FLORIDA

LAWTON CHILES
GOVERNOR

DEPARTMENT OF TRANSPORTATION

605 Suwannee Street, Tallahassee, Florida 32399-0450

BEN G. WATTS
SECRETARY

STRUCTURES DESIGN OFFICE
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MEMORANDUM

DATE: October 25, 1991

TO: All Holders of the FDOT Structures Design Guidelines Manual

FROM: Antonio M. Garcia, State Structures Design Engineer

SUBJECT: FDOT Structures Design Guidelines Manual, Revision "g"

This copy of the subject manual, Revision "g", replaces the following specific pages of the previous Revision "f":

1. New First Page dated November 1, 1991 showing revision status by topic number and effective date.

2. Page 4 of the Table of Contents for Articles 8.10 thru 10.16.

3. Chapter 10 SUPERSTRUCTURE AND APPROACH SLAB DESIGN, pages 10.1 thru 10.13 (Topic No. 625-020-110-g) effective November 1, 1991.

In particular the changes to Chapter 10 generally are concerned with modifications to the end-of-beam reinforcing for simple span pretensioned beams; however, several other minor changes have also been included.

AMG/nbh

Attachment

REVISIONS

Topic Number	Effective Date
625-020-101-f	February 1, 1991
625-020-102-f	February 1, 1991
625-020-103-f	February 1, 1991
625-020-104-f	February 1, 1991
625-020-105-f	February 1, 1991
625-020-106-f	February 1, 1991
625-020-107-f	February 1, 1991
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Effective: November 1, 1991
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Responsible Office: Structures Design
Topic No.: 625-020-110-g

Chapter 10

SUPERSTRUCTURE AND APPROACH SLAB DESIGN

10.1 Curb Heights on Bridges

For all bridges that utilize curbs, the curb height and batter shall match the curb height and batter on the roadway approaches. This condition will exist on bridges with sidewalks which are normally on an urban section. The curb will normally be 6" in height. All projects with sidewalks shall have these features incorporated into the construction plans. Where the roadway approaches the bridge with a raised median, the median on the bridge shall match that on the roadway.

10.2 Pilaster Details for Roadway Lighting

See Structure Standards for details to be used in designing bridges that will require light standards.

10.3 Intermediate Diaphragms for Prestressed Concrete I-Girder Construction

Where skew angle is greater than 30° design as per AASHTO.

Where skew angle is equal to or less than 30° intermediate diaphragms are not required; however, design live load shall be increased by 5% for the purpose of beam design.

(The FDOT prestressed beam program has available an input code which adds the 5% LL.)

10.4 Cast-in-Place Bridge Deck

The thickness of bridge decks cast-in-place on beams shall be eight inches (8") minimum.

10.5 Grooving Bridge Floors

All contracts for bridges that include the construction of cast-in-place bridge floors and that will not be surfaced with asphaltic concrete shall include the following item in the Summary of Pay Items:

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

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

For widened superstructures the entire deck area as described above shall be grooved if at least one additional lane is being added. For projects with shoulder widening only, a note should be added to the plans specifying that the bridge floor finish shall match that of the existing bridge. The SDO should be contacted to determine the required bridge floor finish for unusual situations. See Article 7.5.C penetrant sealer application requirements for grooved decks.

10.6 Prestressed Pretensioned Components, Beam Design and Construction

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

A. General Design Requirements for Prestressed Construction

1. The center-to-center spacing of strands shall be 2 inches for 1/2" diameter or smaller strands and 2-1/4" for 9/16" diameter strands. Strands of 0.6" diameter are not permitted.

2. Stress computations resulting from prestressing shall be based upon the following lengths of strand transfer and development:

a. For standard diameter strands permitted:

1) Transfer length

a) Fully bonded strands = 80 strand diameters

b) Debonded strands = 100 strand diameters (from the terminus of the debonding material).

2) Development length

a) Fully bonded strands = 1.6 times AASHTO 9.27.1

b) Debonded strands per AASHTO 9.27.1 and 9.27.3

b. For prestressed piling not subjected to significant flexure under service or impact loading, strand development shall be in accordance with AASHTO 9.27.1. Bending that produces cracking in the pile is considered significant.

B. Requirements for Prestressed Beams

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

1. Bridges which contain varying span lengths, skew angles, beam spacings, beam loads or other design criteria may result in very similar individual designs. The designer should consider the individual beam designs as a first trial subject to modification by combining similar designs into groups of designs of common materials and stranding based upon the following priorities:

   a. 28-Day Compressive Concrete Strength (f'c)

   b. Stranding (size, number and location)

   c. Compressive Concrete Strength at Release (f'ci)

   d. Shielding (Debonding)

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

2. In order to achieve uniformity and consistency of design the following parameters shall apply:

   a. 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" are required except for AASHTO Type V and VI beams for which strand patterns with an even number of strands per row shall be utilized.

b. In the end regions of the beam a maximum of 25 percent of the strands may be debonded to satisfy the allowable stress limits but debonded strands shall not exceed 37.5 percent of the total strands in any one row. In applying these percentage limitations the following methodology shall be used:

1) Strands shall be debonded in a pattern that is symmetrical about the vertical axis of the beam.

2) The theoretical number of debonded strands shall be rounded to the closest even number (pairs) of strands except that debonded strands will not be permitted in rows containing three (3) strands or less.

3) Not more than 4 or 40% of the shielded strands, whichever is the greater, shall be terminated at any section. The number of shielded strands being terminated at the section under consideration shall include those for which the strand force has not yet been fully transferred (see Article 10.6.A.2.a.1)b)).

3. The critical stresses at release in the end regions of the beam may be neglected within a distance of 75 percent of the beam height from the centerline of bearing for top of beam tensile stresses and within a distance from the end of the beam equal to the fully bonded strand transfer length for bottom of beam compressive stresses. Stresses beyond these

distances shall comply with the requirements of Article 10.6.B.4. below:

4. In analyzing stresses and/or determining the required length of debonding, stresses shall be limited to the following values:

   a. Tension at top of beam (at release) for straight strand profiles only:

      1) Outer 15 percent of design span = $12\sqrt{f'ci}$

      2) Center 70 percent of design span = $6\sqrt{f'ci}$

   b. Tension at top of beam (at release) for depressed (draped) strand profiles only = $6\sqrt{f'ci}$

   c. Compression at bottom of beam (at release) = 0.6f'ci

   d. Compression at top of beam (service load) = 0.4f'c

   d. Tension at bottom of beam (service load):

      1) Slightly or moderately aggressive environment = $6\sqrt{f'c}$

      2) Extremely aggressive environment = $3\sqrt{f'c}$

5. Exterior strands in any and all rows shall be fully bonded.

6. Reinforcement shall be provided in the top flange of all beams to control tensile cracks prior to, during and after release of the prestressing force. The reinforcement shall consist of a combination of No. 5, ASTM A615, Grade 60, deformed reinforcing bars and 3/8"-diameter (min.), ASTM A416, Grade 250 or 270 prestressing strands initially tensioned to 10,000 lbs. each. The number of bars and strands varies with the beam type as shown in the Structure Design Office's <u>Standard Drawings</u>. Because the majority of the initial tension in the strands is lost prior to the application of the service live load, the effects of these strands on beam stresses after release and the capacity of the beams under service load may be ignored.

7. The strands in the bottom flange shall be confined for a distance from the beam end equal to one and one-half (1.50) times the depth of the beam. The confinement shall be provided by enclosed No. 3 deformed ties, ASTM A615, Grade 40 or 60, spaced 6 inches center-to-center as shown in the Structures Design Office's Standard Drawings. The ties shall be of standard dimensioning and shape for a given beam type regardless of strand pattern. For straight strand designs only, the requirement of AASHTO 9.21.3 to provide 4% of the prestressing force as vertical stirrups at a unit stress of 20,000 psi is waived.

8. The use of "L" shaped longitudinal bars, in the webs and flanges in the end zone areas, is required.

9. The minimum compressive concrete strength at release shall be 4,000 psi. Higher release strengths may be used and specified when required by the designer but generally should not exceed 4,500 psi unless otherwise specifically approved by the FDOT Review Office.

10. Prestressed beams shall be designed and specified to conform to one of the following classes and related strengths of concrete:

Class of Concrete		28-Day Compressive Strength
Class III	(*a)	5,000 p.s.i.
Class IV		5,500 p.s.i.
Class V (Special)	(*b)	6,000 p.s.i.
Class V		6,500 p.s.i.

(*a) Class III Concrete may be used only when the superstructure environment is "slightly aggressive".

(*b) When Class V (Special) is specified, a note shall be added to the beam data sheet stating that: "Class V (Special) Concrete shall conform to all the requirements of Class V Concrete except for 28-day compressive strength."

11. A note shall be shown on the beam sheet in the General Notes stating "Optional stranding shall comply in all respects with the Department's Structures Design Guidelines".

12. All prestressed concrete components shall be designed for stay-in-place metal forms when they are permitted by the Specifications. The weight to be used in the design shall be in accordance with Article 4.4 of these guidelines.
    A note shall be added to the beam detail sheets stipulating the weight allowance included in the beam design for the stay-in-place metal forms and concrete required to fill the form flutes. Additionally, a note shall be added requiring all embedded items and accessories required for their use to be included on the Shop Drawing submittal for the forming system. The design slab thickness shall be provided from the top of the stay-in-place metal form to the finished slab surface and the superstructure concrete quantity shall not include the concrete required to fill the form flutes.
    Refer to Article 10.9 of these Guidelines for stay-in-place metal form quantity calculation and cost notes to be added to the Superstructure drawings.

10.7 Precast Prestressed Slab Units and Double Tee Beams

All bridge structures designed utilizing precast, prestressed slab units or precast, prestressed double tee beams shall conform, in general, with the design criteria and details provided in the Standard Drawings of the Structures Design Office.

10.8 Approach Slabs

On all projects that require epoxy coated reinforcing steel for approach slabs, a note is required on the plans stating that the reinforcing steel in the approach slabs is to be epoxy coated.

10.9 Stay-in-Place (SIP) Slab Forms

For stay-in-place metal and concrete forms see Articles 400-5.7 and 400-5.8 in the FDOT Standard Specifications for Road and Bridge Construction. SIP metal forms shall be designed and detailed for prestressed beam and steel girder superstructures in accordance with Articles 10.6.A.15 and 11.7 of these Guidelines, respectively.

A note shall be placed on the superstructure drawings stating that the quantity of superstructure concrete shown does not include the concrete required to fill the stay-in-place metal form flutes.

Additionally, a note shall be placed on the same superstructure drawings stating that the cost of the stay-in-place metal forms, the concrete required to fill the form flutes, the metal form attachments and accessories and all miscellaneous items required to install the forms shall be included in the contract unit price for the superstructure concrete.

10.10 Prestressed Beam Camber/Build-up Over Beams

In the past, the Department 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 of 2 to 3 times the initial camber at release is not uncommon.

The Department's prestressed beam program provides a table of net camber versus time for the designer's use in estimating the build-up over beams required at the time of slab casting. The values in the program account for creep growth of camber with time and have been closely verified by field measurements.

Until sufficient experience with the predicted versus actual camber has been achieved to permit reliable parameters to be established and utilized, the designer must give consideration to camber growth in the design.

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

10.11 Post-tensioning Strand

Except where otherwise specifically permitted by the Department, strand for post-tensioning applications, either for longitudinal or transverse prestressing, shall be 0.6 inch diameter, ASTM A416, low relaxation strand.

10.12 Dimensional and Location Requirements for Post-Tensioning Ducts

The designer shall show offset dimensions to post-tensioning duct trajectories from fixed surfaces or clearly defined reference lines at intervals not exceeding five (5) feet. When the rate of curvature of a duct exceeds two (2) degrees per foot, offsets shall be shown at intervals not exceeding two and one-half (2.50) feet. In regions of tight reverse curvature of short sections of tendons, offsets shall be shown at sufficiently frequent intervals to clearly define the reverse curve.

Post-tensioning ducts placed within limited section thicknesses, such as slabs and webs of spanning members, shall be limited in outside dimension to 40% of the total section thickness. The thickness shall be measured perpendicular to the centerline plane of the section. Additionally, the clear post-tensioning duct cross-sectional area shall be at least two and one-half (2.50) times the total area of the prestressing steel specified. It is recommended that a clear distance of at least one-half (0.50) times the duct diameter, one and one-half (1.50) times the maximum aggregate size or one and one-half inches (1-1/2"), whichever is the greater, be maintained between ducts running in the same general direction. At joints in segmental construction, it is recommended that the minimum clear distance between ducts be at least two inches (2"). Ducts which cross at right angles or similar large angles may be in contact.

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

10.13 <u>Barrier/Railing Distribution</u>

In lieu of a more refined analysis and for superstructures not less than 40 feet nor more than 110 feet in length, the dead load of barriers and railings attributed to exterior beams or stringers may be determined by the following equation:

$$W_{ext} = \frac{(W)(C_1)(C_2)}{100}$$

Where: W_{ext} = Uniform dead load of the barrier on the exterior beam (lbs./ft.)

W = Total uniform dead load of the barrier (lbs./ft.)

C_1 = Beam spacing coefficient from Figure 10-1

C_2 = Span length coefficient from Figure 10-2

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

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

The distribution methods described above apply to concrete or steel, longitudinal beam or stringer superstructures on which a concrete slab is cast, compositely, prior to installation of the barrier or railing. For superstructures not conforming to the limitations stated above the barrier/railing distribution may be determined in accordance with AASHTO, Article 3.23, or by individual analysis.

10.14 Design of Beams with Variable Spacings

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

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

$$Vx = V_{avg} \sqrt{\text{Beam Spacing at Section/Average Beam Spacing}}$$

10.15 Prestressed Beam Standards and Semi-Standards

Prestressed beam standards and semi-standards included in the Structures Design Office Standard Drawings are available for use in simple span applications only.

10.16 Pretensioned/Post-tensioned Beams

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

A. The pretensioning acting alone shall comply with the minimum steel provisions of AASHTO Article 9.18.2.

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

C. The pretensioning shall be of such a level that the initial midspan camber at release, including the effect of the dead load of the beam, shall be at least one-half (1/2) inch. In computing the initial camber, the value of the modulus of elasticity shall be in accordance with Table 4.1 of this manual for the minimum required strength of concrete at release of the pretensioning force and the pretensioning force in the strands shall be reduced by losses due to elastic shortening and steel relaxation.

10.17 Special Requirements for Bulb-Tee Beams

The recommended web thickness of the Florida Bulb-Tee series of beams shall be:

Pretensioned Beams	6-1/2"
Post-tensioned Beams	7"

In designing concrete cast-in-place slabs supported by bulb-tee beams, the slab design in accordance with AASHTO Articles 3.24 and 8.8 shall be predicated on an effective clear span equal to the actual clear span between edges of the flanges of adjacent bulb-tee beams plus eighteen (18) inches.

When applying the provisions of AASHTO, Article 8.10, the full width of the top flange shall be considered effective.

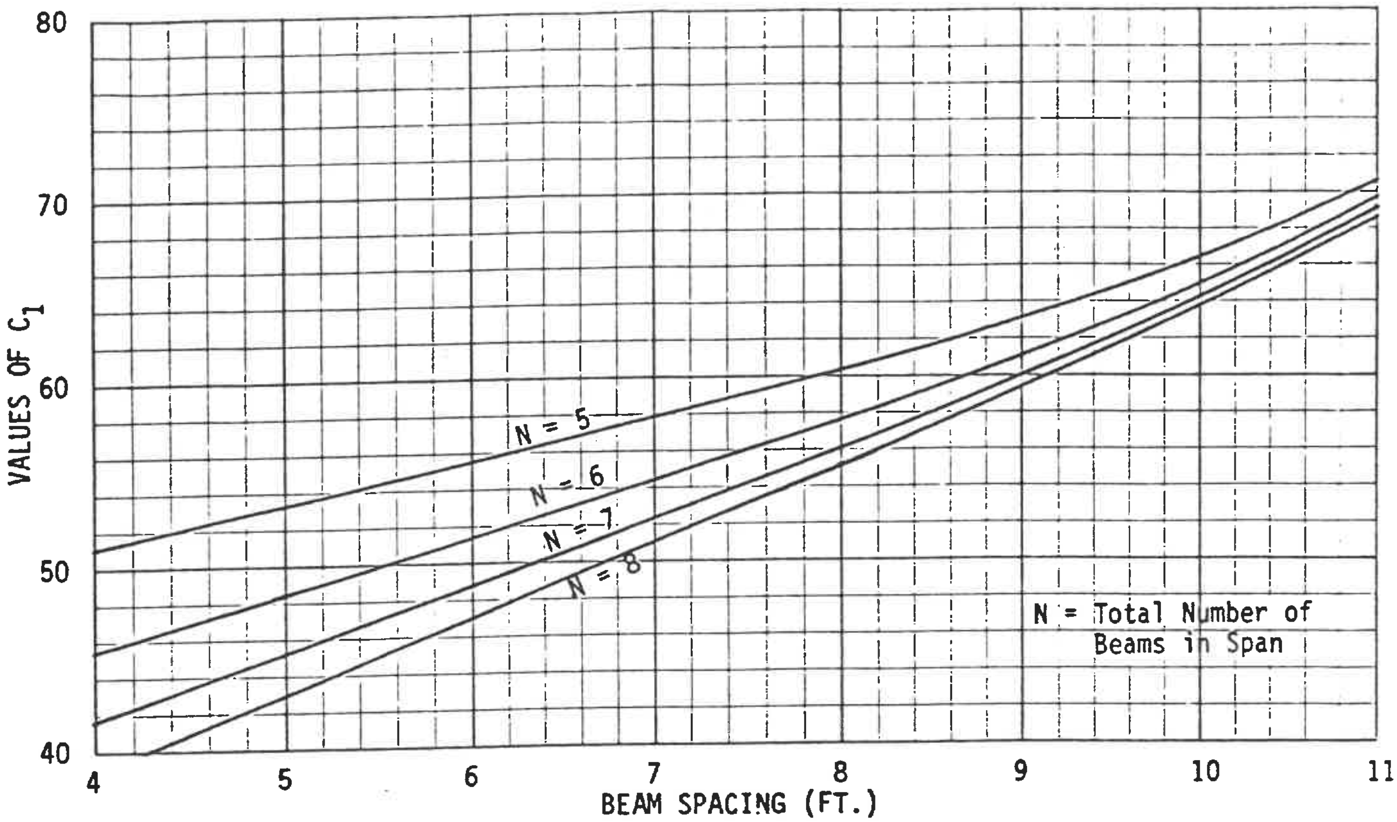


FIGURE 10-1

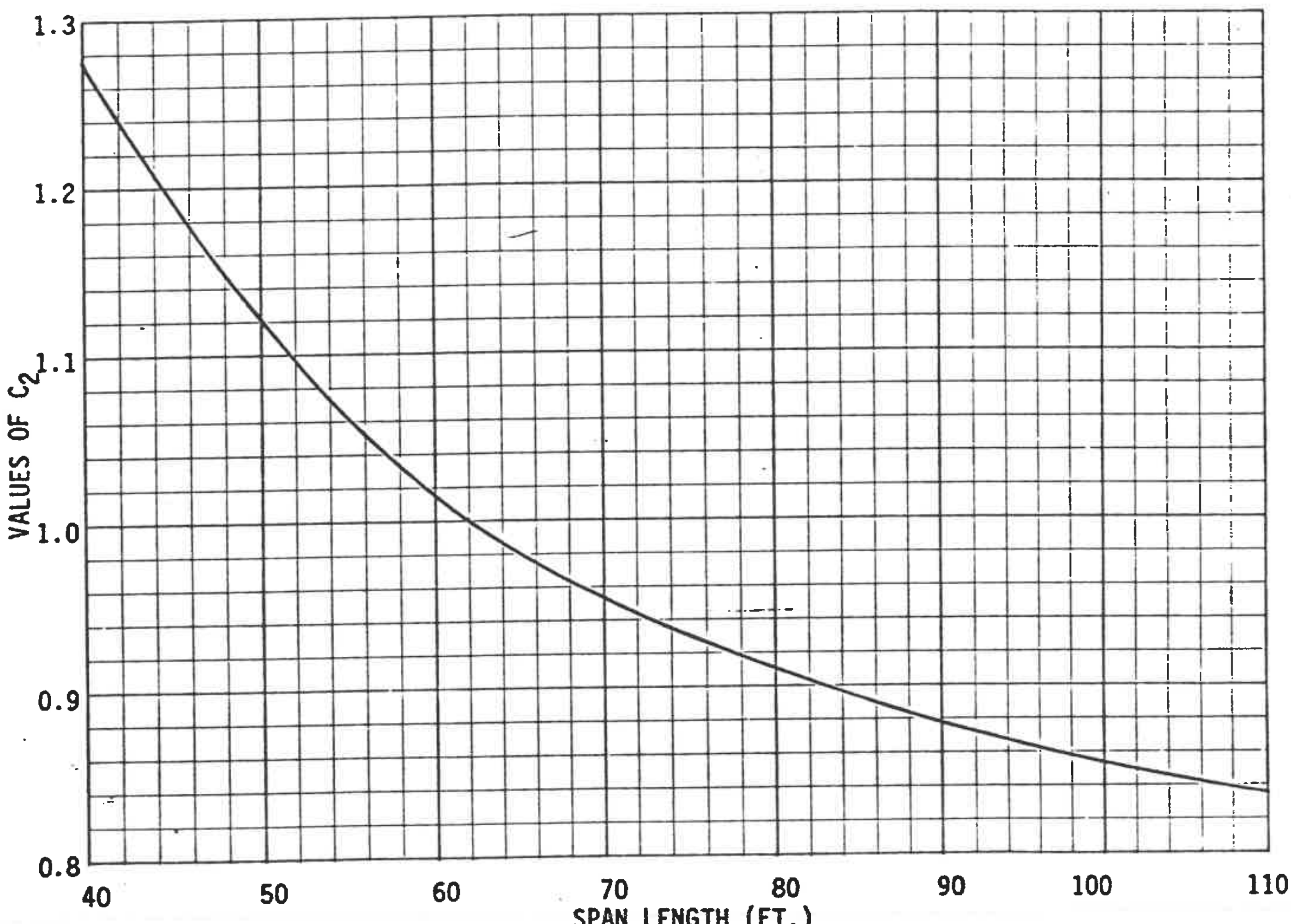


FIGURE 10-2

FLORIDA DEPARTMENT
OF TRANSPORTATION

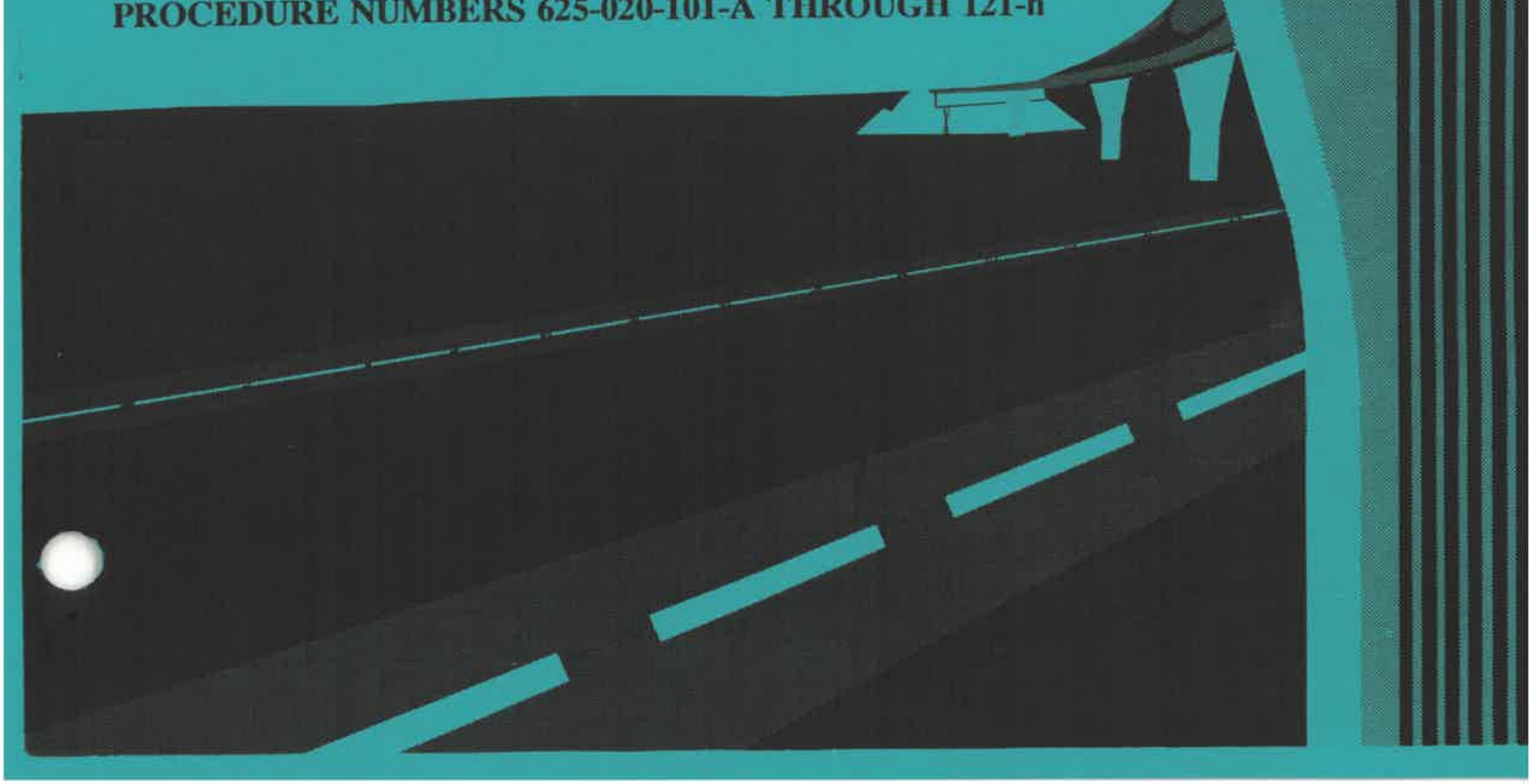
Bureau of Structures Design

Tallahassee, Florida

1987

STRUCTURES DESIGN GUIDELINES

PROCEDURE NUMBERS 625-020-101-A THROUGH 121-h



STRUCTURES DESIGN GUIDELINES

NAME OF REGISTRANT ______________________________

FIRM, AGENCY OR
FDOT AFFILIATION
(INCLUDE DISTRICT NO.) ______________________________

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MAILING ADDRESS ______________________________

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TELEPHONE ______________________________

Please remove this sheet from your Guidelines and complete the requested information so that addenda and revisions may be forwarded as necessary.

Please return to:

Florida Department of Transportation
Structures Design Office
Mail Station 33
605 Suwannee Street
Tallahassee, FL 32399-0450

PREFACE

The structural designer must clearly communicate his ideas and concepts through the plans and specifications he prepares. In this task he should be guided by experience, accepted practice, existing codes, these guidelines and the FDOT's general requirements.

These Structures Design Guidelines express the Structures Design Office's basic practices and philosophy. The manual is intended to provide the structural engineer with a common source of FDOT structural design criteria, standard formats and general information about items for which the structural engineer is usually responsible. It also addresses questions about Florida's specific requirements either not found in or in lieu of the latest AASHTO Standard Specifications for Highway Bridges and other AASHTO standards used in the design.

A bridge is a unique structure accommodating a particular need, using current materials and techniques, conforming to accepted analytical methods and adapted to a site's topography and geology. As such, the primary elements of Structural Design are:

Efficiency	-	the most effective use of materials and design techniques to achieve the required structural performance and maintain the public's safety.
Economy	-	the minimum cost of constructing the structure with proper consideration of details, fabrication, construction and reduced maintenance.
Elegance	-	maximizing aesthetic expression.

Constructibility - the relative ease by which components and elements of the structure can be assembled and installed utilizing available manpower, materials and equipment.

Maintainability - the relative ease by which a structure and its components and elements may be periodically inspected, maintained and/or replaced and the maximizing of the timeframe for such maintenance and replacement; i.e., durability.

These elements are tempered by both safety and scheduling. That is, safety requires defining the limits of risk. Factors of safety, load factors, etc. are a reflection of the confidence we have in our analytical tools, the materials we use and the construction techniques of the time; hence, they reflect the limits of our knowledge. Schedules, however, are artificial restrictions imposed, too often, arbitrarily. An unrealistically short schedule causes efficiency to suffer because a designer will tend to be more conservative thus losing economy and sacrificing elegance. Conversely, an extended schedule may result in an efficient and elegant structure but may impose excessive delays on the public thus indirectly invalidating the resulting economy of design.

Therefore, the Structures Design Office's basic philosophical principle is to encourage designers to satisfy, to the best of their ability, the primary elements listed above by working within the existing body of knowledge, being innovative, exposing unduly restrictive schedules and always seeking to maximize efficiency, economy, and elegance.

By design, this is a "living" document and we intend to update these guidelines continually as criteria changes and improvements dictate. We encourage and welcome input from all bridge designers without whose input such improvements cannot be as effectively accomplished as we would like.

STRUCTURES DESIGN GUIDELINES

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Effective: November 2, 1992
Sequence No.: 0160
Responsible Office: Structures Design
Topic No.: 625-020-101-h

Chapter 1

FDOT STRUCTURES DESIGN OFFICE

1.1 Responsibility

The Structures Design Office (SDO) is a subdivision of the Office of Design under the State Highway Engineer and the Assistant Secretary for Transportation Policy. The SDO is under the direction of the State Structures Design Engineer, is located in Tallahassee and is in direct charge of a majority of structural designs under the Florida Department of Transportation's jurisdiction. Each District, including the Turnpike which operates as a District, has a staff of structural engineers comprising the District Structures Design Office (DSDO) and under the direction of the District Structures Design Engineer (DSDE).

There are bridges being designed either for the State or through Authorities that do not come under the direct responsibility of the Structures Design Office. These include Jacksonville Transportation Authority, Northwest Hillsborough Expressway Authority, Orlando Expressway Authority and others.

The SDO is responsible for developing, updating, revising, amending and modifying the Structures Design Guidelines, the Detailing Manual and the book of Standard Drawings. These three documents, along with both the latest AASHTO Standard Specifications for Highway Bridges and FHWA Directives, constitute the basic documents guiding all structures design work for the Department. The SDO also has certain responsibilities over the DSDE's.

The FDOT is currently under a Certification Acceptance (CA) program. The criteria for and limitations of projects eligible for CA are defined in Chapter 24 of the Department's Plans Preparation Manual. As it applies to bridge design, CA is limited to Federally Funded structures located on the national highway system and off the interstate system. The SDO and District Offices authorized as previously described are certifying agents under the CA program. The SDO also will perform Quality Assessment (QA) of authorized CA activities in District Offices.

At present all Category 2 bridge plans and Category 1 bridge plans for District Offices not authorized by the SDO for review responsibility must be reviewed by the SDO. The review will normally occur for all stages of the design phase from Scope of Services to the 100% submittal and must include a review of the 100% submittal. Some District Offices may also have direct charge of structural design and, in this event, the SDO's responsibility is limited to that of QA.

The structures under each category are as follows:

Category 1	Category 2
Box Culverts	Steel box and plate girders
Short span bridges (continuous reinforced slab or prestressed slab)	Continuous bridges
Simple span bridges (rolled steel beam or concrete)	Cast-in-Place concrete box bridges
Retaining Walls	Concrete Segmental bridges
Roadway signing and lighting	Steel Truss bridges
Bridge widenings for the above listed types	Cable Stayed bridges
Overhead Sign Structures	Complex geometry or rehabilitation details
	Movable bridges (Specifically, the electrical and mechanical components. The approach structures could be Category 1.)
	Any structure of design concept or which contains components, details or construction techniques with a history of less than five (5) years.

1.2 Organization

In general, the SDO's work and responsibilities are in the following areas:

A) Computer Support: The SDO is responsible for creating, acquiring and maintaining structural programs for technical computer support and for the various computer systems the Department supports.

B) CADD Support: This involves developing and maintaining standards for the use of CADD in preparing structural drawings, in particular, the publication of the CADD User Manuals.

C) Standards: The SDO publishes the Structures Design Guidelines, the Detailing Manual and the Standard Drawings.

D) Quality Assurance: The SDO reviews all DSDE offices for compliance with all policy, procedures and standards.

E) Consultant Plans Review: The SDO reviews all Category 2 designs and Category 1 structures for those districts not yet doing Category 1 reviews.

F) In-House Design: Designs for Category 2 bridges are prepared in the SDO.

G) Geotechnical: Standards, procedures and specifications relating primarily to bridge foundations and retaining walls are provided.

H) Electrical/Mechanical: The SDO provides mechanical and electrical design and review for moveable bridge design and rehabilitation.

I) Structures Research and Testing: Testing of existing and new bridge structural components is provided in an effort to improve the Department's bridges. Also, testing is provided to facilitate load rating of bridges.

1.3 Administration

All administration of the SDO will be handled by the State Structures Design Engineer (SSDE) and his administrative assistants. The SSDE oversees the activities of the Office as well as administering and resolving matters of structural design, policy, and procedure.

Approved: Jerry J Potter

Effective: November 2, 1992
Sequence No.: 0160
Responsible Office: Structures Design
Topic No.: 625-020-102-h

Chapter 2

GEOMETRIC DESIGN CRITERIA

2.1 Geometric Design Criteria

The geometric design of highway structures shall comply with Index No. 700 of the FDOT's Roadway and Traffic Design Standards, "Design Criteria Related to Highway Safety".

The minimum clear zone requirements in the standard and throughout this Chapter are based on the embankment slope criteria as shown in the standard and may not apply for freeways, rural arterials and high-speed (greater than 45 MPH) rural collectors if different slope criteria is used. The AASHTO Roadside Design Guide is to be used to the maximum extent practical for determining the clear zone requirements for freeways, rural arterials and high-speed rural collectors when slope criteria other than that shown on the standard is used and for those cases where design for an operating speed greater than 60 MPH may be desirable. The clear zone should be adjusted on the outside of horizontal curvature in either case when practical to do.

The clear zone requirements shown on the standard or in the Roadside Design Guide (if applicable) shall be used for new construction and on reconstruction projects to the extent that economic and environmental considerations and R/W limits will allow.

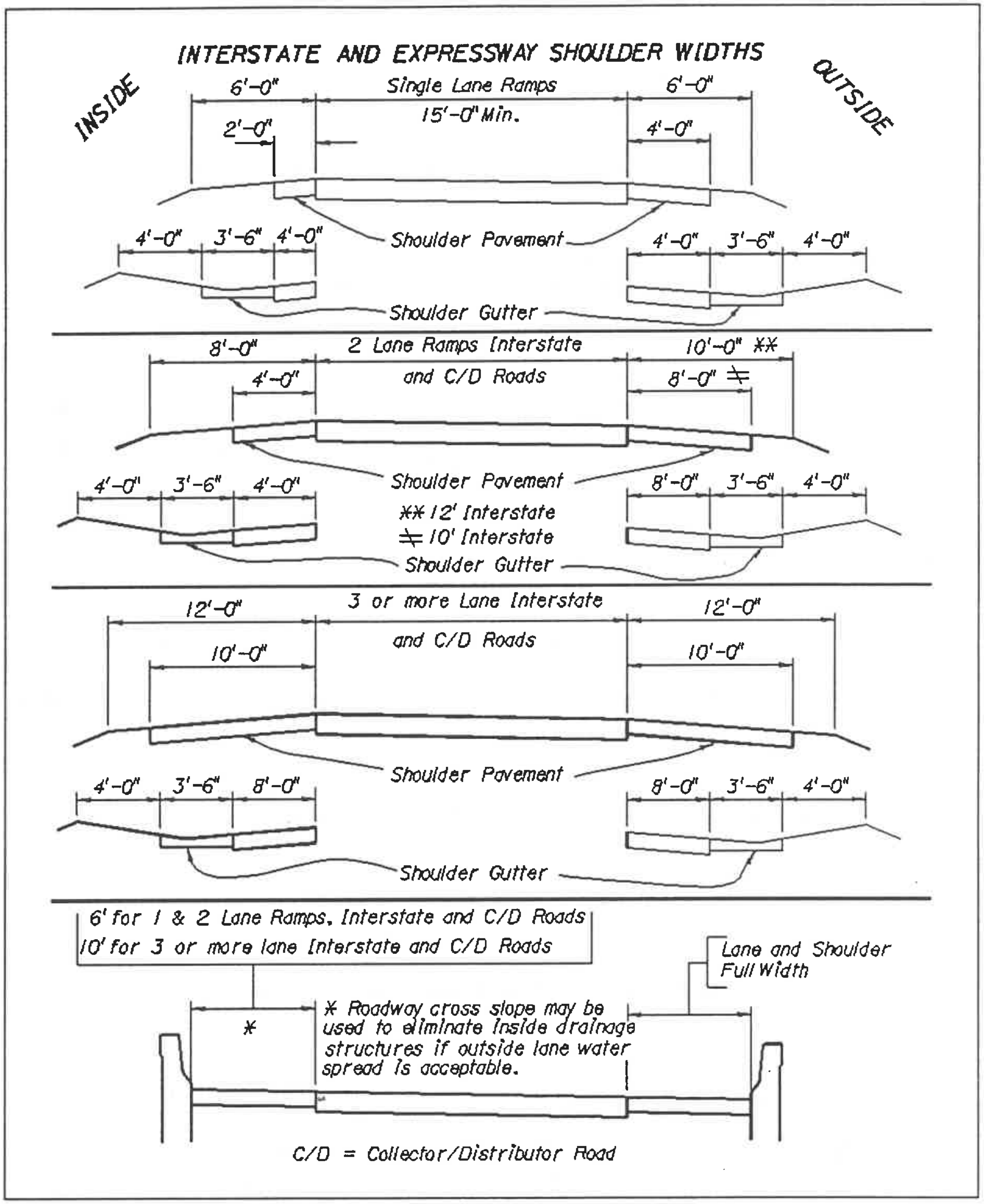
For the minimum clearances required for bridges over railroads, see Chapter 13.

2.2 Geometric Design Criteria for Bridge Widths

The following chart and roadway sections show shoulder widths for Interstate and Expressway facilities. There have been several occasions where questions have arisen in regard to the proper widths for two-lane ramps and collector-distributor roads. Deviations from the widths shown must be approved by the Roadway Design Office.

For divided highways, the District will determine the desired distance between structures. The figures on Pages 2-4 and 2-6 for divided highways indicates that a full deck is recommended if the open space between the bridges is 20 ft. or less (10 ft. each side of the ℄).

Each District Office, in deciding on a single structure deck or twin bridges, must take into account the inspection and maintenance capabilities of its personnel and equipment. If the total width for a single structure exceeds the capacity of district maintenance equipment (approximately 60 foot reach), twin structures may be specified and the open distance between structures determined by the practical capability of the maintenance and inspection equipment. This is particularly important for girder superstructures because those areas that cannot be reached by topside equipment might require catwalks, ladders or other access features. Such features will add to the cost of superstructures and must be accounted for in the initial selection of alternates.



INTERSTATE AND EXPRESSWAY SHOULDER WIDTHS
INSIDE
OUTSIDE
6'-0"
Single Lane Ramps
15'-0" Min.
6'-0"
2'-0"
4'-0"
Shoulder Pavement
4'-0" 3'-6" 4'-0"
4'-0" 3'-6" 4'-0"
Shoulder Gutter
8'-0"
2 Lane Ramps Interstate
and C/D Roads
10'-0" **
4'-0"
8'-0" ≠
Shoulder Pavement
4'-0" 3'-6" 4'-0"
8'-0" 3'-6" 4'-0"
** 12' Interstate
≠ 10' Interstate
Shoulder Gutter
12'-0"
3 or more Lane Interstate
and C/D Roads
12'-0"
10'-0"
10'-0"
Shoulder Pavement
4'-0" 3'-6" 8'-0"
8'-0" 3'-6" 4'-0"
Shoulder Gutter
6' for 1 & 2 Lane Ramps, Interstate and C/D Roads
10' for 3 or more lane Interstate and C/D Roads
Lane and Shoulder Full Width
*
* Roadway cross slope may be used to eliminate inside drainage structures if outside lane water spread is acceptable.
C/D = Collector/Distributor Road

Figure 2-1

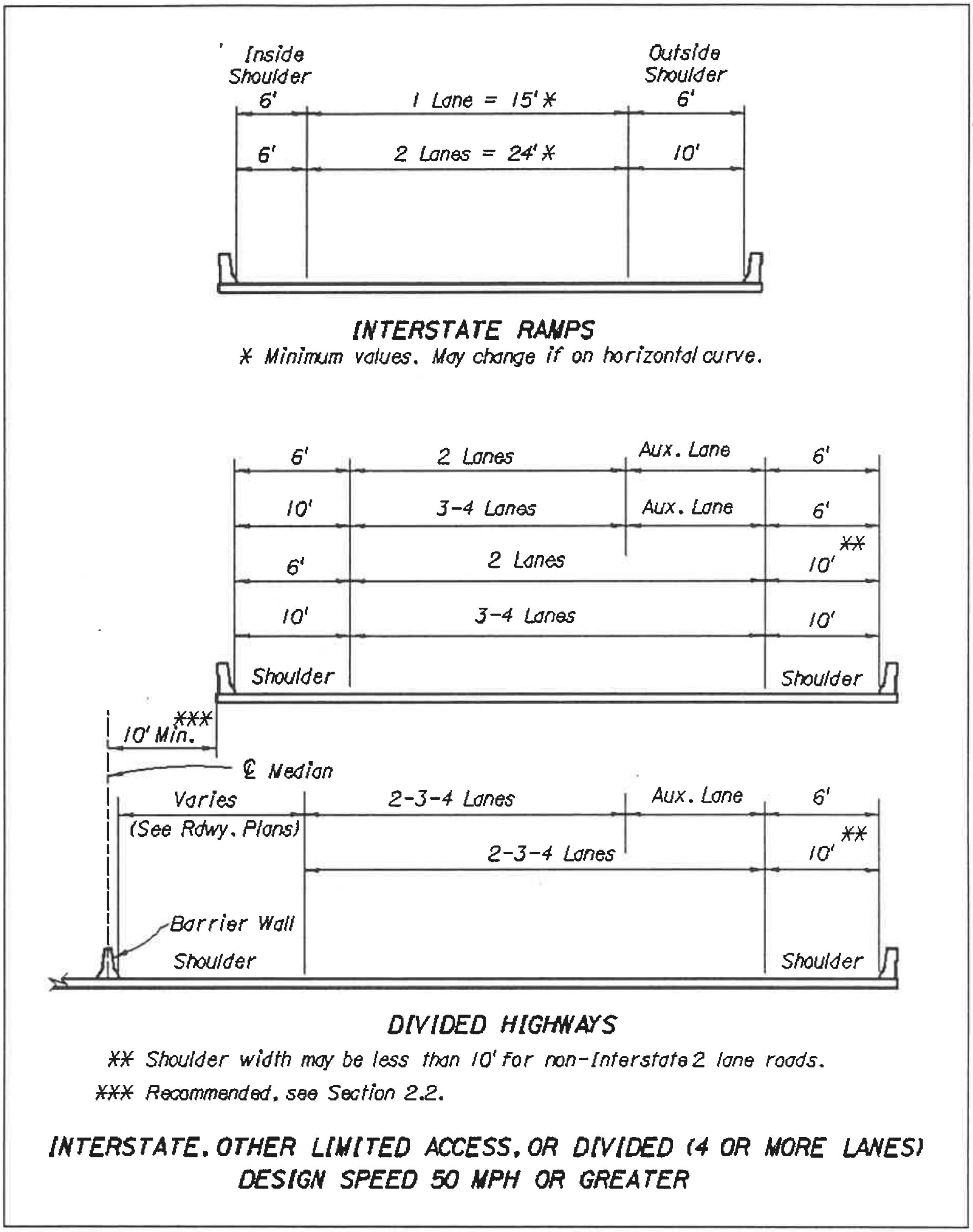

Inside Shoulder
Outside Shoulder
6'
1 Lane = 15' *
6'
6'
2 Lanes = 24' *
10'
INTERSTATE RAMPS
* Minimum values. May change if on horizontal curve.
6'
2 Lanes
Aux. Lane
6'
10'
3-4 Lanes
Aux. Lane
6'
6'
2 Lanes
** 10'
10'
3-4 Lanes
10'
Shoulder
Shoulder
*** 10' Min.
℄ Median
Varies (See Rdwy. Plans)
2-3-4 Lanes
Aux. Lane
6'
2-3-4 Lanes
** 10'
Barrier Wall
Shoulder
Shoulder
DIVIDED HIGHWAYS
** Shoulder width may be less than 10' for non-Interstate 2 lane roads.
*** Recommended, see Section 2.2.
INTERSTATE, OTHER LIMITED ACCESS, OR DIVIDED (4 OR MORE LANES)
DESIGN SPEED 50 MPH OR GREATER

Figure 2-2

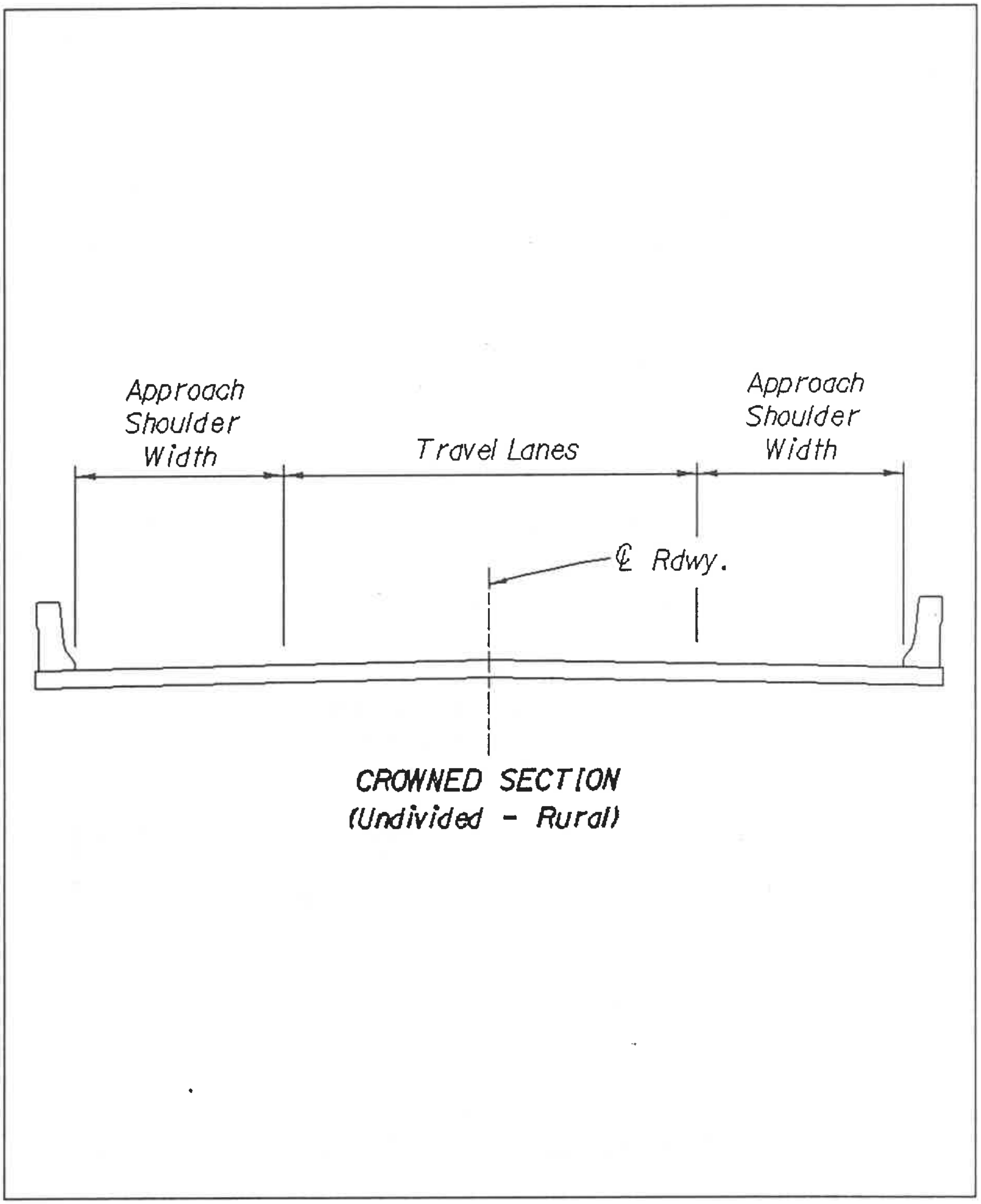

Approach
Shoulder
Width
Travel Lanes
Approach
Shoulder
Width
℄ Rdwy.
CROWNED SECTION
(Undivided - Rural)

Figure 2-3

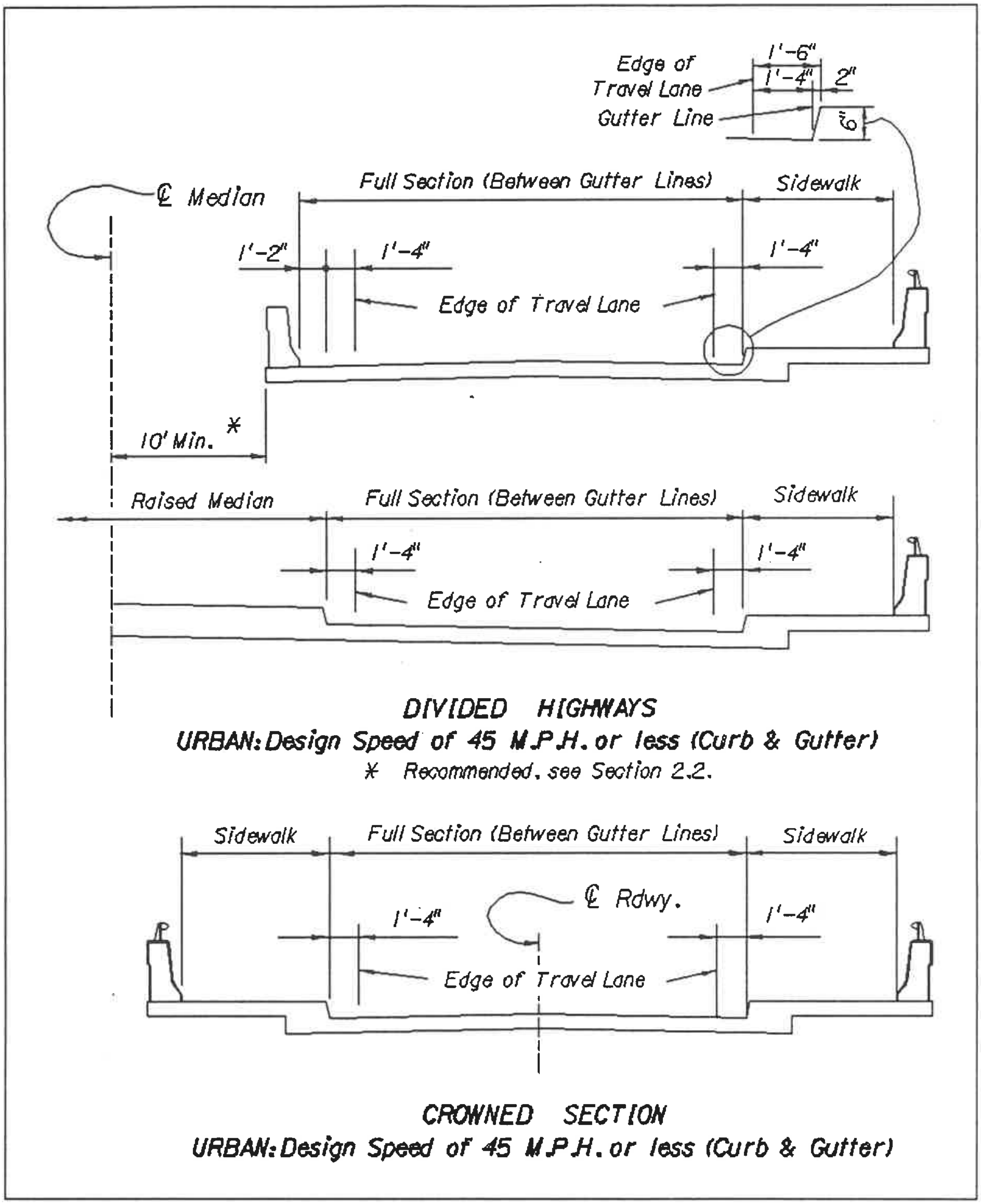
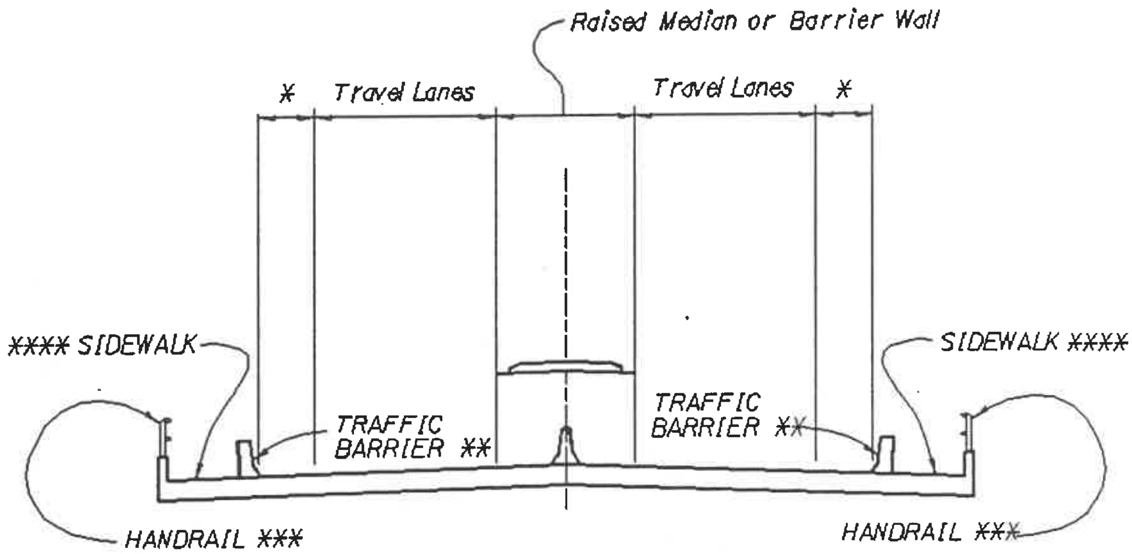

Edge of
Travel Lane
Gutter Line
1'-6"
1'-4"
2"
6"
℄ Median
Full Section (Between Gutter Lines)
Sidewalk
1'-2"
1'-4"
1'-4"
Edge of Travel Lane
10' Min. *
Raised Median
Full Section (Between Gutter Lines)
Sidewalk
1'-4"
1'-4"
Edge of Travel Lane
DIVIDED HIGHWAYS
URBAN: Design Speed of 45 M.P.H. or less (Curb & Gutter)
* Recommended, see Section 2.2.
Sidewalk
Full Section (Between Gutter Lines)
Sidewalk
℄ Rdwy.
1'-4"
1'-4"
Edge of Travel Lane
CROWNED SECTION
URBAN: Design Speed of 45 M.P.H. or less (Curb & Gutter)

Figure 2-4



DIVIDED HIGHWAYS

URBAN SECTION :Roadway Design Speed = 45 M.P.H. or less

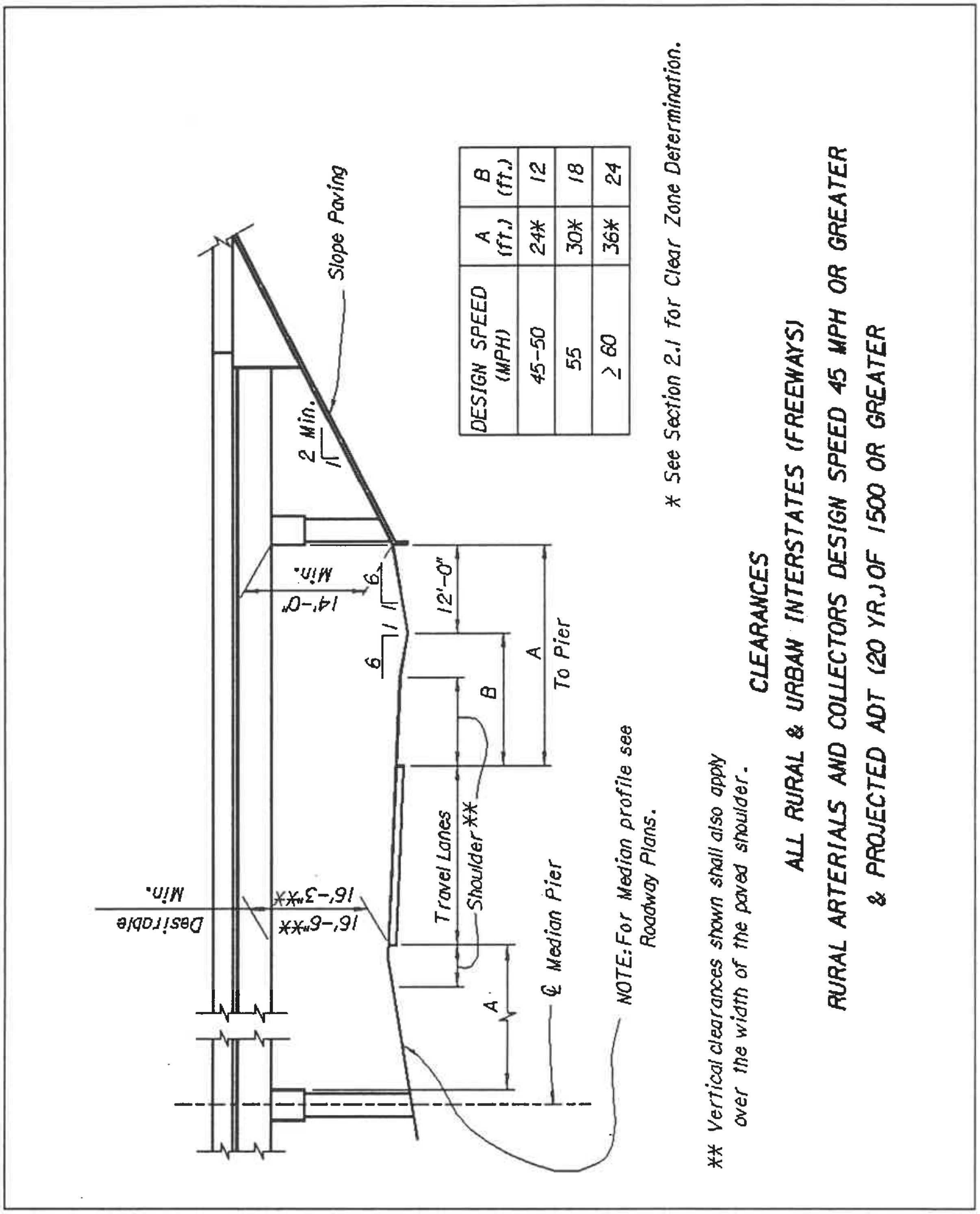
* 2'-6" Minimum, 4'-0" desirable
8'-0" minimum for long bridges (500 ft. or greater) and or high level bridges.

