foot.

8.) Expansion joints; cost per linear foot.
- Strip seal $106
- Finger joint <6”. $315
- Finger joint >6” $473
- Modular 6” $500
- Modular 8” $700
- Modular 12” $900

C. Retaining Walls

1.) MSE Walls; Cost per square foot
- Permanent $23
- Temporary $8

D. Noise Wall; Cost per square foot $18

E. Detour Bridge; Cost per square foot $10*

*Using FDOT supplied components. The cost is for the bridge proper and does not include approach work, surfacing, or guardrail.

9.2.3 Design Aid for Determination of Reinforcing Steel

In the absence of better information, use the following quantities of reinforcing steel per cubic yard of concrete.

Pile abutments	135
Pile Bents	145
Single Column Piers; Tall (>25 ft)	210
Single Column Piers; Short (<25 ft)	150
Multiple Column Piers; Tall (>25 ft)	215
Multiple Column Piers; Short (<25 ft)	195
Bascule Piers	110
Deck Slabs; Standard	205
Deck Slabs; Isotropic	125
Concrete Box Girders; Pier Segment	225
Concrete Box Girders; Typical Segment.	165
Concrete Box Girders; Flat Slabs (30 ft x 15” deep)	220

Step Two:
After developing the total cost estimate utilizing the unit cost, modify the cost to account for site condition variables. If appropriate, the cost will be modified by the following variables:

1. For rural construction decrease construction cost by 6 percent.
2. For urban construction (Broward, Dade, Duval, Hillsborough, Orange, Palm Beach and Pinellas counties), increase construction cost by 6 percent.
3. For construction over water increase construction cost by 3 percent.
4. For phased construction (over traffic or construction requiring multiple phases to complete the entire cross section of the bridge), add a 20 percent premium to the affected units of the structure.

Step Three:
The final step is a comparison of the cost estimate with historic bridge cost per square foot data. These total cost numbers are calculated exclusively for the bridge cost as defined in the General Section of this chapter. Price computed by Steps 1 and 2 should be generally within the range of cost of as supplied herein. If the cost falls outside the provided range, good justification must be provided.

Bridge Superstructure Type	**Total Cost per Square Foot**
Reinforced Concrete Flat Slab; Simple Span	$50-65*
Reinforced Concrete Flat Slab; Continuous Span	$60-80*
Steel Deck/Girder; Simple Span	$62-75*
Steel Deck/Girder; Continuous Span	$70-90*
Prestressed Concrete Deck/Girder; Simple Span	$50-70*
Prestressed Concrete Deck/Girder; Continuous Span	$65-110*
Post-tensioned, cast-in-place Concrete Box Girder	
Cast on scaffolding; span length <=240 ft	$75-110
Steel Box Deck/Girder	
Span range from 150 ft to 280 ft	$76-120
For curvature add a 15 percent premium	
Segmental Concrete Box Girders	
Span range from 150 ft to 280 ft	$80-110
Movable Bridges; bascule spans & piers	$900-1500
Demolition of existing bridges	
Typical	$9-15
Bascule spans & piers	$63

* Increase the cost by twenty percent for phased construction.

9.3 Historical Bridge Costs

The unadjusted bid cost for selected bridge projects are provided as a supplemental reference for estimating costs. The costs have been stripped of all supplemental items such as mobilization, so that only the superstructure and substructure cost remain.