** Use traffic barrier and double rail pedestrian handrail if heavy pedestrian traffic is anticipated or facility is near a school, or design speeds on the bridge are 50 M.H.P. or greater.

*** If fishing is to be allowed but bicycle traffic is prohibited. the concrete parapet shall be 9" high.

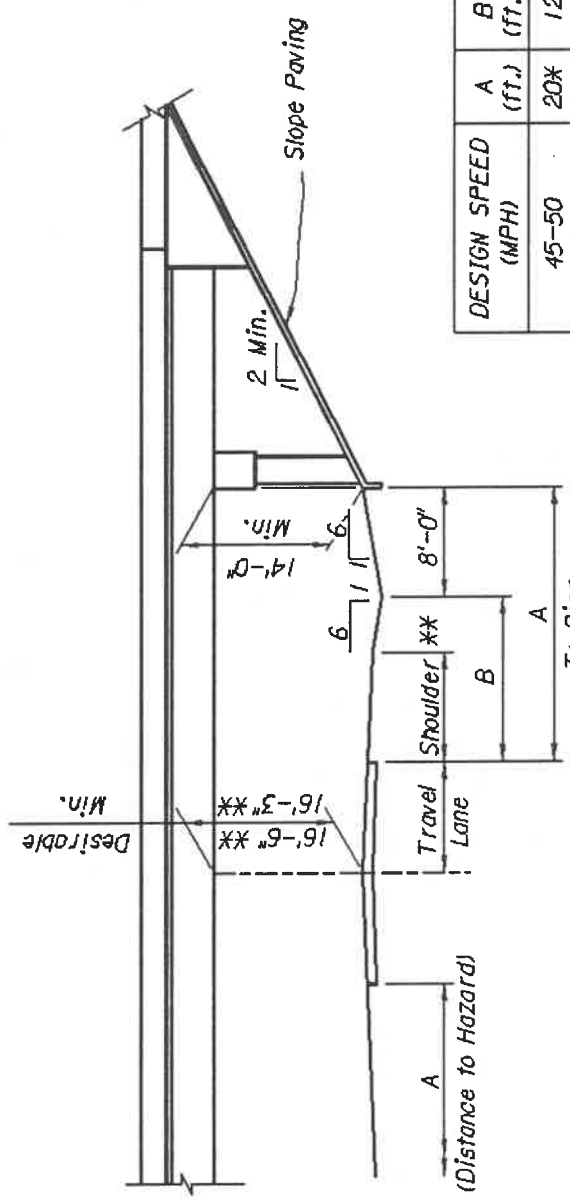
**** Sidewalks shall be a minimum of 5'-0" in clear width and may be located along one side of the bridge only.

Figure 2-5



DESIGN SPEED (MPH)	A (ft.)	B (ft.)
45-50	24*	12
55	30*	18
≥ 60	36*	24

Figure 2-6



DESIGN SPEED (MPH)	A (ft.)	B (ft.)
45-50	20*	12
55	24*	16
≥ 60	30*	22

* See Section 2.1 for Clear Zone Determination.

** Vertical clearances shown shall also apply over the width of the paved shoulder.

CLEARANCES

RURAL ARTERIALS AND COLLECTORS DESIGN SPEED 45 MPH OR GREATER & PROJECTED ADT (20 YR.) LESS THAN 1500

Figure 2-7

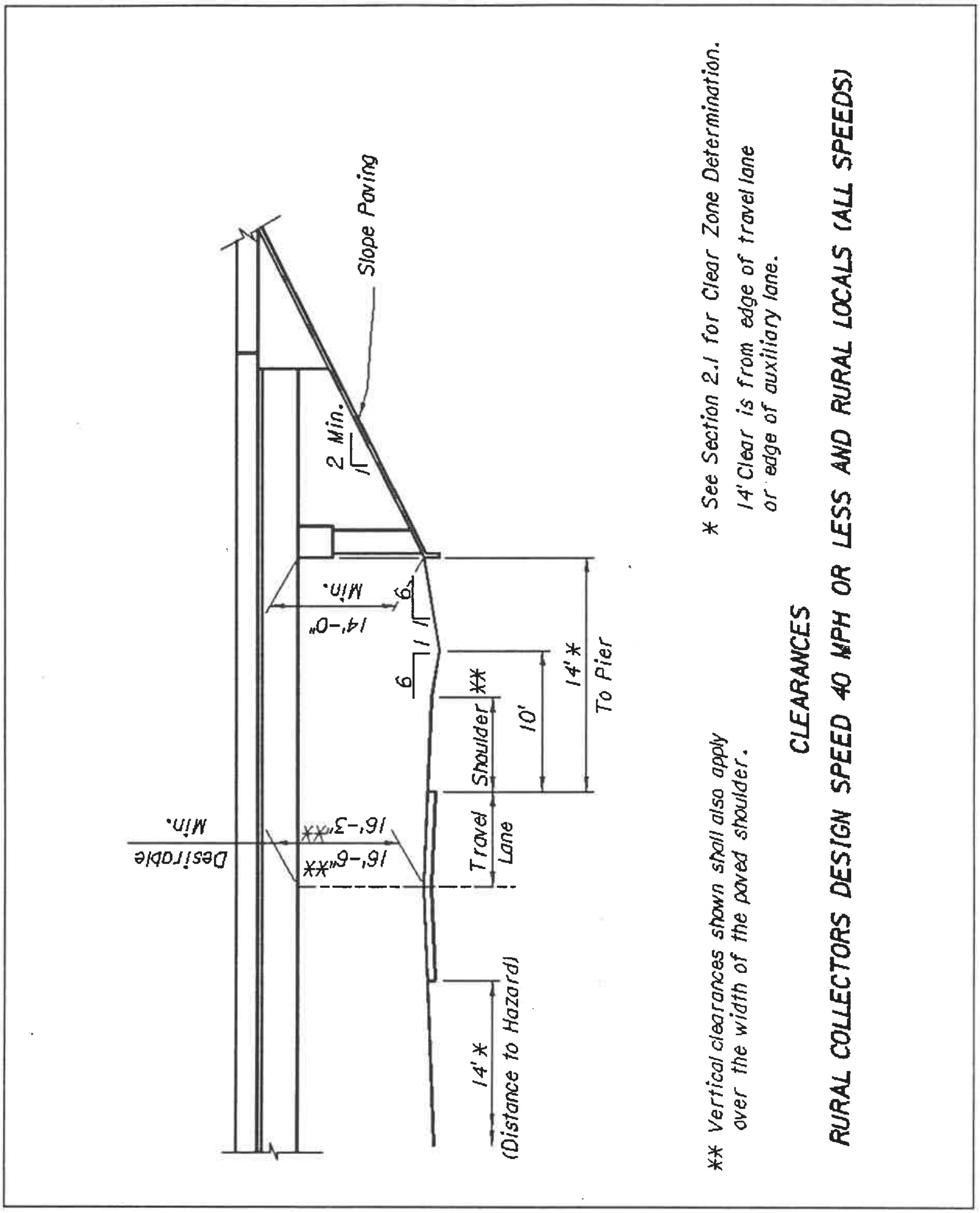
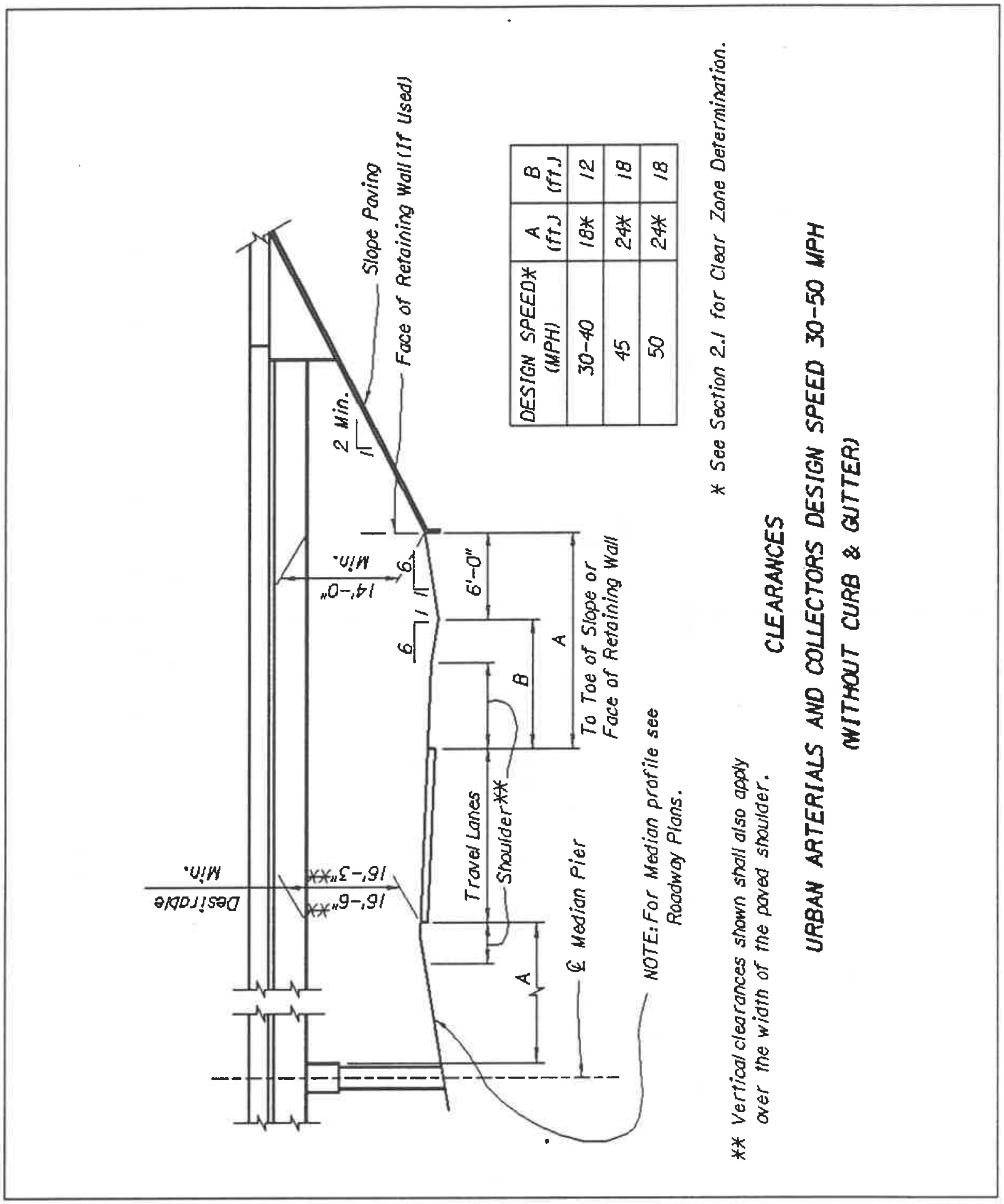

Slope Paving
2 Min.
1
14'-0" Min.
6
1
1
6
Shoulder **
10'
14' *
To Pier
Travel Lane
16'-3"** Min.
16'-6"** Desirable
14' * (Distance to Hazard)
* See Section 2.1 for Clear Zone Determination.
14' Clear is from edge of travel lane or edge of auxiliary lane.
** Vertical clearances shown shall also apply over the width of the paved shoulder.
CLEARANCES
RURAL COLLECTORS DESIGN SPEED 40 MPH OR LESS AND RURAL LOCALS (ALL SPEEDS)

Figure 2-8



DESIGN SPEED* (MPH)	A (ft.)	B (ft.)
30-40	18*	12
45	24*	18
50	24*	18

Figure 2-9

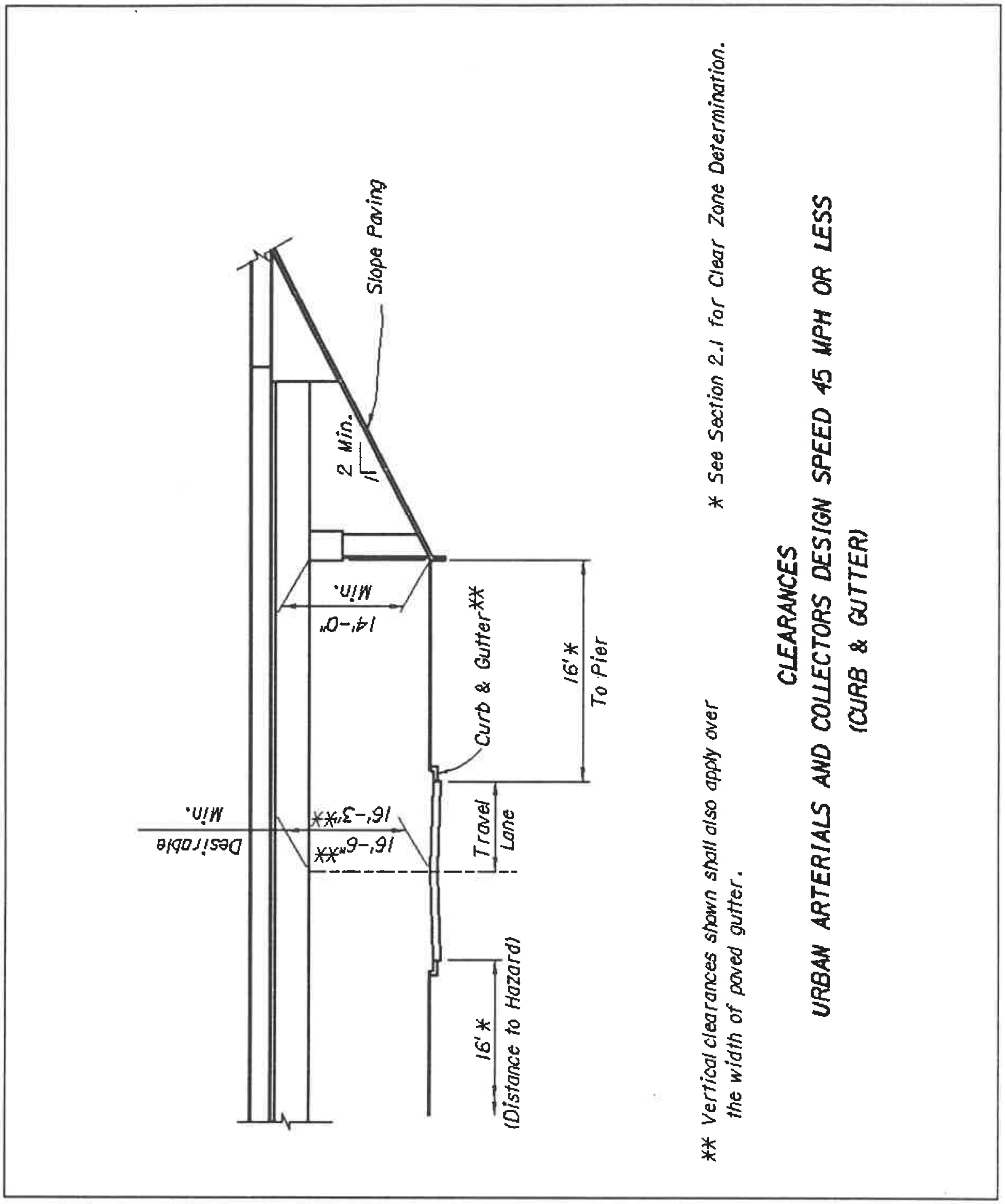

Slope Paving
2 Min.
1
14'-0"
Min.
Curb & Gutter **
16' *
To Pier
Travel
Lane
16'-3"**
Min.
16'-6"**
Desirable
16' *
(Distance to Hazard)
** Vertical clearances shown shall also apply over the width of paved gutter.
* See Section 2.1 for Clear Zone Determination.
CLEARANCES
URBAN ARTERIALS AND COLLECTORS DESIGN SPEED 45 MPH OR LESS
(CURB & GUTTER)

Figure 2-10

2.3 Vertical Clearance over Waters

Whenever possible the plans for the widening of an existing bridge shall be prepared to provide the desirable vertical clearance stipulated hereinafter; however, in any event, there must be no encroachment on the existing vertical clearance.

Normally, the superstructure for a new bridge over water shall have a vertical clearance from the water sufficient to satisfy all of the following requirements:

A. Environment:

1. For concrete superstructures classified as slightly aggressive, the desirable vertical clearance is six (6) feet minimum over the mean annual flood or control water elevation.
2. For concrete superstructures classified as moderately aggressive or extremely aggressive, the desirable vertical clearance is twelve (12) feet minimum over the mean annual flood, control water elevation or spring tide, as applicable.
3. For steel superstructures, the desirable minimum vertical clearance shall be obtained from the District Maintenance Engineer.

B. Drainage: The minimum vertical clearance requirement shall also conform with the FDOT Drainage Manual, Volume 1 - Standards, latest edition.

C. Navigation: The minimum vertical clearance for navigational purposes shall be determined and provided by the Coast Guard and/or other agencies having jurisdiction.

Information on the mean annual flood, control elevation or spring tide can be obtained from the appropriate District Drainage Engineer.

In the event that the District Office determines that the minimum clearance determined from the criteria above is not practical, economical or feasible at a given location, the District, in coordination with the State Materials Office in Gainesville, shall establish the vertical clearance criterion and advise the appropriate Structures Design Office.

Approved: Jerry J Potter

Effective: November 2, 1992
Sequence No.: 0160
Responsible Office: Structure Design
Topic No.: 625-020-103-h

CHAPTER 3

BRIDGE PROJECT DEVELOPMENT

3.1 Purpose

This Chapter will define, clarify and list the information necessary to produce an acceptable and reproducible set of contract documents (special provisions, bridge contract drawings, etc.) ready for advertisement and construction. The total bridge project development consists of the following:

- o Checklist Information (Section 3.2)
- o Bridge Development Report (BDR) (Section 3.3)
- o Design and Plans Preparation (Section 3.4).

In general, some of the Checklist Information and a complete BDR occur during the Project Development and Environmental (PD&E) phase as applicable. The Design and Plans Preparation phase occurs after approval of the BDR as applicable.

3.2 Checklist Information

Input by other disciplines either in the PD&E phase or concurrently during the Design phase directly affects the overall project. This information is required prior to starting the bridge design or done concurrently with the BDR work. The "Bridge Design Submittal Checklist" in Chapter 4 of the Structures Detailing Manual covers these items and includes:

A. Roadway Design:

Approved grades, alignment and roadway cross sections should be provided to the designer when the work is authorized to begin. Early in the Bridge Development Report phase or on the 30% Plans Stage, as applicable, the designer should verify that the grades, superelevation, etc., fulfill clearance requirements for the type of bridge under consideration. Also see AASHTO Art. 2.2 thru 2.6.

A typical bridge section, approved by the District Design Engineer, should be in hand and reviewed by the designer before completing the Bridge

Development Report or 30% Plans stage as applicable.

The designer should coordinate his bridge plans with the roadway plans paying particular attention to staged construction details and Maintenance of Traffic (MOT) requirements.

B. Hydraulics

1. Bridge Hydraulics: For additional information used for design purposes, refer to the FDOT Drainage Manual, Volume 1 - Standards, latest edition. The hydraulics work consists of the following two items:

a. The Location Hydraulics Study (LHS) is required as part of the PD&E phase and establishes the approximate length of bridge. In a vast majority of cases, this can be used in preparing the BDR with little or no modifications. In a few cases, the LHS will state that additional information is needed to finalize the length of bridge.

Depending on the magnitude and importance of the project, the District should decide if the following item should be included in the PD&E phase or if a conservative bridge length can be established and used in the BDR.

b. The Bridge Hydraulics Report (BHR) and the Bridge Hydraulics Recommendation Sheet (BHRS) both constitute a continuation and refinement of the LHS and are done in the Design phase. In a majority of cases, they serve as confirmation of the LHS. Some Districts may, as a normal routine, require that this work be done in the PD&E phase. If scour is expected to have a significant effect on the foundation design, the BHR should be completed concurrently with either the BDR or 30% Plans Stage, as applicable.

2. Deck drainage requirements will be provided by the drainage

designer. The design will be based on criteria and guidelines given in the FDOT Drainage Manual and the specific requirements of the stormwater permits; however, free-fall discharge of bridge scuppers shall be excluded over the main channel of navigable waterways. Recommendations will be made for the minimum grades, the type and spacing of deck drains and the need for an enclosed drainage system.

C. Environmental Classification - see Chapter 7, Section 7.2 of these Guidelines.

D. Right-of-Way

The designer shall take note of the proposed right-of-way limits and construction easements ensuring their adequacy for the project's needs including both the contractor's construction operations (work trestles, etc.) and future maintenance operations.

E. Geotechnical

See FHWA Checklist and Guidelines for Review of Geotechnical Reports and Preliminary Plans and Specifications available through the District Geotechnical Engineers.

The foundation report shall include recommendations as to type and design parameters. To expedite development of the bridge design, every effort should be made to secure the foundation report at the earliest possible time. If the report is not available, assumptions made by the designer for the foundation design and BDR should be clearly documented.

Where foundation data may affect bridge length and span arrangements, the Foundation Report is required prior to submittal of the Bridge Development Report or 30% Plans Stage, as applicable. Upon request, the Department will review and comment on the consultant's proposed layouts prior to the acquisition of the subsoils information. However, the Department will not approve the Bridge Development Report nor the 30% Plans Stage submittal, as applicable, when inadequate foundation data could cause the inappropriate selection of the type of bridge and/or span arrangements (see FHWA Checklist and Guidelines for Review of Geotechnical Reports and Preliminary Plans and Specifications).

F. Utilities

Major utilities that may affect the spans, pier locations or other aspects of the proposed structure should be investigated and taken into account.

If utilities (water, sewage, gas, electric, telephone, etc.) may be attached to a bridge, the Florida Department of Transportation "Utility Accommodation Guide" should be followed. Obtain this Guide from the Maps and Publications Office, Haydon Burns Building, Tallahassee.

3.3 Bridge Development Report (BDR)

A. Intent

The BDR is intended to establish all the basic parameters that will affect the work done in the Design and Plans Preparation phase. Once approved, the BDR either will define the continuing work by the PD&E consultant or establish the scope of services for the bridge design by a new consultant. Changes to the parameters after the BDR is approved could result in schedule delays and supplemental agreements. Therefore, it is critical that the Districts, FHWA (if involved), other agencies, and the Structures Design Office recognize the purpose and importance of the BDR.

The BDR should contain only supportable and defendable statements. Subjective opinions or unsubstantiated statements are not acceptable. All arguments must be clearly and logically defensible with calculations, sketches, or other technical data.

The work necessary, in this phase, depends on the project's complexity and will be determined during development of the conceptual report in the PD&E phase. However, this work will typically consist of the following:

1. Bridge Widenings: The Bridge Development Report (BDR) phase may not be necessary and the work effort should be so noted in the Scope of Work. The District Structures Design Engineer should be involved in making the decision. However, structural possibilities and economics should be thoroughly investigated to determine if it would be more appropriate to replace rather than

widen the bridge. This is particularly true at sites where the existing bridge condition is marginal, where there has been a record of serious flooding or scouring, when the widening is part of a route improvement with a high potential for attracting traffic, if the existing bridge has a history of problems including ship impacts, or the inventory rating is less than required by AASHTO (i.e., HS20 or HS15 for collection facilities) and cannot be improved.

The BDR, if required, will be submitted to the FDOT for approval prior to start of the Design and Plans Preparation phase.

2. Minor Grade Separations or Small Water Crossings: The BDR phase may not be necessary but the work effort will be noted in the design scope of work since the viable types of superstructures and substructures could be limited. The BDR if required will be submitted to the FDOT for approval prior to the start of the Design and Plans Preparation phase.

3. Major Bridges (including Movable) and Major Interchanges: Unless otherwise directed by the Department, the BDR is required and all possible structure types must be considered. The BDR must be submitted to the FDOT for approval before starting the Design and Plans Preparation phase.

For movable bridges, the designer will provide studies evaluating operator visibility and line-of-sight data and/or sketches showing marine traffic approaching the bridge as well as vehicular traffic under all conditions of span opening thus optimizing the location of the bridge tender's house.

Unless otherwise directed by the Department, the BDR for moveable bridges must also consider at least one fixed bridge alternate during the PD&E phase.

B. Report Contents: The BDR when required will establish what alternative(s) will be carried forward in the Design and Plans Preparation phase.

1. Purpose: The BDR will consider any number of possible substructure and superstructure alternatives and will support those options carried forward and developed in the Design and Plans Preparation phase. Cost effective span lengths shall be finalized allowing early start of additional

geotechnical exploration.

When alternate designs are considered, equality between the alternates is essential in ensuring equitable competition and optimum cost-effectiveness. This equality includes achieving uniformity of design criteria, material requirements and development of unit costs.

2. Format: The report shall use 8-1/2x11 pages with drawings on larger sheets, if necessary, but folded to fit the report size. The report shall be neatly written and the contents presented in a logical sequence with narrative, as required, to explain the section contents. An Executive Summary shall compare the relative features and costs of the alternates considered and recommend alternate(s) to be carried forward into the Design and Plans Preparation phase.

The BDR should be as self contained as possible by including all arguments that establish, justify, support, or prove the conclusions. It is, nevertheless, acceptable to make reference to other documents that will be included in the final submittal package; however, any documentation that will help emphasize a point, support a statement or clarify a conclusion should be used. Such documentation may include drawings, clear and concise views, or other such illustrated information that can assist in presenting design intents and solutions.

3. Contents: Information other than cost, affecting the selection of an alternate, shall also be included. This requires timely completion and submittal of the Bridge Design Submittal Checklist (Structures Detailing Manual, Chapter 4) items such as hydraulics reports including potential scour problems, consideration of available geotechnical survey data, life cycle maintenance, construction time and staging assumptions, constructibility, maintenance of traffic, etc. Various methods of handling traffic during construction should be thoroughly investigated.

In addition to the material requested in the Checklist, general notes regarding the project's history or other information useful to the reviewer should be submitted. Reference should be made to available existing documents or reports with which the reviewer may not be aware.

The data provided by others should be thoroughly reviewed by the bridge designers and if deemed insufficient or in error should be brought to the attention of the provider. Some common examples of insufficient data are vague location of existing utilities, lack of existing ground profiles, missing proposed right-of-way lines, and unspecified clearances on existing bridges. The data in the foundation and hydraulic reports contain recommendations; however, these must be reviewed and challenged if deemed inappropriate for the designer's purposes.

The major items considered in this phase shall be:

a. Span arrangement, proportion, and bridge length: Column and/or pier locations are subject to vertical and horizontal clearance requirements (AASHTO Bridge Specs. and Chapter 2 of the Structures Design Guidelines) and/or hydraulic considerations. After these considerations are met, span lengths are governed by economics and aesthetic considerations.

Piers located in a divided highway median must be protected from traffic when located within the clear zone. Generally this will be accomplished by guardrail or concrete barrier; however, the specific requirements for each occurrence must be coordinated with the District Design Office.

Superstructure depths (grade separation structures in particular) shall be kept to the minimum consistent with good engineering practice. Recommended depth/span ratios are shown in AASHTO Bridge Specs.

The length of the bridge will be determined by:

1) stream opening required by the Bridge Hydraulic Report or environmental considerations.

2) railroad clearances and cross-sections (over crossing) (see AASHTO Bridge Specs. and Chapter 13 of these Guidelines) (Contact the railroad company to verify clearances.)

3) width and cross-section of lower roadway (see AASHTO Bridge Specs. Art. 2.3.1), use or non-use of retaining walls and/or

embankment slopes.

b. Statical System: The economic and engineering advantage of simple span vs continuous spans will be addressed.

c. Superstructure: Consider prestressed concrete girders (AASHTO or Florida Bulb-Tee); double-tee sections, slabs, if appropriate; steel rolled sections or plate girders; steel or concrete box girders; other approved sections.

d. Combinations: The above in combination with various foundation types, such as piles, drilled shafts and/or spread footings. For piles and drilled shafts, assume size, length, and capacities from available geotechnical information. For spread footings, allowable bearing pressure should also be assumed from available geotechnical information.

e. Quantity estimates: For minor bridges rough quantities (such as reinforcing steel based on pounds per cubic yard of concrete) may be sufficient. However, for major and complex bridges the degree of accuracy may require more exact calculations keeping in mind that the intent is to establish relative costs between alternates and not necessarily to the accuracy required for an Engineer's Final Estimate. Also, for major and complex structures it may be necessary to develop unit costs from an analysis of fabrication, storage, delivery and erection costs.

f. Unit costs: Data available from the FDOT or contractors and suppliers should be used to arrive at unit costs. The sources of all price data should be recorded for later reference. The historical unit price information from bid item information can be obtained on a fiscal year basis from the Department's Construction Contract History and is available from:

Florida Department of Transportation
Map and Publication Sales
Mail Station 12
605 Suwannee Street
Tallahassee, FL 32399-0450
Telephone (904)488-9220

All designers with an active user identificaiton (USERID) for the Department's mainframe computer can access the same database from which the Construction Contract History report is compiled. The designer, however, can select from a wide range of specified parameters to limit results to as small as one item in one district for one particular letting or as large as a statewide use report. The designer's District Contract Estimating System (CES) Coordinator can provide specific information on and help in using the mainframe estimating database.

g. Develop cost curves: For each alternative establish the most economical span arrangement, i.e., minimum combined superstructure and substructure cost.

C. Aesthetics: Any bridge design must integrate three basic elements: efficiency, economy and elegance. Regardless of size and location, the quality of the structure and its aesthetic attributes should be carefully considered by the designers in compliance with 336.045, Florida Statutes, as amended by Section 5, Chapter 92-152, Laws of Florida (CS/WB 2439). Achieving the desired results involves:

- full integration of the three basic elements listed above
- the designer's willingness to accept the challenge and opportunity presented.

A successful bridge design will then be elegant or aesthetically pleasing in and of itself and compatible with the site by proper attention to form, shapes and proportions. Attention to details is of primary importance in achieving a continuity of form and lines.

1. The designer must consider the totality of the structure as well as its individual components. A disregard for this continuity or lack of attention to detail can spoil the best intent. Formulas cannot be established but authors such as David P. Billington, the ACI's Aesthetic Considerations for Concrete Bridges and the TRB's Bridge Aesthetics Around the World can guide the designer. In bridge aesthetics the designer is dealing with the basic structure itself; not with enhancement, additions, or other superficial touches.

The challenge differs for a major and a minor structure. Indeed, the challenge may be greater the smaller the project. Major structures, because of their longer spans, taller piers, or curving geometry offer opportunities not available for minor bridges.

Some basic guidelines where aesthetics may play a more important role are:

a. Bridges highly visible to large numbers of users (maritime and/or motorists).

b. Bridges located in or adjacent to parks, recreational areas, or other major public gathering points.

c. Pedestrian bridges.

d. Bridges in urban areas in or adjacent to commercial and/or residential areas.

e. Multi-bridge projects, such as interchanges, or corridors should attain a conformity of theme and unifying appearance. Avoid abrupt changes in structural features.

2. Based on the above, the District Structures Design Engineer (or other person requested by the District Secretary) in conjunction with the PD&E office, should determine the level of aesthetic effort warranted on a project early in its development. When significant aesthetic expense is proposed, such as is the case with Level Three below, Federal projects require legitimate written justification.

3. Three levels of aesthetic effort are suggested below that should be developed as part of the BDR phase or 30% Plans Stage as applicable.

a. <u>Level One</u> Consists of cosmetic improvements to conventional Department bridge types, such as using color pigments in the concrete, texturing the surfaces, modifications to facia walls and beams, and surfaces, or more pleasing shapes for columns and/or caps.

b. <u>Level Two</u> The emphasis is on full integration of efficiency, economy and elegance in all bridge components and the structure as a whole. Consideration should be given to structural systems that are inherently more pleasing, such as hammerhead or "T" shaped piers, oval or polygonal shaped columns, integral caps, piers in lieu of bents, smooth transitions at superstructure depth change locations, etc. Level Two is preferred for Category 2 bridges.

c. Level Three The emphasis in this level applies more to the surroundings at interchanges or projects in highly urbanized areas where landscaping or unique neighborhood features need to be considered. The bridge itself shall comply with Level Two requirements. This level of work may require, at the District's option, a subconsultant (architect to consider the disposal of adjacent building styles, landscape architect, etc.) with the necessary expertise and credentials to perform the desired work.

D. Constructibility and Maintainability

All viable structure concepts shall be evaluated for constructability. Items such as member sizes, handling, fabricating, and transporting members, maintenance of traffic, construction staging, equipment access, equipment requirements, etc. should be considered. Special evaluation shall be made to insure against potential problems that may occur in obtaining permits and equipment to transport long and/or heavy members from point of manufacture to the project site. The Department's Road Use Permits Office should be contacted for questions concurring the feasibility of transporting long and/or heavy structural components. Also, considerations for future maintenance inspection shall be taken into account in the structure's design. Such considerations may include those described in Articles 2.2, 3.7 and 12.5 or the need for 6'-0" minimum headroom inside steel or concrete box girder superstructures. The intent of this requirement is to assure that all special construction and maintenance requirements are identified and appropriately reflected in the consideration of any concept that is to be recommended for design.

E. Conclusion and Intent of the BDR

Following submittal of the BDR and its analysis, the FDOT should have enough data to select either one or more alternates for development in the next stage.

3.4 Design and Plans Preparation

A. General: Within this phase of work, for both Category 1 and 2 structures, there are three stages of work; the 30% Plans Stage (where design is about 60% complete and the contract plans are brought up to approximately 30% of the total work effort), 90% Plans Stage (where the design is complete and the contract plans are brought up to approximately 90% of the total design work effort) and 100% plans stage (which constitutes 100% of the work excluding any further corrections and/or modifications). For some Category 2 structures there may be an additional review, the 75% plans stage, where the design is about 95% complete and the drawings are approximately 75% complete.

After each of the stages, except the 100% plans stage, review comments from the FDOT are sent to the designer by letter and a marked-up set of prints. The designer must address each of the comments in writing and resolve each prior to the next submittal. It is emphasized that each stage, except the 100% Plans Stage, is the preparation for the following stage. Also, for any stage, items and drawings from a preceding stage are automatically included.

B. Plans Submittal:

1. Schedule

The consultant and the District Project Manager (DPM) are responsible for preparing the schedule of submittals early in the project. For Category 1 structures, the DPM shall consult with the District Structures Design Engineer (DSDE) to assure that the schedule is acceptable to both the consultant and the FDOT. For Category 2 structures, the DPM shall consult with the SDO.

2. Submittals

a. Documents (as required by the Scope of Services)

(1) BDR

(2) 30% Plans (see Item 3.4.C below)

(3) 75% Plans (see Item 3.4.D below)

(4) 90% Plans (see Item 3.4.E below)

(5) 100% Plans (see Item 3.4.F below)

a) Check prints

b) Tracings

b. Time

(1) The BDR, 30%, 75%, and 90% Plans are submitted at their scheduled times.

(2) The 100% Plans submittal is divided into two distinct phases. First, prints of the Tracings are submitted 30 days prior to the District's Plans Production Date (PPD) which is established by the DPM. Secondly, once notified by the FDOT, the Tracings and all other documents are submitted to the District at the PPD.

(3) Once all necessary documents are ready, the District shall submit the structure package, along with other work, to Tallahassee at the Plans to Tallahassee (PTT) date. After PTT submittal and prior to Authorization, the consultant may be required to make additional corrections either at his office, the District Office or the Central Office (Tallahassee). Revisions shall be made in accordance with Chapter 20 of the FDOT's Plans Preparation Manual.

C. Thirty Percent (30%) Plans:

The 30% plans shall be in compliance with Chapter 4 of the Structures Detailing Manual with all necessary checking substantiated by initials in the title blocks of plan sheets and on each page of calculations, with appropriate dates, in accordance with Chapter 20 of these guidelines. The Key Sheet (if required), General Notes, Retaining Wall Control Drawings (if required), Plan and Elevation, Bridge Hydraulic Recommendation Sheet (if required), Report of Core Borings and Typical Roadway Section shall be in final form and shall be nearly complete. Thirty percent (30%) plans for superstructure and substructure may be presented in either final or temporary form; however, the minimum information specified in the Structures Detailing Manual for the 30% stage shall be presented on the drawings (see Item No. 2 below). In addition to the drawings discussed above, the 30% plans package shall include the information listed below. If a Bridge Development Report has been submitted, major items should have been resolved and the designer may continue with plans development, incorporating any review comments by FDOT into the final plans. If a BDR has not been submitted, the 30% plans submittal shall include justification for the submitted structure and the designer shall not proceed with final design until review of the 30% plans submittal is complete.

1. Checklist (See Chapter 4, Structures Detailing Manual)

The Checklist, except for the Plans, should be completed and submitted, if not previously submitted, with the BDR. Those items not previously completed must be completed and included with this submittal. The District Project Manager shall ensure that the properly executed "Bridge Design Submittal Checklist" accompanies the 30% bridge plans submitted.

2. The list of drawings and information that constitutes, as a minimum, the 30% Plan Submittal is given in Chapter 4 of the Structures Detailing Manual.

Thirty percent (30%) plans for movable bridges will include evaluations of alternative drive and control systems. The designer will recommend the system to be selected, based upon considerations of reliability, life-cycle and first cost comparisons, maintainability and constructability, and effect upon maintenance of traffic, both during construction and during the life of the bridge.

D. Seventy-five Percent (75%) Plans Stage

The list of drawings and what constitutes, as a minimum, the 75% Plans Stage is given in Chapter 4 of the Structures Detailing Manual. Plans for this submittal may be required for Category 2 structures only (see Chapter 1). Generally the design at this stage should be 95% complete and the drawings approximately 75% complete. Furthermore, all comments from the 30% Plans Stage shall have been resolved.

E. Ninety Percent Plans Stage:

After submittal of the plans at the 30% Plans Stage for Category 1 Structures or 75% Plans Stage for Category 2 structures, the designer should continue developing the project up to the 90% plans stage. Prior to submittal of the 90% Plans, the designer should have received and resolved all previous review comments. The contents of this submittal are explained in Chapter 5 of the Structures Detailing Manual.

F. One Hundred Percent (100%) Plans Stage:

After resolution of the 90% Plan Stage comments, the designer shall make all authorized changes necessary to complete the project. Any changes to the design and/or drawing found by and made by the consultant after receipt of 90% review comments should be noted and submitted to the FDOT with the check set submittal. The intent is to avoid surprises and/or objections by FDOT that would impact authorization and delay the project.

The 100% Plans Stage submittal constitutes the completion of the work and is referred to as the 100% stage. As previously mentioned and unless otherwise directed, a set of check prints shall be submitted (the consultant retains the original drawings) to the DSDE for Category 1 structures or the SDO for Category 2 structures. THE 100% SUBMITTAL MUST BE RECEIVED BY THE FDOT REVIEW OFFICE AT LEAST 30 DAYS PRIOR TO THE PDD DATE. NO CHANGES TO THE PLANS CAN BE MADE AFTER PRINTING FOR THIS SUBMITTAL UNLESS SO AUTHORIZED BY THE FDOT REVIEW OFFICE.

Within the 30 day period allotted, the designer will receive notification of either additional changes/corrections or that the Final Plans can be submitted as is.

If after receipt of changes/corrections the designer finds additional changes/corrections he must notify and discuss these with either the DSDE or the SDO.

Once all changes/corrections are made or if no changes/corrections are necessary, the designer should submit all his work to the District prior to or at the PPD.

Submittal of this stage of the work shall include the original tracings plans (each sheet signed but not sealed), one record set of prints with each sheet signed and sealed, quantities book assembled as specified in the Department's Computation Manual (available from Maps and Publications Sales in Tallahassee), signed and sealed special provisions (if required) and the design calculation books (signed and sealed).

3.5 Plans Assembly

A. General

The Structures Detailing Manual should be consulted for plans assembly, materials, content of submittals, and other drafting information. The following items are covered in the Structures Detailing Manual under the Chapter indicated:

1. Drafting Material: Chapter 1
2. Standard Drawings: Chapter 5
3. Drawing Numbers: Chapter 5
4. Sheet Numbers: Chapter 5
5. Approach Slab Sheets: Chapter 5
6. Work Program Item No. (W.P.I. No.): Chapter 2
7. Bridge Numbers: Chapter 2
8. Summary of Bridge Pay Items: Chapter 3
9. Engineer of Record and Consultant Name: Chapter 2
10. Scales: Chapter 2

3.6 Overhead Sign Structures

Normally the Engineer of Record (EOR) will be responsible for the design of all overhead sign structures whether ground mounted or supported on a structure. The responsibility is for the entire sign structure design, including the supports and foundations, as well as all details necessary to allow preparation of conventional shop drawings by the Contractor. The Engineer of Record will be responsible for the shop drawing review in accordance with Chapter 19.

When the scope of services does not designate the overhead sign structure design and detailing to the Engineer of Record, the EOR will still be responsible for the design of any support built integrally with the structure (i.e., supported on a bridge or retaining wall) and shall show all details (i.e., anchor bolt size, bolt circle, bolt length, etc.) as well as the design assumptions (wind loads on sign face and sign support structure, moments, torsional loads, dead loads, etc.) used in the design. In this event, the Contractor shall be wholly responsible for the design of the sign structure and will submit shop drawings prepared, signed and sealed by his Specialty Engineer and reviewed by the EOR.

Additional requirements for overhead sign structures are also included in Chapter 21.

3.7 Review for Constructibility and Maintainability

A. Purpose: To provide reasonable and practical use of fabrication and construction techniques and equipment without overloading and/or overstressing components, provide for proper material handling, provide safe maintenance of traffic, provide a safe and secure construction sequence, etc. Additionally, provide features which will retard bridge deterioration, permit reasonable access to all parts of the bridge for inspection and performance evaluation, and provide features to facilitate replacement of worn, damaged and/or deteriorated bridge components.

B. Responsibility: For Category 1 Structures, it will be the responsibility of the project manager or his designate to coordinate a review of the 30% plan submittal by the appropriate construction and maintenance personnel for constructibility and maintainability.

For Category 2 Structures, it will be the responsibility of the Structures Design Office to coordinate a review of the 30% plans submittal by appropriate construction and maintenance personnel for constructibility and maintainability.

The comments received from the constructibility and maintainability reviews shall be implemented or justification provided for any other action taken.

C. Generally, the following types of bridge will require these reviews:

1. Movable Bridges (See Chapter 15)

2. Complex and innovative designs
3. Bridges with span lengths in excess of 250 feet
4. Bridges in extremely aggressive environments which have a total bridge length of 1,000 feet or greater.
5. Bridges over saltwater where vertical clearance between superstructure and water is 12 feet or less.
6. Bridges with unusual foundation problems.
7. Bridges with complex maintenance of traffic involvement.

These reviews and the scheduling of meetings should be established when preliminary plans are submitted for review thereby allowing adequate time to incorporate the desired features in the final plans.

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Chapter 4

STRUCTURAL LOADINGS, WEIGHTS AND MATERIAL PROPERTIES

4.1 Loadings

For Florida bridge designs, the following bridge loadings shall be applied in addition to those required by AASHTO:

A.	Allowance for future surfacing	15 lbs. per sq. ft.
B.	Prestressed I-Girder Construction where skew angle is equal to or less than 30^0 (beam design only)	5% AASHTO Live Load Increase (See Art. 10.3)

4.2 Earthquake Provisions

A. New Construction

All bridge elements for new construction shall be designed in accordance with the latest AASHTO Standard Specifications for Seismic Design of Highway Bridges. Florida falls within the zone with an acceleration coefficient of less than 0.05 and a Seismic Performance Category (SPC) of A for both the Importance Classification (IC) of I and II.

B. Widening

All bridge elements for the widening of an existing bridge structure classified as a major widening as defined in Article 16.6 shall comply with the earthquake provisions of Article 4.2.A above.

Earthquake provisions for the widening of an existing structure classified as a minor widening by Article 16.6 shall be considered by the Department on an individual basis.

4.3 Wind Loads

A. Wind Loads on Bridges

In accordance with AASHTO Article 3.15, bridges shall be designed for wind pressures that increase with the height of the superstructure above the average existing ground (water) elevation in the immediate vicinity of the bridge. The elevation of the superstructure shall be taken as the profile grade elevation at the centerline of the substructure. All substructure elements for a given superstructure unit may be designed for the average elevation of the piers in that unit. The design wind pressures for various heights above the average existing ground (water) elevation are:

Height (ft)	Pressure (psf)
0 <H ≤ 35	50
35 <H ≤ 45	55
45 <H ≤ 60	60
60 <H ≤ 75	65

To account for the higher wind velocities which occur in South Florida, the wind pressures shown above shall be increased by 20% for bridges located in Palm Beach, Broward, Dade and Monroe counties.

Wind pressures for bridges over 75' high or with unusual structural features shall be considered by the Department on an individual basis.

B. Wind Loads on Other Structures

Wind loads for miscellaneous highway related structures are specified in Chapter 21 of these guidelines (see Figure 21-3).

4.4 Weights

Unless a more refined analysis is performed by the designer, the following weights of structural materials and elements may be used:

Item	Unit*	Weight
Barrier, Traffic Railing	PLF	418
Barrier, Median	PLF	483
Concrete, Counterweight	PCF	146
Concrete, Structural	PCF	150
Soil, compacted	PCF	115
Stay-in-Place metal forms	PSF	20**

* NOTE: PLF = pounds per linear foot

PSF = pounds per square foot

PCF = pounds per cubic foot

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

4.5 Material Properties

Unless otherwise stipulated or obtained from testing, the modulus of elasticity of concrete at 28 days shall be taken from Table 4.1.

TABLE 4.1 Modulus of Elasticity

	Normal Aggregate Concrete Density (pcf)				Florida Limerock Aggregate Concrete Density (pcf)			
	140	145 *	150	155	140	145 *	150	155
Strength (psi)								
3400	3.2	3.4	3.5	3.7	2.9	3.0	3.2	3.3
4000	3.5	3.5	3.8	4.0	3.1	3.3	3.5	3.6
4500	3.7	3.9	4.1	4.3	3.3	3.5	3.7	3.8
5000	3.9	4.1	4.3	4.5	3.5	3.7	3.9	4.1
5500	4.1	4.3	4.5	4.7	3.7	3.9	4.1	4.3
6000	4.2	4.5	4.7	4.9	3.8	4.0	4.2	4.4
6500	4.4	4.7	4.9	5.1	4.0	4.2	4.4	4.6
7000	4.6	4.8	5.1	5.3	4.1	4.3	4.6	4.8

All values X 10^{6} psi.

* These columns shall normally apply.

NOTE: Densities shown are densities of the concrete mix itself excluding any and all embedded items such as reinforcing, ties, inserts, etc.

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Chapter 5

FOUNDATIONS AND GEOTECHNICAL DATA

5.1 Core Borings For Structures

This report (drawings), which reflects the foundation data acquired from field investigations, is prepared by the Geotechnical section of the District Materials Engineer or by a Geotechnical consulting firm contracted with the FDOT.

5.2 Foundation Report

The District Geotechnical Engineer or the contracted Geotechnical firm shall issue a Foundation Report for most projects. This report shall describe the soil conditions in detail and recommend suitable foundation types. The report shall include appropriate design parameters. The designer should select the most cost effective design for the structure foundation. The report shall be done in accordance with the Department's "Soils and Foundations Manual" which is available through the Maps and Publications Office.

Conformance 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 is required when preparing Geotechnical Reports. Contact the District Geotechnical Engineer to receive a copy of this document. The appropriate checklists provided in the subject document are <u>required</u> to be properly executed and submitted as part of the Foundation Report.

In the event that the Foundation Report is prepared by a contracted Geotechnical firm, it generally will be reviewed by both the State and District Geotechnical Engineer's offices for category 2 structures and the District Geotechnical Engineer for category 1 structures. Final acceptance of the report is contingent upon the District Geotechnical Engineer's approval. Concurrence by the State Geotechnical Engineer's Office is required for all category 2 structures. The contracted Geotechnical firm is to interact in a timely manner with the District Geotechnical Engineer throughout the course of the contract/project. The scope as well as the proposed field and laboratory investigations shall be discussed with the District Geotechnical Engineer prior to commencing any operations.

5.3 Pile Foundations

The design engineer and the geotechnical engineer shall determine the structure load(s); pile section(s); pile type(s); pile size(s); location type and number of test piles; and test pile lengths. The pile capacity shall be verified in the field by means of test piles. See section 5.10 for scour considerations.

The design load for prestressed concrete (PSC) piles shall not exceed the following unless specific justification is provided:

18 inch............................100 tons

20 inch............................120 tons

24 inch............................150 tons

30 inch............................200 tons

The design loads on steel piles shall be coordinated with the Department's District Structures Engineer.

Generally, unless otherwise noted in the Foundation Report, the minimum size of prestressed concrete piles shall be 18-inch square. Exceptions to this minimum size requirement are piles for fender systems and some widenings of existing bridges where smaller piles are feasible and will match existing piles. Pile spacing center-to-center shall be at least three (3) times the least width of the pile.

5.4 Test Piles

Test piles are installed and tested in order to verify the pile capacity, the pile driving system, the pile driveability, to determine production pile lengths and to establish driving criteria. Test piles include static load test piles, dynamic load test piles, and confirmation piles ("unloaded test piles"). As a minimum, one test pile shall be located every 200 feet of bridge length with a minimum of two test piles per bridge structure. These requirements shall apply for each size and pile type in the bridge except at end bents. For bascule piers and high level crossings that require large footings or cofferdam type foundations, a minimum of one test pile shall be driven at each pier. The need for test piles in the end bents should be evaluated on all projects and shall be based upon site conditions, soil profile variability and potential cost savings.

Test piles shall be required for bridge widening and replacement projects unless pile driving records for the existing structure include enough information (i.e., stroke length, hammer type, cushion type, etc.) to adequately estimate the obtained capacity.

When determining the location of test piles, traffic maintenance requirements, required sequence of construction as well as geological conditions and pile spacing must be considered. If phase construction, preferably locate all test piles in the first portion to be constructed. The Geotechnical Engineer shall verify the adequacy of the test pile locations.

Because these piles are exploratory in nature, they may be driven harder, deeper, and to a greater bearing value than required for permanent piling or may be used to establish soil freeze parameters. (See FDOT Specifications Section 455). Therefore, the designer shall take these facts into consideration in preparing estimated lengths. Test piles should be at least 15′ longer than the estimated production piles in addition to the length required by the load frame geometry when static load tests are used. The designer shall coordinate his recommended test pile lengths and locations with the District Geotechnical Engineer, or Geotechnical Consultant, prior to finalization of the plans.

5.5 Load Tests

Load tests include static load tests and dynamic load tests. In general, the more variable the subsurface profile, the less cost effective are static load tests. When site variability is an issue, options include additional field exploration, more laboratory samples, in-situ testing and pullout tests. Both construction phase and design phase load testing should be investigated. Factors to be considered when evaluating the benefits and costs of providing loaded test piles, include, but are not limited to the following: soil stratigraphy, design loads, pile type and number, type of loading, testing equipment and mobilization.

1. Static Load Test

   Test piles can be subjected to static compression, tension or lateral test loads. Static load tests may be desirable when foundation investigations reveal sites where the soils cause concern regarding developing the required pile capacity at the desired depths, and/or the possibility exists that considerable cost savings will result if higher pile capacities can be justified. Also static load tests will reduce driving effort because of lower factor of safety is applied to the design loads in accordance with the specification section 455. If static load test piles are called for on the design plans, the number of required tests, pile type(s), pile size(s) and load(s) shall be shown in the plans.

   As a general rule and for general guidance only, static load testing should be considered when the pile design loads exceed the loads addressed in section 5.3.

   Static load tests shall be designed to test the pile to failure as in the specifications section 455. Consult the District Geotechnical Engineer for the specifics.

2. Dynamic Load Test

   Dynamic load testing consists of instrumenting piles with strain transducers and accelerometers which measure pile force and acceleration, respectively, during driving operations. A pile driving analyzer (PDA) unit is used for this purpose. All piles which will be statically load tested shall first be subjected to dynamic load testing. When dynamic testing is required, <u>all</u> test piles shall have dynamic load tests. The need for dynamic load tests shall be addressed in the geotechnical report. This requirement shall be shown by means of a note on the pile layout sheet.

3. Special Considerations

Load testing of foundations which will be subjected to subsequent scour activity require special attention. The necessity of isolating the resistance of the scourable material from the load test results must be considered. This shall be discussed with the District Geotechnical Engineer.

5.6 Battered Piles

Plumb piles are preferred; however, if the design dictates the use of battered piles, then a single batter either parallel or perpendicular to the centerline of the cap or footing is preferred. If the design dictates that a different batter direction is required then the pile shall be oriented so that the direction of batter will coincide with an axis of the pile. The Designer should evaluate the effects of length and batter on the selected pile size. Longer piles are more susceptible to bending, especially if an obstruction is hit during driving. Also when driving long slender piles on batters, the tendency is for the tip to bend downward due to gravity which creates additional bending stresses. These deflections can be severe enough to cause additional stresses due to buckling when hit with a hammer. The addition of these stresses can lead to pile failure in bending. Hard subsoil layer can also deflect piles outward in the direction battered causing pile breakage in bending. This must be reviewed during the design phase. The following maximum batters shall not be exceeded:

End bents and abutments	2"/ft.
Piers without ship impact	1"/ft.
Intermediate bents	1-1/2"/ft.
Piers with ship Impact	3"/ft.

5.7 Fender Piles

See specifications section 455.

5.8 Drilled Shafts

In the event that the Foundation Report recommends or suggests a drilled shaft foundation, the design engineer and the geotechnical engineer shall determine the: design loads, size(s), number and length(s). The design parameters presented in the report are to be used for such purposes. The designer shall select the combination of size, number and length that will result in the most cost effective drilled shaft. Shaft spacing center-to-center shall be at least three (3) times the shaft diameter. The drilled shaft size(s) and minimum tip elevation(s) shall be clearly shown on the plans. Load testing, if required, will be indicated on the plans. The appropriate information concerning static load tests (section 5.5) is also applicable to drilled shaft testing. If the drilled shafts were designed to be installed by using a particular method of installation and subject to certain criteria being met, such method and criteria shall be shown on the plans.

5.9 Spread Footings

In the event that the Foundation Report recommends or suggests a spread footing foundation, the allowable soil pressure(s), the minimum footing width(s), and the minimum footing embedment(s) will be indicated in the report. The design engineer shall determine the design load(s) and proportion the footing(s) so as to provide the most cost effective design without exceeding the recommended allowable soil pressure(s). The designer shall communicate with the Geotechnical Engineer as necessary to insure that the corresponding settlement(s) does not exceeding the tolerable limits. Sliding, overturning and rotational stability of the footing(s) shall also be verified. Dewatering, when recommended in the Foundation Report, shall be so noted in the plans.

5.10 Scour Considerations for Pile Foundation Design

Sequential steps must be followed when evaluating pile foundation capacities with scour as a design condition. This is a multi-discipline effort involving Geotechnical, Structural and Hydraulics Engineers. This multi-discipline concept is also addressed in Chapter 6. The foundation design must satisfactorily accommodate the various scour conditions and also furnish sufficient information for the contractor to provide adequate driving systems and construction procedures.

The latest version of software based on the RB-121 design methodology shall be used for the static pile capacity analysis. Contact the District Geotechnical Engineer to verify the latest version. The following definitions are provided to ensure compatibility between the designer and specification 455:

Allowable pile capacity (APC), is a Geotechnical term and refers to a safe soil resistance. It includes factors of safety associated with soil side friction and/or soil end bearing.

Required Driving Resistance (RDR) is the soil resistance which must be overcome by the pile driving equipment. This is also the resistance used as input for wave equation analyses.

Scour resistance is an estimate of the ultimate static side friction resistance provided by the scourable soil.

Ultimate Pile Capacity is a theoretical soil resistance calculated by RB-121 and is equal to the sum of the ultimate pile side friction resistance and the ultimate pile end bearing resistance.

Davisson Pile Capacity is an estimated capacity based on Davisson failure criteria, and equals ultimate side friction plus mobilized end bearing (one third of the ultimate pile end bearing resistance).

1. The Structural Engineer provide a preliminary design configuration of a bridge structure utilizing all available geotechnical and hydraulic data.
2. The Hydraulics Engineer provides three scour elevations; (1) El_{100}, corresponding to the design flood event or up to the 100 year scour critical event, (2) El_{500}, corresponding to the 500 year (or less) scour critical event, and (3) the elevation corresponding to 1/2 of the 100 year scour critical event depth.
3. Using the RB-121 methodology, the Geotechnical Engineer develops an allowable pile capacity curve with the soil surface at elevation El_{100}.

4. Using the RB-121 methodology, the Geotechnical Engineer develops a Davisson pile capacity curve with the soil surface at El_{500}.
5. Using the RB-121 methodology, the Geotechnical Engineer develops a Davisson pile capacity curve and a tension capacity curve with the soil surface at the elevation corresponding to 1/2 of the 100 year scour critical event depth.
6. Using the RB-121 methodology, the Geotechnical Engineer develops an ultimate side friction curve based on the present (i.e. unscoured) soil profile. The scour resistance (SCOUR) of the soil between the present ground surface and El_{100} scour elevation is determined.
7. Using the RB-121 methodology, the Geotechnical Engineer develops an ultimate pile capacity curve based on the present (i.e. unscoured) soil profile.
8. For each design load, the Structural and/or Geotechnical Engineer establish a minimum depth of embedment and corresponding tip elevations below EL_{100} which will satisfy AASHTO loadings (exclusive of ship impact). A lateral analysis is done to ensure stability. An axial analysis is done to <u>estimate</u> pile tip elevations using the allowable pile capacity curve established in step 3.
9. For each design load, the Structural and/or Geotechnical Engineer establish a minimum depth of embedment and corresponding tip elevations below EL_{500} which will satisfy AASHTO loadings (exclusive of ship impact) using a load factor of 1.0. A lateral analysis is done to ensure stability. An axial analysis is done to <u>estimate</u> pile tip elevations using the Davisson pile capacity curve established in step 4.
10. For each pile load due to ship impact load case, the Structural Engineer and/or Geotechnical Engineer establish a minimum depth of embedment and corresponding tip elevations below the elevation corresponding to 1/2 of the 100 year scour critical event depth, which will satisfy ship impact criteria using a load factor of 1.0. An axial analysis is done to <u>estimate</u> pile tip elevations using the Davisson and tension pile capacity

curves established in step 5. The design load for this load case is one half of the pile load. If this design load governs the foundation design, then it should be used to re-estimate the pile tip elevations in steps 8 and 9 for axial capacity.

11. Compare the three elevations from the lateral analyses of steps 8, 9, & 10 above with the minimum depth of penetration (required per 455-3.9) below El_{100}. The deepest elevation is shown in the table and plan notes as a minimum tip elevation. No piles may be tipped above this elevation. Compare the tip elevations from steps 8, 9, & 10 to establish the estimated pile quantity value and test pile lengths.

12. The Geotechnical Engineer calculates a tentative Required Driving Resistance (RDR) by combining the scour value from step 6 with the design load as follows:

    RDR = (Design Load x FS) + SCOUR

    Where FS is a factor of safety defined in specification 455-3.12.2. The required driving resistance shall not exceed the following maximum ultimate bearing capacity unless specific justification is provided and accepted by the Department:

    18" pile .... 300 tons 24" pile .... 450 tons

    20" pile .... 360 tons 30" pile .... 600 tons

13. The Geotechnical, Structural and Hydraulics Engineers may be required to repeat steps 1-12 for various pile sizes, design loads, and scour elevations. The Structural Engineer will perform an economic analysis to determine the optimal pile size and design load.

14. The Geotechnical Engineer must provide the Structural Engineer with the maximum required driving resistance from the ultimate pile capacity curve from step 7 associated with tipping the piles at the minimum elevation. If this value exceeds the maximum ultimate bearing capacity, (as specified in step 12 above) the designer and Geotechnical Engineer must develop a method of pile installation to reduce the estimated required driving resistance so as to not exceed the values shown in step 12.

If jetting or preforming is performed to El_{100}, the SCOUR value established in step 6 is assumed to be equal to 0.0 and in this case the Required Driving Resistance is equal to the Design Load times the factor of safety from 455-3.12.2.

5.11 Scour Considerations for Drilled Shaft Foundation Design

Sequential steps must be followed when evaluating drilled shaft capacities with scour as a design condition. This is a multi-discipline effort. The foundation design must satisfactorily accommodate the various scour conditions and also furnish sufficient information for the contractor to provide adequate drilling equipment and construction procedures. Scour is also addressed in Chapter 6 of these Guidelines.

1. The Structural Engineer provides a preliminary design configuration of a bridge structure utilizing all available geotechnical and hydraulic data.
2. The Hydraulics Engineer provides the Geotechnical Engineer with three scour elevations; (1) El_{100}, corresponding to the design flood event or up to the 100 year scour critical event, (2) El_{500}, corresponding to the 500 year (or less) scour critical event, and (3) the elevation corresponding to 1/2 of the 100 year scour critical event depth.
3. With the soil surface at El_{100}, the Geotechnical Engineer provides an axial capacity versus shaft tip elevation curve with a minimum factor of safety of 2.0 applied to the soil properties.
4. With the soil surface at El_{500}, the Geotechnical Engineer provides an axial capacity versus shaft tip elevation curve using ultimate soil properties.
5. With the soil surface at the elevation corresponding to 1/2 of the 100 year scour critical event depth, the Geotechnical Engineer provides an axial capacity versus shaft tip elevation curve using ultimate soil properties.
6. The Structural Engineer and/or Geotechnical Engineer establish a minimum depth of embedment below EL_{100} which will satisfy AASHTO loadings

(exclusive of ship impact). A lateral analysis is done to ensure stability. Using the curve from step 3, a tip elevation is established to accommodate the axial load.

7. The Structural Engineer and/or Geotechnical Engineer establish a minimum depth of embedment below EL_{500} which will satisfy AASHTO loadings (exclusive of ship impact) using a load factor of 1.0. A lateral analysis is done to ensure stability. Using the curve from step 4, a tip elevation is established to accommodate the axial load.

8. The Structural Engineer and/or Geotechnical Engineer establish a minimum depth of embedment below the elevation corresponding to 1/2 of the 100 year scour critical event depth, which will satisfy ship impact criteria using a load factor of 1.0. A lateral analysis is done to ensure stability. Using the curve from step 5, a tip elevation is established to accommodate the axial load.

9. Compare the tip elevations from the lateral and axial analyses of steps 6, 7 & 8. The deepest elevation governs and is the minimum tip elevation. No drilled shafts may be tipped above this elevation.

10. The Geotechnical, Structural and Hydraulics Engineers may be required to repeat steps 1-9 in order to establish a range of capacities for various shaft diameters, lengths, and scour elevations. The Structural Engineer determines the final dimensions based on an economic analysis of all factors.

5.12 Plan Information

1. Piles

The plans shall include the following table and notes, as applicable, for all projects with driven piles.

PILE INSTALLATION TABLE								
Pier or Bent	Pile Size (in)	Design Load (Tons)	Total Scour Resistance* (Tons)	Min. Tip Elev (ft)	Scour Elev (ft)	Reqd. Preform Elev. (ft)	Reqd. Jet Elev. (ft)	Net Scour Resistance** (Tons)

Required Plan Notes:

I. * Total side friction resistance from ground line to the scour elevation.

II. ** Net side friction resistance from the required preformed or jetting elevation to the scour elevation.

III. When a required jetting elevation is shown, the jet shall be lower to the elevation and continue to operate at this elevation until the pile driving is completed. **If jetting or preforming elevations differ from those shown on the table, the Engineer shall be responsible for determination of the required driving resistance.**

Optional Plan Notes:

When required, the following note shall also be included

I. In order to achieve the required minimum tip elevation, it is estimated that a driving resistance higher than the required driving resistance may be encountered in some locations.

2. Drilled Shafts

The plans shall include the following table, as applicable, for all projects with drilled shafts.

DRILLED SHAFT INSTALLATION TABLE				
Pier	Shaft Size (inches)	Design Load (Tons)	Minimum Tip Elev (ft)	Scour Elev (ft)

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Responsible Office: Structures Design
Topic No.: 625-020-106-h

Chapter 6

HYDRAULIC DATA AND U.S. COAST GUARD PERMITS

6.1 Bridge Hydraulics Report (BHR)

A BHR shall be prepared for all structures over water as specified in Chapter 9 of the FDOT Drainage Manual.

6.2 Bridge Hydraulics Recommendation Sheet (BHRS)

A BHRS shall be prepared for structures over water as specified in Chapter 9 of the FDOT Drainage Manual.

Hydraulic data must be shown in the structure plans for all bridges over water. This may be done by either including the approved BHRS in the bridge plans or showing the data labeled as "Hydraulic Design Data" elsewhere on the bridge plans. For bridge culverts, the BHRS or hydraulic data shall be included in either the bridge or roadway plans as appropriate.

6.3 U.S. Coast Guard Permit

A U.S. Coast Guard Permit may be required for the construction of a bridge or causeway over navigable waters. If a water body is tidally influenced, a permit will probably be required. The FDOT District Permit Coordinator will determine when a permit is required for a particular site.

The permit shall be prepared in accordance with the "U.S. Coast Guard Bridge Permit Application Guide". The official instructions for preparing and handling the permit drawings will be issued by the District Permit Coordinator. Additional information is available in Environmental Management Office Procedure No. 650-040-001.

6.4 Scour Considerations

Scour estimates shall be developed using a multi-disciplinary approach involving the hydraulics engineer, the Geotechnical Engineer, and the Structures Engineer. Bridges and bridge culverts shall be designed to withstand the design flood without damage and should withstand the 500 year flood (super flood) without failure. Refer to Chapter 5 for specific foundation design steps.

A. Development of Scour Design Criteria

The extent and the mitigating steps needed to resolve scour problems should be resolved early in the design process. The Bridge Development Report (BDR) or 30% plans submittal whichever is earliest, is a means of addressing and resolving all major design issues early in the total design process and should also define the need for scour considerations, establish the scour parameters and arrive at possible solutions. This can be achieved through the concerted and cooperative efforts of the hydraulics, geotechnical and structural engineer. The necessary steps are as follows:

1. The Drainage Engineer determines the need for scour considerations based on assumed soil conditions, the extent of scour and its severity at the particular site(s) in question. The Drainage Engineer's assumptions and design parameters should be discussed with both the Geotechnical and Structural Engineers. When evaluating scour potential, the recommendations developed from FHWA HEC 18 should be followed as well on the design requirements provided in the FDOT Drainage Manual. This work should take place early in the PD&E study where changes in the alignment could affect the severity of general scour.

2. Given the scour potential and based on known subsoil conditions and where knowledge of the local variability of the subsoil is available, the Geotechnical Engineer will then consider the possible alignments. It may be necessary to conduct exploratory work if variability of subsoil conditions are suspected but not sufficiently defined. The results of exploratory investigations should be discussed with both the Hydraulics and Structural Engineer and any previous scour assumption verified and/or modified.

3. The Structural Engineer should provide approximate span ranges, pier configurations and pier locations necessary for the different alternates. In addition, possible foundation types and approximate size should be developed

such that the Hydraulics Engineer can estimate local scour potentials. Conditions considered are:

a. The extent and severity of scour along the alignment must be developed. For example, for bridges over a wide body of water, general scour could vary in extent and severity. It may be reasonable, therefore, to consider fewer foundations in the most severe areas (i.e., span the problem) or take appropriate steps to assure the structural integrity of the foundation in those locations.

b. The pile driving resistance which must be overcome at the time of construction may be greater than the ultimate pile capacity at a later date due to subsequent scour activity.

c. Likewise, design drilled shaft capacity must account for the possibility that ultimate capacity will be reduced as a result of future scour activity.

4. The three Engineers (Drainage, Geotechnical and Structural) shall develop the scour potential and rate each location. The results are furnished to the District Environmental Management Office (EMO) Engineer for consideration in establishing the recommended alignment(s).

5. The preferred alignment is established by others.

6. The Structural Engineer develops more detailed calculations showing possible span arrangements, types and size of foundations.

7. The three Engineers review the proposed configuration to assure that the scour problem has been properly addressed. (The Drainage Engineer reviews both the general and local scour potential and recommends continuation or changes.)

8. The Structural Engineer finalizes his configuration and proceeds with an even more detailed analysis of the foundation. All three Engineers should review and concur.

The final results are then incorporated into the BDR or 30% Plans Stage as applicable. The eight (8) steps described above are shown as a flow diagram in Figures 6-1 and 6-2.

B. Submittal Requirements for Scour Design

During the review of the 30% and the 90% plans stages, both the Drainage and Geotechnical Engineers should review the design to assure compliance with the results of the scour calculations. This review activity is shown diagrammatically in Figure 6-2.

6.5 Debris Accumulation

Debris accumulation on the upstream side of substructure units can significantly affect the flow of water and cause significant scour. The designer shall evaluate the type of vegetation upstream from the bridge and consider the probability of debris accumulation in establishing types and locations of substructure units. Special consideration shall be given to mitigating debris accumulation on substructure units.

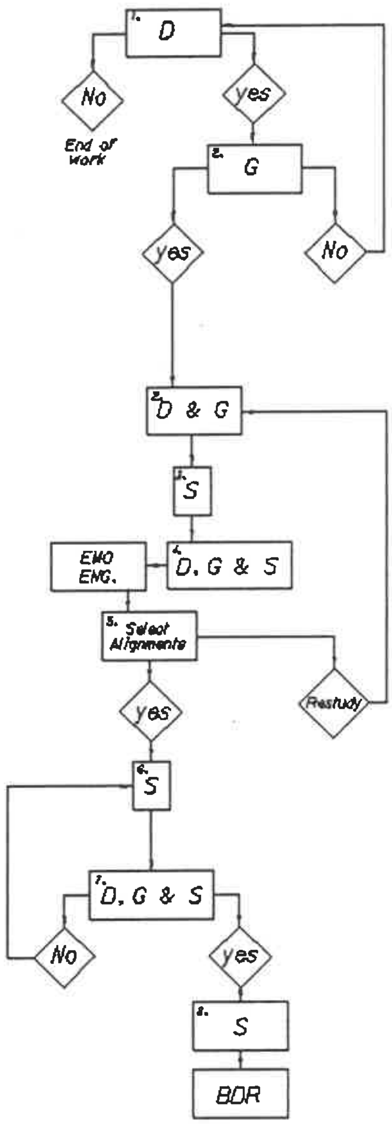
6.6 Widening

The design for scour as described in Article 6.4 above must be included in the widening of an existing bridge structure classified as a major widening as defined in Article 16.6.

The requirement to include scour potential in the design of the widening of an existing structure classified as a minor widening as defined in Article 16.6 shall be considered by the Department on an individual basis.

SCOUR DESIGN

STEPS | EXPLANATION



The following notation are used.

G: Geotechnical Engineer
S: Structures Engineer
D: Drainage Engineer
EMO: Environmental Management Office

Drainage engineer establishes scour potential and extent based on assumed soil paramaters and other hydraulics data.

Geotechnical engineer reviews existing soil profiles and parameters.

yes: Soil data as assumed by D is correct, no further work.
No: G. Provides corrected soil data to D. D re-evaluates scour depth.

Define the extent of general scour expected for each alignment.

Bridge types, span arrangements, possible foundation types established.

Develop recommendations for EMO Engineer.

Structural Engineer refines calculations including pier & foundation types. Develop data for local scour analysis.

Drainage engineer reviews data for conformance to general scour condition and reviews local scour problems.

Structural Engineer continues work and completes BDR.

BDR finalized.

Note: Number in upper left hand corner of some of the activity boxes refer to the Sub-article number of Article 6.4.A.

Figure 6-1

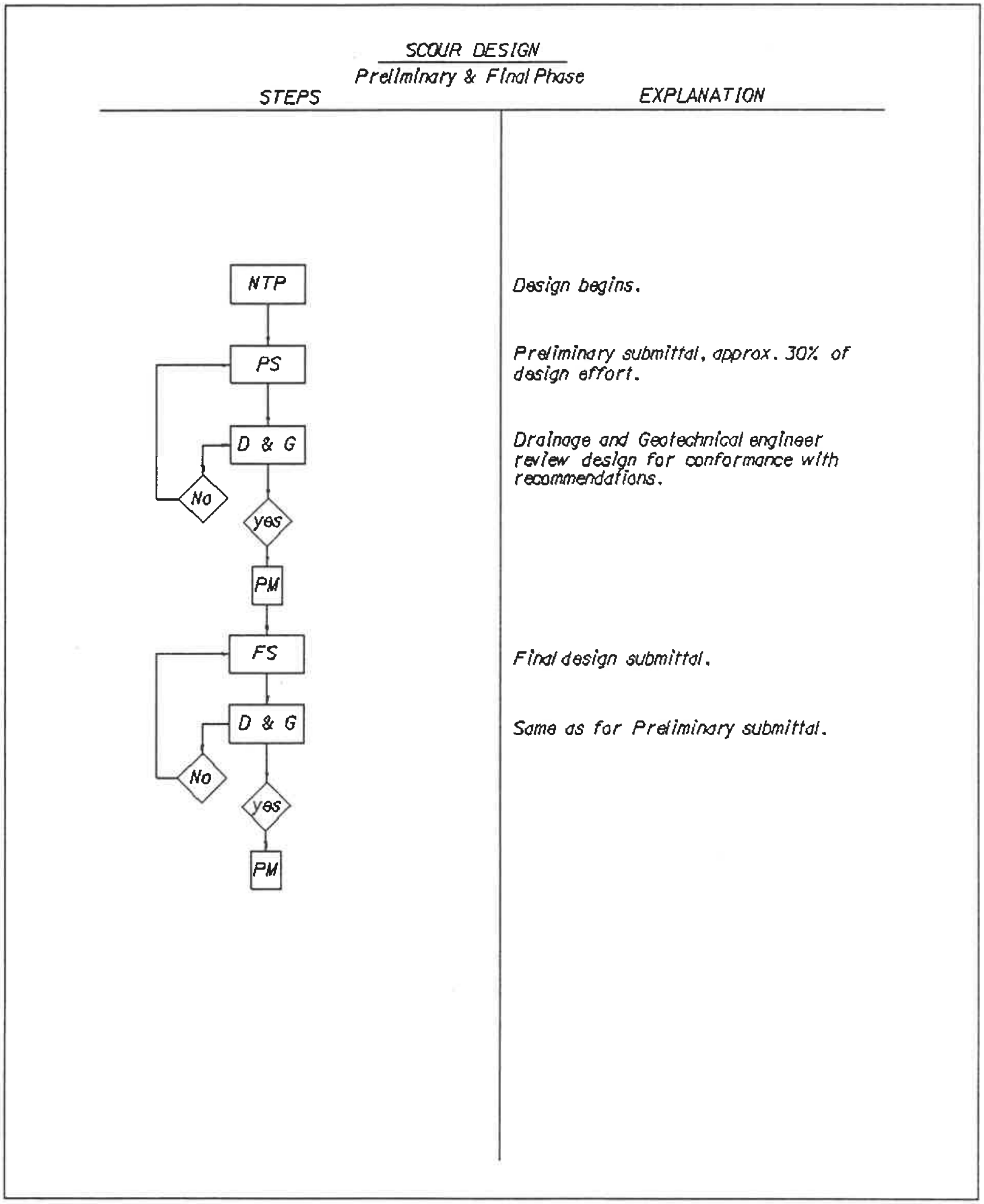

SCOUR DESIGN
Preliminary & Final Phase
STEPS
EXPLANATION
NTP
PS
D & G
No
yes
PM
FS
D & G
No
yes
PM
Design begins.
Preliminary submittal, approx. 30% of design effort.
Drainage and Geotechnical engineer review design for conformance with recommendations.
Final design submittal.
Same as for Preliminary submittal.

Figure 6-2

Approved: Jerry L Potter

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Sequence No.: 0160
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Topic No.: 625-020-107-h

Chapter 7

FLORIDA - CONCRETE DESIGN, ENVIRONMENTAL CLASSIFICATION AND CONSTRUCTION CRITERIA

7.1 Mass Concrete

The designer shall be responsible for indicating which portions of the concrete in a bridge shall be designated as Mass Concrete.

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

When the minimum dimension of the concrete exceeds two (2) feet and the ratio of volume of concrete to the surface area is greater than one (1) then mass concrete shall be required. (The surface area for this ratio shall include the summation of all the surface areas of the concrete component being considered, including the full underside (bottom) surface of footings, caps, construction joints, etc.). Volume and surface area calculations shall be in units of feet.

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

The designer shall also be responsible for taking precautionary measures to reduce the likelihood of cracking of the concrete. These precautionary measures are especially needed in the design of bascule bridge substructures and other structural components requiring the casting of large volumes of concrete.

To prevent or control cracking, some considerations, such as more and/or better reinforcing steel distribution and the judicious placing of construction joints as well as other methods as outlined in ACI 207, ACI 244 and ACI 308, should be taken in all designs involving mass concrete.

7.2 Environmental Class

A. All new bridge sites shall be environmentally classified by the District Materials Engineer or the Consultant. Additionally, the need for environmental classification shall be considered for all widenings and, generally, required for major widenings as defined in Chapter 16. The work shall be accomplished prior to or during the development of the Bridge Development Report (BDR) or 30% Plans Stage, as applicable, and the results included in the documents. The bridge site shall be tested and separate classifications for superstructure and substructures shall be provided.

B. The bridge plans' General Notes shall include the environmental class for both the superstructure and substructure as follows:

1. Slightly Aggressive
2. Moderately Aggressive
3. Extremely Aggressive

As an example, for a proposed bridge located in a swampy area where the substructure is determined to be in an extremely aggressive environment and the superstructure in a slightly aggressive environment, the format on the bridge plans will be as follows:

ENVIRONMENTAL CLASSIFICATION:

Substructure: Extremely Aggressive

Superstructure: Slightly Aggressive

7.3 <u>Environmental Classification Parameters</u>

A. For soil and/or water.

1. <u>Slightly Aggressive</u>

Ph greater than 6.6

Resistivity greater than 3,000 ohm-cm

Sulfates less than 150 ppm

Chlorides less than 500 ppm

NOTE: All conditions above must exist for a slightly aggressive area.

2. Moderately Aggressive

This classification contains all sites not meeting requirements for classifications 1 or 3.

3. Extremely Aggressive

Ph less than 6.0

Resistivity less than 500 ohm-cm

Sulfates greater than 1,500 ppm

Chlorides greater than 2,000 ppm

NOTE: Only one of the above listed conditions need to be met for an area to be extremely aggressive.

B. For bridge substructures the environmental classification is the same as the soil and/or water environmental classification listed in 7.3.A. above.

C. For bridge superstructure, the following environmental classifications apply:

1. Slightly Aggressive

Any superstructure situated in an environment that does not fall into the category of Moderately or Extremely Aggressive, shall be classified as Slightly Aggressive (see Table 7-1).

2. Moderately Aggressive

Any superstructure located within one-half mile 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.

Other specific conditions and/or locations described in Table 7-1.

3. Extremely Aggressive

Any superstructure situated such that a combination of environmental factors indicate that a significant corrosion potential exists and other specific locations and/or conditions described in Table 7-1.

7.4 Chloride Content Determinations:

1. The chloride content of the water and soil at the proposed bridge site will be determined by the District Materials Engineer using established procedures and test methods.
2. The chloride content of major bodies of water within one-half mile of the proposed bridge site will, in most instances, be available in the Department's Bridge Corrosion Analysis. In the unlikely event chloride values are unavailable, the major body of water must be tested unless it is known to be a freshwater body.
3. Generally, all superstructures that are situated line-of-sight and within one-half mile of the Atlantic Ocean or the Gulf of Mexico are subject to increased chloride intrusion rates on the order of 0.016 lbs/c.y./year at 2" concrete depth. The intrusion rate decreases rapidly with distance from open waters and/or when obstacles such as rising terrain, foliage or buildings alter wind patterns. In order to optimize the materials selection process, the Designer and/or District Materials Engineer shall have the option of obtaining representative cores for determination of chloride intrusion rates for any superstructure situated within one-half mile of any major body of water containing more than 12,000 ppm chlorides. Cores shall be taken from bridge superstructures in the immediate area of the proposed superstructure and in sufficient number as to represent the various deck elevations. The sampling plan for such structures shall be coordinated with the State Corrosion Engineer. The District Materials Engineer shall be responsible for obtaining cores. The chloride content and intrusion rates of core samples shall be determined by the State Materials Office, Corrosion Laboratory where a data base of such data will be maintained. In

the absence of core samples and testing all such superstructures shall be classified as extremely aggressive.

TABLE 7-1

SUPERSTRUCTURE CRITERIA

DIST.[2] (miles)	CHLORIDE CONTENT[3] (ppm)	OVERLAND/ OVERWATER	HEIGHT (feet)[4]	CHLORIDE CONTENT[5] (ppm)	CLASSIFICATION
> 0.5	N.L.	OVERLAND	N.L.[6]	N.L.	SLIGHTLY
		OVERWATER	N.L.	< 2,000	SLIGHTLY
		OVERWATER	< 12	2,000 TO 6,000	MODERATELY
				> 6,000	EXTREMELY
		OVERWATER	> 12	2,000 TO 6,000	SLIGHTLY
				> 6,000	MODERATELY
< 0.5	> 12,000	N.L.	N.L.	N.L.	EXTREMELY(1)
	6,000 TO 12,000	OVERLAND	N.L.	N.L.	MODERATELY
	< 6,000	OVERLAND	N.L.	N.L.	SLIGHTLY
	6,000 TO 12,000	OVERWATER	< 12	2,000 TO 6,000	MODERATELY
				> 6,000	EXTREMELY
			> 12	N.L.	MODERATELY
	< 6,000	OVERWATER	SEE "> 0.5 MILES" ABOVE		

NOTES: (1) Concrete cores may be obtained from existing representative structurs in lieu of the above "moderately" or "extremely" environmental classifications. The following table correlates the core results (chloride intrusion rate at a depth of 2 inches in lbs./c.y./year) with the classification.

CHLORIDE INTRUSION RATE	CLASSIFICATION
> 0.16	EXTREMELY
< 0.16	MODERATELY

(2) Distance, in miles, from a major body of water with the chloride content shown in the adjacent column.

(3) Chloride content of major body of water within 0.5 miles of proposed bridge site.

(4) Height, in feet, refers to the distrance between the lowest superstructure elevation and the water surface at mean high water.

(5) Chloride content of the water immediately under the bridge. This is different than the chloride content of the nearest major body of water as called for in the second column.

(6) N.L. signifies "No Limitation" for the particular element indicated.

7.5 Concrete Class, Additives for Corrosion Protection, Penetrant Sealers and Stay-in-Place Metal Forms

A. Concrete Class Requirements

When the environmental classificaitons for a proposed structhre have been determined, then those portions of the structure located in each classification shall use the class of concrete at the location and for the intended use described in Table 7-2 unless otherwise directed and approved by the Department.

B. Additives for Corrosion Protection in Extremely Aggressive Environments

The use of additives to concrete for corrosion protection in extremely aggressive environments will be established by the District Materials Engineer and the State Corrosion Engineer with concurrence from the State Structures Design Engineer or District Structures Design as applicable. Such additives as calcium nitrite and/or micro-silica shall be provided for cast-in-place bridge decks and shall be considered for all other cast-in-place concrete.

For Federal-Aid Projects on the National Highway System, the use of additional corrosion protection systems are considered "experimental" and must have FHWA approval on a case-by-case basis.

C. Penetrant Sealers

When a new bridge, bridge widening or portion of a precast member is located over water containing more than 2,000 ppm chlorides, then all exposed surfaces within 12 feet of high tide, except as noted below, shall be sealed with a high performance penetrant sealer. Exceptions to this requirement are traffic barriers, curbs, sidewalks, riding surfaces, other surfaces above the sides of superstructures and piling that will be completely submerged in water such as when embedded in pier footings. A note and/or sketch describing the concrete surfaces to be coated with penetrant sealer shall be included in the "General Notes" sheet of the bridge plans. Those wearing surfaces which do not meet the concrete cover requirement over reinforcing and are part of a widening project, either as existing or new construction, shall be sealed after grooving in accordance with Articl 10.5.

In no event shall penetrant sealers be specified for box culverts nor the earthn side of retaining wall systems or bulkheads, except for sheet piling as noted above.

D. Stay-in-Place Metal Forms

Stay-in-Place Metal Forms shall not be used for superstructures located over water containing more than 2,000 ppm chloride. The following note shall be included in the General Notes for the structure:

"Stay-in-Place Metal Forms will not be permitted on this bridge."

TABLE 7-2

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(2)
SUPERSTRUCTURE	CAST-IN-PLACE BRIDGE DECKS (INCLUDING DIAPHRAGMS)	CLASS II (BRIDGE DECK)	CLASS IV	CLASS IV(2)
SUPERSTRUCTURE	PRECAST OR PRESTRESSED	CLASSES III, IV, V OR V (SPECIAL)	CLASSES IV, V OR V (SPECIAL)	CLASSES IV, V OR V (SPECIAL)
SUBSTRUCTURE	CAST-IN-PLACE (OTHER THAN SEALS)	CLASS II	CLASS IV	CLASS IV(2)
SUBSTRUCTURE	CAST-IN-PLACE SEALS	CLASS III (SEAL)	CLASS III (SEAL)	CLASS III (SEAL)
SUBSTRUCTURE	PRECAST OR PRESTRESSED (OTHER THAN PILING)	CLASSES III, IV, V OR V (SPECIAL)	CLASSES IV, V OR V (SPECIAL)	CLASSES IV, V OR V (SPECIAL)
SUBSTRUCTURE	PILING	CLASS V (SPECIAL)	CLASS V (SPECIAL)	CLASS V (SPECIAL)
SUBSTRUCTURE	DRILLED SHAFTS	CLASS III (DRILED SHAFTS)	CLASS IV (DRILLED SHAFTS)	CLASS IV (DRILLED SHAFTS)

NOTES:

(1) Penetrant sealer may be required, see Article 7.5.C.

(2) Additives for corrosion protection shall be required in accordance with Article 7.5.B.

7.6 Substructure and Superstructure Definitions for Environmental Consideration

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

A. Box culverts, retaining wall systems, and bulkheads shall be considered as substructures.

B. Abutments shall always be governed by the most stringent condition.

C. If at a given site, the superstructure is classified moderately or extremely aggressive and the substructure is classified slightly aggressive, all portions of the component exposed to the atmosphere down to the next structural element shall be considered superstructure.

D. All piles and caps for bridges with bent type substructures shall be included in the most severe classification.

E. Approach slabs shall be considered as superstructure.

F. Piles for pile bents located in water with a chloride content greater than 2,000 ppm shall be 24-inch square or larger.

The rationale is that in time, after corrosion and spalling of the concrete cover has occurred, a central concrete core of 12 in. square remains if an 18-inch pile was used originally and 18 in. square if a 24-inch pile was used originally. There is a structural advantage in having the larger core.

Using a pile less than 24 in. square can be acceptable if the District Maintenance Engineer concurs. This decision is dependent on site specific conditions and history of piles in the location. Basically, if pile bents are exposed to wet/dry cycles such that jacketing will be required in the future, then a 24-inch square pile should be used. If, however, pile jacketing is not expected, a smaller pile can be used.

G. For MSE walls, refer to Chapter 18 for additional consideration.

7.7 Epoxy Coated Reinforcing Steel

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

7.8 Concrete Cover

The requirements for concrete cover over reinforcing steel are listed in Table 7-3.

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 applying the design covers of Table 7-3.

7.9 Construction Joints in Tall Piers

Construction joints shall be detailed on the plans for tall piers or columns to limit the concrete lifts to 25 feet; except, a maximum lift of 30 feet may be allowed to avoid successive small lifts (less than approximately 15 feet) which could result in vertical bar splice conflicts or unnecessary splice length penalties.

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

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

TABLE 7-3

CONCRETE COVER REQUIREMENTS

ITEM	CONCRETE COVER FOR DESIGN & DETAILING (Inches)	RESULTANT MINIMUM CLEARANCES** (Inches)
Superstructure (Precast)		
Internal and external surfaces (except riding surface) of segmental concrete boxes also all surfaces of prestressed beams except the top surface	1-3/4"	1-1/2"
Top Surface Riding Surface	2-1/2"*	2-1/4"
For all components and all external surfaces not included above including beam top and barriers	2"	1-3/4"
Superstructure (C.I.P.)		
All External Surfaces, Internal Surfaces and All Top Surface Riding Surfaces	2"	1-3/4"
Substructure (C.I.P.)		
External Surfaces Cast Against Earth and Surfaces in Water	4"	3-1/2" (3" Footing)
External Surfaces Formed	3"	2-1/2" (2" Footing)
Internal Surfaces	3"	2-1/2"
Substructure (Precast)	3"	2-3/4"
Prestressed Piling	3"	2-3/4"
Drilled Shafts	6"	5"
Retaining Walls (C.I.P.)	3"	2-3/4"
Retaining Walls (Precast)	2"	1-3/4"
Culverts (C.I.P. and Precast)	2"*** 3"****	1-1/2" 2-1/2"
Bulkheads (C.I.P.)	4"	3-1/2"

* Includes a 1/2 inch allowance for milling.
** This is given FOR INFORMATION ONLY and accounts for placement tolerance.
*** For slightly aggressive environments only.
**** For moderately and extremely aggressive environments.

7.10 Control of Shrinkage Cracking

For long walls and similar construction where one lift of concrete is placed on top of an existing one the designer is advised to consider the following:

A. The minimum percentage of reinforcement to control shrinkage/thermal cracking should be at least 0.1% per face based upon the full cross section area. This minimum reinforcement applies in each direction.

B. Limit the length of a lift to about 24-30 ft. maximum between vertical construction joints. See Article 7.9 for limits of concrete pours in tall piers.

C. Required construction joints shall be clearly detailed on the plans.

D. Provide details for keyways (trapezoidal in shape for ease of forming and stripping) to key one side of the joint to the other. (For example, a typical vertical joint should have a keyway about 2" deep and about 6" wide, or one third the thickness of the member if less than 18" thick, running the full height.)

E. Construction or expansion joints where water leakage has to be prevented should be fitted with a water barrier.

7.11 Adhesive Anchors

Adhesive Anchors may be used to attach new construction to existing concrete structures when dowelling from or anchoring to previously cast concrete is required. Anchors may be either reinforcing bars or anchor bolts depending upon the application. Adhesive anchor systems approved by the Department are now listed on the Qualified Products List (QPL) with their assigned class.

Unless special circumstances dictate otherwise, anchorages should be limited to a maximum diameter of 1 inch and designed for a ductile failure. A ductile failure requires embedment sufficient to insure that failure will occur by yielding of the steel. To produce this condition adequate embedment may be assumed to be that which will develop 125 percent of the yield strength or the minimum ultimate tensile strength whichever is less.

The adhesive properties associated with the four classes on the QPL are as follows:

QPL CLASS	ADHESIVE PROPERTIES U_{O}(PSI)	U_{MAX}(PSI)	λ'
I	900	900	0.055
II	1200	1250	0.065
III	1450	1550	0.075
IV	1700	1800	0.085

The anchorage capacity, P, is calculated according to Equation 7-1 or 7-2 as applicable:

For $L/\sqrt{d} \leq 8$

$$P = \phi \times u_o \times \pi \times d \times (L-2) \qquad [Eq.7\text{-}1]$$

For $L/\sqrt{d} > 8$

$$P = \phi \times u_{\max} \times \frac{d^{1.5}}{\lambda'} \times \tanh\left[\lambda' \times \frac{(L-2)}{\sqrt{d}}\right] \qquad [Eq.7\text{-}2]$$

Where:

L	=	embedment length in inches
d	=	hole diameter (anchor diameter + 1/8 inch) in inches
Ø	=	0.8
P	=	nominal ultimate anchorage capacity in pounds
u_o	=	adhesive property depending on class
u_{max}	=	adhesive property depending on class
λ'	=	adhesive property depending on class

No reduction in the capacity of anchors calculated from the above equations is required for spacings of 10 hole diameters or greater. For spacings of 5 to 10 hole diameters a reduction in capacity of 15 percent is recommended. At present no recommendations are available for anchor spacings of less than 5 hole diameters. A minimum edge distance of 3 inches should be provided.

The steel bolt or reinforcing bar may be considered fully anchored and effective in resisting the tensile load, P, calculated according to Eq. 7-1 or 7-2 as applicable. When the anchor is thusly embedded, it is also fully effective in shear.

Design Example 1

Problem Statement --> 1 inch diameter A36 threaded rod with 10 kips of tension on it. Edge distance and spacing are greater than required for full capacity.

Yield strength (Fy) of A36 material = 36 ksi

Minimum ultimate tensile strength (Fu) = 58 ksi

Fy * 1.25 = 45 ksi (controls)

Such that the design force P = 0.7854 in^2 * 45 ksi * 0.75

NOTE: The 0.75 factor accounts for the tensile stress area assumed to be 75 percent of the nominal area.

Design Values

d = 1.125 in hole diameter

ϕ = 0.8

u_o = 1700 psi (Class IV value)

u_{max} = 1800 psi (Class IV value)

λ' = 0.085 $in^{-0.5}$ (Class IV value)

Solve for L (embedment), trying first Equation 7-1

$$L = \frac{P}{\phi \times u_o \times \pi \times d} + 2$$

L = 7.515 inches

$$\frac{L}{\sqrt{d}} = 7.085 \quad (< 8 \Rightarrow OK)$$

Design Example 2

Problem Statement --> 3/4 inch diameter A193 B7 rod with 18 kips of tension on it. Edge distance and spacing are greater than required for full capacity.

Yield strength (Fy) of A193 B7 material = 100 ksi

Minimum ultimate tensile strength (Fu) = 125 ksi

Fy * 1.25 = 125 ksi (controls for either case)

The design force P for this bolt shall be:

P = 0.4418 in^2 * 125 ksi * 0.75 = 41.419 kips

(NOTE: See Design Example 1 for 75 percent factor.)

Use the same design values as before except the hole size is 0.875 inches in diameter.

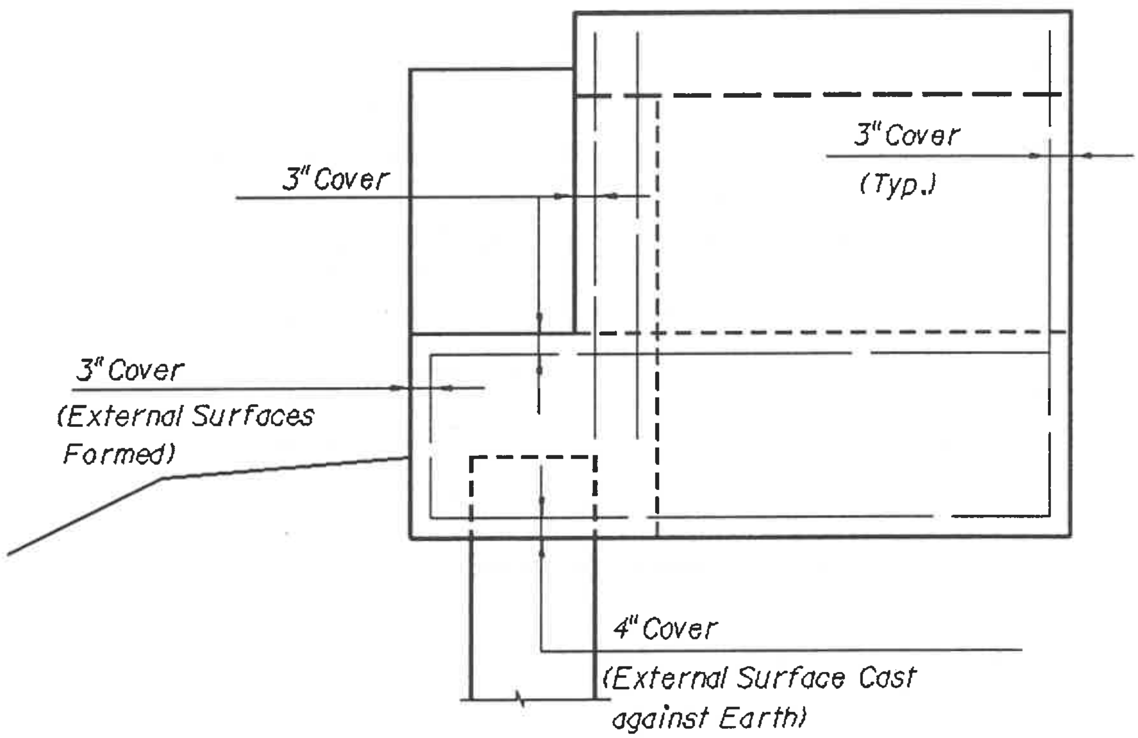
Try Equation 7-1 first

$$L = 13.079 \text{ inches}$$

however,

$$\frac{L}{\sqrt{d}} = 13.982 \quad (> 8 \Rightarrow NG)$$

Equation 7-1 failed, thus Equation 7-2 must be solved for L using trial and error or another method. This results in a required length of 22.25 inches.