9.3.1 Deck/Girder Bridges

Project Name and Description	Letting Date	Deck Area (SF)	Cost per SF
Jenson Bch Cswy (890145)	01/02	150,679 78” Bulb-tee, simple span	$59.00
SR 417/Turnpike (770616)	99/00	5,270 AASHTO Type VI	$50.39
US 98/Thomas Dr.(460111)	02/03	167,492, U-Beam	$66.50
SR 704 over I-95 (930183 & 930210)	97/98	14,804 each AASHTO Type IV Simple span	$60.66
SR 700 over C-51 (930465)	97/98	7,153 AASHTO Type II Simple Span	$46.46
SR 807 over C-51 (930474)	98/99	11,493 AASHTO Type III Simple Span	$48.77
SR 222 over I-75 (260101)	00/01	41,911 AASHTO Type III & IV	$63.59
SR 166 over Chipola River (530170)	00/01	31,598 AASHTO Type IV	$48.52
SR 25 over Santa Fe River (260112)	00/01	17,118 AASHTO Type IV	$52.87
SR 71 over Cypress Creek (510062)	00/01	12,565 AASHTO Type III	$49.64
SR 10 over CSX RR (580175)	00/01	12,041 AASHTO Type IV	$54.91
SR 291 over Carpenter Creek (480194)	00/01	7,760 AASHTO Type IV	$59.41
SR 54 over Cypress Creek (140126)	00/01	6,010 AASHTO Type III	$51.48
SR 400 Overpass (750604)	00/01	27,084 AASHTO Type VI	$48.15
Palm Beach Airport Interchange over I-95 (930485)	99/00	9,763 Steel	$85.50
Turnpike Overpass (770604)	98/99	7,733 Steel 179' Simple Span	$79.20
SR 686 (150241)	99/00	63,387 Steel	$73.31
SR 30 RR Overpass (480195 & 480196)	00/01	6,994 each	$118.35

9.3.2 Post - tensioned Concrete Box Girder, Segmental Bridges

Project Name and Description	Letting Date	Deck Area (SF)	Cost per SF
A1A over ICWW St. Lucie River Evans Crary (890158)	97/98	297,453 Span by Span	$80.50
Palm Beach Airport Interchange at I-95 (930480)	99/00	77,048 Balanced Cantilever	$100.73
Palm Beach Airport Interchange at I-95 (930477)	99/00	20,925 Balanced Cantilever	$96.31
Palm Beach Airport Interchange at I-95 (930479)	99/00	69,233 Balanced Cantilever	$88.49
Palm Beach Airport Interchange at I-95 (930482)	99/00	47,466 Balanced Cantilever	$104.96
Palm Beach Airport Interchange at I-95 (930482)	99/00	81,059 Balanced Cantilever	$101.44
Palm Beach Airport Interchange at I-95 (930483)	99/00	90,926 Balanced Cantilever	$101.57
Palm Beach Airport Interchange at I-95 (930484)	99/00	41,893 Balanced Cantilever	$115.11
Palm Beach Airport Interchange at I-95 (930478)	99/00	20,796 Balanced Cantilever	$95.16
17th Street over ICWW Ft. Lauderdale (860623)	96/97	135,962 Balanced Cantilever	$74.71
Royal Palm Way SR 704 over ICWW (930507 & 930506)	00/01	43,173 each C-I-P on Travelers	$163.88
US 92 over ICWW Broadway Bridge Daytona (790188)	97/98	145,588 Balanced Cantilever	$81.93
US 92 over ICWW Broadway Bridge Daytona (790187)	97/98	145,588 Balanced Cantilever	$81.93
SR 789 over ICWW Ringling Bridge (170021)	00/01	329,096 Balanced Cantilever	$81.43
US 98 over ICWW Hathaway Bridge (460012)	00/01	575,731 Balanced Cantilever	$87.72

9.3.3 Post-tensioned Cast-in-place Concrete Box Girder Bridge (low level overpass)

Project Name and Description	Letting Date	Deck Area (SF)	Cost per SF
SR 858 over ICWW Hallandale Beach (860619 & 860618)	97/98	29,888 each	$83.25
SR 858 Flyover Hallandale Beach (860620)	97/98	21,777	$81.99
4th Street over I-275	94/95	12,438	$75.21

9.3.4 Bascule Bridge Cost

Deck area is calculated to be coping-to-coping width times overall bascule length including both bascule pier lengths and main span. Costs include all cost for moveable span, gates and bascule piers.

Closed Deck Bascule Bridges

Project Name and Description	Letting Date	Deck Area (SF)	Cost per SF
SR 45 over ICWW Venice (170170 & 170169)	99/00	8,785 each	$768
Royal Palm Way SR 704 over ICWW (930507 & 930506)	00/01	11,535 each	$1089
SR 858 over ICWW Hallandale Beach (860618 & 860619)	97/98	14,454 each	$811
Ocean Ave. over ICWW ICWW Boynton Beach (930105)	98/99	11,888	$1157
17th Street over ICWW Ft. Lauderdale (860623)	96/97	34,271	$865
2nd Avenue over Miami River (874264)	99/00	29,543	$1080