END BENT

Figure 7-1

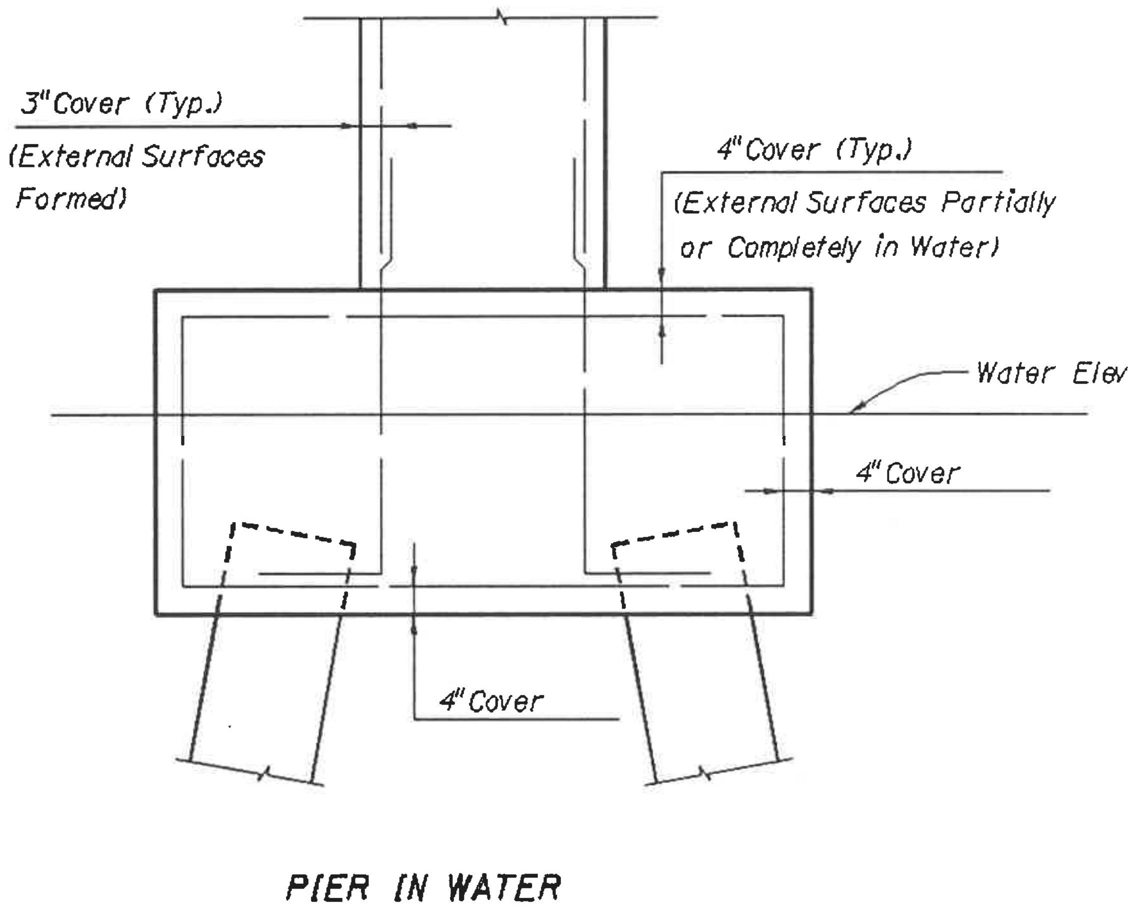

3" Cover (Typ.)
(External Surfaces
Formed)
4" Cover (Typ.)
(External Surfaces Partially
or Completely in Water)
Water Elev.
4" Cover
4" Cover
PIER IN WATER

Figure 7-2

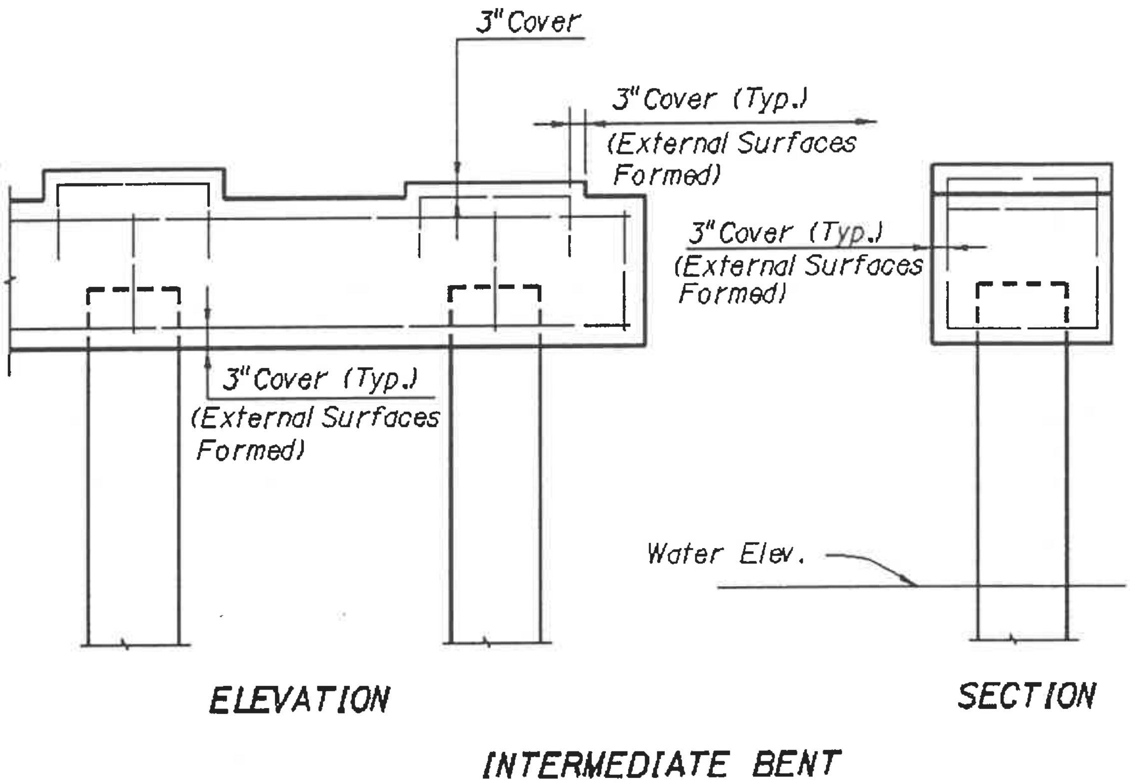
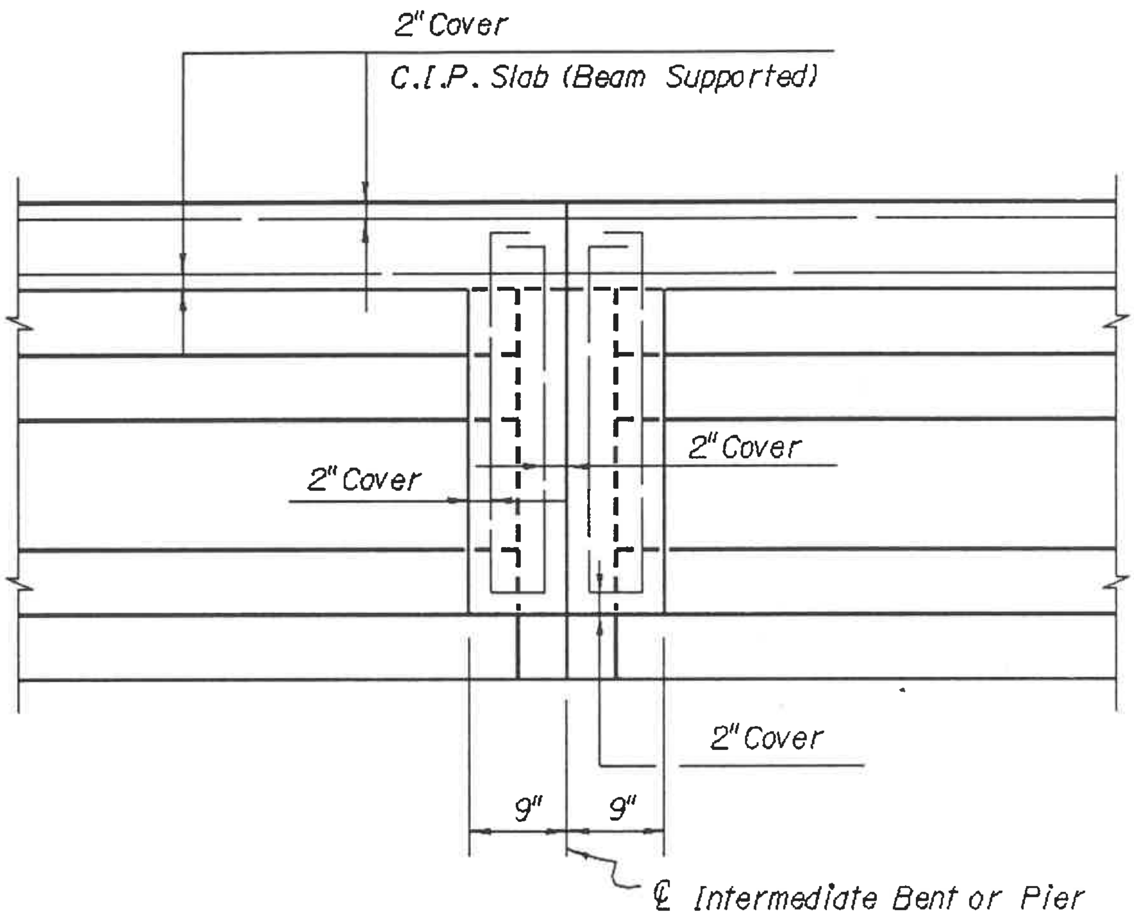

3" Cover
3" Cover (Typ.)
(External Surfaces Formed)
3" Cover (Typ.)
(External Surfaces Formed)
3" Cover (Typ.)
(External Surfaces Formed)
Water Elev.
ELEVATION
SECTION
INTERMEDIATE BENT

Figure 7-3



SUPERSTRUCTURE

Figure 7-4

Approved: Jerry L Potter

Effective: November 2, 1992
Sequence No.: 0160
Responsible Office: Structures Design
Topic No.: 625-020-108-h

Chapter 8

DESIGN CRITERIA FOR SEGMENTAL BOX GIRDER BRIDGES

8.1 Introduction

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

Comments or explanatory notes in this chapter are quoted in parentheses.

Specific requirements regarding post-tensioning duct size and alignment are described in Chapter 10, Article 10.11.

Contract drawings shall be prepared in accordance with Method A as explained in Section 29.1.1 of the Guide Specification for Design and Construction of Segmental Concrete Bridges, 1989. In addition, only Type A joints as defined in Section 8.3.5 of that same specification will be allowed.

8.2 Specifications

The "Guide Specifications for Design and Construction of Segmental Concrete Bridges", AASHTO, 1989 edition, and subsequent interim Specifications shall apply except where modified by the Florida Department of Transportation "Standard Specifications for Road and Bridge Construction", latest edition, and supplements thereto, the FDOT "Supplemental Special Provisions for Segmental Bridges" and the contents of this document. The AASHTO "Standard Specifications for Highway Bridges", latest edition, should be consulted for items not specifically addressed in the above.

8.3 Notation

Unless otherwise noted, the notations used in these criteria correspond to the notations contained within the codes upon which these criteria are based (i.e. AASHTO, CEB-FIP, etc.). The codes are specified where necessary.

8.4 Design Loading

A. General

All loadings shall be in accordance with the referenced specifications, except as provided below.

B. Dead Loads

1. Unit weight of concrete (including reinforcement) - 150 pcf.
2. Wearing surface (future provision for) 15 psf or as determined for project.
3. Appurtenances (e.g. sound barriers) as determined for project.
4. Utilities - as determined for project.

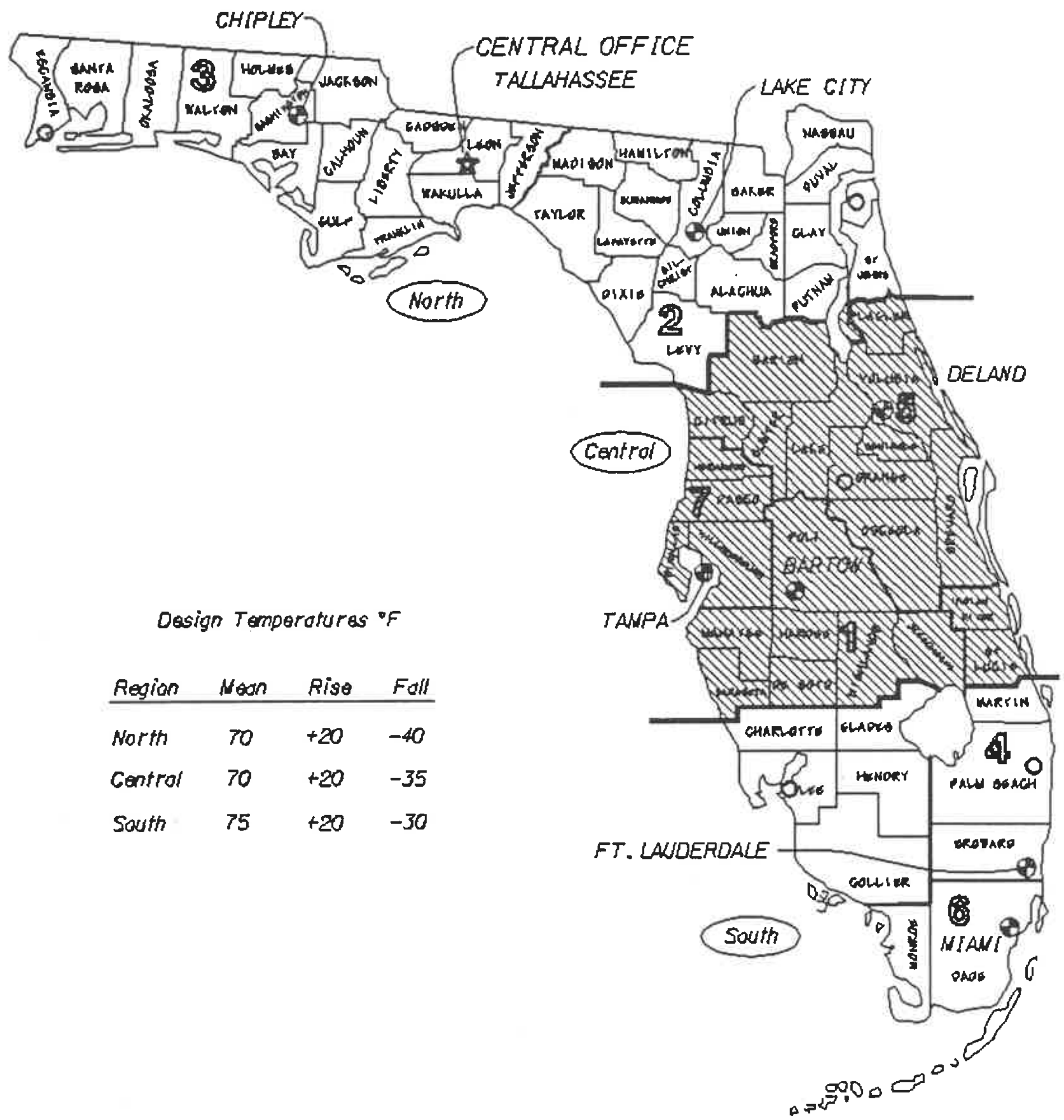
C. Ice - not applicable.

D. Wind Load on Structure

Wind loading shall be in accordance with the more stringent of the referenced specification or, for high or exceptionally exposed structures, as determined from the basic wind speeds for the location and drag coefficients appropriate to the type and shape of the structure or in accordance with approved model tests or similar procedures. This shall also apply to wind loads on special erection equipment.

E. Thermal Effects

1. Normal Mean Temperature: For detailing purposes, the normal mean temperature for the location shall be used (Fig. 8-1).
2. Seasonal Variation
   a. For the purposes of design:
      Except as stated in Section 12.3, the maximum and minimum over-all temperatures for the location shall be used (Figure 8-1).
   b. Thermal Coefficient: (see Article 12.3)
   c. The temperature setting variations for bearings and expansion joints shall be stated on the bridge plans.



Design Temperatures °F

Region	Mean	Rise	Fall
North	70	+20	-40
Central	70	+20	-35
South	75	+20	-30

Temperature Variation for Florida

Figure 8-1

See Section 12.3 "Design of Neoprene Pads and Joint Seals to Accommodate Movement".

3. Differential Temperature

a. Gradients

As per "AASHTO Guide Specifications, Thermal Effects in Concrete Superstructures, 1989".

b. Transverse Effects

For hollow box girder superstructures, provision shall be made for a reversible differential temperature between the inside and outside of the box of 10^{o} F.

F. Creep and Shrinkage

Creep and shrinkage strains and effects shall be calculated in accordance with the CEB-FIP Model Code for Concrete Structures with the exceptions noted below. The following shall apply:

1. Relative Humidity

For all areas (inland and coastal) the relative humidity for Florida may be taken as 75%.

2. Notional Thickness (h_o).

To calculate the notional thickness for precast or cast-in-place concrete, the perimeter in contact with the atmosphere shall be taken as:

a. For hollow sections with internal surfaces in contact with atmosphere, regardless of the degree of ventilation in the final structures... the sum of the internal and external perimeters.

b. For hollow sections made with permanent impervious void formers... the external perimeter only.

3. Creep: Terms of the Creep Factor (ϕ):

The creep factor should be determined in accordance with the 1978 version of the CEB-FIP Code which includes the "$\beta_a(t_o)$" term. However, this term may be omitted, in accordance with the 1974 version of the Code, provided a suitable compensating increase is made to the creep factor. One method would be to increase the value for the φd term as follows:

- o 74 for concrete loaded at 14 days
- o 66 for concrete loaded at 28 days
- o 58 for concrete loaded at 56 days.

4. Modulus of Elasticity

   For the purpose of applying the CEB-FIP Code, the secant modulus of elasticity at 28 days may be taken from Table 4.1.

5. Shrinkage: Terms of the Shrinkage Function, $s(t_1\ t_0)$:

   a. For concrete made from Florida limerock aggregate the "s2" term and the "s" curves may be taken for a notional thickness of 0.7 times that determined in 8.4.F.2 above.

   b. For normal concrete made from non-limerock aggregate no modification to the CEB-FIP Code is required.

7. Moment Redistribution: Creep strains and prestress losses which occur after closure of the structure, for continuity, result in a redistribution of the static moments. On cabled stayed or cast-in-place structures, the effects of the maturity of the concrete on the loads shall be carefully considered. Stress envelopes shall be calculated for these effects based on an assumed early and late construction schedule. The Designer's assumed construction schedules shall be submitted with the preliminary plans for approval by the Department.

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

1. Losses: For determining final prestress forces, prestress losses shall be calculated for both the early and late construction schedule approved by the Department. (See Section 8.4.F.7 above.) The final required prestress shall be

based on the most severe condition at each location along the structure.

2. Secondary Effects: Prestress tendon profiles, especially on curved structures, can result in substantial secondary effects.

   For example, draped web tendons will produce transverse bending stress in the box cross-section due to the lateral component of forces in a curved structure. All such effects must be considered.

3. Deck Slab: All prestressed box girder deck slabs shall be transversely post-tensioned. Where draped post-tensioning is used in deck slabs, due consideration shall be given to the final location of the c.g.s. within the duct. Critical eccentricities over the webs and at the centerline of box shall be reduced 1/4 inch from theoretical, to account for adverse construction tolerances.

4. Tendon Geometry: The Designer shall take into account the fact that the c.g. of the duct and the c.g.s. are not necessarily coincidental when coordinating design calculations with detail drawings.

5. Required Prestress: Prestress forces are required to be shown on the drawings only at locations where they could be verified during construction; namely at anchorages of the "pull" ends of tendons.

H. Earthquake

Refer to the requirements of Article 4.2.

8.5 Material

A. Concrete: 28-day cylinder strengths shall be:

Precast superstructure (incl CIP joints)	5500 psi.
Precast piers stems	5500 psi.

B. Reinforcement:

Reinforcing steel: ASTM Grade 60 unless otherwise required.

C. Concrete Cover to Reinforcement

Concrete cover shall be in accordance with Chapter 7.

(NOTE: Refer to FDOT Materials and Research Office, Gainesville, for latest requirements on penetrant sealers.)

D. Post-Tensioning Steel

1. Strand: ASTM A 416 Grade 270, low relaxation.

2. Parallel wires: ASTM A 421 Grade 240.

3. Bars: ASTM A 722 Grade 150.

4. Prestressing Parameters:

Apparent modulus of elasticity (strand)		26,500 ksi.
Modulus of elasticity (parallel wires & bars)		29,000 ksi.
Anchor set:	Strand	0.375 in.
	Parallel wires	0.375 in.
	Bars	0

Anchor set shall be verified during construction.

Level of Stress in Strands

Maximum jacking stress	S.R.S. - 0.76 f's
	L.R.S. - 0.81 f's

Maximum anchoring stress:

	At Anchor	End of Seating Zone
S.R.S.	0.70 f's	0.71 f's
L.R.S.	0.70 f's	0.75 f's

Level of Stress in Wires and Bars

Maximum jacking stress	0.80 f's
Maximum anchoring stress	0.70 f's

S.R.S. - Stress relieved strand

L.R.S. - Low relaxation strand

Friction and wobble coefficients: See Table 8.3.

TABLE 8.3

Type of Duct	Strands Friction	Strands Wobble k/ft	Wires Friction	Wires Wobble k/ft	Bars Friction	Bars Wobble k/ft
Metal	-------------AS PER AASHTO-----------------					
Polyethylene Curved	0.15	0.0002	0.10	0.0002	--	--
Polyethylene Straight:						
Embedded	0	0.0002	0	0.0001	--	--
External	0	0	0	0	--	--

8.6 Allowable Stresses

A. Stresses at service after losses have occurred:

In addition to the requirements of Section 9.2.2.2 of the AASHTO Guide Specifications, the minimum bonded auxiliary reinforcement through the joints shall be at least 0.5% of the gross area of the slab and shall be well distributed.

For sections with circular void formers, this shall be 0.5% of the solid area between the edge of the void forms and the extreme fiber.

B. Minimum bonded reinforcement in flexural members with unbonded tendons:

The amount of continuous bonded reinforcement shall be calculated according to Section 18.9 of the ACI 318-83 Code.

C. Anchorage Stresses:

Section 9.2.3.1 of the AASHTO Guide Specifications shall be amended as follows:

For a confined anchorage zone, that is one which is provided with sufficient reinforcement to contain all the bursting and directional reaction forces from bearing plates, these stresses are limited to 5000 psi and 6875 psi, respectively, as absolute maximum values even if the concrete strength is in excess of 4000 psi at transfer (load application) and 5500 psi at 28 days.

8.7 Shear and Torsion

The requirements of Section 12.6 of the AASHTO Guide Specifications are expanded as follows:

A. In lieu of a more exact analysis, the web/bottom slab interface shall be designed for an ultimate flange shear force equal to the following:

$$V'u_{bs} = \frac{(Wb)_{bs}}{4(bd)_w} \cdot V'u \quad \text{(Where V'u is per web)}$$

$V'u_{bs}$ - bottom slab equivalent ultimate shear

$(Wb)_{bs}$ - width x thickness of bottom slab

$(bd)_w$ - width x thickness of web

$V'u$ - ultimate web shear force

Localized shear effects due to prestress anchorages shall be added to the Ultimate Shear Force computed above.

B. In lieu of the requirements of Section 11.7 of ACI 318-83, the web/bottom slab interface may be designed as follows:

1. The shear strength provided by the concrete shall be as follows:

$$V_{bs} = \frac{(3.5\sqrt{f'c} + 0.3f_c)\ (Wb)_{bs}}{4}$$

fc - average bottom flange compressive stress.

2. The shear strength provided by reinforcing in the bottom slab perpendicular to the web, when required, shall be computed as follows:

$$V_s = \frac{(Av)(fsy)(w_{bs})}{2s}$$

s - longitudinal spacing for reinforcing

NOTE: All above units are inches and pounds.

8.8 Localized Bending in Top Slabs

A. General

Localized bending effects in top slabs shall be determined by normally recognized practices using Pucher or Homberg charts, finite element analysis or other acceptable distribution technique. Local moments shall be determined for transverse and longitudinal bending.

B. Transverse Bending

Transverse bending moments shall be distributed between the top slab and webs according to the "frame action" of the box. Moments at the top of the web shall be distributed the same as for the slabs noted in Section 8.8.A above. Moments at the bottom of the web may be reduced 50 percent to account for further longitudinal distribution of bending effects. The frame action for moment distribution in the box should be restrained against sidesway, where such reactions are transferred to the rigid pier diaphragms. Reinforcement and/or prestress provisions shall be added to that required for shear and/or torsion accordingly.

C. Longitudinal Bending

Longitudinal local bending effects shall be considered for two cases as follows:

1. Between Transverse Joints or at Transverse Joints with Continuous Longitudinal Reinforcement.

   Local longitudinal bending effects in the top slab of a segment between joints shall be taken by a combination of longitudinal residual compressive stresses for the full section and longitudinal reinforcement.

2. At Unreinforced Transverse Joints

   a. Joints with Residual Compressive Stress

      There shall be sufficient residual longitudinal compressive stress from the full cross-section such that there shall be no net tensile stresses when the stresses due to longitudinal bending effects are added. Alternatively the joint may be dealt with as in Section 8.8.C.2.b below.

b. Joints with No Residual Compressive Stress

In these cases the bending must be taken by transverse provisions. It may be assumed that there is a transfer of shear force only across the joint provided the joint contains keyways in the top slab and that epoxy glue is used for joining the segments.

D. Discontinuous Edges

Discontinuous edges of slabs, at expansion joints for example, shall be checked for localized bending and shear and shall be adequately reinforced and/or stiffened with ribs as required. Consideration should be given to impact wheel loading effects. Armoring of corners under expansion joints is recommended.

8.9 Crack Control

The transverse analysis of the box girder shall include an evaluation of crack width for any loads imposed prior to attainment of full design strength.

Crack widths may be evaluated as follows:

$$W = f_s (d_c \times A)^{1/3} \times 10^{-4}$$

where

W = Maximum crack width, inches

f_s = Reinforcement steel stress, ksi

d_c = Thickness of cover from tension fiber to center of bar, inches

A = Area of concrete surrounding a bar $= 2 \times d_c \times s$ (bar spacing), square inches

W Shall be limited to 0.012 inches.

Crack widths shall be checked under service conditions for all critical parts of the structure. Service crack widths shall not exceed 0.012 inches.

8.10 <u>Bridge Bearings</u>

A. General

Bridge bearings shall conform to the latest AASHTO specifications.

B. Replacement

Provisions shall be made in the bridge design for the future replacement of bridge bearings.

C. Bursting and Spalling

Concrete surfaces in contact with bearing shall be adequately reinforced to prevent bursting (splitting) and spalling. This also applies to any jacking pockets or jack locations provided for the bearing replacement.

D. Settings:

Expansion Bearings shall be sized and set at the time of construction for the following conditions:

1. Allowance for movement away from the fixed pier based on the maximum temperature rise given in Section 8.4.E, Figure 8-1, plus 20 degrees for construction variation.

2. Allowance for movement toward the fixed pier based on 1.2 times the total anticipated movement resulting from the combined effect of temperature fall, creep, shrinkage and elastic shortening. Temperature fall shall be based on the maximum value given in Section 8.4.E, Figure 8-1, plus 10 degrees for construction variation. Creep, shrinkage and elastic shortening shall be computed from the time the bearings are installed through day 4000. Installation of bearing shall be based on the early construction schedule established per Section 8.4.F.7.

8.11 Expansion Joints

A. Design

Expansion joints shall be designed for the full range of movement anticipated due to creep, shrinkage, elastic shortening and temperature effects.

B. Settings

The setting of expansion joint recesses and expansion joint devices, including any precompression, shall be clearly stated on the drawings.

Expansion joints shall be sized and set at time of construction for the following conditions:

1. Allowance for closure movements based on the maximum temperature rise given in Section 8.4.E, Figure 8-1.
2. Allowance for opening movements based on 1.2 times the total anticipated movement resulting from the combined effects of temperature fall, creep and shrinkage. Temperature fall shall be based on the maximum value given in Section 8.4.E, Figure 8-1. Creep and shrinkage shall be computed from the time the expansion joints are installed through day 4000. All expansion joints shall be installed after superstructure segment erection is completed for the entire bridge. (A note stating this requirement shall be placed on plans.) Time of installation shall be based on the early construction schedule established per Section 8.4.F.7.
3. To account for the larger amount of opening movement, expansion devices should be set precompressed to the maximum extent possible. Calculations shall allow for an assumed setting temperature of 85^{o} F. The designer should provide a table on the plans giving precompression settings according to the prevailing conditions. Expansion devices shall be sized

and set such that after 4000 days the device shall not remain in tension through the full range of design temperature.

4. A table of setting adjustments shall be provided to account for variation of the temperature at time of installation and the mean temperature given in Section 8.4.E, Figure 8-1. The table shall indicate the ambient air temperature at time of installation and adjustments shall be calculated for the difference between the ambient air temperature and the mean temperature.

C. Armoring

Concrete corners under expansion joint devices shall be provided with adequate steel armoring to prevent spalling or other damage under traffic. It is recommended that the armor be at least 4"x4"x5/8" galvanized angles anchored to the concrete with welded studs or similar devices. Horizontal concrete surfaces supporting the expansion joint device and running flush with the armoring shall have a finish acceptable for the device. All armoring shall have adequate vent holes to assure proper compaction of the concrete under the armor.

8.12 Erection Schedule and Construction System

A typical erection schedule and anticipated construction system shall be incorporated into the design documents in an outline, schematic form. The assumed erection loads along with time of application and removal of erection loads shall be clearly stated in the plans. (See Section 8.13.B for detail of erection schedule.)

8.13 Casting Curves and Erection Elevations

A. General

A "casting curve" is a geometric description of the profile to which the precast segments must be cast in order to compensate for subsequent deflections occurring during the construction and service life of the bridge such that the final bridge profile meets the geometry defined by the highway layout and profile. It is usual to refer to two generic types of casting curves:

1. Camber Curve: This is the curve which compensates only for structural deflections according to the construction process and the over-all structural type, it does not include highway geometry.
2. Casting Curve: This curve includes the highway geometry and the structural deflection compensations.

B. Camber Curves:

Camber curves shall be calculated based on the assumed erection loads used in the design and based on the two construction schedules established in Section 8.4.F.7. Based on these results, the following information shall be shown on the plans:

1. If the results obtained based on the two construction schedules vary by less than 25% or does not exceed 1/2", then only one camber curve and erection schedule shall be shown on the plans, which shall be the average of the two values obtained.
2. If the results obtained based on the two construction schedules vary by more than 25% or exceed 1/2", then both camber curves and erection schedules shall be shown on the plans. A note on the plans shall instruct the contractor to cast segments to an interpolated curve between the values shown, based on his actual construction schedule. For unusual structures, camber curve requirements shall be determined during preliminary studies.

C. Erection Elevations:

For erection methods other than span by span, Erection Elevations shall be shown on the plans for each stage of erection. Erection Elevations shall not be required for span by span erection.

8.14 <u>Final Computer Run</u>

It is required that the final design shall be proven by a full longitudinal analysis taking into account the assumed construction process and final long term service condition, including all time related effects. The analysis shall be made using a version of the B.C. program equivalent to or better than that of the FDOT or alternatively, by an equivalent program or programs which have been previously proven to perform the same functions as the B.C. program, all to the satisfaction of the FDOT.

If the Designer does not have access to such a program, then this final B.C. computer run may be made on the FDOT computer. For this purpose the Designers shall be provided with a copy of the FDOT's B.C. manual and one example input problem and he shall prepare all the data on standard input sheets with up to 80 characters per line or on computer tape compatible with the FDOT equipment. The cost of this final run, including typing data if applicable, shall be billed to the Designer under his final design contract for the project. Details of these services, procedures and costs shall be assessed on a project by project basis. The Designer shall advise the FDOT of his need, or wish, to use this service during negotiations for and before signing of the final design contract. Use of the FDOT computer services shall in no way relieve the Design Consultant of his professional liability.

Approved: Jerry Potter

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Chapter 9

SHIP IMPACT

9.1 Design for Ship Impact

The design of all new bridges over navigable waters must include consideration for possible vessel (generally applicable to barges and oceangoing ships) collisions with supporting members.

The Department will proceed in the following manner to achieve such design:

A. The Engineer of Record shall first 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.
    d. Navigation channel, width, depth and geometry.
    e. Average current velocity.
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 5 years and prediction to end of 25 years in the future).
    c. Weight of vessels (tons).
    d. Weight of cargo (Deadweight tonnage - DWT).
    e. Draft (depth below the waterline) of ships, tugs and barges.
    f. The overall length and speed of tow.
3. Accident reports.

B. Utilizing techniques and data presented in the AASHTO manual, <u>Guide Specification for Vessel Collision Design of Highway Bridges</u>, the Engineer of Record shall develop the equivalent static loads representing vessel collisions with the proposed bridge structure.

Method II of the manual, probability based procedure, shall be the preferred method in determining the vessel collision forces, unless waterways, marine traffic, distribution of vessel sizes or cost effectiveness dictate using other methods.

C. In addition to utilizing the general design recommendations presented in the above described manual, the Engineer of Record shall use the following design methodology:

1. One iteration of secondary effects in columns shall be included: i.e., axial load times the initial lateral deflection.
2. The analysis must include the effect on adjacent piers from the transfer of lateral forces up to the superstructure. Only positive (steel or concrete) connections of the superstructure to the substructure shall be considered in the analysis for transfer of lateral force up to the superstructure. Analysis of force transfer through mechanisms at the superstructure/ substructure interface shall be evaluated using generally accepted theory and practice.
3. Ultimate strength analysis methods shall be used for the ship impact load case. Plastic hinges may be utilized in the analysis to account for redistribution effects.
4. The ultimate capacity of axially loaded piles shall be limited to twice the allowable design load for both compression and tension. Plastic flow of the soil for the purpose of force redistribution effects shall not be used to determine the foundation capacity. That is, the foundation capacity shall be taken as the value obtained by elastic analysis when the axial load of the first pile reaches ultimate load.
5. Location of plastic hinges formed in piles due to lateral bending below the ground or mudline shall be determined by

elastic concepts utilizing a subgrade modulus provided or approved by the Department.

6. The path of impact distribution shall be checked so that the stress in all members and their connections remain within their ultimate capacity.

D. Data Sources

1. U.S. Army Corps of Engineers, "Products and Services Available to the Public," Water Resources Support Center, Navigation Data Center, Fort Belvoir, Virginia.
2. U.S. Army Corps of Engineers, "Waterborne Commerce of the United States (WCUS), Parts 1 & 2"; Water Resources Support Center (WRSC), Fort Belvoir, Virginia.
3. U.S. Army Corps of Engineers, "Waterborne Transportation Lines of the United States," Water Resources Support Center (WRSC), Fort Belvoir, Virginia.
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.

9.2 Pile Bents

Pile bents subject to ship impact shall be designed to remain structurally adequate using the Working Stress Design Method allowing 150% of the AASHTO allowable stresses with any one pile removed and live load applied within the designated, striped traffic lanes. The most critical combination of live load shall be used considering one or more lanes with no reduction for multiple lanes. No direct impact analysis is required.

9.3 Widenings

A bridge structure over navigable water that is classified as a major widening as defined in Chapter 16, Article 16.6 must be designed for ship impact in accordance with Article 9.1 or 9.2 as applicable.

Bridge structures over navigable waters that are classified as minor widenings shall be considered eligible for ship impact design requirements on an individual basis.

9.4 Movable Bridges

Movable bridges shall comply with the requirement of Articles 9.1 through 9.3, without exception.

9.5 Main Span Length for Barges

The length of the main span at the navigable channel shall be at least equal to the channel width (fender to fender) plus the width of two design barges. For movable bridges, exception shall be granted on an individual basis.

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Chapter 10

SUPERSTRUCTURE AND APPROACH SLAB DESIGN

10.1 Curb Heights on Bridges

For all bridges that utilize curbs, the curb height and batter shall match the curb height and batter on the roadway approaches. This condition will exist on bridges with sidewalks which are normally on an urban section. The curb will normally be 6" in height. All projects with sidewalks shall have these features incorporated into the construction plans. Where the roadway approaches the bridge with a raised median, the median on the bridge shall match that on the roadway.

10.2 Pilaster Details for Roadway Lighting

See Structure Standards for details to be used in designing bridges that will require light standards.

10.3 Intermediate Diaphragms for Prestressed Concrete I-Girder Construction

Where skew angle is greater than 30^{o} design as per AASHTO.

Where skew angle is equal to or less than 30^{o} intermediate diaphragms are not required; however, design live load shall be increased by 5% for the purpose of beam design.

(The FDOT prestressed beam program has available an input code which adds the 5% LL.)

10.4 Cast-in-Place Bridge Deck

The thickness of bridge decks that are cast-in-place (CIP) on beams or girders shall be eight inches (8") minimum for all new construction and for widenings defined as major widenings in accordance with Article 16.6.

The thickness of CIP bridge decks on beams or girders for widenings defined as minor widenings in accordance with Article 16.6 shall be handled on an individual basis but, generally, shall match the thickness of the adjoining existing deck.

10.5 Grooving Bridge Floors

All contracts for bridges that include the construction of cast-in-place bridge floors and that will not be surfaced with asphaltic concrete shall include the following item in the Summary of Pay Items:

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

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

For widened superstructures the entire deck area as described above shall be grooved if at least one additional lane is being added. For projects with shoulder widening only, a note should be added to the plans specifying that the bridge floor finish shall match that of the existing bridge. The SDO should be contacted to determine the required bridge floor finish for unusual situations. Penetrant Sealer shall be applied to widening projects when required by and in accordance with Article 7.5.C.

10.6 Prestressed Pretensioned Components, Beam Design and Construction

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

A. General Design Requirements for Prestressed Construction

1. The center-to-center spacing of strands shall be 2 inches for 1/2" diameter or smaller strands and 2-1/4" for 9/16" diameter strands. 0.6" diameter strands are not permitted.

2. Stress computations resulting from prestressing shall be based upon the following lengths of strand transfer and development:

   a. For standard diameter strands permitted:

      1) Transfer length

a) Fully bonded strands = 80 strand diameters

b) Debonded strands = 100 strand diameters (from the terminus of the debonding material).

2) Development length

a) Fully bonded strands = 1.6 times AASHTO 9.27.1

b) Debonded strands per AASHTO 9.27.1 and 9.27.3

b. For prestressed piling not subjected to significant flexure under service or impact loading, strand development shall be in accordance with AASHTO 9.27.1. Bending that produces cracking in the pile, such as that resulting from ship impact loading, is considered significant. Four feet of pile embedment into footings is not considered adequate for this required strand development.

B. Requirements for Prestressed Beams

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

1. Bridges which contain varying span lengths, skew angles, beam spacings, beam loads or other design criteria may result in very similar individual designs. The designer should consider the individual beam designs as a first trial subject to modification by combining similar designs into groups of designs of common materials and stranding based upon the following priorities:

a. 28-Day Compressive Concrete Strength (f'c)

b. Stranding (size, number and location)

c. Compressive Concrete Strength at Release (f'ci)

d. Shielding (Debonding)

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

2. In order to achieve uniformity and consistency in designing strand patterns the following parameters shall apply:

   a. 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" are required except for AASHTO Type V and VI beams for which strand patterns with an even number of strands per row shall be utilized.

   b. In the end regions of the beam a maximum of 25 percent of the strands may be debonded to satisfy the allowable stress limits but debonded strands shall not exceed 37.5 percent of the total strands in any one row. In applying these percentage limitations the following methodology shall be used:

      1) Strands shall be debonded in a pattern that is symmetrical about the vertical axis of the beam.

      2) The theoretical number of debonded strands shall be rounded to the closest even number (pairs) of strands except that debonded strands will not be permitted in rows containing three (3) strands or less.

      3) Not more than 4 or 40% of the total shielded strands, whichever is the greater, shall be terminated at any section. The number of

shielded strands being terminated at the section under consideration shall include those for which the strand force has not yet been fully transferred.

4) Exterior strands in any and all rows shall be fully bonded.

3. The critical stresses at release in the end regions of the beam may be neglected within a distance of 75 percent of the beam height from the centerline of bearing for top of beam tensile stresses and within a distance from the end of the beam equal to the fully bonded strand transfer length for bottom of beam compressive stresses. Stresses beyond these distances shall comply with the requirements stated hereinafter.

4. In analyzing stresses and/or determining the required length of debonding, stresses shall be limited to the following values:

   a. Tension at top of beam (at release) for straight strand profiles only:

      1) Outer 15 percent of design span = $12\sqrt{f'ci}$

      2) Center 70 percent of design span = $6\sqrt{f'ci}$

   b. Tension at top of beam (at release) for depressed (draped) strand profiles only = $6\sqrt{f'ci}$

   c. Compression at bottom of beam (at release) = 0.6f'ci

   d. Compression at top of beam (service load) = 0.4f'c

   d. Tension at bottom of beam (service load):

      1) Slightly or moderately aggressive environment = $6\sqrt{f'c}$

      2) Extremely aggressive environment = $3\sqrt{f'c}$

5. Reinforcement shall be provided in the top flange of all beams to control tensile cracks prior to, during and after release

of the prestressing force. The reinforcement shall consist of a combination of No. 5, ASTM A615, Grade 60, deformed reinforcing bars and 3/8"-diameter (min.), ASTM A416, Grade 250 or 270 prestressing strands initially tensioned to 10,000 lbs. each. The number of bars and strands varies with the beam type as shown in the Structure Design Office's Standard Drawings. Because the majority of the initial tension in the strands is lost prior to the application of the service live load, the effects of these strands on beam stresses after release and the capacity of the beams under service load may be ignored. The No. 5 reinforcing bars in each end of the beam shall be a minimum of 10 feet or 15 percent of the design span length, whichever is greater.

6. The strands in the bottom flange shall be confined for a distance from the beam end equal to one and one-half (1.50) times the depth of the beam. The confinement shall be provided by enclosed No. 3 deformed ties, ASTM A615, Grade 60, spaced 6 inches center-to-center as shown in the Structures Design Office's Standard Drawings. The ties shall be of standard dimensioning and shape for a given beam type regardless of strand pattern. For straight strand designs only, the requirement of AASHTO 9.21.3 to provide 4% of the prestressing force as vertical stirrups at a unit stress of 20,000 psi is waived.

7. The use of "L" shaped longitudinal bars, in the webs and flanges in the end zone areas, is required.

8. Unless otherwise specificaly directed or approved by the Department, the vertical stirrup reinforcement for fully pretensioned beams shall be symmetrically spaced about the centerline of the beams. The stirrups shall be at least No.

4 bars and shall be spaced at not more than the following increments:

Standard Beam Designation	Support to 1/4 of Span	1/4 of Span to Centerline of Span
Type II, III, IV, V, VI and All Bulb-Tee Beams	1 - No. 4 @ 6" o.c.	1 - No. 4 @ 12" o.c.

The preferred detailing practice is to provide several stirrups at a transition spacing (7-1/2" to 10-1/2") past the 1/4-Span to avoid an abrupt change in the stirrup spacing. This also provides a smooth transition when considering the effect of the stirrup's extension into the cast-in-place deck slab.

9. The minimum compressive concrete strength at release shall be 4,000 psi. Higher release strengths may be used and specified when required by the designer but generally should not exceed 4,500 psi unless otherwise specifically approved by the FDOT Review Office.

10. Prestressed beams shall be designed and specified to conform to one of the following classes and related strengths of concrete:

Class of Concrete	28-Day Compressive Strength
Class III	(*a) 5,000 p.s.i.
Class IV	5,500 p.s.i.
Class V (Special)	6,000 p.s.i.
Class V	6,500 p.s.i.

(*a) Class III Concrete may be used only when the superstructure environment is "slightly aggressive".

11. A note shall be shown on the beam sheet in the General Notes stating "Optional stranding shall comply in all respects with the Department's Structures Design Guidelines".

12. All prestressed concrete components shall be designed for stay-in-place metal forms when they are permitted by the Specifications. The weight to be used in the design shall be in accordance with Article 4.4 of these guidelines.

    A note shall be added to the "General Notes" of the bridge plans stipulating the weight allowance included in the beam design for the stay-in-place metal forms and concrete required to fill the form flutes. Additionally, a note shall be added requiring all embedded items and accessories required for their use to be included on the Shop Drawing submittal for the forming system.

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

    Refer to Article 10.8 of these Guidelines for stay-in-place metal form quantity calculation and cost notes to be added to the Superstructure drawings.

10.7 <u>Precast Prestressed Slab Units and Double Tee Beams</u>

All bridge structures designed utilizing precast, prestressed slab units or precast, prestressed double tee beams shall conform, in general, with the design criteria and details provided in the <u>Standard Drawings</u> of the Structures Design Office.

10.8 <u>Stay-in-Place (SIP) Slab Forms</u>

For stay-in-place metal and concrete forms see Articles 400-5.7 and 400-5.8 in the FDOT <u>Standard Specifications for Road and Bridge Construction</u>. SIP metal forms shall be designed and detailed for prestressed beam and steel girder superstructures in accordance with Articles 10.6. and 11.7 of these Guidelines, respectively, except where restricted for use by Article 7.5.

10.9 Prestressed Beam Camber/Build-up Over Beams

In the past, the Department 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 of 2 to 3 times the initial camber at release is not uncommon.

The Department's prestressed beam computer program provides a table of net camber versus time for the designer's use in estimating the build-up over beams required at the time of slab casting. The values in the program account for creep growth of camber with time and have been closely verified by field measurements.

Until sufficient experience with the predicted versus actual camber has been achieved to permit reliable parameters to be established and utilized, the designer must give consideration to camber growth in the design.

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

10.10 Post-tensioning Strand

Except where otherwise specifically permitted by the Department, strand for post-tensioning applications, either for longitudinal or transverse prestressing, shall be 0.6 inch diameter, ASTM A416, low relaxation strand.

10.11 Dimensional and Location Requirements for Post-Tensioning Ducts

The designer shall show offset dimensions to post-tensioning duct trajectories from fixed surfaces or clearly defined reference lines at intervals not exceeding five (5) feet. When the rate of curvature of a duct exceeds two (2) degrees per foot, offsets shall be shown at intervals not exceeding two and one-half (2.50) feet. In regions of tight reverse curvature of short sections of tendons, offsets shall be shown at sufficiently frequent intervals to clearly define the reverse curve.

Post-tensioning ducts placed within limited section thicknesses, such as slabs and webs of spanning members, shall be limited in <u>outside</u> dimension to 40% of the total section thickness. The thickness shall be measured perpendicular to the centerline plane of the section. Additionally, the clear post-tensioning duct cross-sectional area shall be at least two and one-half (2.50) times the total area of the prestressing steel specified. It is recommended that a clear distance of at least one-half (0.50) times the duct diameter, one and one-half (1.50) times the maximum aggregate size or one and one-half inches (1-1/2"), whichever is the greater, be maintained between ducts running in the same general direction. At joints in segmental construction, it is recommended that the minimum clear distance between ducts be at least two inches (2"). Ducts which cross at right angles or similar large angles may be in contact.

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

10.12 <u>Barrier/Railing Distribution</u>

In lieu of a more refined analysis and for superstructures not less than 40 feet nor more than 110 feet in length, the dead load of barriers and railings attributed to exterior beams or stringers may be determined by the following equation:

$$W_{ext} = \frac{(W)(C_1)(C_2)}{100}$$

Where: W_{ext} = Uniform dead load of the barrier on the exterior beam (lbs./ft.)

W = Total uniform dead load of the barrier (lbs./ft.)

C_1 = Beam spacing coefficient from Figure 10-1

C_2 = Span length coefficient from Figure 10-2

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

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

The distribution methods described above apply to concrete or steel, longitudinal beam or stringer superstructures on which a concrete slab is cast, compositely, prior to installation of the barrier or railing. For superstructures not conforming to the limitations stated above the barrier/railing distribution may be determined in accordance with AASHTO, Article 3.23, or by individual analysis.

10.13 Design of Beams with Variable Spacings

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

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

$$Vx = V_{avg} \sqrt{\text{Beam Spacing at Section/Average Beam Spacing}}$$

10.14 Prestressed Beam Standards and Semi-Standards

Prestressed beam standards and semi-standards included in the Structures Design Office Standard Drawings are available for use in simple span applications only.

10.15 Pretensioned/Post-tensioned Beams

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

A. The pretensioning acting alone shall comply with the minimum steel provisions of AASHTO Article 9.18.2.

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

C. The pretensioning shall be of such a level that the initial midspan camber at release, including the effect of the dead load of the beam, shall be at least one-half (1/2) inch. In computing the initial camber, the value of the modulus of elasticity shall be in accordance with Table 4.1 of these guidelines for the minimum required strength of concrete at release of the pretensioning force and the pretensioning force in the strands shall be reduced by losses due to elastic shortening and steel relaxation.

10.16 Special Requirements for Bulb-Tee Beams

The recommended web thickness of the Florida Bulb-Tee series of beams shall be:

Pretensioned Beams	6-1/2"
Post-tensioned Beams	7"

In designing concrete cast-in-place slabs supported by bulb-tee beams, the slab design in accordance with AASHTO Articles 3.24 and 8.8 shall be predicated on an effective clear span equal to the actual clear span between edges of the flanges of adjacent bulb-tee beams plus eighteen (18) inches.

When applying the provisions of AASHTO, Article 8.10, the full width of the top flange shall be considered effective.

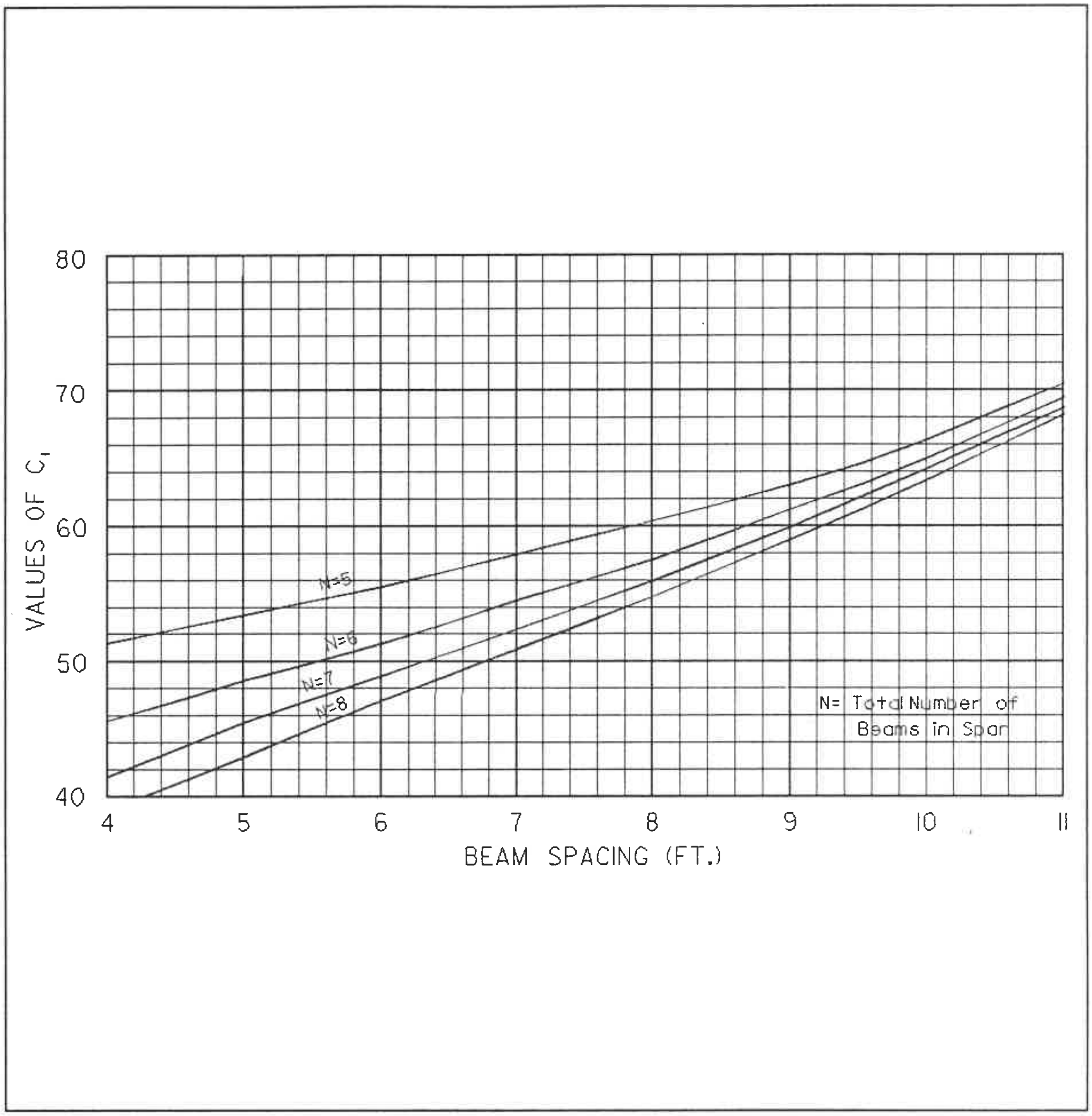
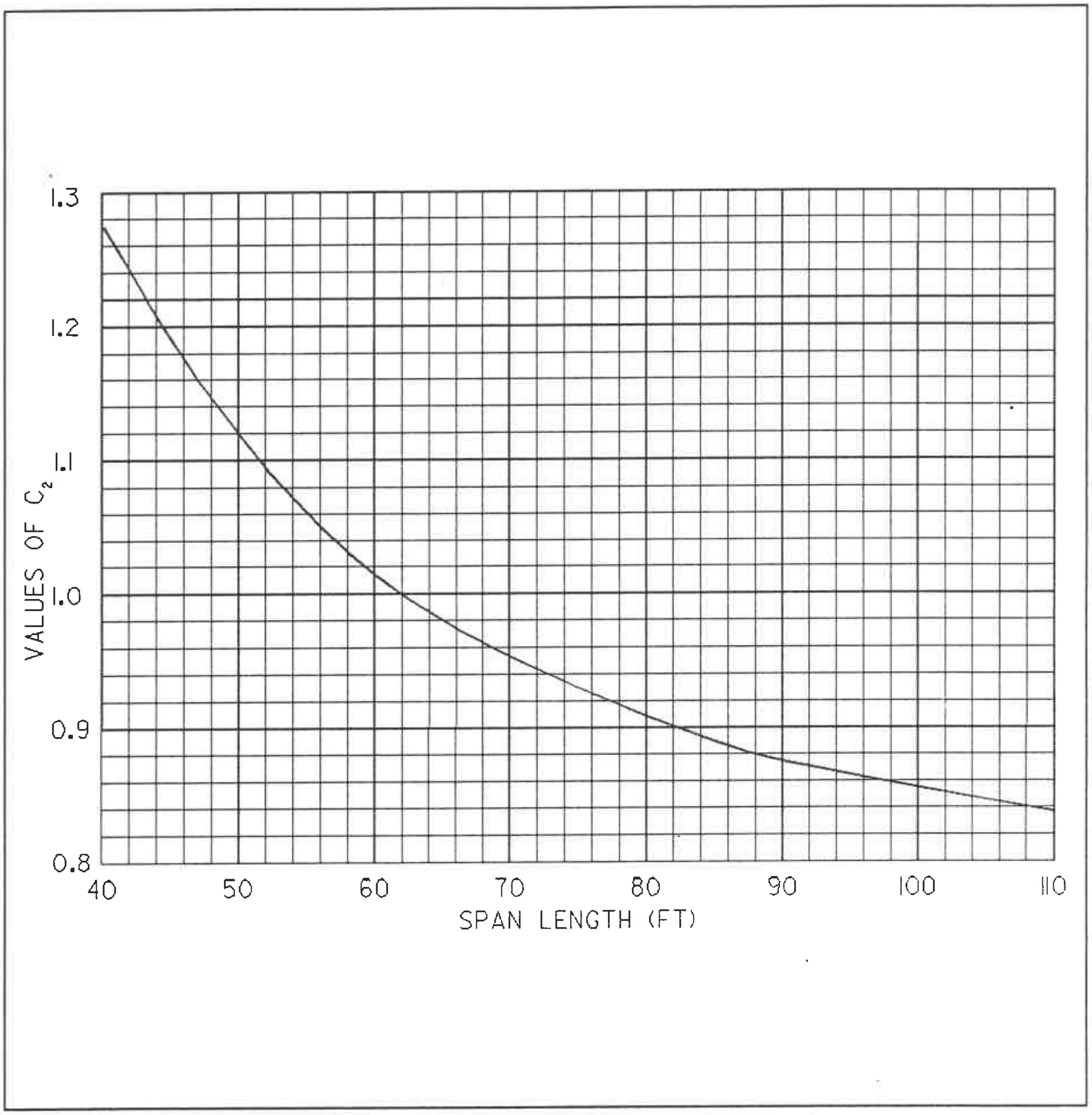

80
70
60
50
40
VALUES OF C_1
N=5
N=6
N=7
N=8
N= Total Number of
Beams in Span
4
5
6
7
8
9
10
11
BEAM SPACING (FT.)

Figure 10-1



1.3
1.2
1.1
1.0
0.9
0.8
VALUES OF C_2
40
50
60
70
80
90
100
110
SPAN LENGTH (FT)

Figure 10-2

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Chapter 11

STRUCTURAL STEEL DESIGN

11.1 Design Criteria for Steel Bridges

Design of steel bridge components shall be in accordance with the AASHTO Standard Specifications for Highway Bridges (1989), and all Interim Specifications currently in effect. Also, horizontally curved plate girder and box girder designs and details shall be in accordance with the AASHTO Guide Specifications for Horizontally Curved Highway Bridges (1980), as updated through the current Interim Specification.

The following general note shall be on the plans for all Category 2 structural steel bridges:

> All structural steel for girder framing (including plate girders, diaphragms, cross bracing, lateral bracing, etc.) shall be fabricated by a certified shop meeting the requirements of AISC Category III certification.

All steel bridges currently under design are subject to the current specifications. Design of new steel girder bridges shall be in accordance with the Strength Design Method - Load Factor Design. If "AutoStress" design is used, the designer must insure that flange plate sizes meet the criteria of Article 11.2 below. Refer to Article 16.1 of these guidelines for design methods applicable to widening of existing structures.

The design of structural steel for miscellaneous items such as ladders, platforms and walkways shall be in accordance with the AISC Manual of Steel Construction, Allowable Stress Design. The designer shall use gravity and lateral live loads for these items in accordance with requirements of the Occupational Safety and Health Act (OSHA).

11.2 Steel Strength and Thickness Limitations

The yield strength of structural steels used in design shall be limited to 50 KSI unless otherwise approved by the Department. ASTM A709, Grade 36 (ASTM A36/A36M) and ASTM A709, Grade 50 (ASTM A572/A572M) steels are used in rolled shape, plate girder and box girder designs. ASTM A709, Grade 50W (ASTM A588/A588M) "weathering steel" is used in bearing assemblies, and ASTM A709, Grade 100 (ASTM A514/A514M) may be approved by the Department in certain cases for use in steel truss chord members. ASTM A709, Grade 70 (ASTM A852/A852M) is a recent addition to AASHTO allowable steels and has not been used in Florida. The Department will consider its use for specific redundant projects, based on recommendation from the steel industry.

For main load-carrying members the minimum thickness of web plates shall be 3/8". The minimum size of flange plates shall be 3/4" x 12". The maximum size of plate girder flange plates shall be 3" x 33". The minimum thickness of box girder bottom flanges shall be 1/2".

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

11.3 Tension Member and Fracture Critical Requirements

Main load carrying member components subjected to tension and stress reversal require Charpy V-Notch (CVN) impact testing on the material used. The CVN testing is required regardless of whether the structure is redundant or non-redundant, even though the base metal CVN values are more stringent for non-redundant structures.

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

Fracture critical steel bridge members shall be designed in accordance with the AASHTO Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members (1978) as updated through the current interim Specification, and the plans shall note that design, fabrication and inspection shall be in accordance with these specifications. The plans shall designate with a symbol all girder components that require CVN testing only (tension component, redundant member), and they shall designate with a different symbol all girder components that are fracture critical (tension component, non-redundant member).

Splice plates that are attached to tension components shall also be designated by the appropriate symbol.

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

11.4 Connection Details and Hardware

The designer shall design and detail bolted attachments for all field connections rather than welded attachments. Field welding is not allowed, except by prior written approval by the Department. All bolted connections, other than anchor bolts, shall utilize ASTM A325 (Type 1) high-strength bolts designated as a slip-critical connections. Bearing-type connections shall be used only with prior approval from the Department. The allowable load for slip-critical connections should be based on a Class A (Slip Coefficient 0.33) contact surface of bolted parts for typical steel components using the three coat painting system.

ASTM A325 bolts of 3/4", 7/8" and 1" diameters should be utilized typically. Larger diameter bolts may be used with prior approval of the

Department. ASTM A490 bolts or load indicating (DTI) washers shall not be called for on the plans unless authorized by the Department.

Anchor bolts and miscellaneous tie-down hardware may conform to ASTM A36 for rolled shapes, plate and threaded bar stock or to ASTM A307 for standard bolt diameters and lengths.

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

Bearing stiffeners for plate or box girders shall be connected to the bottom flange with full penetration groove welds or be shown to receive a "mill to bear" interface with minimum size fillet welds each side. Bearing stiffeners shall have a "tight fit" interface at the top flange except as noted below.

Bearing stiffeners that provide diaphragm connections at interior piers of continuous girders shall be rigidly connected to the top flange by a bolted or welded connection. Bearing stiffeners that provide diaphragm connections at the end spans (negative moment not present) of girders shall receive properly sized fillet welds at the top flange interface.

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

Refer to Article 11.6 for additional information regarding connection details for diaphragms and lateral bracing.

11.5 Welding and Nondestructive Testing Requirements

All welding design, fabrication and inspection shall be in accordance with the 1988 AASHTO/AWS Bridge Welding Code (AASHTO/AWS 01.5-88) and, where applicable, the fracture critical guide specifications noted above. The contract plans shall use welding symbols as described in Section 2.1 of the Code. The designer should not show a specific prequalified joint designation on the plans unless a certain type of groove weld ("V", 'J", "U", etc.) is required. The shop drawings ("detail drawings") shall detail the exact weld type proposed by the fabricator, and it is subject to approval of the designer and the Department at that time.

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

When welding is required during rehabilitation or widening of an existing structure, the plans shall designate the type of existing base metal. If the designer cannot ascertain the type or if the type is not an approved base metal included in the Bridge Welding Code, then a welding investigation must be performed. The designer is responsible for contacting the District Office so that the Materials and Research office and the Department's independent testing firm can obtain this information necessary to define the welding and welding inspection requirements for the project.

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

11.6 Diaphragm and Lateral Bracing Requirements

Internal and external diaphragms shall be typically detailed such that the diaphragm is a shop fabricated "frame" (or rolled beam) that is field connected to the steel beams or girders at stiffener locations. A "K-frame' diaphragm is preferred when the girder depth is less than the girder spacing; an "X-frame" configuration may be considered for deeper girders.

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

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

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

11.7 Provisions for Stay-in-Place Metal Forms

The superstructures and all supporting structural steel components including beams, plate girders and box girders shall be designed and detailed for stay-in-place metal forms in accordance with Article 10.8 of these guidelines, when forms are permitted by the FDOT Standard Specifications for Road and Bridge Construction subject to the environmental restrictions of Article 7.5. The weight to be used in the design shall be in accordance with Article 4.4 of these guidelines. Stay-in-place metal forms shall also be designed and detailed for use as the interior (unexposed) soffit of all steel box girders even when generally otherwise prohibited by the Specifications.

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

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

Refer to Article 10.8 of these guidelines for stay-in-place metal form quantity calculation and cost notes to be added to the superstructure drawings.

11.8 Corrosion Protection of Structural Steel and Hardware

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

All anchor bolts, tie-down hardware and miscellaneous steel (ladders, platforms, grating, etc.) are usually hot-dip galvanized; however, high-strength bolts may or may not receive a galvanized (electroplated) coating, when used with elements to be painted.

The designer is advised to contact the Department's Structures Design Office for clarification regarding corrosion protection for individual items.

Approved: Jerry Potter

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Topic No.: 625-020-112-h

Chapter 12

EXPANSION AND CONTRACTION DEVICES IN BRIDGE STRUCTURES

12.1 General Policy

Bridge structures must be designed to accommodate movements produced by such factors as temperature, elastic shortening, creep and shrinkage. Structural deterioration due to leaking expansion joints, and expansion bearings incapable of movement constitute major bridge maintenance problems. Therefore, bridge designers should not accommodate these movements by using unnecessary bridge deck expansion joints.

The FDOT has a preference for structural continuity of superstructure and, when possible, pinned or fixed connections of substructure to superstructure. The present policy is to design and construct bridges without any bridge deck expansion joints unless absolutely necessary. Generally, this is accomplished with continuous superstructure slabs and either fixed or integral bearings at the piers and end bents . When expansion joints are necessary, attempt to limit these to the bridge ends only.

12.2 Expansion Joint Provisions

Expansion joints shall be avoided whenever possible. However, when a joint is required, it should satisfy as many of the following criteria as possible:

A. Accommodate the full range of structure movements without exceeding the manufacturer's recommended clear span at deck surface level when at maximum opening.

B. Provide proper anchorage and structural capacity to resist the anticipated loads.

C. Have good riding qualities

D. Should not impart undue stresses to the structure due to structure expansion and/or contraction.

E. Be reasonably silent and free of vibration.

F. Facilitate maintenance repair, removal and replacement.

G. Be leakproof with the sealing element continuous for the entire structure width.

H. Be corrosion resistant.

I. Not be a catalyst or vehicle for electrolytic action.

An open joint with provision for diverting drainage without damage to the bridge bearings or other structural elements is also acceptable.

12.3 Design of Neoprene Pads and Joint Seals to Accommodate Movement

The movement from variations in temperature shall be calculated from an assumed temperature of 70^{o} F at the time of erection. The range of temperature shall be that as shown in the Table below:

Structural Material	Temperature of the Structure (In Degrees F)			
	Mean	Rise From Mean	Fall From Mean	Range
Concrete only	70	+25	-25	50
Steel Girder (Plate, WF or Box) with Concrete Deck	70	+35	-35	70
Steel only (Moveable, Arch, Truss Orthotropic)	70	+50	-35	85

The coefficients for thermal expansion by structure type shall be:

Steel Structures: 0.0000065 per F^{o}
Concrete Structures: 0.000005 per F^{o}

Consideration shall be given to creep and shrinkage, the rack caused by skew movements, as well as the effects of temperature range. In no case shall a nominal uncompressed width of seal joint greater than four (4) inches be used. The use of strip seals is preferred in lieu of compression seals.

The Calculation for the Required Opening Range shall include the effects of Temperature, creep and shrinkage. If post-tensioned concrete components are used, the total calculated movement due to temperature, creep, and shrinkage shall be increased by 20% for the purpose of joint selection.

The recommended widths of joints by joint type are:

Joint Type	Joint Width
Poured rubber	to 1/2" (max.)
Strip seal	9/16" to 3"
Modular or finger joint	greater than 3"

The use of stacked standard neoprene bearing pads to create a bearing to accommodate a movement that is more than can be accomplished through one standard neoprene bearing pad by itself is prohibited. A special neoprene bearing pad shall be specifically designed in this eventuality.

12.4 Joints on Bridge Widening Projects

Existing Type Bridge Joints:

Group 1: Compression seals, open joints, poured rubber and copper water-stop.

Group 2: Sliding Plate, finger joint, patented or proprietary systems.

Treatment of Joints:

Group 1: Install compression seals that are continuous from gutter to gutter on the widened bridge. Provision for seal installation shall be made as required by saw-cutting the existing decks.

Group 2: The joints in this group that are in good condition shall remain in place and the same type joints shall be used in the bridge widening.

The joints in this group that are not in good condition shall be inspected by the designer to determine the corrective action necessary to fix the joints. The type joints used shall extend from gutter to gutter of the widened bridge.

12.5 Maintainability Requirements - Bearings

With the exception of slab spans that span cap to cap, all bridge bearings shall be accessible for inspection and maintenance and those for new bridges shall be replaceable without damage to the structure and without removing anchorages permanently attached to the structure. Provisions for future bearing replacement must be made in the design and shown on the bridge plans (i.e., jacking locations, jacking sequence, jack load, etc.).

For conventional concrete girder structures, future bearing replacement has been relatively simple as jacking can be accomplished between the end diaphragm and bent cap. For these bridges, a note may describe the future bearing replacement procedure. The plans shall state that jacking equipment is not a part of the contract and does not need to be provided by the contractor building the bridge. For other structure types, a separate drawing and description may be required.

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

Service Load Design: Group I, except 140% allowable stress

Load Factor Design: Group I, except $\beta\ (L+I)_n = 1.30$.

The District Maintenance Engineer will determine if bearing replacement of a bridge widening must be included in the design plans.

Chapter 13

RAILROAD CROSSINGS

13.1 General

Railroad grade crossings should be avoided by either underpassing or overpassing the railway. For underpasses, the bridge carries the railway and must be designed and constructed to carry railway loadings in conformance with the American Railway Engineering Association (AREA) Code. For overpasses,the bridge carries highway traffic and must be designed and constructed to carry highway loadings. In either case, adequate clearances between the facilities must be provided.

Clearances, geometrics, utilities, provisions for future tracks, and maintenance road requirements for off-track equipment will involve negotiations with the railroad company. The railroad's review and approval, including need for and location of crash walls shall be based on the completed 30% plans prepared by the Structures Design Office, District Structures Design Engineer or their Consultant.

There are presently four major railroad companies operating in the State of Florida:

CSX Transportation, Incorporated

Florida East Coast Railway Company

Burlington Northern Railroad Company

Norfolk Southern Corporation

13.2 Criteria

The 30% plans shall be prepared in accordance with the criteria obtained from the Railroad company and as shown in these Guidelines and in the Detailing Manual.

See Figure 13-1 for dimensions which must be obtained from the railroad company before preparing the 30% plans.

The Transit/Rail Office of the Florida Department of Transportation is an additional resource available to the designer.

13.3 Bridge Width

For overpasses, the highway bridge width is determined from the approved typical section for the proposed bridge. Details for underpasses will depend on the specific project.

13.4 Horizontal Clearances to Face of Structures

Horizontal clearances shall be measured in accordance with Figure 13-1(dimensions "A" and "B"). The minimum and preferred horizontal clearances measured from the ℄ of outside track to the face of pier cap, bent cap or any other adjacent structure are as follows:

	Normal Section	With 8' req'd. clr. for Off-track Equip.	Temporary Falsework Opening
Minimum Clearance	18 feet	22 feet	10 feet
Preferred Clearance	25 feet	25 feet	-

These clearances shall be adjusted in accordance with the following:

A. When track is on a curve the minimum clearance shall be increased at a rate of 1-1/2" for each degree of curvature.

B. Columns or piles should be kept out-of-the ditch to prevent obstruction of drainage.

The preferred horizontal clearance should be provided to avoid the need for crash walls unless extenuating circumstances dictate otherwise.

Figure 13-1 also shows the maximum allowable distances from ℄ track to the intersection of a horizontal plane at the rail elevation and the embankment slope. This criteria may be used to establish preliminary bridge length.

However, surrounding topography, hydraulic conditions, and economic or structural considerations may warrant a decrease or an increase of the above clearances.

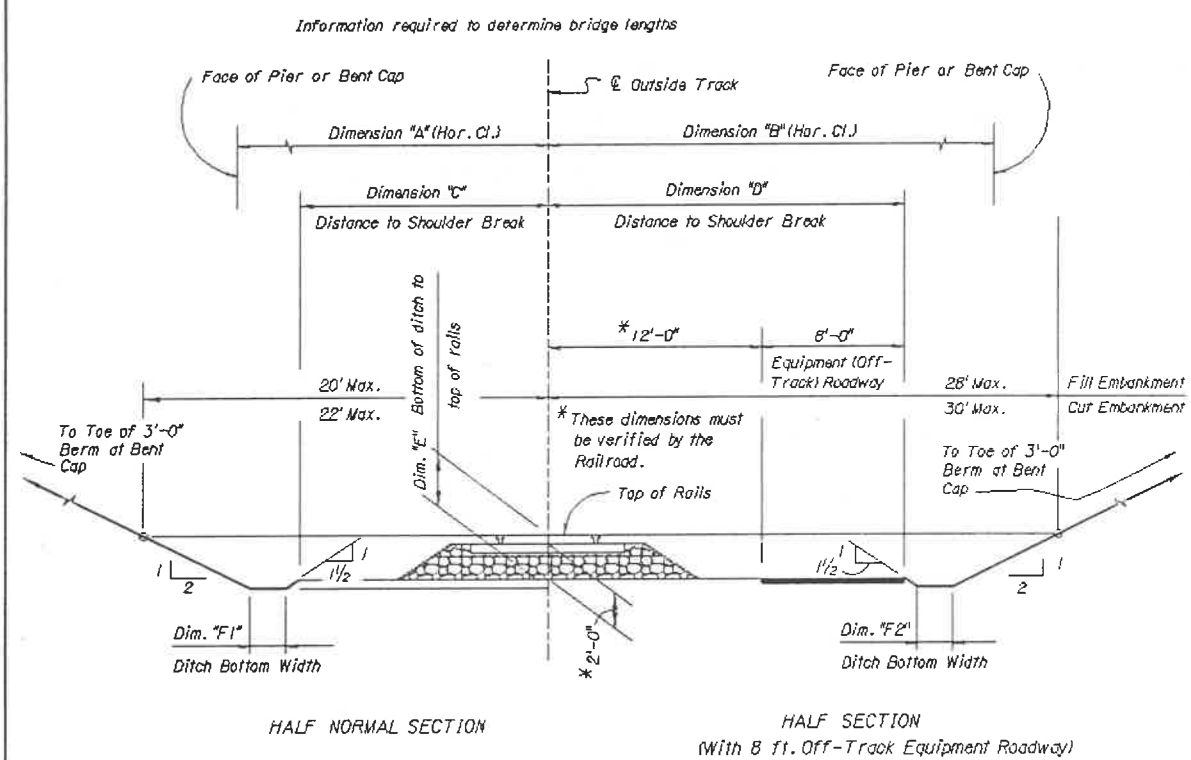

Information required to determine bridge lengths
Face of Pier or Bent Cap
℄ Outside Track
Face of Pier or Bent Cap
Dimension "A" (Hor. Cl.)
Dimension "B" (Hor. Cl.)
Dimension "C"
Distance to Shoulder Break
Dimension "D"
Distance to Shoulder Break
Dim. "E" Bottom of ditch to top of rails
*12'-0"
8'-0"
Equipment (Off-Track) Roadway
20' Max.
22' Max.
28' Max.
30' Max.
Fill Embankment
Cut Embankment
To Toe of 3'-0" Berm at Bent Cap
*These dimensions must be verified by the Railroad.
To Toe of 3'-0" Berm at Bent Cap
Top of Rails
1
2
1
1½
1
1½
1
2
*2'-0"
Dim. "F1"
Ditch Bottom Width
Dim. "F2"
Ditch Bottom Width
HALF NORMAL SECTION
HALF SECTION
(With 8 ft. Off-Track Equipment Roadway)

Figure 13-1

The additional 8 foot of horizontal clearance for off-track equipment shall be provided only when specifically requested in writing by the railroad. In case of doubt, check with the Transit/Rail Office of the Florida Department of Transportation.

Sometimes the Railroad company will accept a waiver of a normal clearance requirement if justified. Therefore, it is possible to use something less than the established standards; for example, for designs involving widening or replacement of existing overpasses. The Transit/Rail Office of the Florida Department of Transportation should be consulted if such action is being considered.

13.5 Crash Walls

Except as stated below, crash walls are required for all bridges over railroads in which any part of the substructure above the ground is to be constructed closer than 25 feet from the centerline of the track (See Figure 13-2). Multiple column piers with columns 4 feet in diameter or greater (or equivalent area) do not require crash walls.

The crash wall shall be constructed integral with the pier or bent and shall have a smooth face. The top of the wall shall be 6 feet higher than the top of the rail. Piles for the crash wall shall be driven to the minimum penetration required by the FDOT Specifications.

Existing piers or bents with crash walls which are lengthened to accommodate bridge widening, shall have the crash walls extended. The new crash wall portion shall meet the requirements for new construction.

Existing piers or bents with less than 25 feet of clearance and without crash walls, which are lengthened to accommodate bridge widening shall have separate short crash walls constructed at the ends of the pier or bent. The crash walls should be added to both ends of the pier or bent even if the lengthening of the pier or bent was confined to one end only.

13.6 Vertical Clearance

For new construction, the minimum vertical clearance that should be provided at an overpass is 23'-6". This is the least distance between the bottom of the superstructure and the top of the highest rail utilized anywhere within a 12'-0" wide corridor centered on the centerline of the track. If a track is identified as an electrified railroad, the minimum vertical clearance shall be 26.0 feet. This provision is based on FHWA policy for a 50 KV line. In addition to existing electrified railroads, this provision applies to tracks identified as candidates for future electrification.

13.7 Special Considerations

A. Shoring and Cribbing requirements during construction should be accounted for in the preparation of the preliminary plans to assure compliance with the clearance criteria set forth herein. See Figure 13-3.

NOTE: Anything (e.g., cofferdams, footings, excavation, etc.) encroaching within 10 feet of ℄ of the track requires approval of the Railroad.

B. Overpasses for electrified railroads may require protection screens.

C. Sometimes the substructure supports may be located between tracks or an outside track and the off-track equipment road.

13.8 Widening of Existing Overpasses

A. The requirements of the Railroad companies for widening existing overpasses are as follows:

1. If existing horizontal or vertical clearances are less than those required for a new structure, it is required that the new portion of the structure be designed so as not to encroach into the existing clearances.

2. Permanent vertical clearances will have to take into account the track grade and the cross slope of the bridge

superstructure. Therefore, it is generally more desirable to widen on the ascending side of the bridge cross slope.

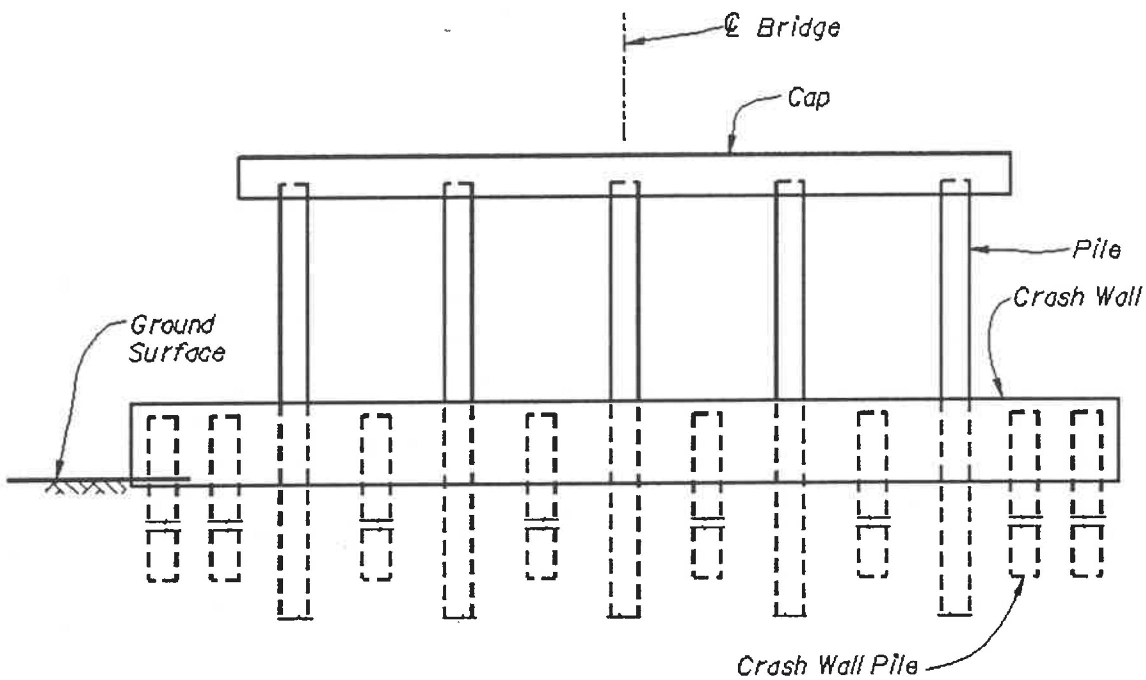
3. Permanent horizontal clearances will have to take into account horizontal curves and substructures that are not presently parallel to the track.

4. Temporary construction clearances are particularly critical where existing clearances are already substandard. If clearances less than 22 feet vertical and 10 feet horizontal are necessary, they will have to be approved on an individual basis. On high volume main lines, it may not be possible to reduce already restricted vertical clearances.

5. If widening requires construction of new widened approach fills, it is required that the same consideration be given to drainage design as required on new bridges. If new substructures provide less than 25 feet horizontal clearance from center line of track, they must be designed with crash wall protection except as stated in 13.5.

B. Thirty percent (30%) Bridge plans should show a cross section at right angles to the centerline of the track where centerline of bridge intersects the centerline of track. In situations where the substructure is not parallel to the track or the track is curved, sections perpendicular to the centerline of tracks shall be furnished at each substructure end.

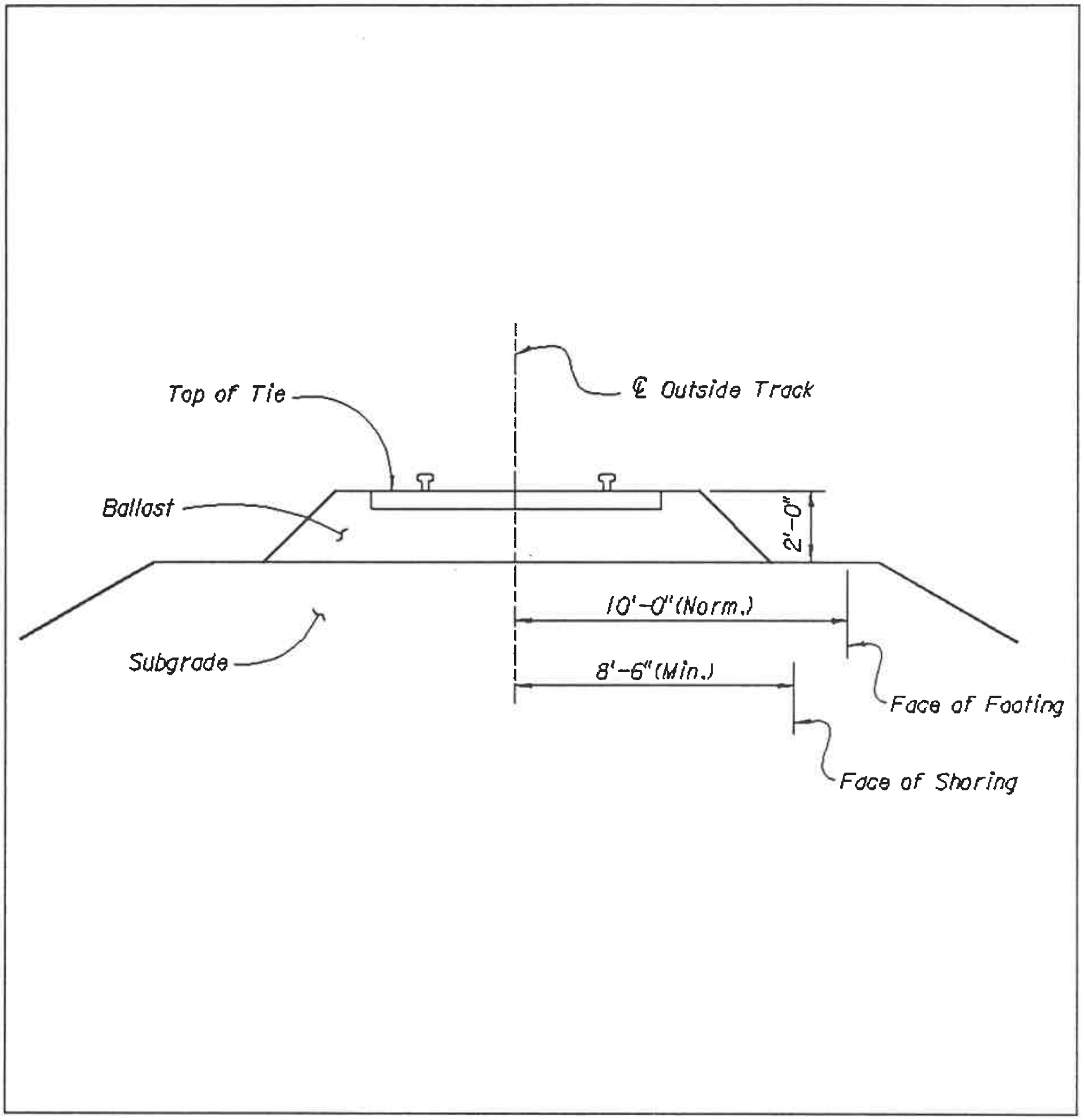
C. If the Railroad is in an existing cut section, plan approvals will be considered by the railroads on an individual location basis. Factors to be considered will be the length, depth, and type material of the existing cut section, in addition to all of the previously mentioned factors.



ELEVATION
(Int. Bent Shown)

NOTES:
1. Crash Wall is required if clearance to ℄ track is less than 25'.
2. Wall thickness shall be equal to cap thickness but not less than 3'.
3. Crash wall piling shall be 18" prestressed driven to minimum penetration.
4. Top of wall shall be 6' higher than top of rail.
5. Bottom of wall shall be 6" minimum below ground.
6. For additional details see Detailing Manual.

Figure 13-2



Top of Tie
℄ Outside Track
Ballast
2'-0"
10'-0"(Norm.)
Subgrade
8'-6"(Min.)
Face of Footing
Face of Shoring

Figure 13-3

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Topic No.: 625-020-114-h

Chapter 14

PEDESTRIAN OVERPASSES AND BIKEWAY BRIDGES

14.1 Purpose

To provide bridges exclusively for pedestrians, bicycles or combination pedestrian/bicycle traffic.

14.2 Pedestrian Bridges

A. The current edition of AASHTO Standard Specifications for Highway Bridges with Interims, with applicable modifications by sections of these Guidelines shall govern.

B. Live load design shall be 85 pounds per square foot. (AASHTO 3.14.1.3)

C. Minimum vertical underclearances shall be 17.00 feet over all roads and 23.50 feet over Railroads.

D. Ramp type approaches shall be used. Maximum grade of ramps shall be 8.33 percent. Intermediate level platforms 5' long shall be provided, at maximum 30 foot intervals. Additionally, a level platform 5 feet long at the top and 6 foot long at the bottom shall be provided. Stairs will be permitted only at locations with unusual circumstances where the limits of right of way are extremely tight and only when permission is obtained (FHWA approval is required on Federal-Aid projects).

E. The minimum walkway width on a pedestrian overpass is 8.00 feet.

F. Chain link fence (enclosed type) shall be provided across the entire walkway. Use of chain link fence on ramps of the pedestrian bridges will be determined on a project to project basis. See Figure 14-1 for typical detail.

G. Walkway Clearance. See Figure 14-1.

14.3 Bicycle Bridges

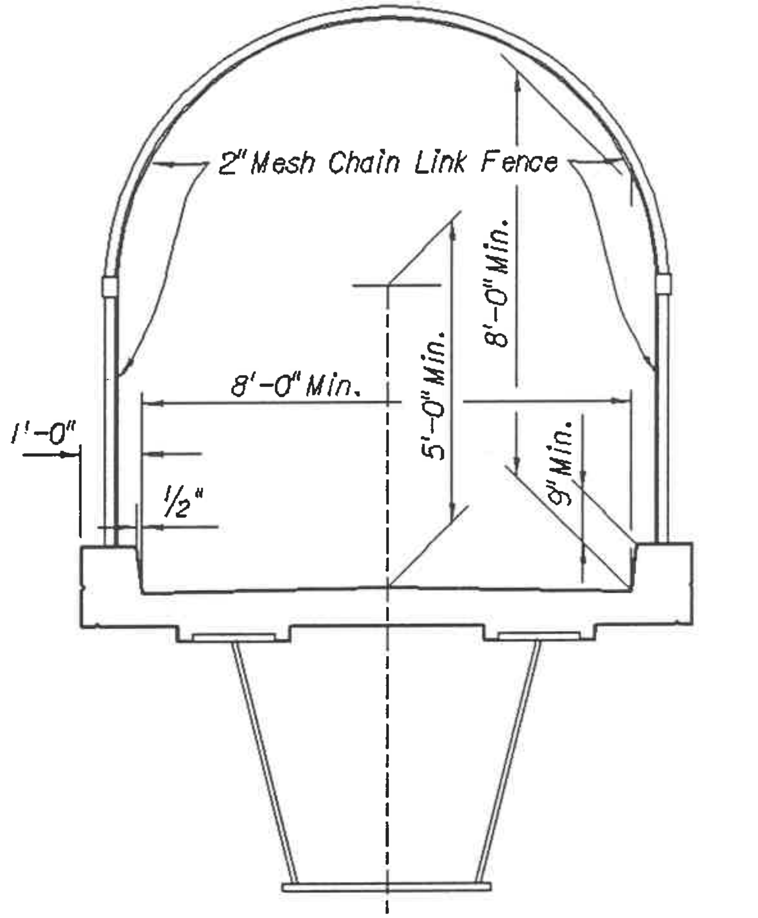
A. The design shall be in accordance with the criteria established in 14.2, Pedestrian Bridges.

B. Reference is also directed to the 1974 AASHTO GUIDE FOR BICYCLE ROUTES for additional guidelines.

14.4 Pedestrian Underpasses

Pedestrian underpasses are generally undesirable; however, if one is required, the geometrics and lighting requirements should be discussed with the District Project Manager. Local law enforcement personnel may need to be consulted to assure public safety, emergency accessibility and other desirable features.

14.5 The MANUAL OF UNIFORM MINIMUM STANDARDS FOR DESIGN, CONSTRUCTION AND MAINTENANCE FOR STREETS AND HIGHWAYS for the State of Florida should also be referenced for Pedestrian and Bicycle Bridges and Underpasses.



TYPICAL SECTION
(Other Superstructure Configurations may be used provided an 8'-0" minimum headroom is maintained)

Figure 14-1

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Chapter 15

BASCULE BRIDGE MAINTAINABILITY

15.1 Policy

The technical policy of the Department is to incorporate maintainability features in the design of bascule bridges consistent with application of sound engineering principles and reasonable application of engineering judgment.

This policy is being implemented through the following Design Guide for Maintainability of Bascule Bridges. This policy is applicable to both new bridges and rehabilitation plans for existing bridges on which construction has not been initiated. Variations from this policy for existing bridges being rehabilitated may be authorized, by approval in writing, by the Structures Design Office.

15.2 Trunnion Bearings

Trunnion bearings shall be designed so that replacement of bushings can be accomplished with the span jacked 1/2 inch and in a horizontal position. Suitable jacking holes or puller grooves are to be provided in bushings to permit extraction. Jacking holes shall utilize standard bolts pushing against the housing which supports the bushing.

Trunnion bushings and housings shall be of a split configuration. The bearing cap and upper half bushing (if an upper half bushing is required) shall be removable without span jacking or removal of other components.

15.3 Span Jacking of New Bridges

NOTE: The following definitions describe elements of the span jacking system.

A. Span jacking surface -- an area located under the trunnion on the bottom surface of the bascule girder.

B. Span stabilizing connector point (forward) -- an area adjacent to the live load shoe point of impact on the bottom surface of the bascule girder.

C. Span stabilizing connector point (rear) -- an area at the rear end of the counterweight on the lower surface of the counterweight girder. (NOTE: For bascule bridges having tail locks, the span stabilizing connector point may be located on the bottom surface of the lockbar receiver located in the counterweight.)

D. Stationary jacking surface -- this surface is located on the bascule pier under the span jacking surfaces. The stationary jacking surface provides an area against which to jack for lifting the span.

E. Stationary stabilizing connector points are located on the bascule pier. These points provide a stationary support for stabilizing the span, by connection to the span stabilizing connector points. One set of span jacking surfaces shall be located under the trunnions (normally, this will be on the bottom surface of the bascule girder). A second set shall be located on the lower surface at the rear end of the counterweight.

Span stabilizing connector points shall be located on the bascule girder forward and back of the span jacking surfaces. The stationary stabilizing connector point (forward) shall be in the region of the Live Load Shoe. Stationary stabilizing connector points (rear) are to be located on the cross girder support at the rear of the bascule pier. Connector points shall be designed to attach stabilizing structural steel components. (NOTE: The stationary jacking surface shall be positioned at an elevation as high as practical so that standard hydraulic jacks can be installed.)

15.4 Trunnion Alignment Features

Center holes shall be installed in trunnion shafting to allow measurement and inspection of trunnion alignment. Span structural components shall not interfere with complete visibility through the trunnion center holes. Trunnions shall be individually adjustable for alignment.

A permanent walkway or ladder with work platform shall be installed to permit inspection of trunnion alignment.

15.5 Lock Systems

Center locks are to be accessible from the bridge sidewalk through a suitable hatch or access door. Under the deck, and in the region around the centerlocks, a work platform suitable for servicing of the lockbars shall be provided.

Lock systems shall be designed so that an individual lock may be disabled for maintenance or replacement without interfering with operation of other lockbars on the bascule leaf.

Tail locks, when required, shall be designed so that the lockbar mechanism is accessible for repair without raising the leaf. The lockbar drive mechanism shall be accessible from a permanently installed platform within the bridge structure.

Lockbar clearances shall be adjustable for wear compensation.

15.6 Machinery Drive Systems

All machinery drive assemblies shall be individually removable from the drive system without removal of other major components of the drive system. For example, a speed reducer assembly shall be removable by breaking flexible couplings at the power input and output ends of the speed reducer.

15.7 Lubrication Provisions

Bridge system components requiring lubrication shall be accessible without use of temporary ladders or platforms. Permanent walkways and stairwells shall be installed to permit free access to regions requiring lubrication.

Lubrication fittings shall be visible, clearly marked and easily reached by maintenance personnel.

If specified by the Department, automatic lubrication systems shall be provided for bearings and gears. Designs for automatic lubrication systems shall provide for storage of not less than three (3) months supply of lubricant without refilling. Refill shall be accomplished within a period of 15 minutes through a vandal-proof connection box located on the bridge sidewalk, clear of the roadway. Blockage of one traffic lane during this period is permitted.

If specified by the Department, self-lubricating bushings shall be incorporated in bridge designs.

15.8 Drive System Bushings

All bearing housings and bushings in open machinery drive and lock systems shall utilize split bearing housings and bushings and shall be individually removable and replaceable without affecting adjacent assemblies.

15.9 Local Switching

"Hand-Off-Automatic" switching capability shall be provided for maintenance operations on traffic gate controllers and brakes and motors for center and tail lock systems.

"On-Off" switching capability shall be provided for maintenance operations on span motor and machinery brakes, motor controller panels and span motors.

Remote switches shall be lockable for security against vandalism.

15.10 Service Accessibility

A service area not less than 30 inches wide shall be provided around system drive components.

15.11 Service Lighting

Machinery and electrical rooms shall be lighted as necessary, with a minimum lighting level of 20 foot-candles, to assure adequate lighting for maintenance of equipment. Switching shall be provided so that personnel may obtain adequate lighting without leaving the work area for switching. Master switching shall be provided from the control tower.

Each work area shall be provided with receptacles for supplementary lighting and power tools such as drills, soldering and welding equipment.

15.12 <u>Communications</u>

A. Intercommunications - Permanent communications equipment shall be provided between the control tower and areas requiring routine maintenance (machinery drive areas, power and control panels locations traffic gates and waterway).

B. Marine Communications - a marine radio/telephone transceiver shall be provided in a location convenient to the bridge tender. Location adjacent to the control console is preferred.

15.13 <u>Wiring Diagrams</u>

Wiring diagrams shall be provided for each electrical panel inside the panel door. Diagrams shall be enclosed in glass or plastic of optical quality.

15.14 <u>Diagnostic Reference Guide for Maintenance</u>

Diagnostic instrumentation and system fault displays shall be installed for mechanical and electrical systems. Malfunction information shall be presented on control system monitor located in the bridge control house. Data shall be automatically recorded. System descriptive information, such as ladder diagrams and wiring data, shall be available on the system memory to enable corrective actions on system malfunctions and to identify areas requiring preventative maintenance.

15.15 <u>Navigation Lights</u>

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

15.16 Working Conditions for Improved Maintainability

When specified by the Department, new and rehabilitated bascule bridge designs shall call for enclosed machinery and electrical equipment areas. Areas containing electronic equipment shall be air-conditioned to protect the equipment as required by the equipment manufacturer and the Structures Design Office.

15.17 Weatherproofing

New and rehabilitated bascule bridge designs should incorporate details to prevent water from draining into the machinery area. To provide this weatherproofing additional appendages such as gutters may be required. Since some water may reach these areas anyway, particular care should be given to avoid details that trap dirt and water. This may require adding drain holes, attaching partial enclosures, providing sloped floors, etc. Also, on particular jobs, the Department may require the designer to consider provisions to discourage birds and other unwanted guests from taking up residence.

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Chapter 16

BRIDGE WIDENINGS

16.1 Analysis and Design

A. General

1. Appearance

The widening of a structure should be accomplished in a manner such that the existing structure does not look "added onto". When this is not possible, consideration should be given to enclosure walls, cover panels, paint, or other aesthetic treatments. Where possible and appropriate, the structure appearance should be improved by the widening.

2. Materials

Materials used in the construction of the widening shall preferably have the same thermal and elastic properties as the materials used in the construction of the existing structure. Except when prohibited by the specifications or when specifically prohibited by the designer (by notes on the plans and/or in the Special Provisions), widenings utilizing prestressed beams (Figures 16-2 and 16-3) or steel beams shall be designed and detailed for the use of stay-in-place metal forms in accordance with Chapter 10 and Chapter 11, respectively.

3. Load Distribution

The main load carrying members of the widening should be proportioned to provide similar longitudinal and transverse load distribution characteristics as the existing structure. Normally, this can be achieved by using the same cross-sections and member lengths as were used in the existing structure. The construction sequence and degree of interaction between the widening and the existing structure after completion, shall be fully considered in determining the distribution of the dead load for design of the widening and stress checks for the existing structure. The distribution of live load shall be in accordance with AASHTO criteria. Where precast-prestressed beams are used to widen an existing cast-in-place concrete box girder or T-Beam bridge, the factor for live load distribution for interior beam(s) of the widening shall be S/5.5.

4. Specifications

The design of the widening shall conform to the current AASHTO Specifications, Florida DOT Standard Specifications for Road and Bridge Construction, these guidelines, and any applicable Structures Design Office criteria. In general the method of design for the widening shall be the same as the original design. That is, if the original design was performed by the Service Load method, then the design of the widening shall be performed by the Service Load method. The same applies to the Load Factor method.

5. Overlay

It should be established at an early stage of widening design if an asphalt overlay will be required. The elimination of asphalt overlays on bridge decks is preferred.

6. Stresses in the Existing Structure

A review of stresses in the main members of the existing structure shall be made for construction conditions and the final condition; that is, after attachment of the widened portion. In some cases when computations indicate large overstresses in members of the existing structure (primarily in columns) designed by Service Load methods, a review by Load Factor Design methods should be made. Using this comparison and good engineering judgment, a decision can be reached by the Engineer of Record and the FDOT review office as to the amount of overstress that can be tolerated.

7. Special Considerations

All widening shall be designed for HS-20 loading, regardless of the loading used in the original design. Where large differential cambers are encountered, joints or other accommodations should be considered between the existing structure and the widening. Longitudinal joints in the riding surface are a safety hazard and not preferred. In general, if alternate, efficient methods are available, falsework should not be supported from the existing structure without special approval. The reason for this is to avoid the adverse effects of transmitting live load vibrations from the existing structure remaining in service and under traffic to the new widening construction. When a superstructure slab is widened without additional beams or other support, the resulting cantilever should be carefully checked for increased stresses. Precast slab or double tee members may be used to widen existing cast-in-place, flat slab structures. This method is especially useful when the horizontal or vertical clearances during construction are insufficient to use cast-in-place members. The diaphragms for widenings shall be in accordance with Section 10.3. When using battered piles, estimate their tip elevations and insure that they will have ample clearance from all other existing piles, utilities, or other similar obstructions.

B. Substructure

1. Selection of Foundation

The type of foundation to be used to support the widening should generally be the same as that of the existing structure unless otherwise recommended by the District Geotechnical

Engineer. The effects of possible differential settlement between the new and the existing foundations shall be considered.

Existing bridge site conditions should be considered in determining the new foundation locations. The conditions include scour potential, overhead clearance for pile driving equipment, horizontal clearance requirements, working room, pile batters, channel changes, utility locations, existing embankments, and other similar conditions.

2. Scour and Drift

Added piles, columns, etc. for widenings at water crossings may alter stream flow characteristics. This may result in pier scouring to a greater depth than experienced with the existing piers. Added substructure may also increase the possibility of trapping drift. The District Drainage Engineer should be consulted concerning potential problems related to scour and drift.

16.2 Attachment of Widening to Existing Structure

A. General

1. Drilling into Existing Structure

When drilling into heavily reinforced areas, exposure of the main reinforcing bars by chipping should be specified. Drilled holes should have a minimum clearance of 3" from the edge of the concrete, and 1" clearance from the existing reinforcing bars in the structure.

When holes larger than 1-1/2 inch, or deeper than 12" are required, core drilling shall be specified. If it is necessary to drill through reinforcing bars, core drilling should be specified.

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

   a. The allowable stresses in the reinforcing steel shall be reduced by the ratio of the actual embedment divided by the required embedment.

   b. Adhesive anchors designed in accordance with Chapter 7 of these Guidelines may be used.

3. Preparation of existing surfaces for concreting shall be in accordance with "Removal of Existing Structures" in the Standard Specifications for Road and Bridge Construction.

B. Connection Details

The figures at the end of this chapter are samples of details which have been used for widening of bridges. They are for general information and are not intended to restrict the designer in his judgment.

1. Flat Slab Bridges

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

2. T-Beam Bridges

   The connection detail shown in Figure 16-2 for the slab connection is recommended. Limits of slab removal is at the discretion of the design engineer and subject to the Department's approval.

3. Prestress Concrete Girder Bridges

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

16.3 Expansion Joints

See Chapter 12.

16.4 Construction Sequence

A construction sequence detail shall be shown on the preliminary bridge plans submittal for all projects utilizing phase construction. In addition to this detail, the final plans shall include a complete outline of the order of construction.

16.5 Bridge Widening Rules

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

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

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

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

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

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

1. Bridge location, pier location, skew angle, stationing
2. Span lengths
3. Number and type of beams
4. Wingwall, pier, and abutment details
5. Utilities (if any) supported on the bridge
6. Finished grade elevations
7. Vertical and horizontal clearances
8. Other features critical to the widening

F. Any discrepancies which, in the designer's opinion, are critical to the continuation of the widening design should be brought to the FDOT Project Manager's attention at the earliest possible time. Either a solution should be agreed to or the District should replace the structure rather than continue the widening.

G. The widened portion shall either match the existing structure as explained in other sections of this Chapter or be designed as if it were a new structure and therefore comply with the most recent requirements. The decision shall be made as follows:

1. If the widening requires one additional beam line or less (or 12 feet or less for a slab), then the widened portion shall match the existing.

2. If the widening requires two or more additional beam lines (more than 12 feet for a slab), then the widened portion shall be designed following the guidelines for a new structure.

3. Voided slab bridges require special attention. If widening is being proposed, the SDO should be contacted.

16.6 Bridge Widening Definitions

Bridge structures to be widened shall be classified as either major or minor widenings according to the following definitions:

- Major Widening - New construction work to an existing bridge structure such that either the resulting number of traffic lanes or the bridge deck area will be at least twice that of the unwidened bridge.
- Minor Widening - New construction work to an existing bridge structure resulting in an extent of widening less than described above.

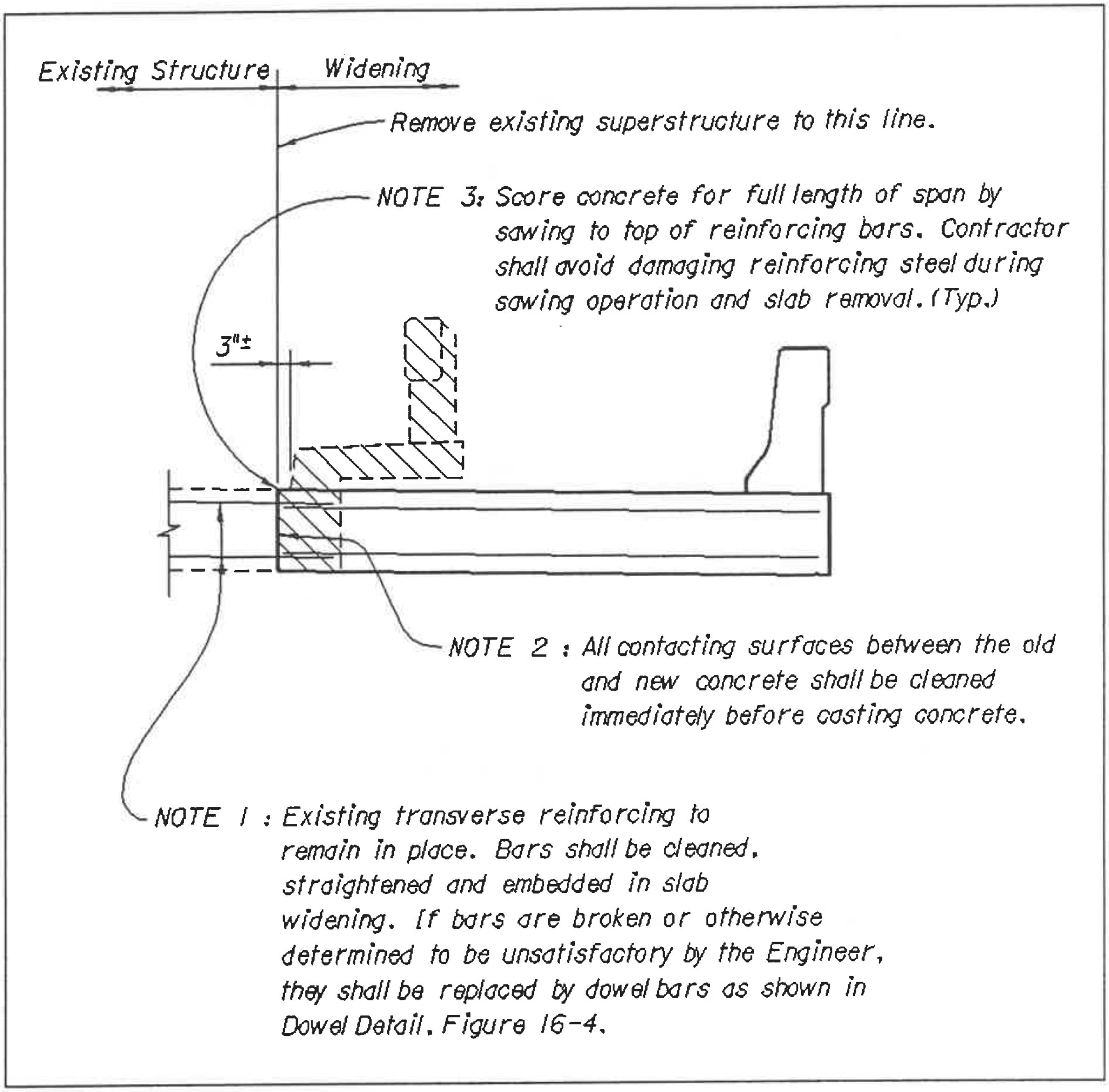

Existing Structure
Widening
Remove existing superstructure to this line.
NOTE 3: Score concrete for full length of span by sawing to top of reinforcing bars. Contractor shall avoid damaging reinforcing steel during sawing operation and slab removal. (Typ.)
3"±
NOTE 2 : All contacting surfaces between the old and new concrete shall be cleaned immediately before casting concrete.
NOTE 1 : Existing transverse reinforcing to remain in place. Bars shall be cleaned, straightened and embedded in slab widening. If bars are broken or otherwise determined to be unsatisfactory by the Engineer, they shall be replaced by dowel bars as shown in Dowel Detail, Figure 16-4.

Figure 16-1

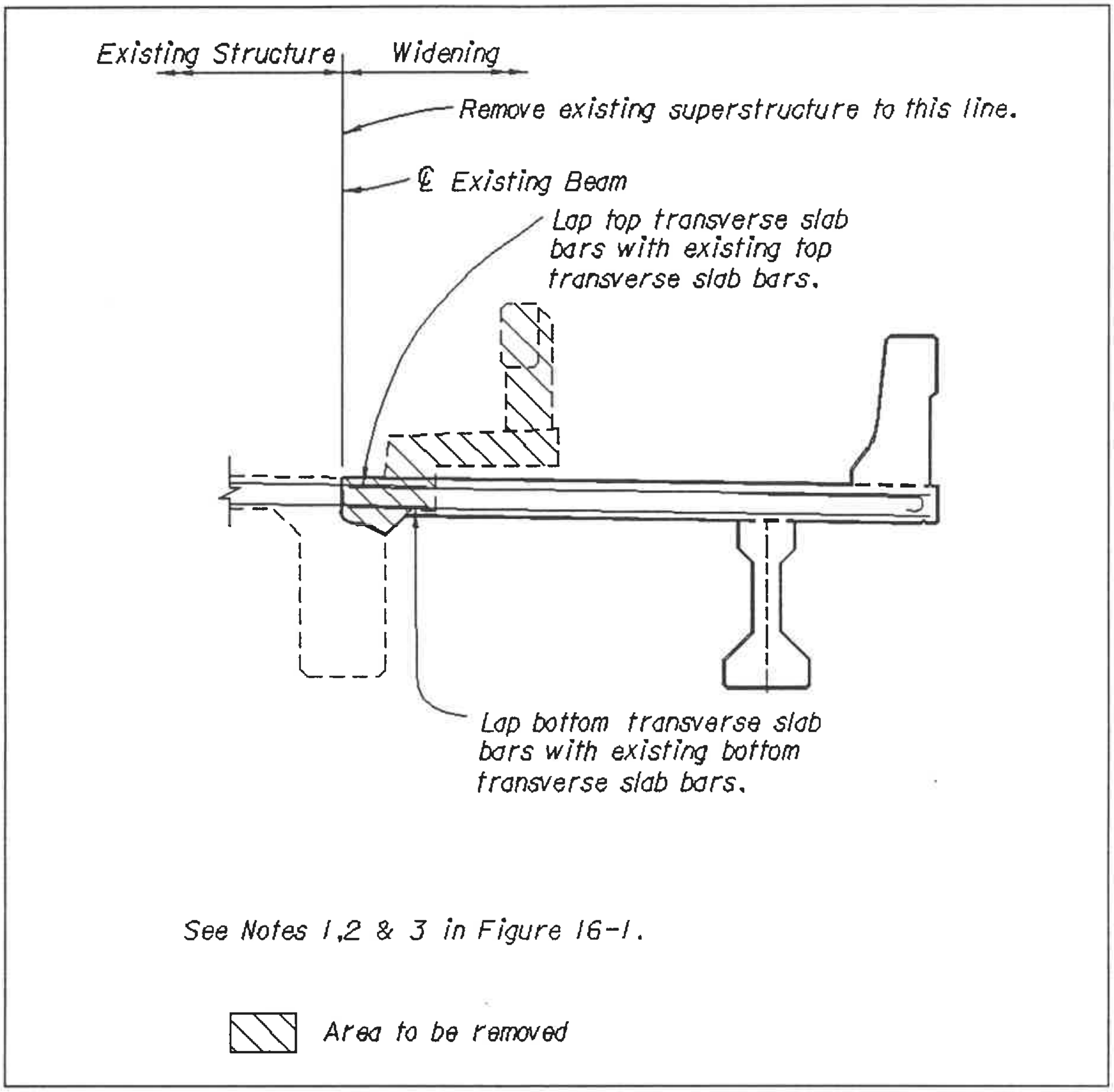

Existing Structure
Widening
Remove existing superstructure to this line.
℄ Existing Beam
Lap top transverse slab bars with existing top transverse slab bars.
Lap bottom transverse slab bars with existing bottom transverse slab bars.
See Notes 1,2 & 3 in Figure 16-1.
Area to be removed

Figure 16-2

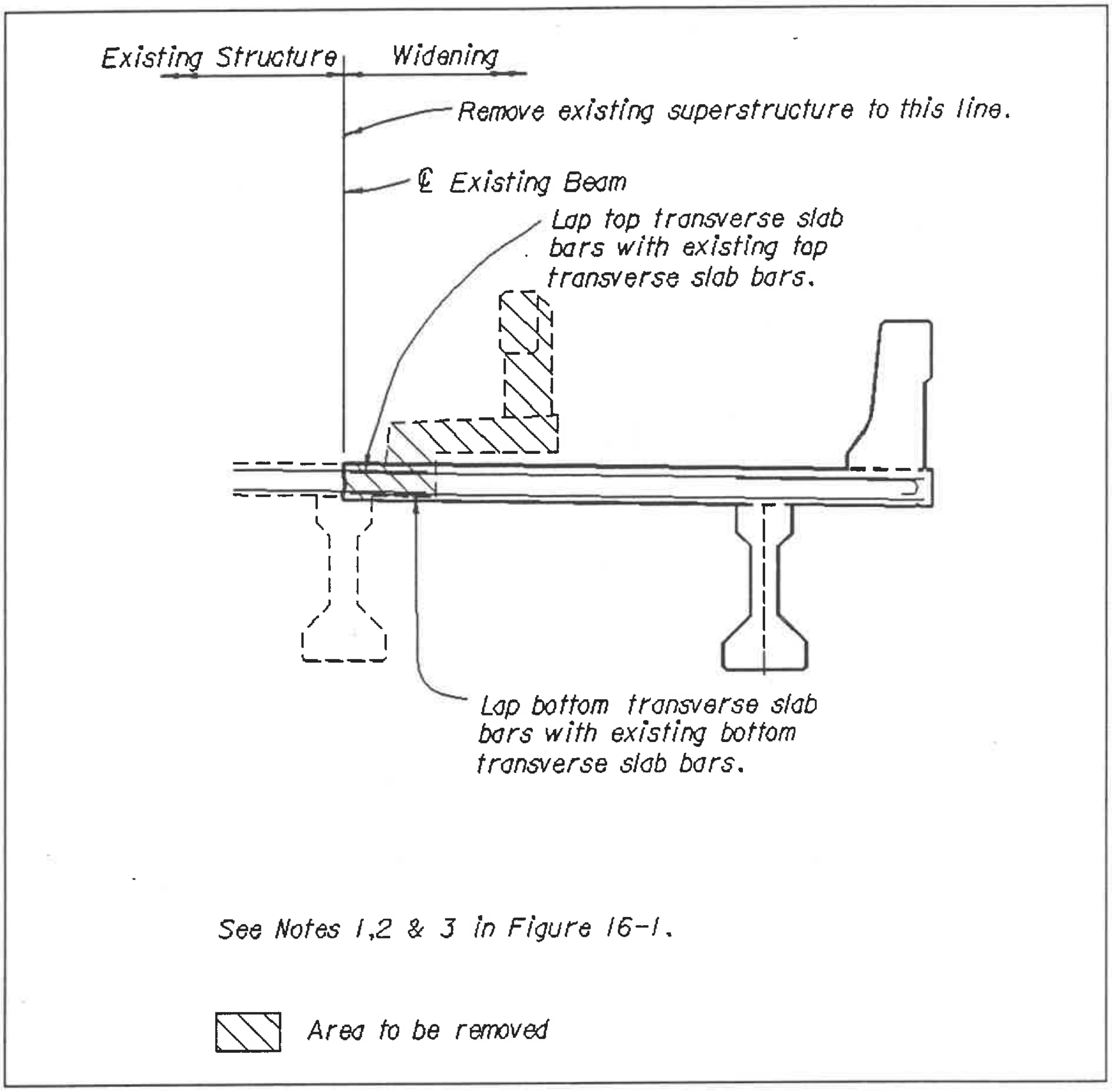
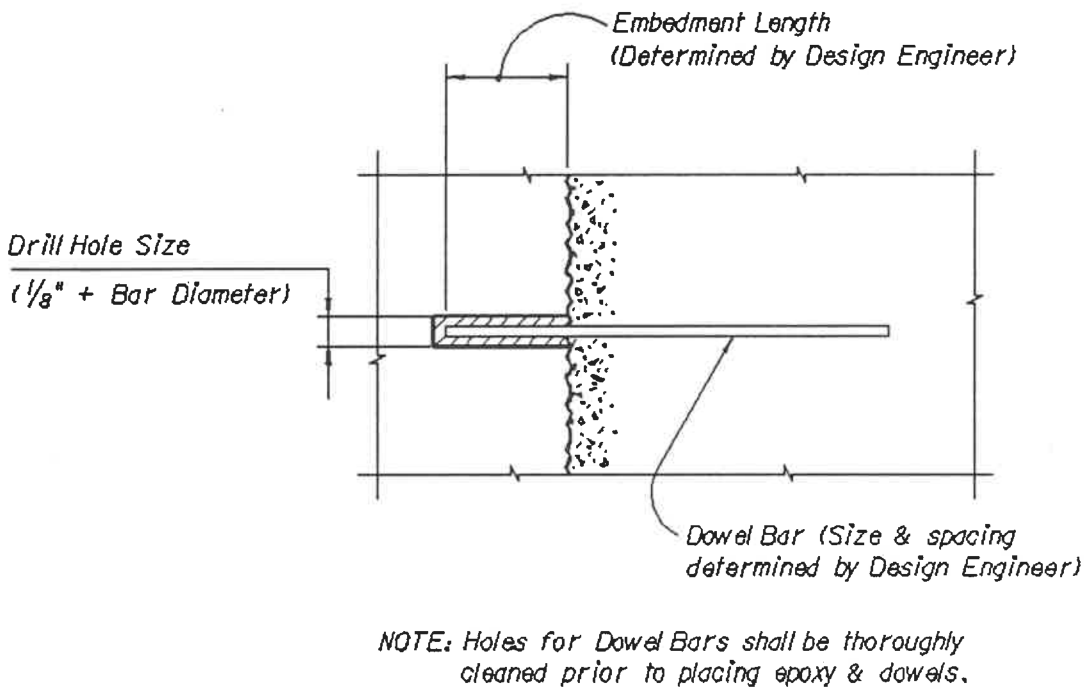

Existing Structure
Widening
Remove existing superstructure to this line.
℄ Existing Beam
Lap top transverse slab bars with existing top transverse slab bars.
Lap bottom transverse slab bars with existing bottom transverse slab bars.
See Notes 1,2 & 3 in Figure 16-1.
Area to be removed

Figure 16-3



Note to Designers: Either specify the class of adhesive anchor in accordance with Chapter 7 if this detail is generic to a project, or specify the appropriate class at each different detail location in the project.

Figure 16-4

Approved: Jerry Potter

Effective: November 2, 1992
Sequence No.: 0160
Responsible Office: Structures Design
Topic No.: 625-020-117-h

Chapter 17

COMPUTER PROGRAMS FOR STRUCTURAL DESIGN AND ANALYSIS

17.1 General Information

The Department is in the process of migrating from the mainframe and the vax system as the platforms for structural design and analysis programs. Our future platforms will be microcomputers and unix work stations. Some of the programs presently available on the mainframe will be ported to these systems and should be available over a network.

A number of the programs listed in this chapter have been listed with an asterisk (*) preceding their name, thus indicating programs fully supported by the Structures Design Office or other Central Offices which are the preferred program for use in each particular area of expertise.

17.2 Available Programs on the IBM Mainframe

Following is a list of the computer programs available through the Florida Department of Transportation mainframe. These programs with their alias in parentheses are classified in the following categories:

A. Integrated Civil Engineering System (ICES)

1. *COGO: Coordinate GeOmetry System, solution of geometric problems. COGO Version 4.4 was developed by MIT/McDonnell Douglas.
2. STRUDL: STRUctural Design Language, general structural design and analysis. STRUDL PRO and interactive graphics are also included in this package. STRUDL Version 5.4 was developed by MIT/McDonnell Douglas.
3. *CEAL COGO: Coordinate geometry system; solution of geometric problems. CEAL was developed by MIT/CLM Systems.
4. GTSTRUDL: Georgia Institutes of Technology STRUctural Design Language, general structural design and analysis program. GTSTRUDL Version 87021 IBM/MVS/XAVF was developed by MIT/GIT.

B. Related Highway Structures Programs

1. Roadway Ground Signs (PSTSGNO1): Design of ground mounted sign structures with two or three columns. PSTSGNO1 was developed by the Florida DOT.

2. Steel Truss (PSTSGNO2): Design of overhead steel signing truss. PSTSGNO2 was developed by the Florida DOT.

3. Aluminum Truss (PSTSGNO3): Design of overhead aluminum signing truss. PSTSGNO3 was developed by the Florida DOT.

4. Aluminum Cantilever (PSTSGNO5): Design of 4 post aluminum cantilever overhead signing truss. PSTSGNO5 was developed by the Florida DOT.

5. Multi-Post Ground Sign-Steel (PSTSGNO6): Design of multi-post ground signs in conjunction with Index Drawing 9535. STSGNO6 was developed by the Florida DOT.

6. Aluminum Butterfly (PSTDSN32): Design of 4 post aluminum butterfly overhead signing truss. PSTSGN32 was developed by the Florida DOT.

7. *Wyoming Pole Design (PSTDSN43): Design or analysis of cantilevered high mast lighting pole and base plate. PSTDSN43 was developed by the Wyoming DOT. Version 2.0.

C. Structures Elevation and Geometry

1. *Bearing Pad Elevation (PSTGMY54): Tabulation of the four corner elevations of bearing areas on substructures.

2. *Skewed Bridge (PSTGMYO7): Three dimensional bridge geometry. PSTGMYO7 was updated from PSTGMYO1 to create CADD file for final plans. PSTGMYO1 was developed by the Georgia DOT.

D. Geotechnical Pile and Foundation Programs

1. BSHAFT (PSTDSN45): Predicts the ultimate capacity of drilled shafts in clay and in sand. Ultimate bearing capacity is obtained from a direct correlation with blowcount data.

2. *COM624 (PSTDSN44): Soil/structure interaction program. Provides deflection, moment and shear values for laterally loaded single piles. Allows for a variety of boundary conditions.

3. *WEAP86 (PSTANA61): Wave Equation Analysis Program. Predicts ultimate capacities as well as stresses for driving piles. Models a wide variety of driving systems.

4. RB-121 (PSTDSN10): Based on Research Bulletin RB-121 originally developed in 1967 by University of Florida and Florida DOT. Predicts ultimate and allowable static concrete pile capacity.

5. RB-121/Plot (PSTDSN10): Plots the pile capacity curve of RB-121 results.

6. Reinforced Concrete Cantilever Retaining Wall/Plo (PSTDSN61): Performs a detailed design and a stability analysis of cantilever concrete retaining walls. Plots of the results are available. PSTDSN61 was developed by the University of Iowa.

7. PILGP2 (PSTDSN62): 3-D pile group program to predict axial load vs. settlement and lateral load vs. deflection.

8. *Prediction of Pile Group Performance (PSTDSN67): Predicts vertical pile load - settlement responsible pile groups of any single pile in a group.

9. *LPGSTAN (LPGSTAN): Laterally loaded Pile Group and STructural ANalysis was developed by the University of Florida to analyze a bridge pier composed of columns and column cap supported on a pile cap and piles with a specifically developed nonlinear Finite Element program.

E. Steel Girder Programs

1. *Plate Girder (PSTDSN66): Designs steel plate girder and box girder bridges. SIMON Version 60 was developed by University of Wisconsin/U.S. Steel.

2. VLOAD (PSTANA57): Analysis of horizontally curved steel plate girders. PSTANA57 was developed by U.S. Steel.

3. California Curved Bridge (PSTANA59): Analysis of horizontally curved steel plate girders. PSTANA59 was developed by the California DOT.

4. Mississippi Curved Bridge (PSTANA60): Analysis of horizontally curved steel box girders. PSTANA60 was developed by the Mississippi DOT.

5. *DASH (PSTDSN60): Analysis and design of continuous steel plate girder bridges with working stress or load factor design. PSTDSN60 was developed by the University of Maryland.

F. Structures Concrete and General Beam Programs

1. Continuous Prestressed Girder Analysis (PSTANA12): Analysis of post-tensioned continuous prestressed concrete girders with a maximum of five spans. The cables must be a continuous series of second degree parabolas. PSTANA12 was developed by the California DOT.

2. *Continuous Beam (PSTANA03): Analysis of a continuous beam or slab for a highway bridge with two to eight spans. PSTANA03 was developed by the Georgia DOT.

3. *Continuous Beam (PSTANA3N): Same as PSTANA03, except the input of moment of inertia may be greater than or equal to 1,000,000 and less than or equal to 9,999,999 in^4.

4. *Continuous Beam (PSTANA58): Same as PSTANA03, except the output of moment values are load factored.

5. NEB Double-Tee (PSTDSN50): Design or analysis of simple span concrete double-tees using stress-relieved strands. PSTDSN50 was developed by the Nebraska Department of Roads.

6. NEB Double-Tee (PSTDSN53): Design or analysis of simple span concrete double-tees using ASTM A416, Grade 270, low-

relaxation, seven-wire strands. PSTDSN53 was developed by the Nebraska Department of Roads.

7. *Prestressed Concrete Beam (PSTDSN65): Design and analysis of prestressed concrete beams with either a depressed or a straight strand pattern in accordance with Chapter 10 of this manual. PSTDSN65 Version 2.3.1 was developed by the Florida DOT.

8. *Prestressed Double-Tee/Solid Slab (PSTANA70): Analysis of simple span prestressed double-tee or solid slabs. PSTANA70 was developed by the Florida DOT.

G. Structures Pier and Column Programs

1. *Mississippi Column (PSTDSN47): Analysis of conventionally reinforced or prestressed concrete columns. Design of conventionally reinforced concrete columns. Calculates and plots the points of the axial load versus moment interaction diagrams for all column cross sections analyzed by this program. PSTDSN47 was developed by Mississippi State Highway Department.

2. PCA R/C Column (PSTDSN31): Analysis and design of reinforced concrete columns based on ultimate strength theories. PSTDSN31 was developed by the Portland Cement Association.

3. *Multiple Column Piers (PSTANA05) Version 4.2: Design and analysis of bridge piers and foundations. PSTANA05 was developed by the Georgia DOT.

H. Miscellaneous Programs

1. OverLoad 1-20 Axles (PSTANA01): Computes the maximum simple span moment for a moving live load consisting of from 1 to 20 axles. PSTANA01 was developed by the Florida DOT.

2. *Reinforcing Steel (PSTQTY02): Tabulates the reinforcing steel sizes, marks, numbers, types, lengths, and costs based on Florida Standard Bar Bending Details. Reproducible plots

of the results are available for inclusion in the contract documents. PSTQTY02 was updated from PSTQTY01 to create a CADD file for final plans. PSTQTY02 was developed by the Florida DOT.

3. *Reinforced Concrete Box Culvert and Wingwall (PSTDSN55): Designs box culverts and wingwalls, and tabulates concrete and reinforcing steel quantities. PSTDSN55 Version 2.0 was codeveloped by the North Carolina and Florida DOT's.
4. Structure Inventory and Appraisal (MTINV17A): Bridge inventory and maintenance record database.
5. BFAST (PSTANA63): Analysis of soil and design and analysis of soil and bridge foundations. PSTANA63 Version 1.6 was developed by the University of Texas.
6. FRAME51 (PSTANA65): Nonlinear analysis of plane frame structures. Laterally loaded piles with non-linear supports (P-Y curves) may be included as in PSTANA64. PSTANA65 Version 2.0 was developed by the University of Texas.
7. PIER3 (PSTANA66): Analysis of reinforced concrete single-pier-bent, beam-column subject to static loads. PSTANA66 was developed by the University of Texas.
8. BMCOL76 (PSTANA64): Analysis of single beam-column members supported by non-linear lateral or axial supports (P-Y curves). PSTANA64 Version 2.0 was developed by the University of Texas.
9. UTEXAS (PSTANA67): Performs stability evaluation for earth slopes. PSTANA67 was developed by the University of Texas.

I. *Structures BC Program: Bridge Construction program. Analysis of post-tensioned, segmental, time-dependent concrete members. Developed by Europe Etudes.

J. Bridge Load Rating and Analysis Programs

1. *SALOD (SALOD): Structural Analysis for LOad Distribution is a hybrid finite element program which creates an influence surface of the simple span bridge superstructure. The influence surface is loaded with standard trucks or any specific truck for determination of live load distribution of design components (beam and slab). SALOD was developed by the University of Florida for the Florida DOT.
2. *BRUFEM (BRUFEM): Bridge Rating Using Finite Element Method program creates a unique bridge model for analysis of bridge superstructures. Bridge superstructure systems which can be modeled with the preprocessor are listed below; simple or continuous span, constant skew, reinforced concrete tee beams, prestressed concrete girders, composite or non-composite steel girders. The post-processor will determine either live load distribution factors or bridge ratings. The post-processor can be switched to general bridge analysis. BRUFEM was developed by the University of Florida for the Florida DOT.

17.3 Available Programs on Minicomputer (VAX 780, VAX 250) (VAX System will be Discontinued in 1993)

A. Integrated Civil Engineering System

1. *GEOPAK: Coordinate geometry system; solution of geometric problems; translation of data to IGDS.Geometric problems; translation of data to IGDS. Developed by Beiswenger, Hoch & Associates
2. CEAL COGO: Coordinate geometry system; solution of geometric problems. CEAL was developed by MIT/CLM Systems.
3. *CTOGEO: Translates CEAL data files into GEOPAK format. CTOGEO was developed by MIT/CLM Systems.

B. Structures Elevation and Geometry

*Skewed Bridge Program Graphics Output: Creates IGDS file from output of Georgia Skewed Bridge Program (PSTGMY01 on mainframe). Was developed by the Florida DOT.

C. Geotechnical Pile and Foundation Programs

Prediction of Pile Group Performance (PILEGRP): Predicts pile load - settlement response for pile groups. Settlement of any pile in a group subjected to vertical load can be predicted. (Same as 17.2.D.8)

D. Structures Concrete and General Beam Programs

1. *Prestressed Concrete Beam: Design and analysis of prestressed concrete beams, which includes depressed and straight strand design and analysis in accordance with Chapter 10, Article 10.6. Version 2.3.1 was developed by the Florida DOT. (Same as 17.2.F.7)

E. Miscellaneous

1. *Reinforced Concrete Box Culvert and Wingwall Version 2.0: Design box culvert and wingwall, tabulate concrete and reinforcing steel quantities. Codeveloped by North Carolina and Florida Departments of Transportation. (Same as 17.2.H.3)
2. *Reinforcing Steel Program Graphical Output: Creates an IGDS file from the output of the Rebar program (PSTQTY01 on mainframe). Developed by the Florida DOT. (See 17.2.H.2)

17.4 Available Programs on Interpro with UNIX Operating System

A. Geotechnical Pile and Foundation Programs

*Prediction of Pile Group Performance (PILEGRP): Predicts pile load - settlement response for pile groups. Settlement of any pile in a group subjected to vertical load can be predicted. (Same as 17.2.D.8)

17.5 Available Programs on Microcomputers

A. Integrated Civil Engineering System

CEAL COGO: Coordinate geometry system for solution of geometric problems. CEAL was developed by MIT/CLM Systems.

B. Related Highway Structures Programs

1. *MPSIGN (PSTSGNO6): Designs the steel posts required for multi-post ground signs. The updated version includes U-channels in the internal section library. MPSIGN Version 2.3.1 was developed by the Florida DOT. (Updated version of 17.2.B.5)

2. *STRAIN POLE (PSTSGNO7): Designs strain poles for supporting traffic signals also calculates cable loads and pole forces and required pole embedment. STRAIN Version 1.4 was developed by the Florida DOT.

3. *CANT (PSTSGNO8): Designs and analyses steel cantilever overhead design structures and their foundations. CANT Version 1.10 was developed by the Florida DOT and the University of Florida.

4. *SPAN (PSTSGNO9): Designs and analyses steel span overhead sign structures and their foundations. SPAN Version 1.10 was developed by the Florida DOT and the University of Florida.

C. Structures Elevation and Geometry

TS$SURVE: Controls total station surveying instruments and collects data. Runs on hand-held or regular PC. Produces fieldbook-style reports. Includes statistical analysis program to produce a 3D model of observations. Converts model into IGDS file.

D. Geotechnical Pile and Foundation Programs

1. Prediction of Pile Group Performance (PILEGRP): Predicts pile load - settlement response for pile groups. Settlement of any pile in a group subjected to vertical load can be predicted. (Same as 17.2.D.8)

2. *Static Pile Bearing Analysis Program (SPT91): Estimates Statics axial capacity of driven displacement piles. Methodology is based on interpretation of Standard Penetration Test results developed in Research Bulletin RB-121.

E. Structures Concrete and General Beam Programs

1. *Prestressed Concrete Beam: Design and analysis of prestressed concrete beams, which includes depressed and straight strand design and analysis in accordance with Chapter 10, Article 10.6. Version 2.3.1 was developed by the Florida DOT. (Same as 17.2.F.7)

2. *Prestressed Double-Tee/Solid Slab: Analysis of simple span prestressed double-tee/solid slab. This program was developed by the Florida DOT. (Same as 17.2.F.8)

3. *Reinforced Concrete Box Culvert and Wingwall (PSTDSN55): Designs box culverts and wingwalls, and tabulates concrete and reinforcing steel quantities. PSTDSN55 Version 2.0 was codeveloped by North Carolina and Florida Departments of Transportation (updated version and manual will be available later).

F. Miscellaneous

1. Section Properties (PSTANA98): Calculates area, coordinate of centroid, moments of inertia, static moments for arbitrary plane domain.

17.6 <u>Sample Input Data of Mainframe Computer Programs</u>

A database has been established with sample input data files for mainframe structures computer programs. These files should help users become familiar with the input requirements and the output features of our mainframe programs.

The name of the database is STDESIGN.DATA. The name of each sample input in this partitioned dataset is the alias of each program without the first three letters; PST. For example, the sample input data for Roadway Ground Signs

(PSTSGN01) is SGN01. If there is more than one sample for any program a two digit extension beginning with "01" will be attached to the member name. For example, the four sample input datasets for the Prestressed Concrete Beam (PSTDSN65) program are DSN6501, DSN6502, DSN6503, and DSN6504.

17.7 Use of Programs

These programs are provided as an aid to the designer in performing structural design and/or analysis services for the Department; however, designers are under no obligation to use any of these programs. The Structures Design Office will assist the designer in obtaining the necessary reference material, technical assistance and computer access required to utilize any of these programs.

The Department makes no warranty or representation, either expressed or implied, with respect to these programs, their documentation, their quality, performance, or suitability for a particular purpose. The Engineer of Record assumes the entire risk as to the quality and performance of these programs for his particular application. The Department would appreciate being informed of any problems, apparent errors or suggested improvements in any of these programs or their execution so that they can be updated and benefit all users.

17.8 Requesting Documentation and Microcomputer Programs

Manuals of mainframe and microcomputer programs as well as microcomputer software developed by Florida Department of Transportation may be requested by filling out the Documentation and Software Request Form included at the end of this chapter.

Documentation and software availability and costs of duplication are listed in Tables 17-1 and 17-2, respectively. There is no charge to Department engineers.

17.9 Requesting the CADD Manual and Information Tape

The Structures CADD Manual and Standard DOT Structures CADD Set-Up tape or diskettes may be requested by filling out the Documentation and Software Request Form included at the end of this chapter.

Available documentation and their cost are listed in Table 17-3.

17.10 Dial-Up Service

The Department currently provides dial up service for microcomputer users to dial into the Florida Department of Transportation's Network (DOTNET). When the user logs on to DOT he will have access to the mainframe computer programs listed under 17.2.

The hardware and software requirements for dial up service and technical support information are described in the Florida Department of Transportation's Hints and Tips for Dial-In Users available from the Office of Information Systems (OIS) in Tallahassee.

The Dial-Up technical support telephone number is (904)488-5013 and the telecommunication numbers for a microcomputer user connecting his computer to DOTNET through modem are listed below:

Jacksonville	(904)695-4148
Miami	(305)470-5607
Tallahassee	(904)487-4769
Tampa	(813)871-7290

Users outside these areas may dial long distance to one of these numbers and be billed by the respective telephone company for the long distance telecommunication service.

Table 17-1
Documentation Cost

Program Name	Description	No. of Pages	Cost/ Notes
BC-D	Bridge Construction	242	$49.00
CEAL-D	Request from MIT/CLM System or review in State/District Structures Design Engineer's office.		(1)
COGO-D	Request from McDonnell Douglas or review in State/District Structures Design Engineer's office.	282	(2)
STRUDL-D	Request from McDonnell Douglas or review in State/District Structures Design Engineer's office.	1370	(2)
GTSTRUDL-D	Request from Georgia Institute of Technology or review in State/District Structures Design Engineer's Office		(6)
PSTANA01-D	Over Load 1-20 Axles	1	$1.00
PSTANA03-D	Continuous Beam	154	$31.00
PSTANA05-D	Multiple Column Piers	135	$27.00
PSTANA3N-D	Continuous Beam	154	$31.00
PSTANA12-D	Concrete Prestressed Concrete Girder Analysis	110	$22.00
PSTANA57-D	VLOAD	68	$14.00
PSTANA58-D	Continuous Beam	154	$31.00
PSTANA59-D	California Curved Bridge	214	$43.00
PSTANA60-D	Mississippi Curved Bridge	176	$35.00
PSTANA61-D	WEAP86, request from FHWA		(3)
PSTANA63-D	BFAST, request from the Univ. of Texas		(4)
PSTANA64-D	BMCOL76, request from the U. of Texas		(4)
PSTANA65-D	FRAME51, request from the U. of Texas		(4)
PSTANA66-D	PIER3, request from the U. of Texas		(4)
PSTANA67-D	UTEXAS, request from the U. of Texas		(4)
PSTANA70-D	Prestressed Conc. Double-Tee/Solid Slab	80	$16.00

Table 17-1 (Continued)
Documentation Cost

Program Name	Description	No. of Pages	Cost/ Notes
PSTDSN10-D	RB-121,request from Office of Materials & Research, Florida DOT	150	$30.00
PSTDSN31-D	PCA R/C Column	28	$6.00
PSTDSN32-D	Aluminum Butterfly Overhead Sign Truss	5	$1.00
PSTDSN43-D	Wyoming Pole Design	102	$20.00
PSTDSN44-D	COM624, request from McTrans Center		(7)
PSTDSN45-D	BSHAFT, request from the Univ. of Texas		(4)
PSTDSN47-D	Mississippi Column	63	$13.00
PSTDSN50-D	NEB Double-Tee stress-relieved strands	57	$11.00
PSTDSN53-D	NEB Double-Tee low-relaxation strands	104	$21.00
PSTGMY54-D	Bearing Pad Elevation	2	$1.00
PSTDSN55-D	Reinf. Conc. Box Culvert and Wingwall	80	$16.00
PSTDSN60-D	DASH	42	$8.00
PSTDSN61-D	Reinforced Concrete Retaining Wall	70	$14.00
PSTDSN62-D	PILGP2, request from FHWA		(3)
PSTDSN65-D	Prestressed Concrete Beam	159	$32.00
PSTDSN66-D	SIMON	159	$32.00
PSTDSN67-D	Pile Group Performance, request from Univ. of Florida		(5)
PSTDSN69-D	Static Pile Bearing Analysis	47	$9.00
PSTGMY01-D	Skewed Bridge	186	$37.00
PSTQTY01-D	Reinforcing Steel Quantity	20	$4.00
PSTSGN01-D	Roadway Ground Signs		$1.00
PSTSGN02-D	Overhead Steel Signing Truss	1	$1.00
PSTSGN03-D	Overhead Aluminum Signing Truss	1	$1.00

Table 17-1 (Continued)
Documentation Cost

Program Name	Description	No. of Pages	Cost/ Notes
PSTSGN05-D	Aluminum Cantilever Overhead Sign Truss	6	$1.00
PSTSGN06-D	MPSIGN, Steel Multi-Post Ground Signs	10	$2.00
PSTSGN07-D	STRAIN, Strain Pole	18	$4.00
PSTSGN08-D	CANT, Steel Cantilever Ovrhd. Sign Truss	21	$4.00
PSTSGN09-D	SPAN, Steel Span Ovrhd. Sign Structure	19	$4.00
MTINV17A-D	Structure Inventory and Appraisal, request from Maintenance Office, Florida DOT	150	$30.00
SALOD-D	Structural Analysis for Load Distribution	79	$16.00
BRUFEM-D	Bridge Rating Using Finite Element Method	179	$36.00
LPGSTAN-D	Laterally Loaded Pile Group and Structural Analysis	29	$ 6.00

Notes:

(1) CLM/Systems, Inc.
4023 S. Dale Mabry
Tampa, Florida 33611
(813) 831-7090
Attn. Dr. C. L. Miller, P.E.

(2) McDonnell Douglas
Box 516
Saint Louis, Missouri 63116
(800) 634-4990

(3) National Tech. Information Service
Springfield, Virginia 22161

(4) Center for Transportation Research
Bureau of Engineering Research
The University of Texas at Austin
Austin, Texas 78712-1075

(5) University of Florida
Dept. of Civil Engineering
345 Weil Hall
Gainesville, Florida 32611
Att: Dr. M. C. McVay

(6) Georgia Inst. of Technology
GTICES Systems Laboratory
Atlanta, Georgia 30332-0355

(7) McTrans Center
University of Florida
512 Weil Hall
Gainesville, Florida 32611

Table 17-2 Microcomputer Software Cost

Program Name	Description	Cost
PSTANA70-S	Prestressed Double-Tee / Solid Slab (1.0)	$25.00
PSTANA98-S	Section Properties (1.0)	$25.00
PSTDSN65-S	Prestressed Concrete Beam (2.3.1)	$25.00
PSTSGN06-S	MPSIGN, Steel Multi-Post Ground Signs (1.2)	$25.00
PSTSGN07-S	STRAIN, Strain Pole (1.65)	$25.00
PSTSGN08-S	CANT, Steel Cantilever Ovrhd. Sign Truss (1.12)	$25.00
PSTSGN09-S	SPAN, Steel Span Ovrhd. Sign Structure (1.30)	$25.00
PSTDSN55-S	Reinforced Concrete Box Culvert & Wingwall Design Computer Program (2.3)	$25.00
PSTDSN69-S	Static Pile Bearing Analysis (1.1)	$25.00

Table 17-3
CADD Documentation and Information Cost

Description	Cost
Structures CADD Manual	$25.00
Standard DOT Structures CADD Set Up on 9 track 1600 bpi tape * (Not Available After 1/1/93)	$50.00
Standard DOT Structures CADD Set Up on 5 1/4 in. or 3 1/2 in. High Density Double Side diskette *	$50.00
Standard DOT Structures Standard Drawings on 9 track 1600 bpi tape * (Not Available After 1/1/93)	$50.00
Standard DOT Structures Standard Drawings on 5 1/4 in. High Density Double Side diskette *	$50.00

* If tapes or diskettes are not provided with request. The cost is $50.00 per tape or $4.00 per diskette in addition to the $50.00 charge listed above. Fifteen formatted double sided, high density diskettes are required for the complete set up.

NOTE: Structures CADD Set-ups are also available in the Unix Format. All prices are the same as the P.C. Versions. Please provide pre-formatted media.

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE
Structures Computer Program
Documentation and Software Request Form

Firm Name	
Attention	
Address	
City, State, Zip	
Phone ( )	Vendor No.

Item No.	Program Name	Description	Quantity	Unit Cost	Total
1					
2					
3					
4					
5					

Payable to: Florida Department of Transportation Remit to: Office of Information Systems Florida Department of Transportation 605 Suwanee Street, MS-14 Tallahassee, Florida 32399-0450 **ATT: Elena Davenport** (904)488-3503	Subtotal Florida residents add Sales Tax (or T.E.#) Shipping (5% of subtotal) Processing Total Amount Check No. ______ enclosed	 $4.00

Check Diskette Type: _____ 5.25" LD 360K _____ 3.5" LD 720k
_____ 5.25" HD 1.2mb _____ 3.5" HD 1.44mb

For technical assistance call: Structures Computer Applications (904)488-6351.

Disclaimer : Users of these programs represented by the undersigned engineer-in-charge understand that no warranty, expressed or implied, is made by the Florida Department of Transportation as to the accuracy and functioning of the program text or the results it produces, nor shall the fact of distribution constitute any such warranty, and no responsibility is assumed by the Florida Department of Transportation in any connection therewith. The Engineer of Record assumes the entire risk as to the quality and performance of these programs for his particular application. These programs are for applications in the State of Florida Only.

Name of Engineer-in-Charge (please print or type)	
Signature:	Date:

Approved: Jerry Potter

Effective: November 2, 1992
Sequence No.: 0160
Responsible Office: Structures Design
Topic No.: 625-020-118-h

Chapter 18

RETAINING WALLS

18.1 Purpose

The purpose of this chapter is to give the designer an understanding of where to use retaining walls, design parameters and in some situations how to design retaining walls. A step by step method to develop and organize the retaining wall plans is presented. A section on wall types will explain the applicability of each wall type (their strong and weak points) presently being used. A section on design parameters, methodology and AASHTO specification exceptions is included.

Prior to 1977 the types of retaining walls constructed for Florida Department of Transportation (FDOT) projects were somewhat limited. Reinforced cast-in-place concrete and steel sheet piles were the walls of choice and to a lesser extent concrete sheet piles were used primarily as bulkheads.

The advent of Mechanically Stabilized Earth (MSE) walls brought many changes to the way FDOT designed and let to contract retaining walls. After 1977 FDOT began almost exclusively utilizing MSE proprietary retaining walls due to their economic advantage and their ability to articulate and thus withstand a degree of settlement.

If the difference in height between the ground levels to be supported is less than five (5) feet, a gravity retaining wall is generally the most efficient structure to be used. For details of gravity retaining walls see the Roadway Design Standards, Index No. 520.

When the difference in height between the ground levels to be supported exceeds five (5) feet then either a conventional reinforced cast-in-place (CIP) concrete cantilever retaining wall or a proprietary retaining wall is required.

In general, proprietary MSE retaining walls should be utilized for projects when the exposed surface area of the walls exceed 1000 sq. ft. and sufficient room for the earth reinforcement system is available; however, site specific conditions must always be considered when determining the type(s) of wall to be designed.

18.2 Conventional (CIP) Retaining Walls and Proprietary Retaining Walls (Permanent Walls)

The Department's policy is to provide either a set of conventional retaining wall plans or at least two, preferably three, different proprietary retaining wall plans for all projects where walls are not supported on piling. Projects where walls are supported on pilings only require a conventional pile supported wall design or a pile supported proprietary wall design. Omission of conventional retaining walls is possible if adequate justification is provided in the Bridge Development Report (BDR) or 30% Plans Stage submittal as applicable.

Proprietary retaining wall design plans are not required in the contract plans for normal uncomplicated wall projects. If the proprietary walls are experimental, exceed 40 ft. in height, or exhibit some unusual geometric, (i.e. phase construction) environmental or topographic feature, they shall be required to have fully detailed design plans in the contract set.

Step-by-step procedures for developing retaining wall plans are as follows:

A. Retaining Walls (Conventional Design)

1. BDR

   The BDR, if applicable, shall discuss and justify the use/non-use of conventional retaining walls. If the use of conventional retaining walls is applicable to the site and economically justified, it may be the only design required or it may be an alternate to a proprietary design.

2. 30% Plans Stage

   The 30% Plans Stage submittal shall contain a location plan, plan and elevation of walls showing vertical and horizontal alignment, cross sections and details. The plans shall denote location of drainage inlets, utilities, sign structures, lights and barrier joints. Specifically the submittal package shall include:

a) Plan:

A plan view of the wall and footings which indicate pertinent dimensions, boring locations and horizontal alignment.

b) Elevation:

A front view of the wall which indicates pertinent dimensions and elevations, sign and lighting structures locations, drainage structure locations and flow line elevations, location of section views and vertical alignment.

c) Sections:

Sections taken through the wall to better indicate dimensions and elevations.

d) General Notes including:

Design Toe Pressure

Environmental Classification

Concrete - (Strength and Class)

Reinforcing Steel - (Grade)

Design Method

Soil Design Parameters for both the in-situ and backfill materials.

Factors of Safety

3. The 30% Plans Stage shall be submitted for approval. After the 30% Plans Stage submittal has been approved the 90% Plans shall be developed.

4. 90% Plans Stage

The 90% Plans Stage submittal shall be further developed to include, in addition to the information required for the 30% Plans Stage, the following:

a. Plan:

A plan view of the wall and footings which indicates pertinent dimensions; reinforcing steel locations, cover and spacing in footings; boring locations, back of wall drainage details and horizontal alignment.

b. Elevation:

A front view of the wall which indicates pertinent dimensions and elevations; location of section views; reinforcing steel location, cover and spacing; back of wall drainage and flow lines and vertical alignment.

c. Sections:

Sections taken through the wall to better indicate dimensions, reinforcing steel locations, concrete cover for rebars and elevations.

d. Estimated Quantities:

Estimated quantities for the items incorporated in the wall, reinforcing bar list and standard bar bending sheet.

The Structures Design Office has prepared standard drawings of conventional cantilever retaining walls ranging from 2000 psf to 6000 psf design toe pressure and in height from 6 ft. to 30 ft. (See Structures Standard Drawings Index 800 thru 822.) These Structures Standards may be obtained from the Florida Department of Transportation, Maps and Publication Sales, 605 Suwannee Street, MS 12, Tallahassee, Florida 32399-0450.

B. Retaining Walls (Proprietary Design) (Design Required in Contract Plans)

The following procedure for plans preparation should be followed if the walls are experimental, exceed 40 ft. in height, or exhibit some unusual geometric (i.e. phase construction), environmental or topographic feature.

1. BDR

   The BDR, if applicable, shall discuss and justify the use of proprietary retaining walls.

2. 30% Plans Stage

   Prepare Control Plans. The Control Plans shall include but not be limited to the following information (See Figure 18-1 thru 18-9).

   a) Plan and Elevation Sheet:

      1. horizontal and vertical alignment
      2. limits of wall
      3. utility locations
      4. plan view of wall
      5. elevation view of wall (showing existing and proposed ground lines, elevations at 25 ft. intervals at top of wall, wall embedment (maximum elevation at top of leveling pad) and beginning and end of wall stations)
      6. boring locations
      7. quantity (pay area of walls)
      8. table showing soil reinforcement length vs. wall height (for external stability)
      9. general notes
      10. in-situ soil characteristics
      11. design parameters - safety factors
      12. sections thru wall showing offset control point, pay area, ditches, sidewalks and other unusual features.

   b. Soil Profile Sheet

   c. General Details showing:

      1. wall/end bent cap interface
      2. barrier and coping to wall interface

3. pile, inlets and pipe conflicts with soil reinforcement and slip joint details.

d. Preapproved Standard Drawings:

1. Standard drawings of each of the alternate companies shall be included in the control plans. These drawings can be obtained from the
Structures Design Office
605 Suwannee Street, M.S. 33
Tallahassee, FL 32399-0450.

3. The Control Plans shall be reviewed by the Engineer of Record and upon approval sent to the appropriate preapproved (on QPL) proprietary wall companies. The companies shall be provided with a set of control plans, roadway plans and foundation report. The Control Plans shall be sent to the wall companies at the 30% Plans Stage.

4. The proprietary companies shall acknowledge receipt of the invitation package. If they choose to participate they shall provide design plans for the retaining walls and submit the plans for review as prescribed in the invitation letter (see Figure 18-10 and 18-11).

5. 90% Plans Stage
Upon receipt of the proprietary design plans, the designer shall review the design and incorporate the wall plans into the contract set. The plans from the wall companies, control plans and wall company standard drawings shall constitute the final plans.

C. Retaining Walls (Proprietary Design) (Design Not Required in Contract Plans - Control Plans Only)

The following procedure for plans preparation should be followed for normal, uncomplicated wall projects.

1. BDR

   The BDR, as applicable, shall discuss and justify the use of proprietary retaining walls.

2. 30% Plans Stage

   Prepare Control Plans. The Control Plans shall include but not be limited to the information as shown in Section 18.2.B.2.

3. The Control Plans shall be reviewed by the Engineer of Record and upon approval sent to the appropriate preapproved (on QPL) proprietary wall companies. The Control Plans are sent to the wall companies at the 30% Plans Stage.

4. The proprietary wall companies shall acknowledge receipt of the control plans package as prescribed in the transmittal letter (See Figures 18-12 and 18-13).

5. 90% Plans Stage

   The Control Plans and the preapproved proprietary wall standard drawings shall be incorporated into the 90% Plans Stage submittal.

   NOTE: The preapproved proprietary wall standard drawings are available from the State Structures Design Office.

The success of this method of producing and letting wall plans is highly dependent on complete, accurate and informative Control Plans. The importance of the Geotechnical Engineer's role in this scheme cannot be emphasized enough.

The Geotechnical Engineers' responsibilities are as follows:

1. Borings
2. Soils Report
3. Wall Type recommendation
4. If MSE Wall; reinforcement length vs. wall height for external stability.
5. Review of internal stability design as provided by the wall

companies.

6. Establish allowable bearing pressures.

The normal failure modes to be investigated are as shown in Figure 18-14.

18.3 Critical Temporary Walls

A critical temporary wall is one that is necessary to maintain the safety of the traveling public or structural integrity of nearby structures and utilities for the duration of the construction contract.

Critical temporary walls shall be designed in accordance with AASHTO Specifications and the Structures Design Guidelines, Section 18 including the soil reinforcement lengths, size and stress level requirements for permanent walls. For temporary MSE walls using geogrid soil reinforcement, the allowable reinforcement tension shall be based on AASHTO requirements for permanent MSE walls and the following:

1. The overall factor of safety, F_S, shall be at least 1.5
2. The reduction factor for construction site damage, F_C, shall be at least 1.25, if supported by applicable test data. If no test data is available $F_C = 3.0$.
3. The reduction factor for durability, F_D, may be reducted to 1.1 if supported by applicable test data. If no test data is available $F_D = 2.0$.
4. The allowable reinforcment tension, T_a, shall not exceed one half the facing connection strength at 3/4" movement.
5. The value of Limit State Tensile Load, T1, shall preclude brittle or ductile failure of the geogrid within the design life based on stress rupture testing of the geogrid product. The testing and data extrapolation shall be in accordance with ASTM D-2837 or GRI GM-5; results shall be submitted with the design calculations.

   In lieu of junction strenth criteria, the long term pullout interaction coeffieient may be determined from pullout data from testing the geogrid product in a representative soil material with transverse ribs severed.

The design of critical temporary walls shall be included in the contract set of plans.

18.4 Experimental Wall Projects

The Department maintains a Qualified Products List (QPL) of all wall systems that have been approved. The proprietary wall companies must comply with the Department's "Guidelines for Selection and Approval of Proprietary Retaining Wall Systems" to be placed on the QPL. One of the requirements is to build a wall that may be instrumented and shall be monitored. Special instructions for package design and plans package preparation shall be obtained from the State Structures Design Office.

18.5 Shop Drawing Review

Conventional C-I-P retaining walls do not require shop drawings.

Proprietary retaining walls require shop drawings to be submitted in accordance with Chapter 19.

The shop drawing reviewer (EOR) shall be experienced in the requirements, design and detailing of proprietary wall plans. The EOR shall review but not be limited to the following items:

A. Verify vertical and horizontal geometry with contract plans.

B. Verify details with MSE wall suppliers standard details in contract plans.

C. Soil reinforcement placement in acute corners shall be detailed.

D. Slip joints shall be at all bin wall and standard MSE wall interface locations.

E. Soil reinforcement shall be detailed at all obstructions. Cutting or kinking of soil reinforcement shall not be allowed. Connection of soil reinforcment to piles or bearing against piles shall not be allowed.

F. Corner panels shall be used at all locations where walls are deflected horizontally 5 degrees or more.

G. Compare proposed reinforced fill characteristics with design fill characteristics. In-place moist weight of backfill may vary by $\pm$ 5 lbs./C.F. from and the internal friction angle may be 1^{0} less than the design values (as shown in control plans) before a check of the wall design is required. If the internal friction angle is greater than the design value then a redesign is not required.

H. Review proprietary wall internal stability design calculations.

I. Verify soil reinforcement lengths for conformance to Section 18.8, the external stability table on the plans, and the internal stability design calculations.

J. Confirm wall embedment.

K. Verify panel types and thickness are consistent with contract plans.

L. Soil reinforcement lengths shall be the same from top to bottom of wall at any section. Soil reinforcement size (thickness, diameter, etc.) shall be the same throughout any wall section (i.e. wall reinforcement shall be all W11 or all W20, etc.). The diameters of the longitudinal and transverse bars of any given mesh reinforcement shall be equal. The cross-section of any soil reinforcement shall not vary along its length (i.e., "2W11" reinforcement shall not be spliced to "4W11).

M. Calculated maximum applied bearing pressure shall not be more than the allowable bearing pressure based on the ultimate bearing capacity divided by a factor of safety of 2.5.

N. Check stress level in soil reinforcement and connections.

18.6 Bidding Procedure

The conventional C-I-P walls shall be bid as Concrete (Retaining Wall) and Reinforcing Steel (Retaining Wall). Conventional walls may be bid as an alternate to proprietary walls if the site conditions justify conventional walls.

Proprietary Walls shall be bid as alternates using a unique bid item number for each wall company. A note shall be on the plans stating, "The wall alternate bid shall be the alternate built. No substitution of other alternates or other companies' walls shall be allowed."

18.7 Wall Types (Site Applications)

There are a number of types of retaining walls presently being used by the Department.

Wall site considerations, economics, aesthetics, maintenance and constructability should all be considered when determining wall type to be used.

The following list of wall types are being used or considered for use by the Department:

- o Conventional C-I-P
- o Pile Supported Walls
- o MSE Walls
- o Steel Sheet Piles (Cantilevered and Tied-back)
- o Concrete Sheet Piles
- o Precast Counterfort Walls
- o Soil Nails (temporary walls)
- o Wire Faced Walls (Temporary Walls)
- o Geogrid Reinforced MSE Walls (Temporary Walls)
- o Soldier Pile/Panel Wall
- o Modular Block Walls (Under consideration)
- o Permanent - Temporary Wall Combination

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

CIP walls are normally used in either a cut or fill situation. These walls are sensitive to foundation problems. The foundation soil must be capable of withstanding the design toe pressure and must exhibit very little differential settlement. Judgement should be exercised to assure that during the life of the structure the bearing capacity of the soil will not be diminished (i.e., french drains in close proximity to the walls) thus requiring pile supported walls. This type wall has an advantage over (MSE) walls because they can be built with conventional construction methods even in extremely aggressive environments. Another advantage over MSE walls is on cut/widening projects where the area behind the wall is not sufficient for the soil reinforcing (See Figures 18-15 and 18-16).

The relative cost of CIP walls is greater than MSE walls when the site and environment is appropriate for each wall type. This is assuming the area of wall is greater than 1000 SF and greater than 8 ft. high.

B. Pile Supported Walls

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

Pile supported retaining walls are extremely expensive compared to CIP cantilevered and MSE walls and are only appropriate when foundation soil conditions are not conducive for CIP or MSE walls. Pile supported walls are appropriate for cut or fill sites. Temporary sheeting may be required in cut sites (See Figures 18-17 and 18-18).

C. Mechanically Stabilized Earth (MSE) Walls

MSE walls are not a cure-all for poor foundation soil and are not appropriate for all sites. MSE walls are very adaptable to both cut and fill conditions and will tolerate a greater degree of differential settlement than C-I-P walls. Because of their adaptability MSE walls are being used almost exclusively. The design of MSE walls, 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 extremely aggressive waters or from heavy fertilization. When soil reinforcement is proposed for use in areas where the water is classified as an extremely aggressive environment and the 100-year flood can infiltrate the backfill, the reinforcement shall be designed and/or protected by a method approved by the SSDE.

MSE walls are generally the most economic of all wall types when the area of retaining wall is greater than 1000 sq. ft. and the wall is greater than 8 ft. high (See Figures 18-19 and 18-20).

D. Precast Counterfort Walls

Precast counterfort walls are applicable in cut or fill locations. Their advantage is in cut locations such as removing front slopes under existing bridges. Also, their speed of construction is advantageous in congested areas where maintenance of traffic is a problem. This type wall is also applicable in areas where the backfill is or can become classified as extremely corrosive.

This type wall is generally not as economical as MSE walls but is competitive with CIP walls (See Figure 18-21).

E. Steel Sheet Pile Walls (Permanent and Temporary)

Steel sheet pile walls are applicable for use in permanent locations (i.e., bulkheads) but their more common use is for temporary use (i.e., phase construction). Generally steel sheet pile walls can be designed as cantilevered walls up to approximately 15 ft. in height. Over 15 ft. steel sheet pile walls are tied back with either prestressed soil anchors or tiebacks (deadmen).

Steel sheet piles walls are relatively expensive initially and require periodic maintenance (i.e. painting, cathodic protection). This type of wall should only be used if there are no other more economical alternates (See Figure 18-22).

F. Concrete Sheet Piles

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

This type wall is relatively costly and should only be used when less costly alternates are not appropriate or when the environment is appropriate (See Figure 18-23).

G. Wire Faced Walls (Temporary)

Wire faced walls are applicable in temporary situations, phase construction where large amounts of settlement are anticipated or where surcharges are required to accelerate settlement. This type wall is a form of MSE wall. The soil reinforcement may be either steel or geogrid. The designer shall verify the systems he intends to use are on the Department's Qualified Products List (See Figure 18-24).

H. Soil Nails (Temporary Walls)

Soil nails are a relatively new technology and at the present time the FHWA is writing a procedure for the design methodology of soil nailing. The Department is allowing the use of soil nails in temporary walls only.

The relative cost of this type wall has not been determined because no contractors have bid this alternate (See Figure 18-25).

I. Soldier Pile/Panel Walls

This type wall is applicable in bulkheads and retaining walls where the environment is extremely aggressive and/or rock is relatively close to the ground surface.

The cost of this type wall is very competitive with concrete sheet pile walls (See Figure 18-26).

J. Modular Block Walls

Modular blocks consist of drycast unreinforced blocks which are sometimes used as a gravity wall and sometimes used as a wall facing for a MSE variation normally utilizing a geogrid for soil reinforcement. The Department has not approved any systems of this type and any future approval is likely to be limited to low-height, non-structural applications for the following reasons:

1. The blocks are subject to cracking due to differential settlement.
2. The long term durability of the blocks is in question.
3. The connection of the soil reinforcement to the facing block has inadequate safety.
4. The long term durability and allowable design strength of geogrids is uncertain.

K. Geogrid Soil Reinforcement for Retaining Walls

Geogrids are presently in the process of being approved for use in temporary walls only. Their use in permanent walls is not approved due to the many questions about the design and durability of the geogrid. 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 of safety to ensure an allowable stress condition at the end of the structure's service life.

L. Permanent - Temporary Wall Combination

As more highways are widened, we have encountered a problem at existing grade separation structures. The existing front slope at the existing bridge is required to be removed to accommodate a new lane and a retaining wall must be built under the bridge. The trick is how do you remove the existing front slope and maintain the stability of the remaining soil? Several methods have been used. One method is to excavate slots or pits in the existing fill to accommodate soldier beams. The soil is then excavated and timber lagging is placed horizontally between the vertical soldier beams.

The soldier beams are tied-back by the use of prestressed soil anchors. This procedure will maintain the soil while the permanent wall is built. The permanent wall should be designed to accept all appropriate soil, dead and live loads. The temporary wall shall not contribute to the strength of the permanent wall (See Figure 18-27).

18.8 Design

Retaining Walls shall be designed in accordance with the AASHTO Specifications and latest interims except as noted in this section.

A. Factors of Safety and Assumed Design Parameters

The following are minimum factors of safety and assumed design values. If actual values are different than the assumed, then the design should be reevaluated by the designer.

1. SAND FILL

Overturning	= 2.0
Sliding	= 1.5
Internal pullout	= 1.5 (Allowable Deformation 0.75 inches)
Bearing capacity	= 2.5
Overall stability	= 1.5
Steel Soil Reinforcement (Bars and Straps)	= 0.55 Fy at end of design life, and 0.50 Fu at net section of bolted connection
Steel Soil Reinforcement (Grids and Bar Mats)	= 0.47 Fy at end of design life
Steel Connections	= See AASHTO Specifications
Design Based ϕ	= 30^{o}
Backfill Density	= 105 lbs/C.F. moist wt. (This is the assumed in place compacted wt. of backfill.)
Maximum Pullout Factors:	
Ribbed Strips, f^*_{MAX}	= 1.5
Grids and Bar Mats, $N_{P\,MAX}$	= 30

2. Limerock Fill

Overturning	= 2.0
Sliding	= 1.5
Internal pullout	= 1.5 (Allowable Deformation 0.75 inches)
Bearing capacity	= 2.5
Overall stability	= 1.5
Steel Soil Reinforcement (Bars and Straps)	= 0.55 Fy at end of design life, and 0.50 Fu at end section of bolted connection
Steel Soil Reinforcement (Grids and Bar Mats)	= 0.47 Fy at end of design life

Steel Connections	= See AASHTO Specifications
Design Based ϕ	= 34^{0}
Backfill Density	= 115 lbs/C.F. moist wt. (This is the assumed in place wt. of backfill.)

Maximum Pullout Factors:	
Ribbed Strips, f^*_{MAX}	= 1.5
Grids and Bar Mats, $N_{P\ MAX}$	= 30

B. Corrosion Rates

The following corrosion rates 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

C. Soil Reinforcement

1. Corrosion rates for reinforcement in extremely aggressive environments shall be in accordance with Article 18.8.D above and the reinforcment designed and/or protected in accordance with the requirements of Article 18.7.C.

2. Soil Reinforcement and connections for permanent walls shall be designed for a design life of 75 years except for walls supporting abutments on spread footings which shall be designed for a design life of 100 years and for that portion of the walls across the face of the abutment or along that length of wall within a horizontal distance equal to the vertical wall height at the abutment.

3. Soil Reinforcement for temporary walls shall be designed for a design life of not less than the contract time of the project or three years whichever is greater.

4. The steel soil reinforcement shall not exceed Grade 65 and the yield stress used in the design shall not exceed 65 KSI.

5. Vertical stresses at each reinforcement level shall include consideration for local equilibrium of all forces to that

level only and shall include the effects of any eccentricity.

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

7. Epoxy coated and passive metal (i.e., stainless steel, aluminum alloys) soil reinforcements are not permitted.

8. Geosynthetic geogrids if approved and on the QPL, may be used in temporary walls only.

9. The stress level in the soil reinforcement and connections (including facing panel embedded connection devices) at the end of the design life shall not exceed:

Bars and Straps	= 0.55 Fy at reduced gross section and 0.50 Fu at net section of bolted connections
Grids and Bar Mats	= 0.47 Fy at all sections

10. Soil reinforcement shall not be skewed more than 15 degrees from a position normal to the wall panel except as necessary and clearly detailed for acute corners.

11. Soil reinforcement lengths, measured from the back of the facing element, shall not be less than the greater of the following:

MSE Walls and Walls in Front of Abutments on Piling

a. L = 8' min. length

b. L = .7 H_l, H_l = mechanical height of wall, See Figure 18-28 and 18-29).

c. L = length required for external stability design

MSE Walls In Front of Abutments on Spread Footings

a. L = 22 feet

b. L = 0.6 H_l + 6 feet

c. L = 0.7 H_l

As a rule of thumb, for an MSE sructure with reinforcment 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 reinforcment length for external stability to assure that the allowable bearing pressure specified equals or exceeds this value.

12. External Stability of MSE Structures

H_1, the mechanical height, of an MSE structure shall be taken as the height measured to the point where the potential failure plane (line of maximum tension) intersects the ground surface, as shown in Figure 18-28 and 18-29.

The mechanical height H_1 can be calculated using the equation provided in Figure 18-28.

In addition to the two conditions shown in AASHTO, horizontal backslope and infinite sloping backfill, we are including Figures 18-28 and 18-29 to illustrate broken back slope conditions with and without traffic surcharge. If the break in the slope behind the wall facing is located within two times the height of the wall (2H), then the broken back slope design shall apply. If a traffic surcharge is present and is located within 0.5H of the back of the reinforced soil mass, then it shall be included in the analysis.

13. Acute corners less than 75 degrees. (MSE Walls)

When two walls intersect forming an internal angle of less than 75 degrees, the nose section shall be designed as a bin wall. The calculations for this special design shall be submitted with the plans for review and approval.

Structural connections between the wall elements within the nose section shall be designed to create an at-rest bin effect without eliminating the flexibility of the facing to tolerate differential settlements. The design for connections to facing elements and allowable stresses in the connecting members shall conform to the requirements for the wall design in general. The nose section shall be designed to settle differentially from the remainder of the structure with a slip joint. Facing panel overlap or interlock or rigid connection across that joint shall not be allowed.

Soil reinforcements shall be designed to restrain the nose section by connecting directly to each of the facing elements in the nose section and running back into the backfill of the main reinforced soil mass at least to a plane 10 ft. beyond a Rankine failure surface. Design for facing connections, obstructions, pullout and allowable stresses in the reinforcing elements shall conform to the requirements of the wall design in general (See Figure 18-30).

14. Minimum Wall embedment in the soil shall be determined by consideration of both scour and bearing capacity. The District Drainage and Geotechnical Engineers shall be consulted in determining the elevation of the top of leveling pad which must also conform to a minimum embedment to the top of leveling pad of 1'-6" as determined in Figure 18-31.

15. Apparent Coefficient of Friction (F*)

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

16. Connections (MSE Walls)

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

17. Abutments Behind MSE Walls

    A. Abutments on Piling

        1. No battered piles should be in abutments.

        2. The abutment face shall be a minimum distance of 2'-0" behind the front face of the wall. (2'-0" minimum between face of piling and leveling pad)

    B. Abutments on Spread Footings

        1. The spread footing shall be sized so that the bearing pressure shall not exceed 4 KSF.

        2. The footing shall be a minimum distance of 1 ft. behind the facing panel.

        3. The minimum distance between the centerline of bearing on the abutment and the back of the facing panel shall be 4 ft.

18. Facing Panels (MSE Walls)

    Full-height facing panels shall not exceed 8 feet in height.

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

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3	FLA.		W-1

RETAINING WALL GENERAL NOTES

1. Area of retaining wall to be used for payment shall be the area bounded by the top of Traffic Railing Barrier (or top of coping in areas without traffic barrier), the proposed final ground line or finished top of concrete slope pavement at the front face of wall and the begin and end wall limits as shown on control plans.

   Proprietary Retaining Walls and Traffic Railing Barrier shall be paid for at the contract unit price per square foot of retaining wall under Pay Item No. 523-70 Alt. AA, Reinforced Earth Wall or Pay Item No. 545-70 Alt. AB, Retained Earth Wall.

   The cost of all foundations, fill material and excavation (required specifically for wall construction outside of roadway cross section template) and walls below the proposed final ground line shall be included in the unit cost for Retaining Walls, Pay Item No. 523-70 Alt. AA or 545-70 Alt. AB.

   The cost of installation of utility foundations (light poles on proprietary walls) and accomodations for drainage structures shall be included in the unit cost for Retaining Walls, Pay Item No. 528-70 Alt. AA or 545-70 Alt. AB.

   The wall alternate bid shall be the alternate built. No substitution of other companies walls shall be allowed.

2. Panel surface treatment shall be a 2" raised finish similar to Reinforced Earth's Type I.

3. Traffic Railing Barrier (or coping in areas without Traffic Railing Barrier) on top of retaining wall shall receive a Class 5 Applied Finish Coating.

4. Piles within the Reinforced Earth Wall and Retained Earth Wall Volume shall be driven prior to the construction of the retaining wall. Piles shall be wrapped with 2 layers of 6 mil polyethylene above level of existing ground.

5. Walls shall be designed for the following:

   Maximum Allowable Bearing Pressure = 4000 psf

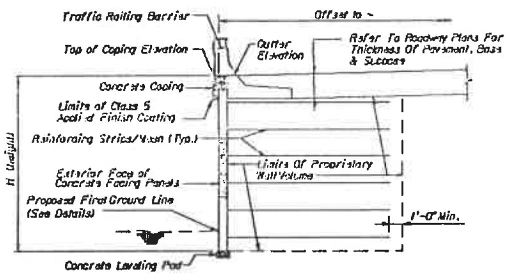
   Anticipated Settlements are between 1" and 4", the majority of which should occur during placement of backfill material. (See table of "Required Minimum, Reinforcing Strip/Mesh Lengths" below)

   Anticipated differential settlement is ___ in 100'.

Overturning	Min. F.S. = 2.0
Sliding	Min. F.S. = 1.5
Bearing Capacity	Min. F.S. = 2.5
Overall Stability	Min. F.S. = 1.5
Pullout	Min. F.S. = 1.5 (Allowable Deformation = 0.75 In.)

	Reinforced Soil & Random Backfill	Loose Fine Sand	Firm Fine Sand	Loose Clayey Fine Sand	Firm Clayey Fine Sand	Soft Clay
Unit Weight	105 PCF (Moist Weight)	115 PCF	115 PCF	115 PCF	120 PCF	95 PCF
Cohesion	0	0	0	400 PSF	400 PSF	600 PSF
Internal Friction	30°	30°	32°	18°	18°	10°

If the unit weight of the fill proposed by the Contractor differs from the above, the Project Engineer shall contact both the District Geotechnical Engineer and Wall Designer for a possible redesign.



PROPRIETARY RETAINING WALL
WITH TRAFFIC RAILING BARRIER
TYPICAL WALL SECTION
(N.T.S.)

INDEX OF PROPRIETARY WALL CONTROL DRAWINGS

NOTE:
See Roadway plans for complete drainage details.

DRAINAGE PIPE DETAIL

REQUIRED MINIMUM REINFORCING STRIP/MESH LENGTHS FOR EXTERNAL STABILITY
(Walls 1, 3, 4 and 5)

Wall Height (Ft.)	6	8	10	12	14	16	18	20	22	24	26
Minimum Strap Length (Ft.)	8	8	8	9	10	11	13	14	16	17	18
Estimated Settlement (In.)	1.0	1.4	1.7	2.1	2.4	2.8	3.1	3.4	3.8	4.1	4.5

Designer Informa... Min. strap length ... be 8' or 7H or length r... d for external stability ... is longest, should be shown in table.

Straps lengths provided in table are the minimum strap length which will assure external stability of wall. Strap lengths may increase due to internal stability requirements of proprietary wall system. Refer to Retaining Wall General Notes for internal stability requirements. Wall height shown is the distance measured from the Top of Coping Elevation to the Proposed Final Ground Line.

ESTIMATED QUANTITIES (SF)*		
Wall 1	A	17469
	B	2236
	C	30382
	Total Wall 1	50087
Wall 3	A	13911
	B	3337
	C	20487
	Total Wall 3	37735
Wall 4	A	15321
	B	2586
	C	13581
	Total Wall 4	31488
Wall 5	A	16317
	B	2645
	C	18758
	Total Wall 5	37730
	TOTAL	157040

* See Retaining Wall General Note 1.

REVISIONS

	Names	Dates
Drawn by	MHS	4-91
Checked by	CDW	4-91
Designed by	CDW	4-91
Checked by	GFM	4-91
Approved by	G.F. Myers	

ENGINEER OF RECORD | LOGO | SEAL

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO. | COUNTY | PROJECT NO.

SHEET TITLE: PROPRIETARY WALL CONTROL DRAWINGS GENERAL NOTES AND SECTIONS

PROJECT NAME | Drawing No. | Index No.

Figure 18-1

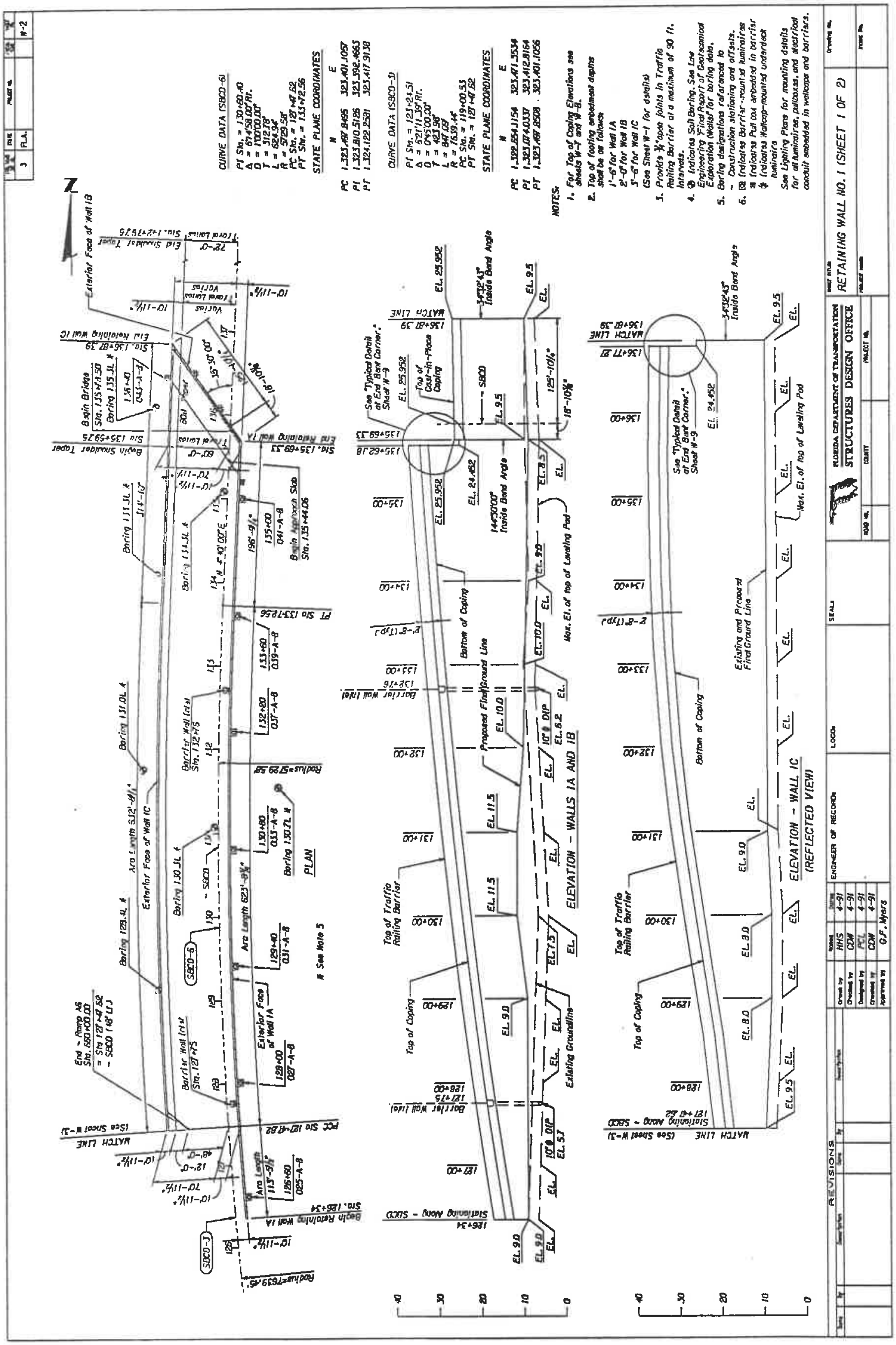

PLAN
ELEVATION - WALLS 1A AND 1B
ELEVATION - WALL 1C
(REFLECTED VIEW)
FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE
RETAINING WALL NO. 1 (SHEET 1 OF 2)

Figure 18-2

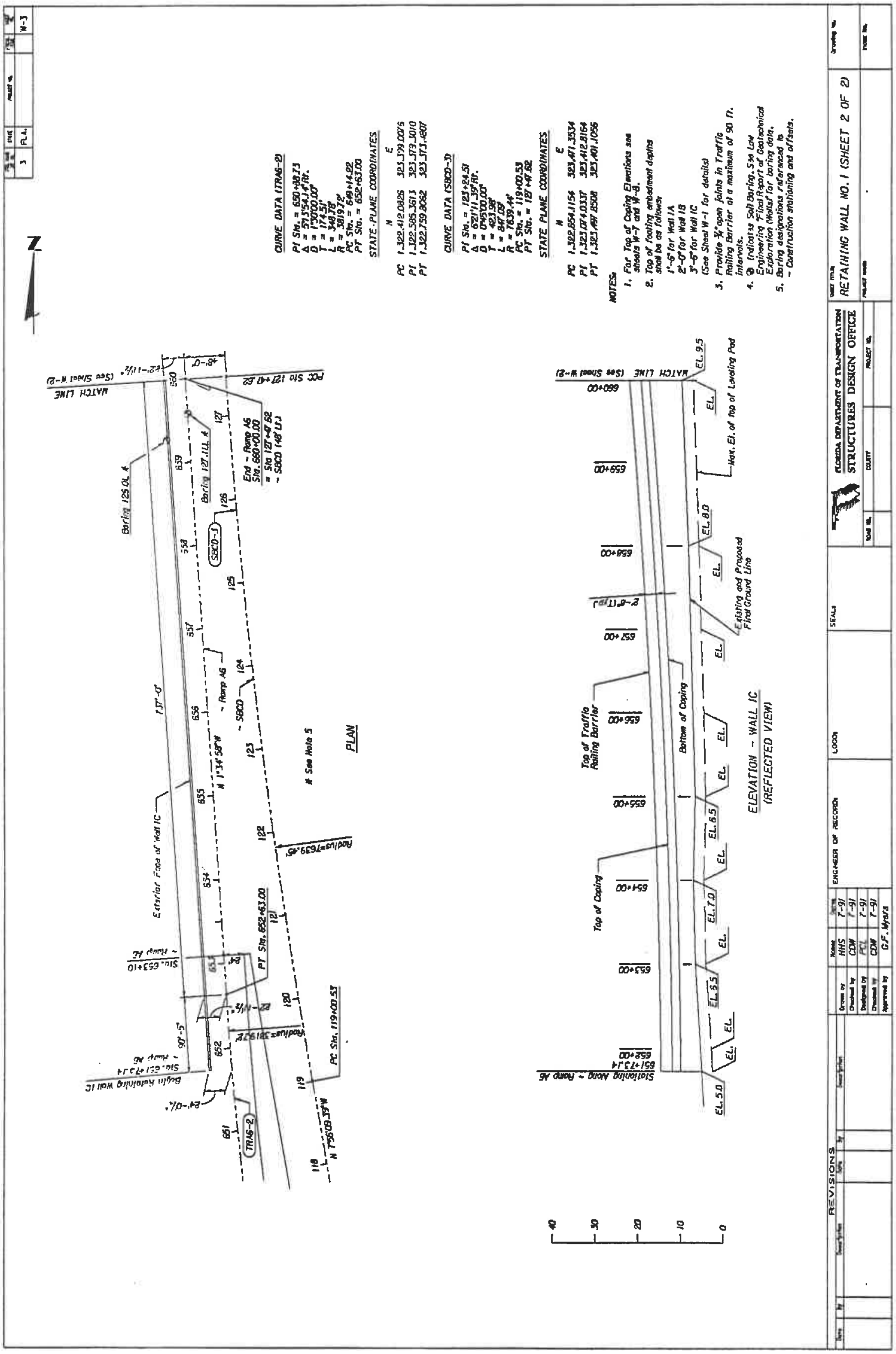

PLAN
ELEVATION ~ WALL 1C
(REFLECTED VIEW)
CURVE DATA (TRA6-2)
PI Sta. = 650+88.73
Δ = 5°13'54.14" Rt.
D = 1°30'00.00"
T = 174.51'
L = 348.78'
R = 3819.72'
PC Sta. = 649+14.22
PT Sta. = 652+63.00
STATE PLANE COORDINATES
PC 1,322,412.0826 323,399.0075
PI 1,322,585.3613 323,379.3010
PT 1,322,759.8062 323,373.4807
CURVE DATA (SBCD-3)
PI Sta. = 123+24.51
Δ = 6°21'11.39" Rt.
D = 0°45'00.00"
T = 423.98'
L = 847.09'
R = 7639.44'
PC Sta. = 119+00.53
PT Sta. = 127+47.62
STATE PLANE COORDINATES
PC 1,322,854.1154 323,471.3534
PI 1,323,074.0337 323,412.8164
PT 1,323,497.8508 323,401.1056
NOTES:
1. For Top of Coping Elevations see sheets W-7 and W-8.
2. Top of footing embedment depths shall be as follows:
1'-6" for Wall 1A
2'-0" for Wall 1B
3'-6" for Wall 1C
(See Sheet W-1 for details)
3. Provide 3/4" open joints in Traffic Railing Barrier at a maximum of 90 ft. intervals.
4. Indicates Soil Boring. See Law Engineering "Final Report of Geotechnical Exploration (Walls)" for boring data.
5. Boring designations referenced to ~ Construction stationing and offsets.
FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE
RETAINING WALL NO. 1 (SHEET 2 OF 2)
REVISIONS
W-3

Figure 18-3

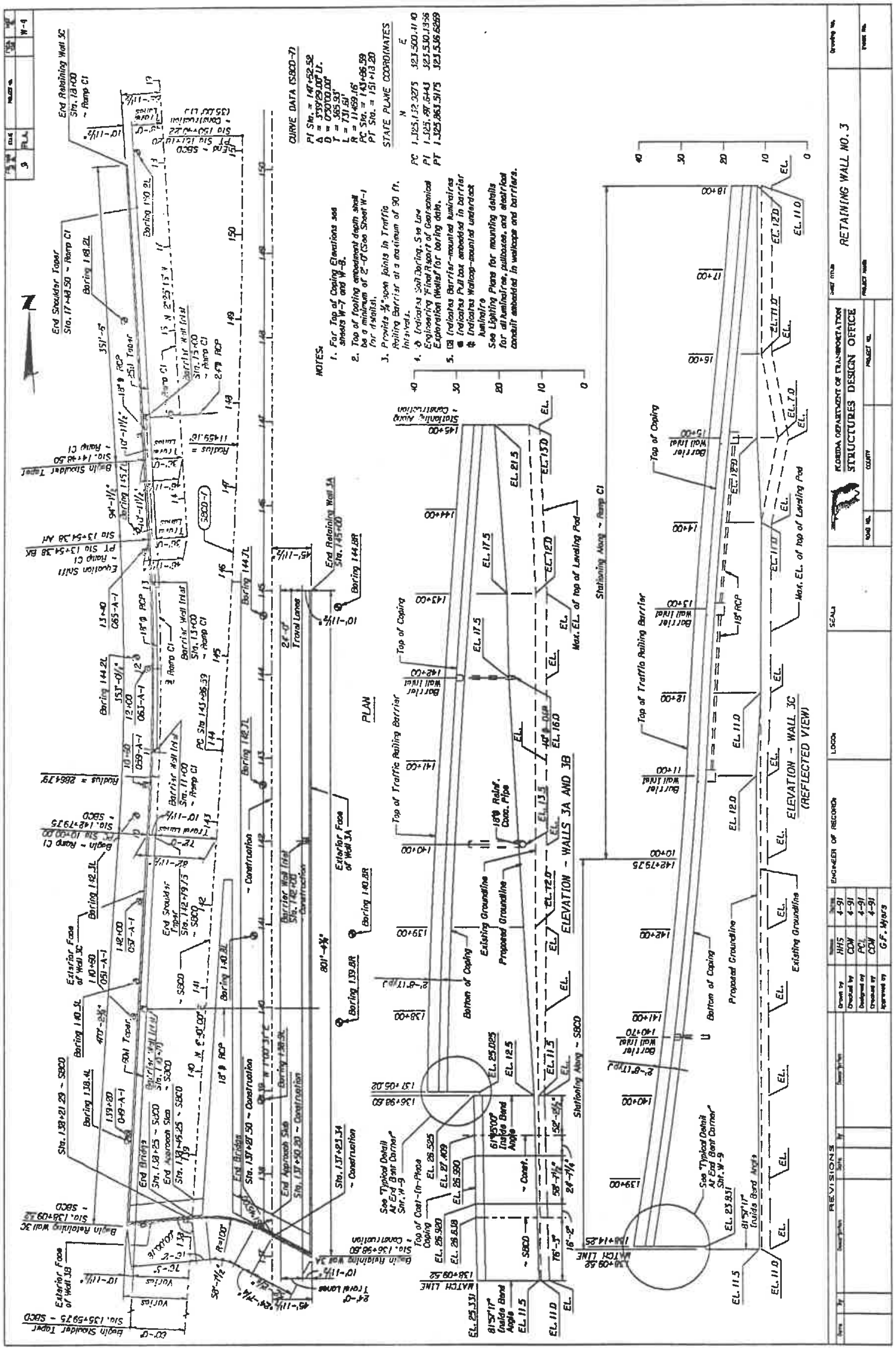

PLAN
ELEVATION - WALLS 3A AND 3B
ELEVATION - WALL 3C
(REFLECTED VIEW)
NOTES:
CURVE DATA (SBCD-1)
STATE PLANE COORDINATES
FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE
RETAINING WALL NO. 3
REVISIONS

Figure 18-4

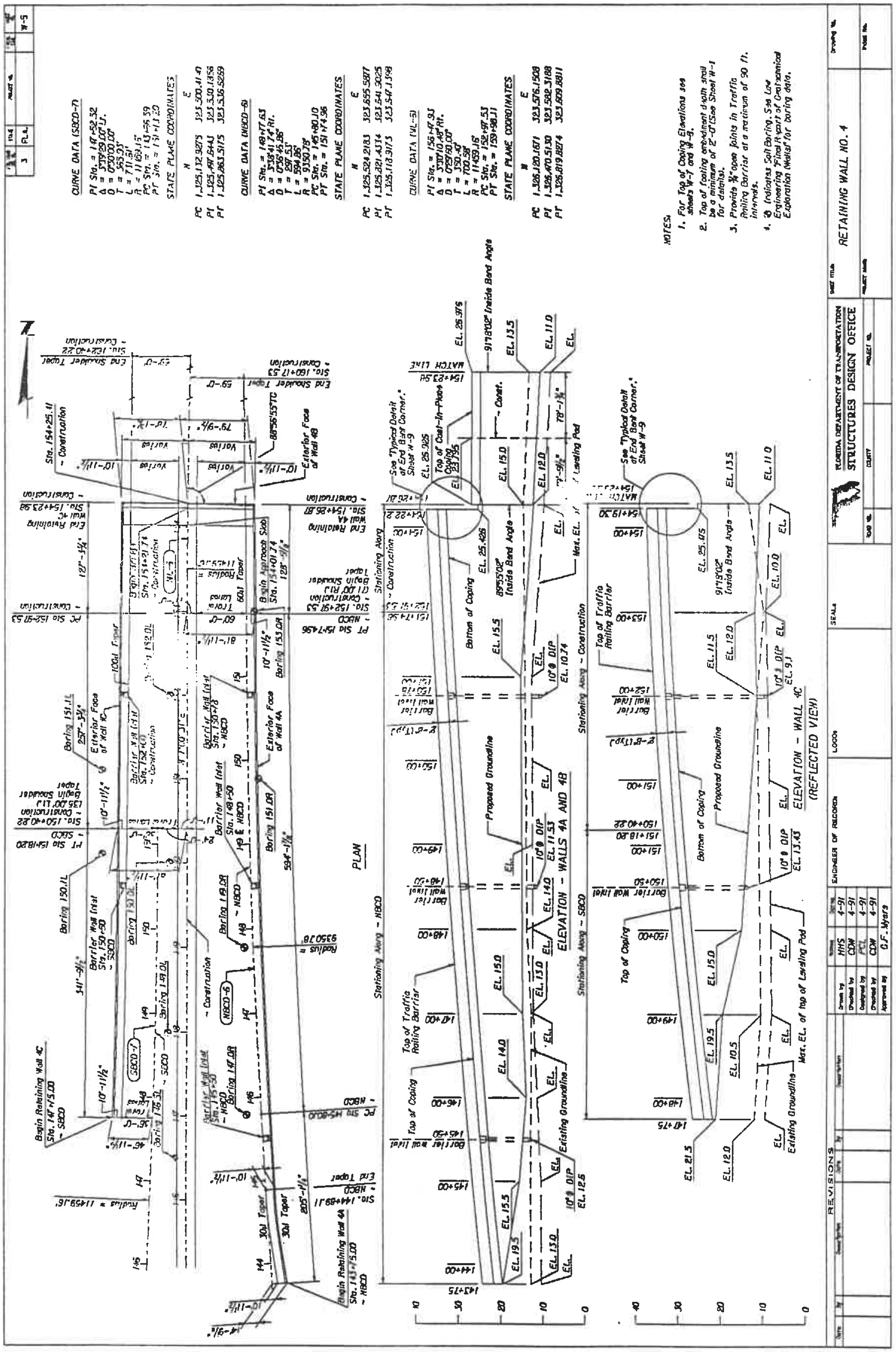

PLAN
ELEVATION - WALLS 4A AND 4B
ELEVATION - WALL 4C
(REFLECTED VIEW)
RETAINING WALL NO. 4
FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE
REVISIONS

Figure 18-5

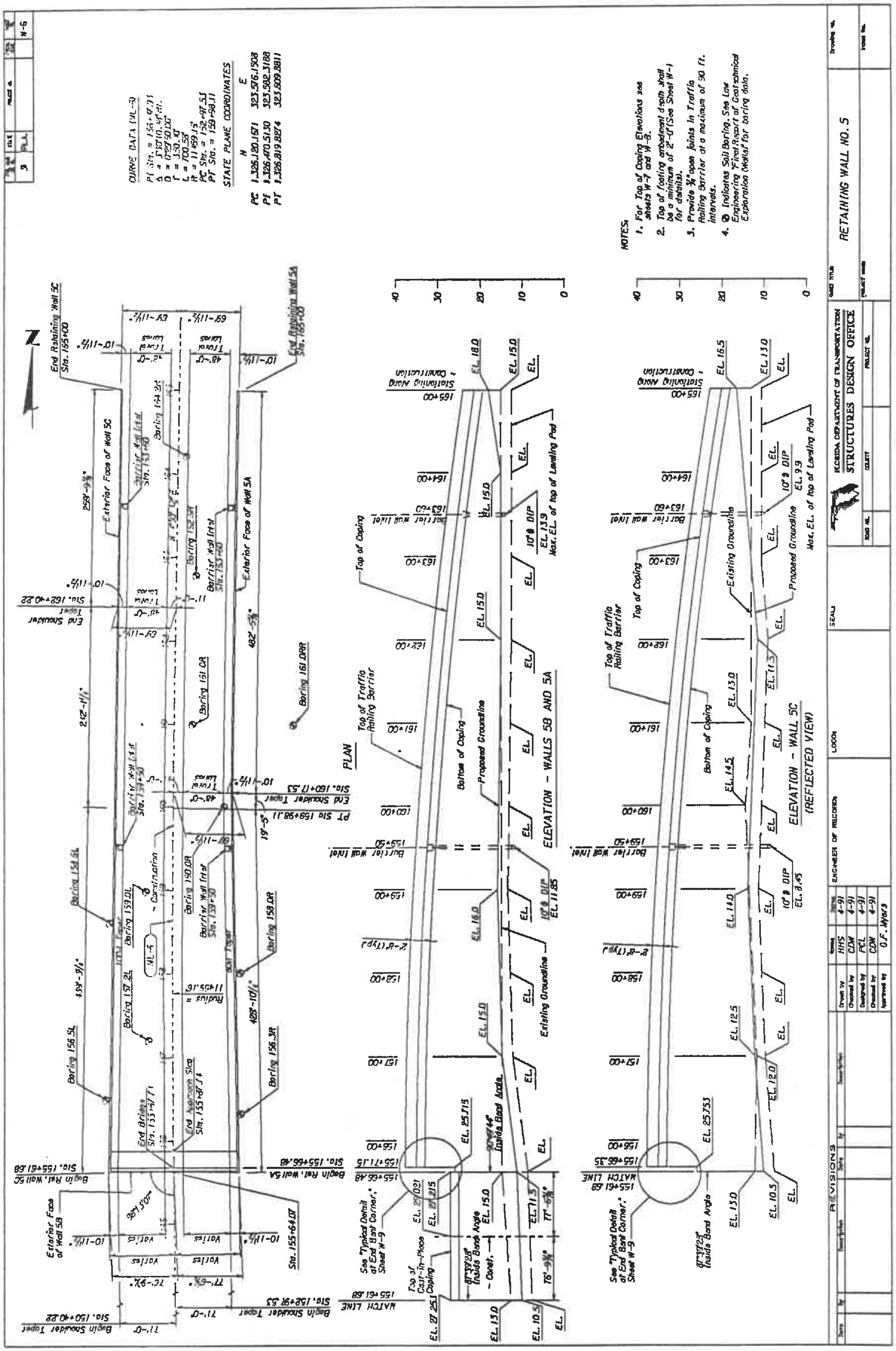

PLAN
ELEVATION - WALLS 5B AND 5A
ELEVATION - WALL 5C
(REFLECTED VIEW)
NOTES:
RETAINING WALL NO. 5
FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE
CURVE DATA (WL-5)
STATE PLANE COORDINATES
REVISIONS

Figure 18-6

ROAD DIST. NO.	STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS
3	FLA.			W-1

WALL Nos. 1C

~ Ramp A6 Station	Wall 1C Offset ~ Ramp A6 (ft)	~ SBCD Station	Wall 1C Offset ~ SBCD (ft)	Top of Coping Elevation @ Wall 1C (ft)
651+73.14	24.022			11.548
651+75.00	23.978			11.559
652+00.00	23.481	11.706		
652+25.00	23.149			11.857
652+50.00	22.981	11.996		
652+63.00	22.958			12.070
652+75.00	22.958			12.133
653+00.00	22.958			12.233
653+25.00	22.958			12.426
653+50.00	22.958			12.586
653+75.00	22.958			12.746
654+00.00	22.958			12.906
654+25.00	22.958			13.066
654+50.00	22.958			13.226
654+75.00	22.958			13.386
655+00.00	22.958			13.546
655+25.00	22.958			13.706
655+50.00	22.958			13.866
655+75.00	22.958			14.026
656+00.00	22.958			14.186
656+25.00	22.958			14.346
656+50.00	22.958			14.506
656+75.00	22.958			14.666
657+00.00	22.958			14.826
657+25.00	22.958			14.986
657+50.00	22.958			15.146
657+75.00	22.958			15.331
658+00.00	22.958			15.564
658+25.00	22.958			15.847
658+50.00	22.958			16.180
658+75.00	22.958			16.561
659+00.00	22.958			16.992
659+25.00	22.958			17.471
659+50.00	22.958			19.000
659+75.00	22.958			18.573
660+00.00	22.958	127+47.62	70.958	19.205
		127+50.00	70.958	19.280
		127+75.00	70.958	20.079
		128+00.00	70.958	20.926
		128+25.00	70.958	21.790
		128+50.00	70.958	22.660
		128+75.00	70.958	23.410
		129+00.00	70.958	24.160
		129+25.00	70.958	24.899
		129+50.00	70.958	25.617
		129+75.00	70.958	26.313
		130+00.00	70.958	26.987
		130+25.00	70.958	27.640
		130+50.00	70.958	28.272
		130+75.00	70.958	28.882
		131+00.00	70.958	29.470
		131+25.00	70.958	30.036
		131+50.00	70.958	30.581
		131+75.00	70.958	31.105
		132+00.00	70.958	31.607
		132+25.00	70.958	32.087
		132+50.00	70.958	32.546
		132+75.00	70.958	32.817
		133+00.00	70.958	33.066
		133+25.00	70.958	33.293
		133+50.00	70.958	33.499
		133+75.00	70.958	33.683
		134+00.00	70.958	33.846
		134+25.00	70.958	33.987
		134+50.00	70.958	34.106
		134+75.00	70.958	34.204
		135+00.00	70.958	34.281
		135+25.00	70.958	34.336
		135+50.00	70.958	34.369
		135+59.75	70.958	34.376
		135+60.72	70.975	34.376
		135+75.00	71.213	34.380
		136+00.00	71.629	34.363
		136+25.00	72.046	34.333
		136+50.00	72.463	34.274
		136+75.00	72.879	34.191
		136+77.27	72.918	34.183
		136+87.39	73.086	-

WALL Nos. 1A

~ SBCD Station	Wall 1A Offset ~ SBCD (ft)	Top of Coping Elevation @ Wall 1A (ft)
126+34.00	10.958	14.518
126+50.00	10.958	14.809
126+75.00	10.958	15.301
127+00.00	10.958	15.840
127+25.00	10.958	16.426
127+47.62	10.958	16.996
127+50.00	10.958	17.059
127+75.00	10.958	17.738
128+00.00	10.958	18.455
128+25.00	10.958	19.210
128+50.00	10.958	19.960
128+75.00	10.958	20.710
129+00.00	10.958	21.460
129+25.00	10.958	22.199
129+50.00	10.958	22.917
129+75.00	10.958	23.613
130+00.00	10.958	24.287
130+25.00	10.958	24.940
130+50.00	10.958	25.572
130+75.00	10.958	26.182
131+00.00	10.958	26.770
131+25.00	10.958	27.336
131+50.00	10.958	27.881
131+75.00	10.958	28.405
132+00.00	10.958	28.907
132+25.00	10.958	29.387
132+50.00	10.958	29.846
132+75.00	10.958	30.283
133+00.00	10.958	30.699
133+25.00	10.958	31.093
133+50.00	10.958	31.465
133+75.00	10.958	31.816
134+00.00	10.958	32.146
134+25.00	10.958	32.454
134+50.00	10.958	32.853
134+75.00	10.958	33.259
135+00.00	10.958	33.644
135+25.00	10.958	34.007
135+50.00	10.958	34.255
135+59.75	10.958	34.336
135+62.18	10.958	34.356
135+69.33	10.958	-

WALL No. 3C

~ SBCD Station	Wall 3C Offset ~ SBCD (ft)	~ Ramp C1 Station	Wall 3C Offset ~ Ramp C1 (ft)	Top of Coping Elevation @ Wall 3C (ft)
138+09.52	75.122			-
138+14.25	75.201	33.579		
138+25.00	75.380			33.556
138+50.00	75.796			33.439
138+75.00	76.213			33.400
139+00.00	76.630			32.711
139+25.00	77.046			32.457
139+50.00	77.463			32.233
139+75.00	77.880			32.120
140+00.00	78.296			31.919
140+25.00	78.713			31.676
140+50.00	79.130			31.452
140+75.00	79.546			31.187
141+00.00	79.963			30.900
141+25.00	80.380			30.591
141+50.00	80.796			30.261
141+75.00	81.213			29.909
142+00.00	81.630			29.535
142+25.00	82.046			29.140
142+50.00	82.463			28.724
142+75.00	82.880			28.285
142+79.75	82.958	10+00.00	10.958	28.050
		10+25.00	10.958	27.550
		10+50.00	10.958	27.000
		10+75.00	10.958	26.480
		11+00.00	10.958	25.930
		11+25.00	10.958	25.360
		11+50.00	10.958	24.870
		11+75.00	10.958	24.200
		12+00.00	10.958	23.740
		12+25.00	10.958	23.430
		12+50.00	10.958	23.180
		12+75.00	10.958	23.000
		13+00.00	10.958	22.820
		13+25.00	10.958	22.610
		13+50.00	10.958	22.550
		BK 13+54.38	10.958	22.550
		AH 13+54.38	46.958	22.540
		13+75.00	46.958	22.470
		14+00.00	46.958	22.400
		14+25.00	46.958	22.327
		14+48.50	46.958	22.097
		14+50.00	47.019	22.079
		14+71.50	47.879	21.807
		14+75.00	48.019	21.758
		15+00.00	49.019	21.383
		15+25.00	50.019	20.954
		15+50.00	51.019	20.470
		15+75.00	52.019	19.932
		16+00.00	53.019	19.340
		16+25.00	54.019	18.694
		16+50.00	55.019	18.015
		16+75.00	56.019	17.341
		17+00.00	57.019	16.668
		17+25.00	58.019	15.994
		17+50.00	58.958	15.321
		18+00.00	58.958	14.014

WALL No. 3A

~ Const. Station	Wall 3A Offset ~ Const. (ft)	Top of Coping Elevation @ Wall 3A (ft)
136+94.60	45.958	-
137+05.02	45.958	34.920
137+25.00	45.958	34.910
137+31.00	45.958	34.904
137+50.00	45.958	34.802
137+75.00	45.958	34.650
138+00.00	45.958	34.477
138+25.00	45.958	34.284
138+31.00	45.958	34.234
138+50.00	45.958	34.145
138+75.00	45.958	34.010
139+00.00	45.958	33.853
139+25.00	45.958	33.676
139+50.00	45.958	33.478
139+75.00	45.958	33.260
140+00.00	45.958	33.020
140+25.00	45.958	32.760
140+50.00	45.958	32.478
140+75.00	45.958	32.176
141+00.00	45.958	31.853
141+25.00	45.958	31.510
141+50.00	45.958	31.145
141+75.00	45.958	30.760
142+00.00	45.958	30.353
142+25.00	45.958	29.926
142+50.00	45.958	29.478
142+75.00	45.958	29.010
143+00.00	45.958	28.520
143+25.00	45.958	28.042
143+50.00	45.958	27.608
143+75.00	45.958	27.217
144+00.00	45.958	26.870
144+50.00	45.958	26.304
145+00.00	45.958	25.920

NOTES:

1. Offsets are given to exterior face of proprietary wall(see Sheet W-1 for detail).
2. Top of Coping Elevation detail shown on Sheet W-1.
3. For existing and proposed ground elevations for all walls and elevations for Walls 1B and 3B through 5B, see Sheets W-2 through W-6.

REVISIONS					
Date	By	Description	Date	By	Description

	Name	Date
Drawn by	HHS	4-91
Checked by	CDW	4-91
Designed by	PCL	4-91
Checked by	GFM	4-91
Approved by	G.F. Myers	

ENGINEER OF RECORD:

LOCO:

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO. | COUNTY | PROJECT NO.

SHEET TITLE: PROPRIETARY WALL ELEVATIONS

PROJECT NAME

Drawing No.

Sheet No.

Figure 18-7

3	FLA.		W-8

WALL No. 4C

~ SBCD Station	~ Const. Station	Wall 4C Offset ~ SBCD (ft)	Wall 4C Offset ~ Const. (ft)	Top of Coping Elevation @ Wall 4C (ft)
147+75.00		46.958		24.920
148+00.00		46.958		25.150
148+25.00		46.958		25.430
148+50.00		46.958		25.730
148+75.00		46.958		26.100
149+00.00		46.958		26.510
149+25.00		46.958		26.930
149+50.00		46.958		27.420
149+75.00		46.958		27.930
150+00.00		46.958		28.470
150+25.00		46.958		29.020
150+50.00		46.958		29.520
150+75.00		46.958		30.000
151+00.00		46.958		30.490
151+18.20	150+40.22	46.958	81.958	30.787
	150+50.00		81.861	30.968
	150+75.00		81.611	31.415
	151+00.00		81.361	31.842
	151+25.00		81.111	32.246
	151+40.22		80.958	32.482
	151+50.00		80.861	32.618
	151+75.00		80.611	32.950
	152+00.00		80.361	33.250
	152+25.00		80.111	33.549
	152+50.00		79.861	33.816
	152+75.00		79.611	34.062
	153+00.00		79.361	34.286
	153+25.00		79.111	34.564
	153+50.00		78.861	34.820
	153+75.00		78.611	35.055
	154+00.00		78.361	35.268
	154+19.50		78.168	35.360
	154+23.98		78.121	-

WALL No. 4A

~ NBCD Station	~ Const. Station	Wall 4A Offset ~ NBCD (ft)	Wall 4A Offset ~ Const. (ft)	Top of Coping Elevation @ Wall 4A (ft)
143+75.00		14.762		21.780
144+00.00		13.929		22.180
144+25.00		13.095		22.580
144+50.00		12.262		23.010
144+75.00		11.429		23.380
145+00.00		10.958		23.770
145+25.00		10.958		24.060
145+50.00		10.958		24.370
145+75.00		10.958		24.650
146+00.00		10.958		24.940
146+25.00		10.958		25.210
146+50.00		10.958		25.520
146+75.00		10.958		25.900
147+00.00		10.958		26.320
147+25.00		10.958		26.790
147+50.00		10.958		27.270
147+75.00		10.958		27.810
148+00.00		10.958		28.400
148+25.00		10.958		28.920
148+50.00		10.958		29.430
148+75.00		10.958		29.930
149+00.00		10.958		30.400
149+25.00		10.958		30.840
149+50.00		10.958		31.250
149+75.00		10.958		31.650
150+00.00		10.958		32.050
150+25.00		10.958		32.400
150+50.00		10.958		32.700
150+75.00		10.958		33.040
151+00.00		10.958		33.100
151+25.00		10.958		33.600
151+50.00		10.958		33.850
151+74.96	152+97.53	10.958	81.958	34.058
	153+00.00		81.918	34.039
	153+25.00		81.501	34.409
	153+50.00		81.084	34.699
	153+75.00		80.668	34.966
	154+00.00		80.251	35.211
	154+22.21		79.881	35.320
	154+26.87		79.803	-

WALL Nos. 5C AND 5A

~ Const. Station	Wall 5C Offset ~ Const. (ft)	Top of Coping Elevation @ Wall 5C (ft)	Wall 5A Offset ~ Const. (ft)	Top of Coping Elevation @ Wall 5A (ft)
155+61.68	76.744	-		
155+66.35	76.697	35.640	77.478	35.617
155+66.48	76.696	35.640	77.476	-
155+71.15	76.649	35.637	77.398	35.615
155+75.00	76.611	35.634	77.334	35.612
156+00.00	76.361	35.600	76.918	35.581
156+25.00	76.111	35.470	76.501	35.458
156+50.00	75.861	35.311	76.084	35.311
156+75.00	75.611	35.145	75.668	35.143
157+00.00	75.361	34.950	75.251	34.955
157+25.00	75.111	34.803	74.834	34.817
157+50.00	74.861	34.646	74.418	34.659
157+75.00	74.611	34.462	74.001	34.480
158+00.00	74.361	34.256	73.584	34.279
158+25.00	74.111	34.029	73.168	34.057
158+50.00	73.861	33.780	72.751	33.813
158+75.00	73.611	33.510	72.334	33.548
159+00.00	73.361	33.218	71.918	33.261
159+25.00	73.111	32.905	71.501	32.953
159+50.00	72.861	32.570	71.084	32.623
159+75.00	72.611	32.214	70.668	32.272
160+00.00	72.361	31.836	70.251	31.899
160+17.53	72.186	31.558	69.958	31.625
160+25.00	72.111	31.437	69.958	31.501
160+50.00	71.861	31.016	69.958	31.073
160+75.00	71.611	30.574	69.958	30.623
161+00.00	71.361	30.110	69.958	30.152
161+25.00	71.111	29.625	69.958	29.659
161+50.00	70.861	29.118	69.958	29.145
161+75.00	70.611	28.590	69.958	28.609
162+00.00	70.361	28.040	69.958	28.052
162+25.00	70.111	27.469	69.958	27.473
162+40.22	69.958	27.110	69.958	27.110
162+50.00	69.958	26.873	69.958	26.873
162+75.00	69.958	26.251	69.958	26.251
163+00.00	69.958	25.608	69.958	25.608
163+25.00	69.958	24.943	69.958	24.943
163+50.00	69.958	24.257	69.958	24.257
163+75.00	69.958	23.549	69.958	23.549
164+00.00	69.958	22.820	69.958	22.820
164+25.00	69.958	22.080	69.958	22.080
164+50.00	69.958	21.340	69.958	21.340
164+75.00	69.958	20.600	69.958	20.600
165+00.00	69.958	19.879	69.958	19.879

NOTES:

1. Offsets are given to exterior face of proprietary wall (see Sheet W-1 for details).
2. Top of Coping Elevation detail shown on Sheet W-1.
3. For existing and proposed ground elevations for all walls and elevations for Walls 1B and 3B through 5B, see Sheets W-2 through W-6.

REVISIONS

Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	HHS	4-91
Checked by	CDW	4-91
Designed by	PCL	4-91
Checked by	GFM	4-91
Approved by	G.F. Myers	

ENGINEER OF RECORD:

LOCO:

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO. | COUNTY | PROJECT NO.

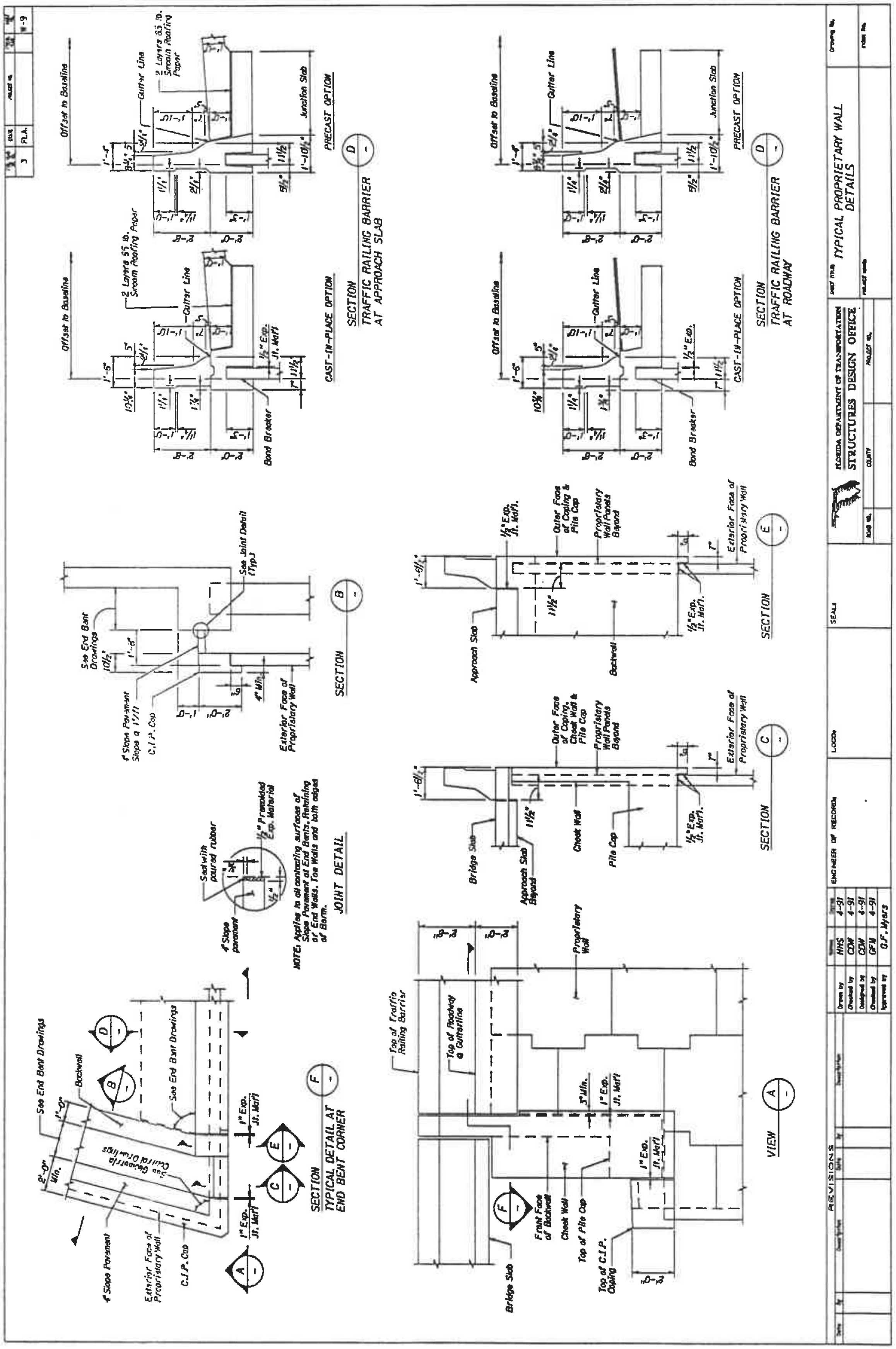
SHEET TITLE: PROPRIETARY WALL ELEVATIONS

PROJECT NAME:

Drawing No.

Index No.

Figure 18-8



FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE
TYPICAL PROPRIETARY WALL DETAILS
SECTION TRAFFIC RAILING BARRIER AT APPROACH SLAB
SECTION TRAFFIC RAILING BARRIER AT ROADWAY
CAST-IN-PLACE OPTION
PRECAST OPTION
JOINT DETAIL
SECTION TYPICAL DETAIL AT END BENT CORNER
VIEW
SECTION
REVISIONS

Figure 18-9

SAMPLE INVITATION LETTER

(When Proprietary Designs are Included in Contract Plans.)

State Project No. ____________________________

____

FAP No. ____________________________

W.P.I. No. ____________________________

____

____

Attn: ____________________________

Gentlemen:

The Florida Department of Transportation would like to extend your company an invitation to participate in alternate designs for the retaining walls to be included in the subject project.

Your participation will involve submitting to us three (3) sets of prints of your final design for our review, and upon our notification of acceptance, the submittal of drawings on polyester film material for inclusion in the final plan assembly. The drawings shall be made on standard size (36" x 24") sheets.

The subject project has been scheduled for letting in ________________; therefore, the final plans and design calculations should be submitted not later than ________________, and the tracings should be submitted not later than ___ ________________ in order to meet this schedule.

We are enclosing the following material for your use:

- o Roadway plans of the area.
- o Foundation information and report.
- o Wall alignment (vertical and horizontal), special details and the locations of sign structures, drainage structures and utilities.

Your design must conform strictly with the above information.

In addition, the following criteria and minimum safety factors shall be used (as applicable).

ALLOWABLE BEARING CAPACITY = ______ lbs./sq.ft.

<u>SAND FILL</u>	
Overturning	= 2.0
Sliding	= 1.5
Internal pullout	= 1.5 (Allowable Deformation 0.75 inches)
Bearing capacity	= 2.5
Overall stability	= 1.5
Steel Soil Reinforcement (Bars and Straps)	= 0.55 Fy at end of design life, and 0.50 Fu at net section of bolted connection
Steel Soil Reinforcement (Grids and Bar Mats)	= 0.47 Fy at end of design life
Design Based ϕ	= 30^{o}
Backfill Density	= 105 lbs/C.F. (Min) Moist wt. in place

Figure 18-10

Maximum Pullout Factors:	
Ribbed Strips, f^*_{MAX}	= 1.5
Grids and Bar Mats, $N_{P\ MAX}$	= 30

<u>Limerock Fill</u>

Overturning	= 2.0
Sliding	= 1.5
Internal pullout	= 1.5 (Allowable Deformation 0.75 inches)
Bearing capacity	= 2.5
Overall stability	= 1.5
Steel Soil Reinforcement (Bars and Straps)	= 0.55 Fy at end of design life, and 0.50 Fu at net section of bolted connection
Steel Soil Reinforcement (Grids and Bar Mats)	= 0.47 Fy at end of design life
Design Based ϕ	= 34^{o}
Backfill Density	= 115 lbs/C.F. (Min) Moist wt. in place

Maximum Pullout Factors:	
Ribbed Strips, f^*_{MAX}	= 1.5
Grids and Bar Mats, $N_{P\ MAX}$	= 30

Your acknowledgement of this invitation and acceptance/or rejection will be appreciated. In the event of non-acceptance, the enclosed material may be discarded. Also, your firm will not be considered for a Value Engineering Concept Proposal for this project at a later date.

If there are any questions, please advise.

Very truly yours,

Enclosures

Figure 18-11

SAMPLE LETTER

(For Control Plans Only.)

State Project No. ____________________________
____
FAP No. ______________________________________
W.P.I. No. ___________________________________

____
____
____

Attn: ____

Gentlemen:

The Florida Department of Transportation is anticipating letting the subject project in ______________.

The enclosed information is being transmitted for your information and bid preparation. If your company has any innovative methods or designs for this project, they should be submitted before __________ for our review.

We are enclosing the following material for your use:

- o Roadway plans of the area.
- o Foundation information and report.
- o Wall alignment (vertical and horizontal), special details and the locations of sign structures, drainage structures and utilities.

Your design must conform strictly with the above information.

In addition, the following criteria and minimum safety factors shall be used (as applicable).

ALLOWABLE BEARING CAPACITY = ______ lbs./sq.ft.

<u>SAND FILL</u>	
Overturning	= 2.0
Sliding	= 1.5
Internal pullout	= 1.5 (Allowable Deformation 0.75 inches)
Bearing capacity	= 2.5
Overall stability	= 1.5
Steel Soil Reinforcement (Bars and Straps)	= 0.55 Fy at end of design life, and 0.50 Fu at net section of bolted connection
Steel Soil Reinforcement (Grids and Bar Mats)	= 0.47 Fy at end of design life
Design Based ϕ	= 30^0
Backfill Density	= 105 lbs/C.F. (Min) Moist wt. in place
Maximum Pullout Factors:	
Ribbed Strips, f^*_{MAX}	= 1.5
Grids and Bar Mats, $N_{P\ MAX}$	= 30

Figure 18-12

Overturning	= 2.0
Sliding	= 1.5
Internal pullout	= 1.5 (Allowable Deformation 0.75 inches)
Bearing capacity	= 2.5
Overall stability	= 1.5
Steel Soil Reinforcement (Bars and Straps)	= 0.55 Fy at end of design life, and 0.50 Fu at net section of bolted connection
Steel Soil Reinforcement (Grids and Bar Mats)	= 0.47 Fy at end of design life
Design Based ϕ	= 34°
Backfill Density	= 115 lbs/C.F. (Min) Moist wt. in place
Maximum Pullout Factors:	
Ribbed Strips, f^*_{MAX}	= 1.5
Grids and Bar Mats, $N_{P\ MAX}$	= 30

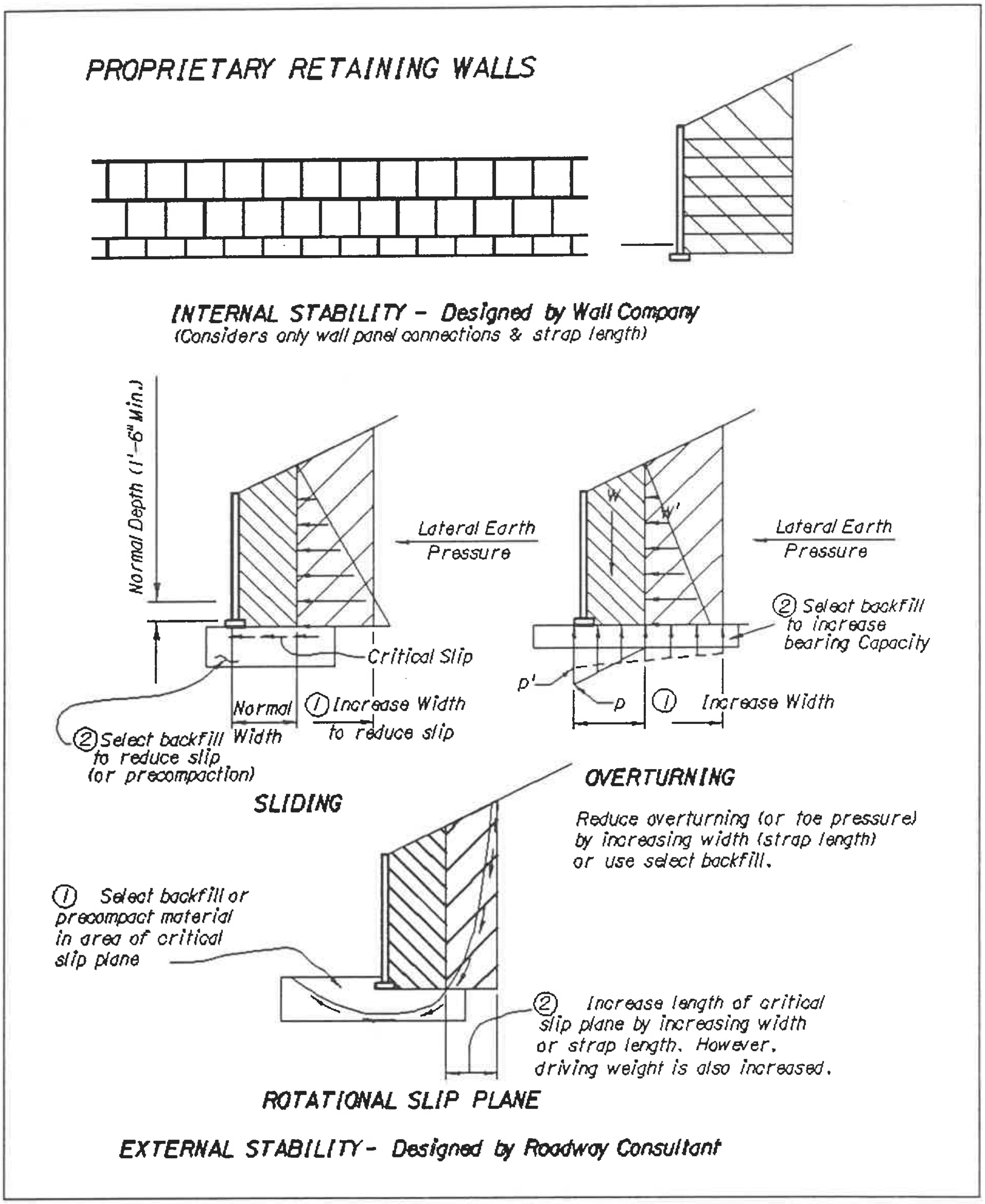
Your acknowledgement of receipt of this package and applicability of your wall for this/these site(s) is appreciated. In the event that you do not wish to participate in this project, the enclosed material may be discarded. Also, your firm will not be considered for a Value Engineering Concept Proposal for this project at a later date.

If there are any questions, please advise.

Very truly yours,

Enclosures

Figure 18-13



PROPRIETARY RETAINING WALLS
INTERNAL STABILITY - Designed by Wall Company
(Considers only wall panel connections & strap length)
Normal Depth (1'-6" Min.)
Lateral Earth Pressure
Critical Slip
Normal Width
① Increase Width to reduce slip
② Select backfill to reduce slip (or precompaction)
SLIDING
W
W'
Lateral Earth Pressure
② Select backfill to increase bearing Capacity
p'
p
① Increase Width
OVERTURNING
Reduce overturning (or toe pressure) by increasing width (strap length) or use select backfill.
① Select backfill or precompact 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 Roadway Consultant

Figure 18-14

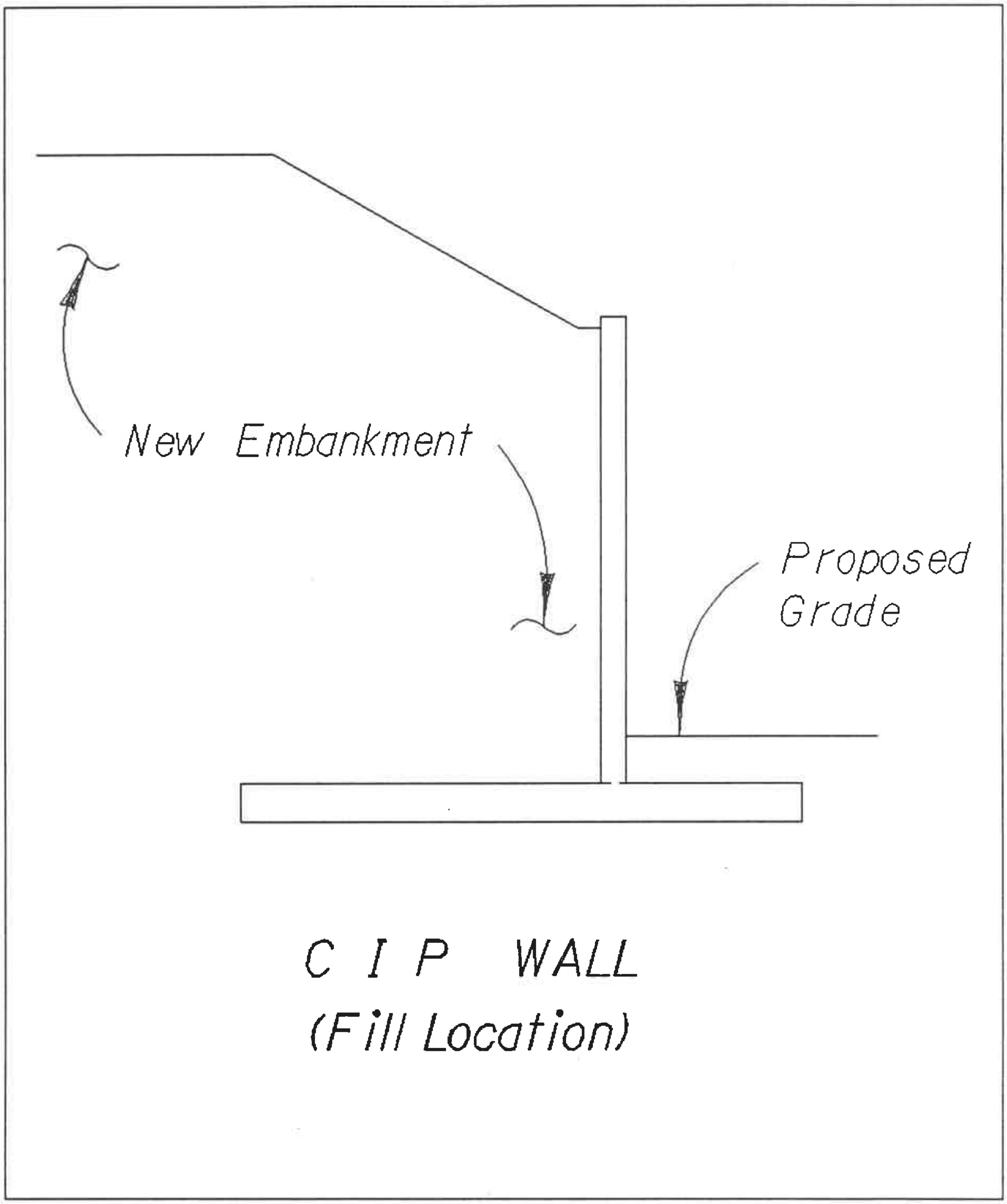


Figure 18-15

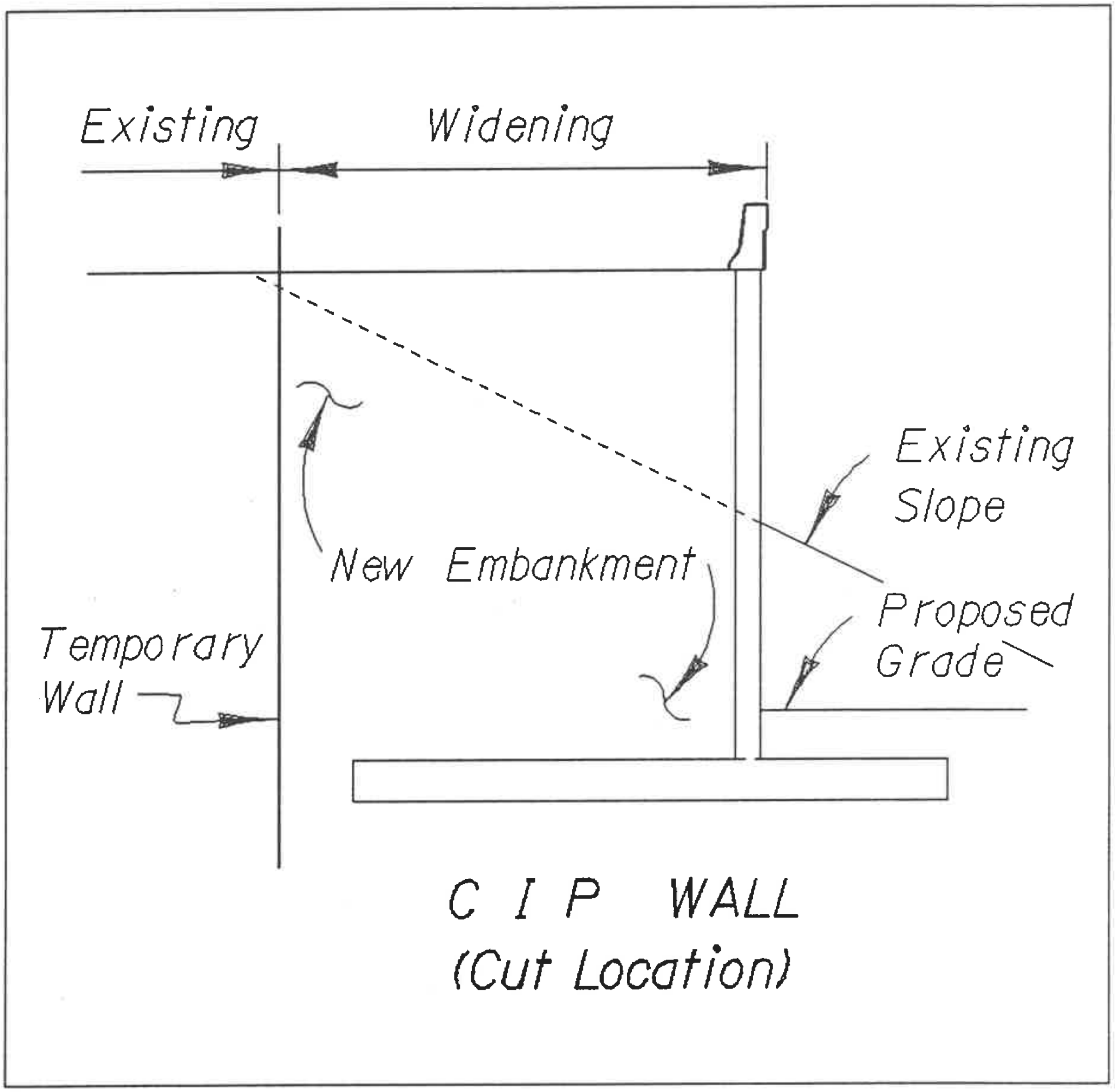

Existing
Widening
Existing
Slope
New Embankment
Proposed
Grade
Temporary
Wall
C I P WALL
(Cut Location)

Figure 18-16

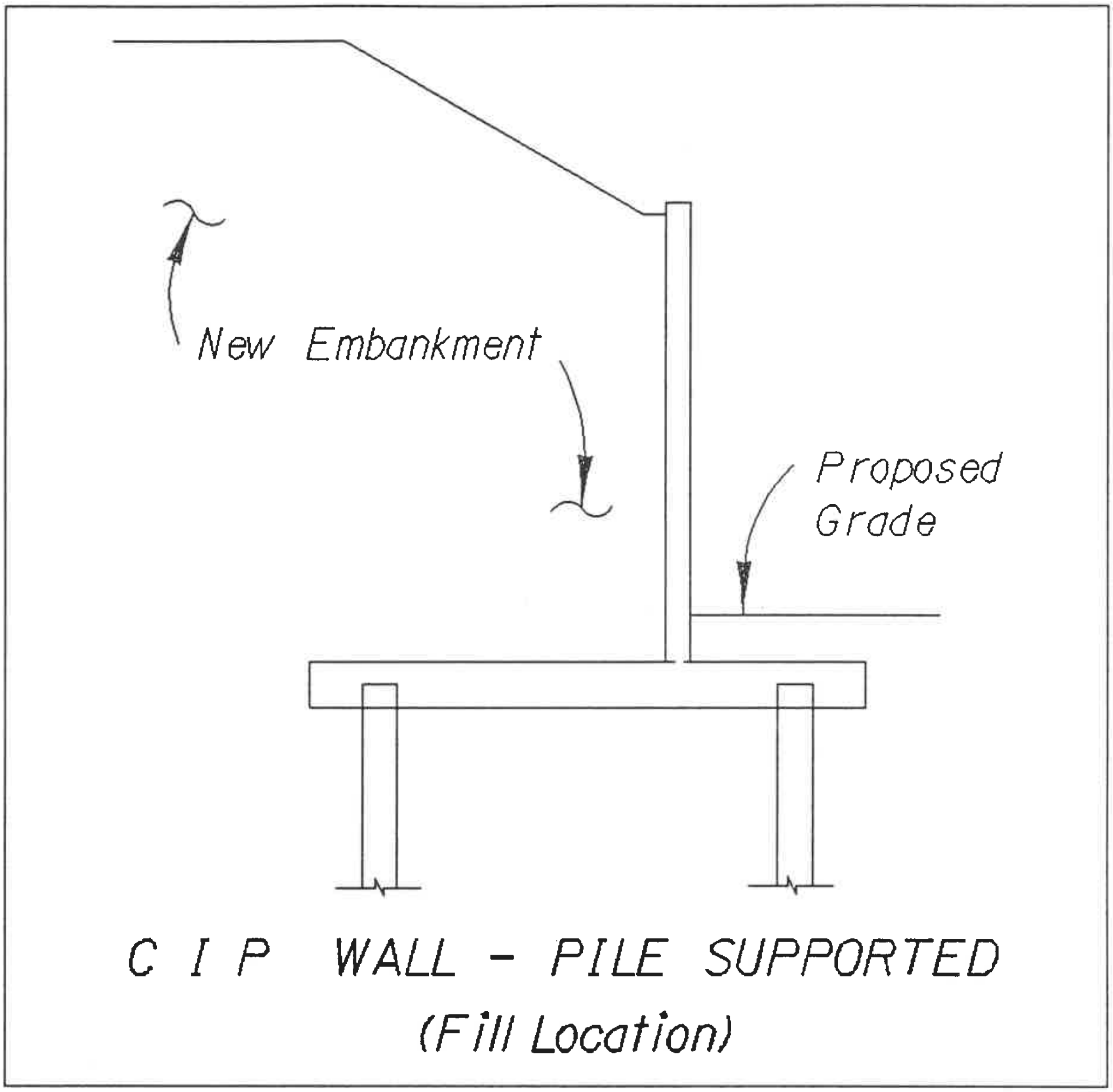
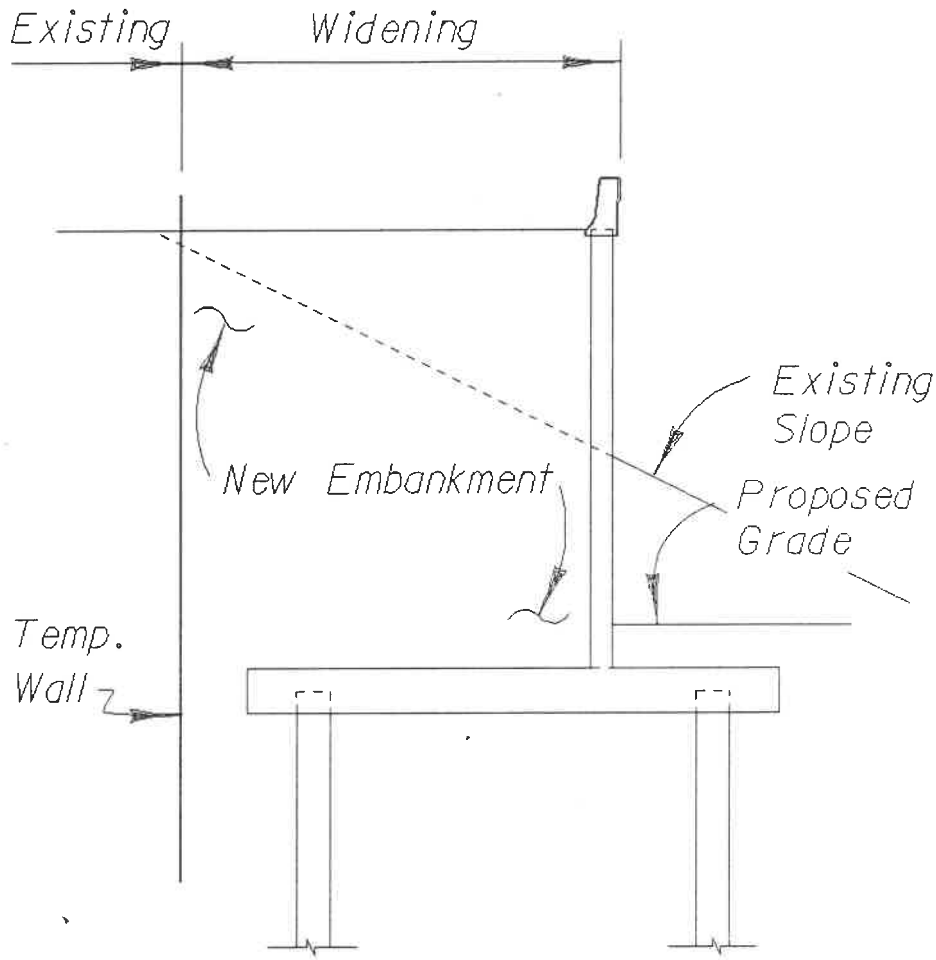

New Embankment
Proposed
Grade
C I P WALL - PILE SUPPORTED
(Fill Location)

Figure 18-17



C I P WALL - PILE SUPPORTED
(Cut Location)

Figure 18-18

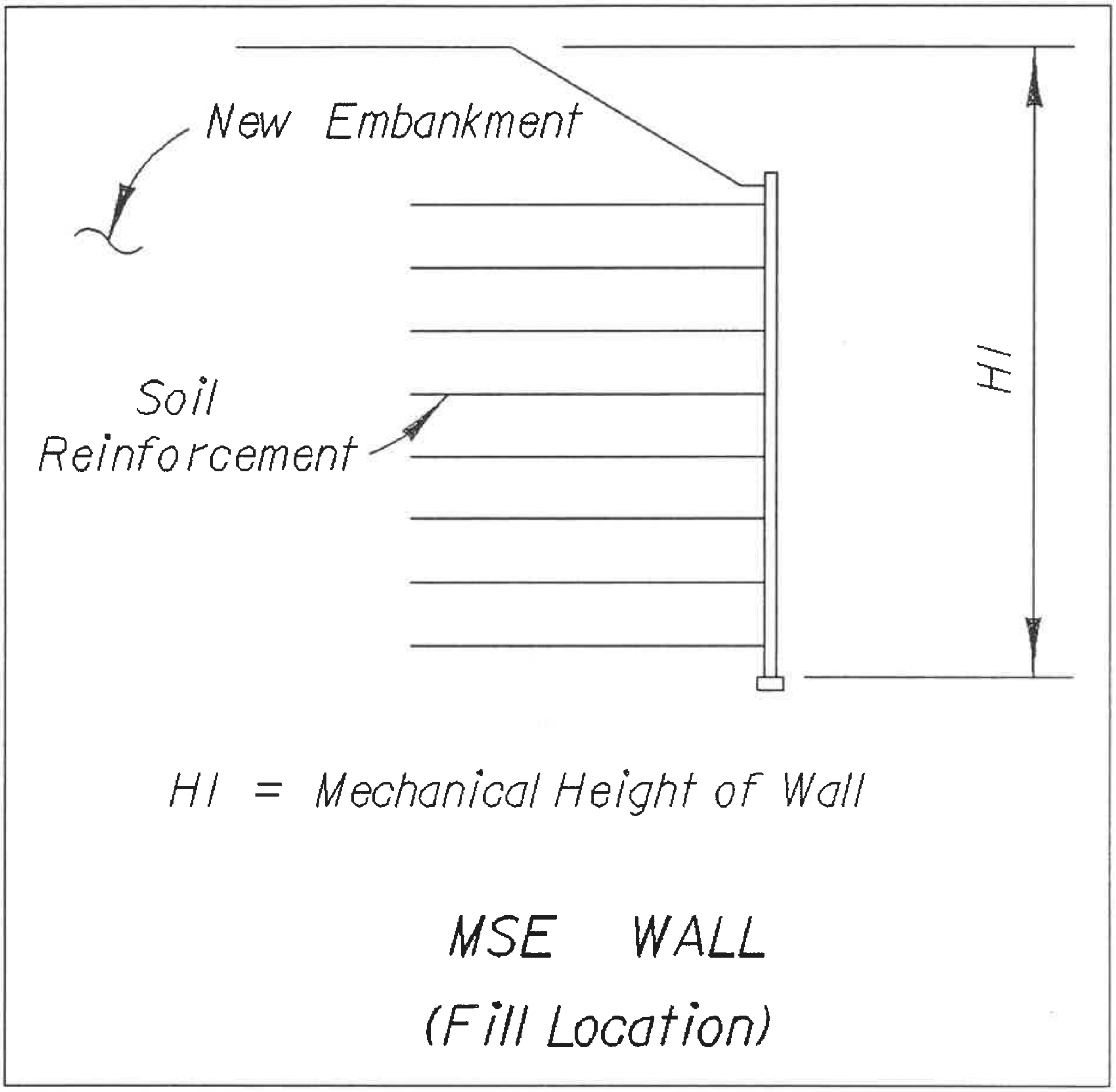

New Embankment
Soil
Reinforcement
HI
HI = Mechanical Height of Wall
MSE WALL
(Fill Location)

Figure 18-19

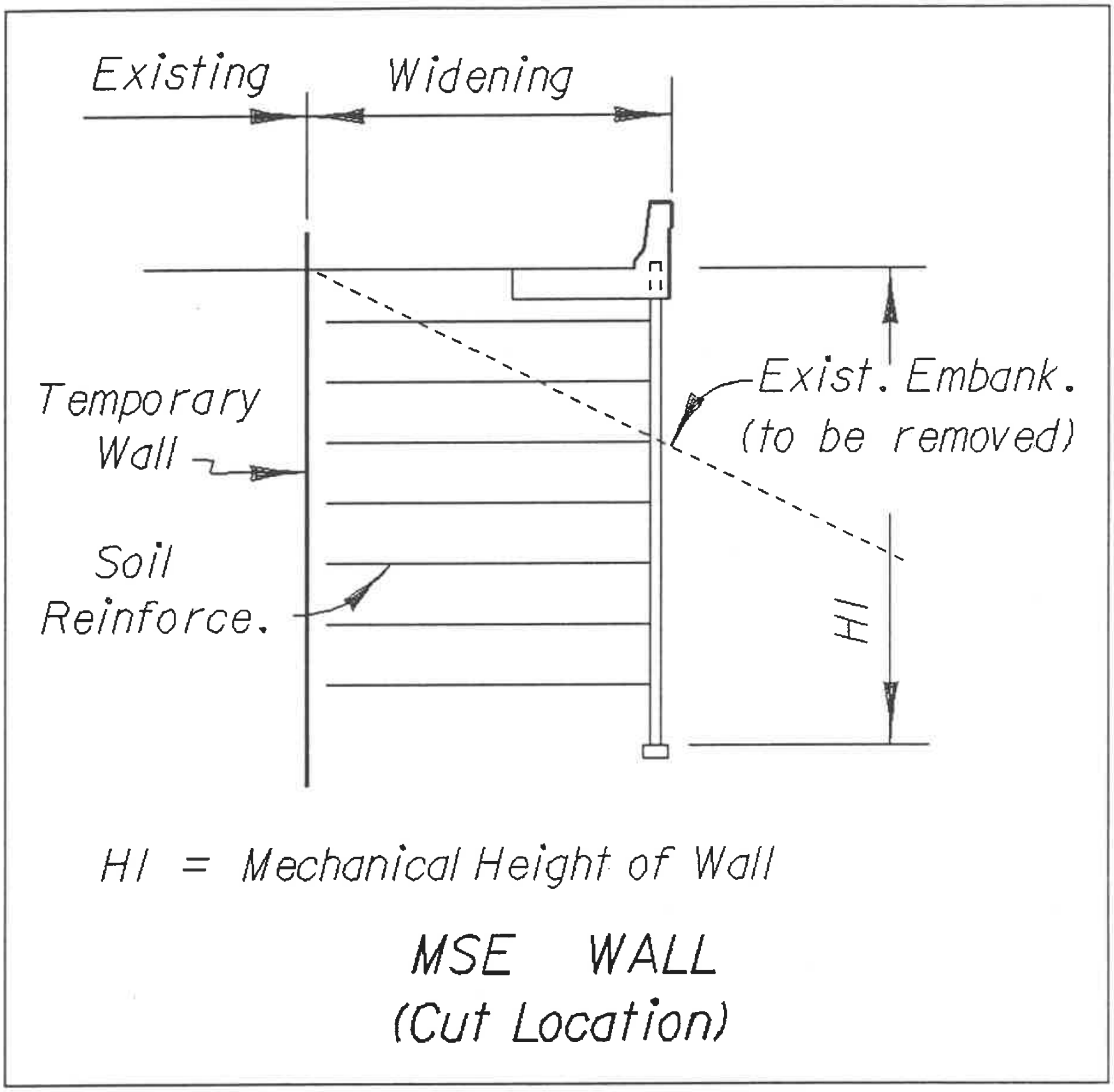

Existing
Widening
Temporary Wall
Soil Reinforce.
Exist. Embank. (to be removed)
HI
HI = Mechanical Height of Wall
MSE WALL
(Cut Location)

Figure 18-20

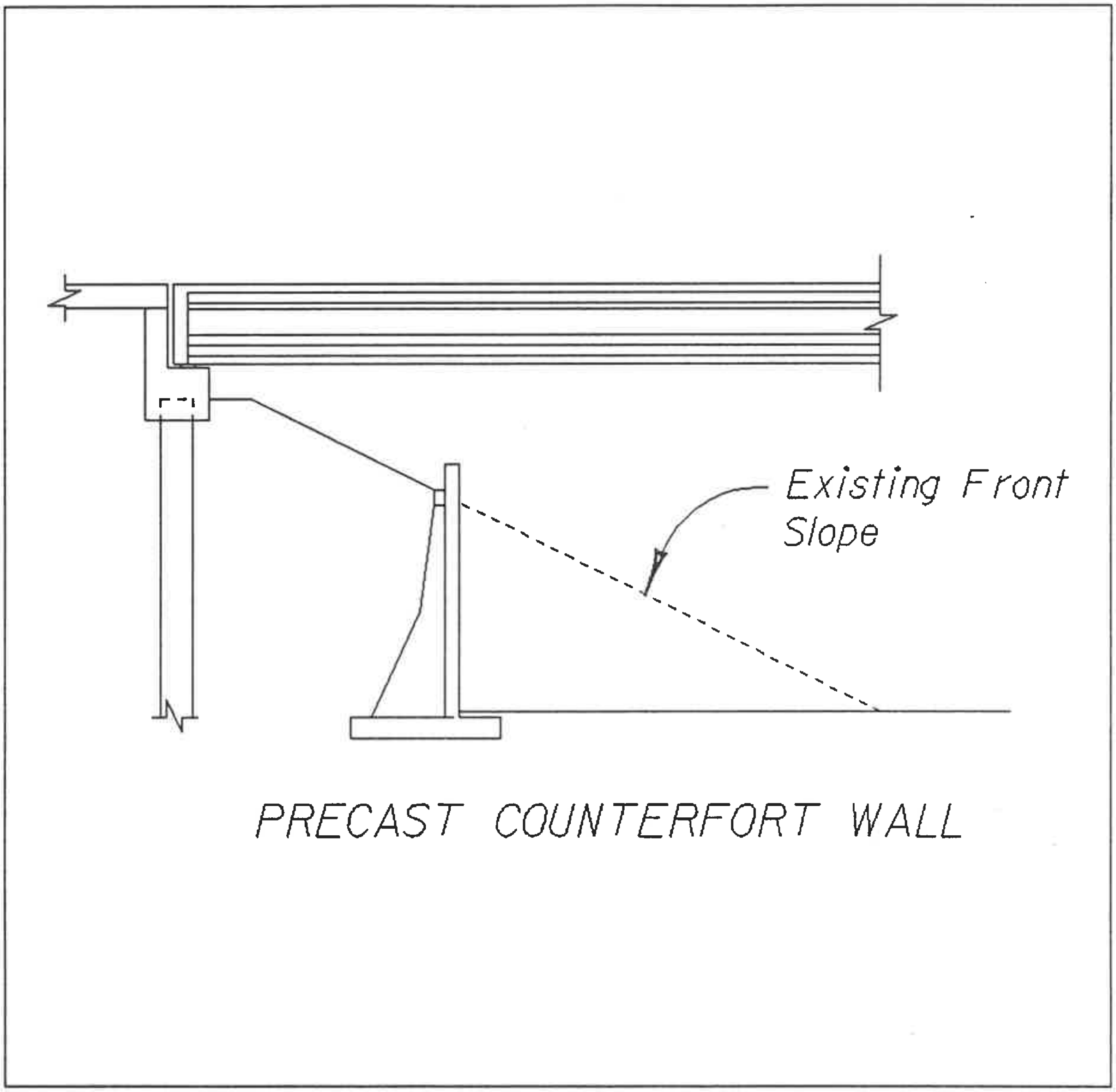
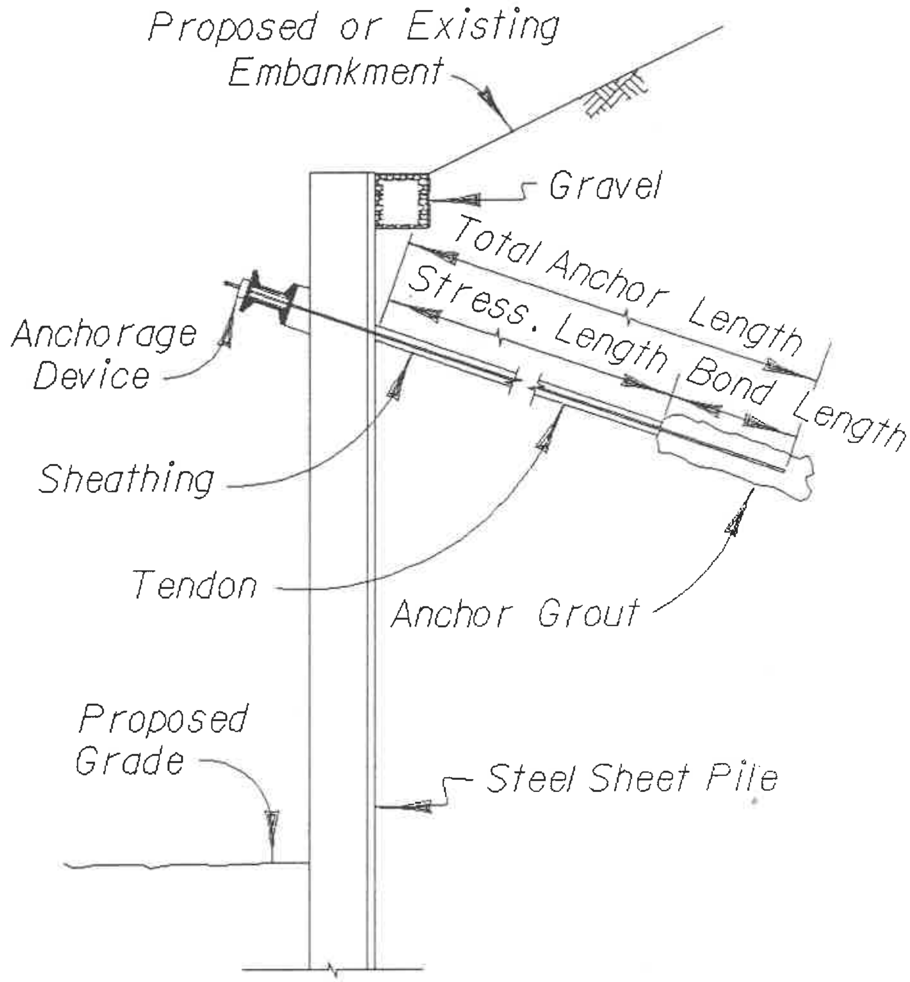


Figure 18-21



TIEBACK COMPONENTS
(Steel Sheet Piles)

Figure 18-22

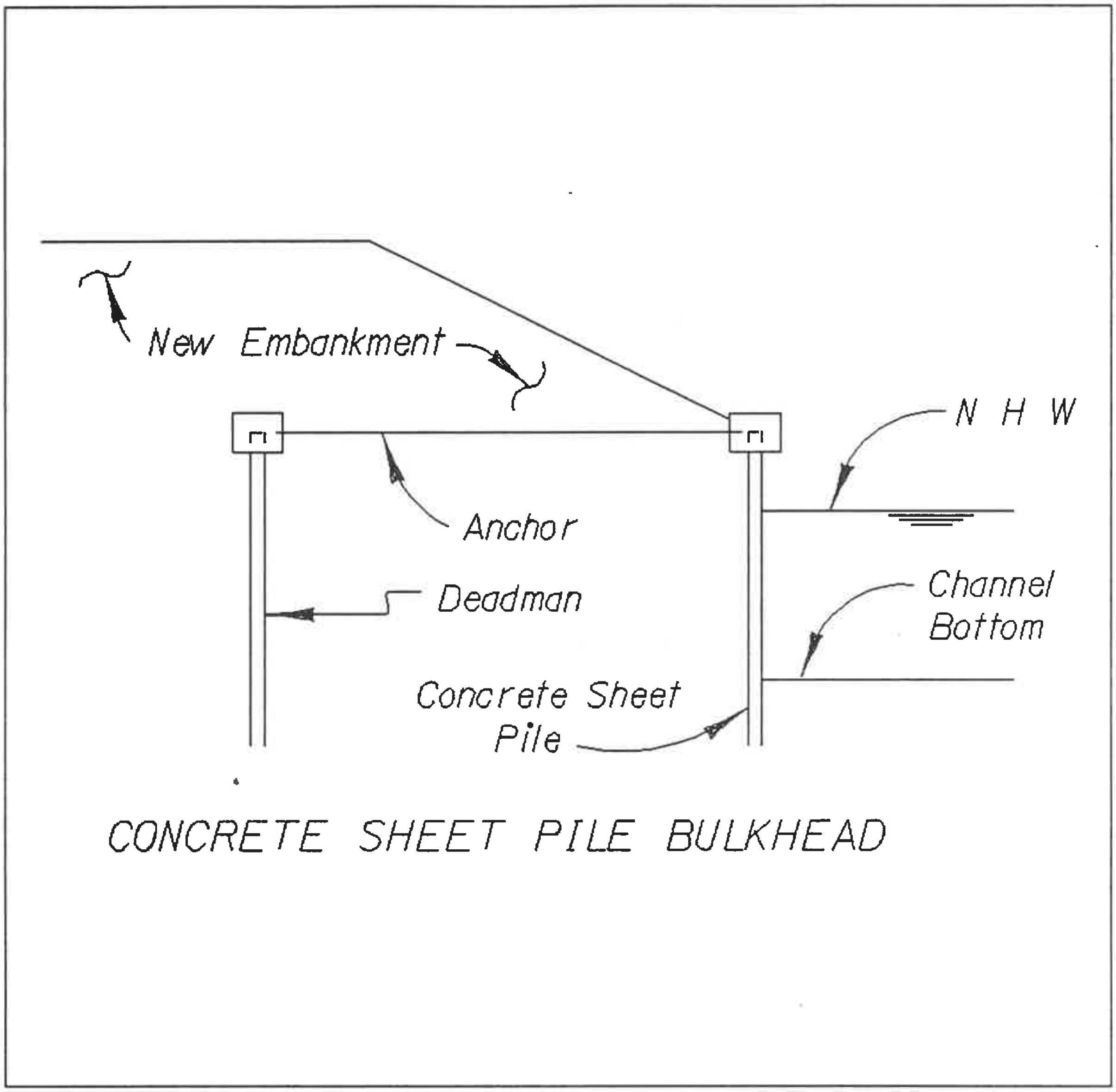

New Embankment
N H W
Anchor
Deadman
Channel
Bottom
Concrete Sheet
Pile
CONCRETE SHEET PILE BULKHEAD

Figure 18-23

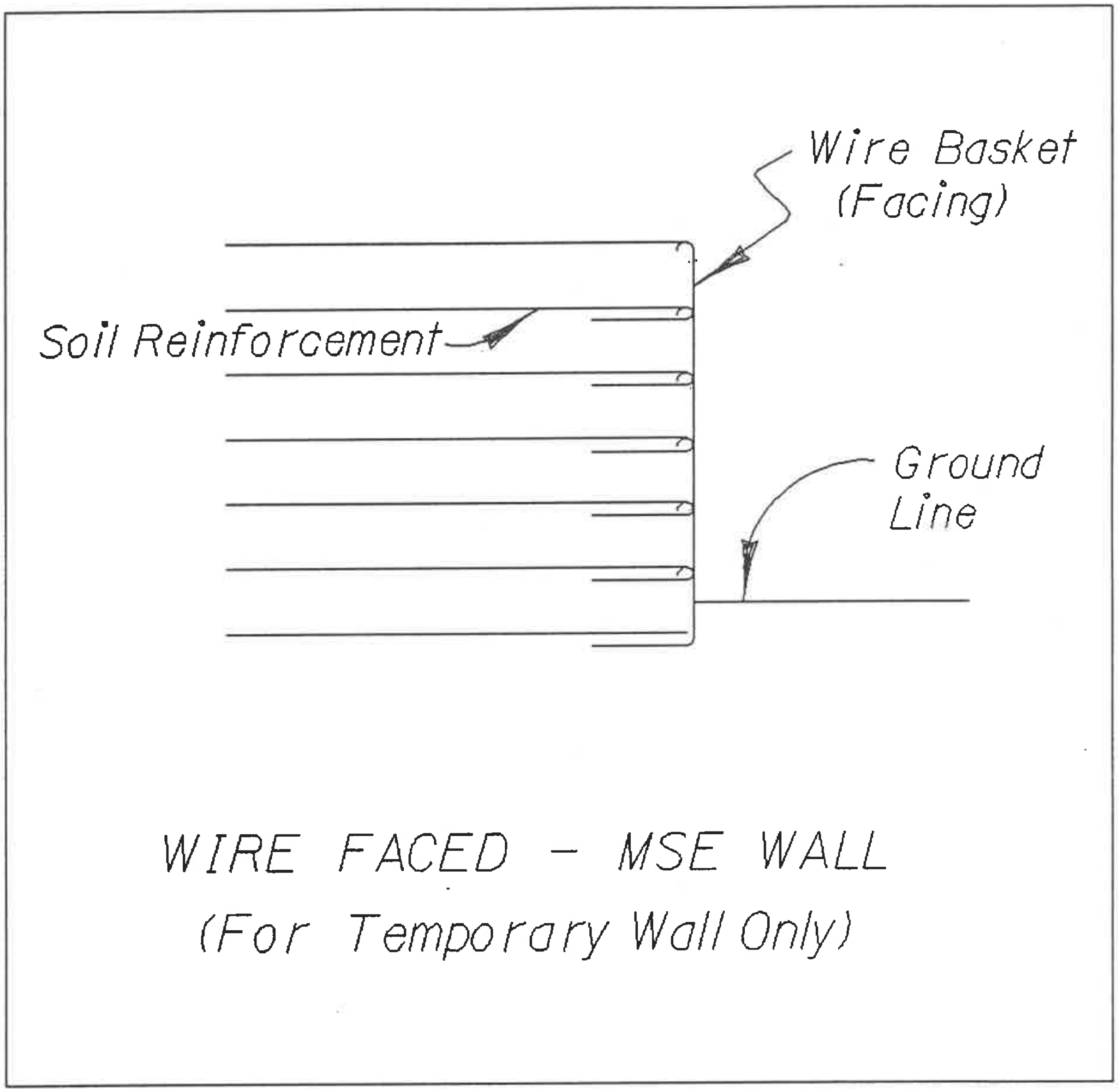

Wire Basket
(Facing)
Soil Reinforcement
Ground
Line
WIRE FACED - MSE WALL
(For Temporary Wall Only)

Figure 18-24

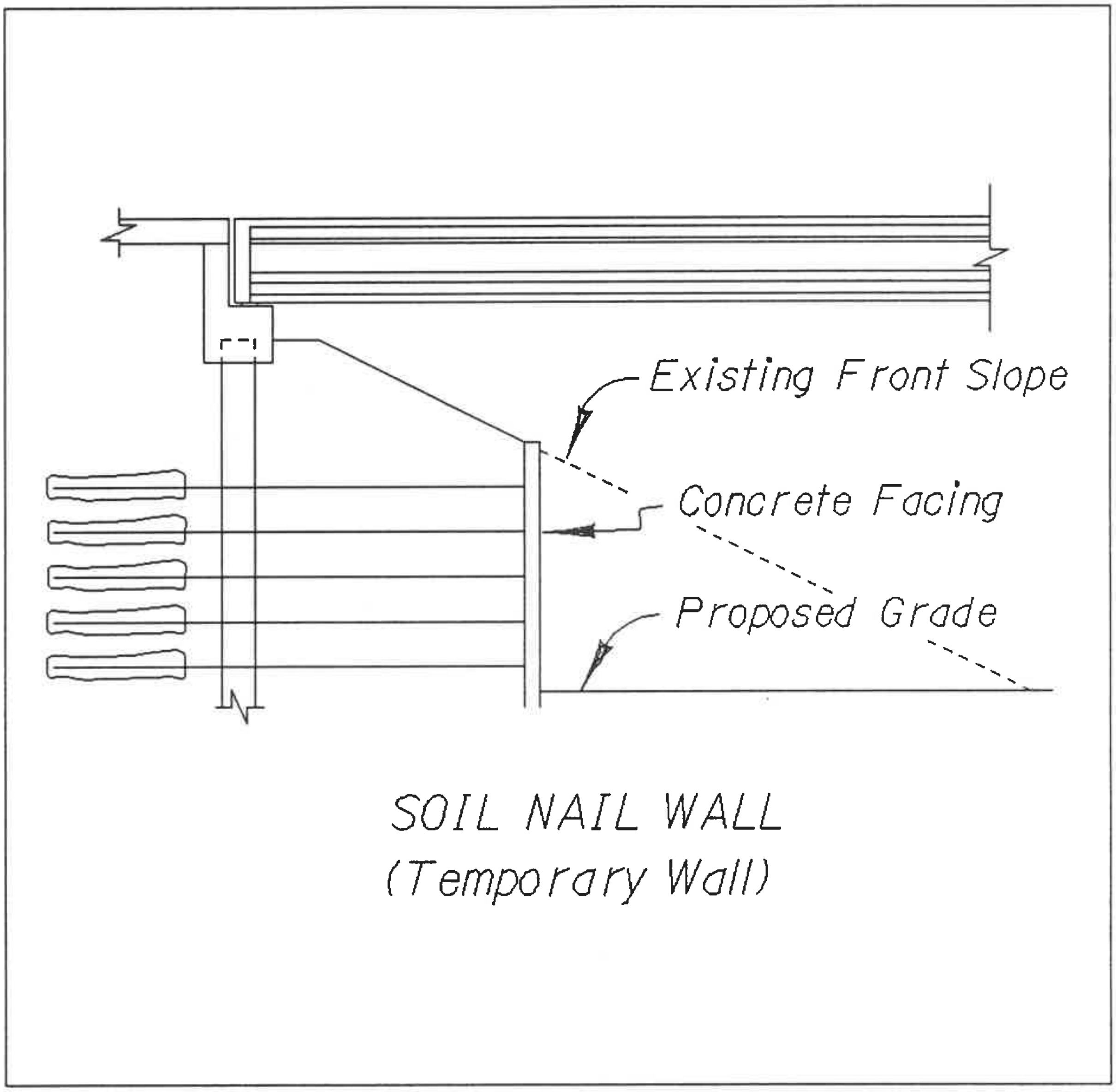

Existing Front Slope
Concrete Facing
Proposed Grade
SOIL NAIL WALL
(Temporary Wall)

Figure 18-25

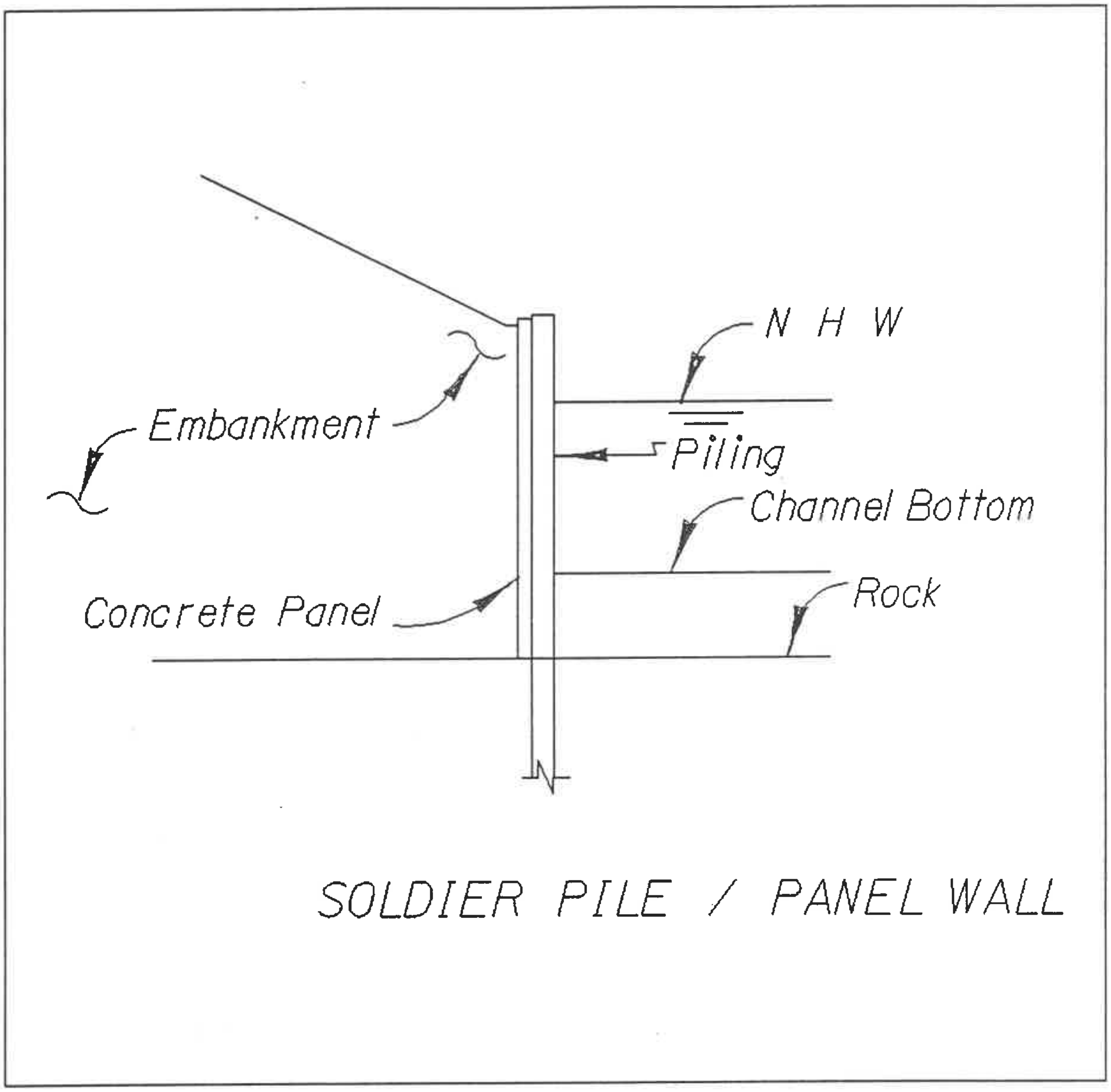

N H W
Embankment
Piling
Channel Bottom
Concrete Panel
Rock
SOLDIER PILE / PANEL WALL

Figure 18-26

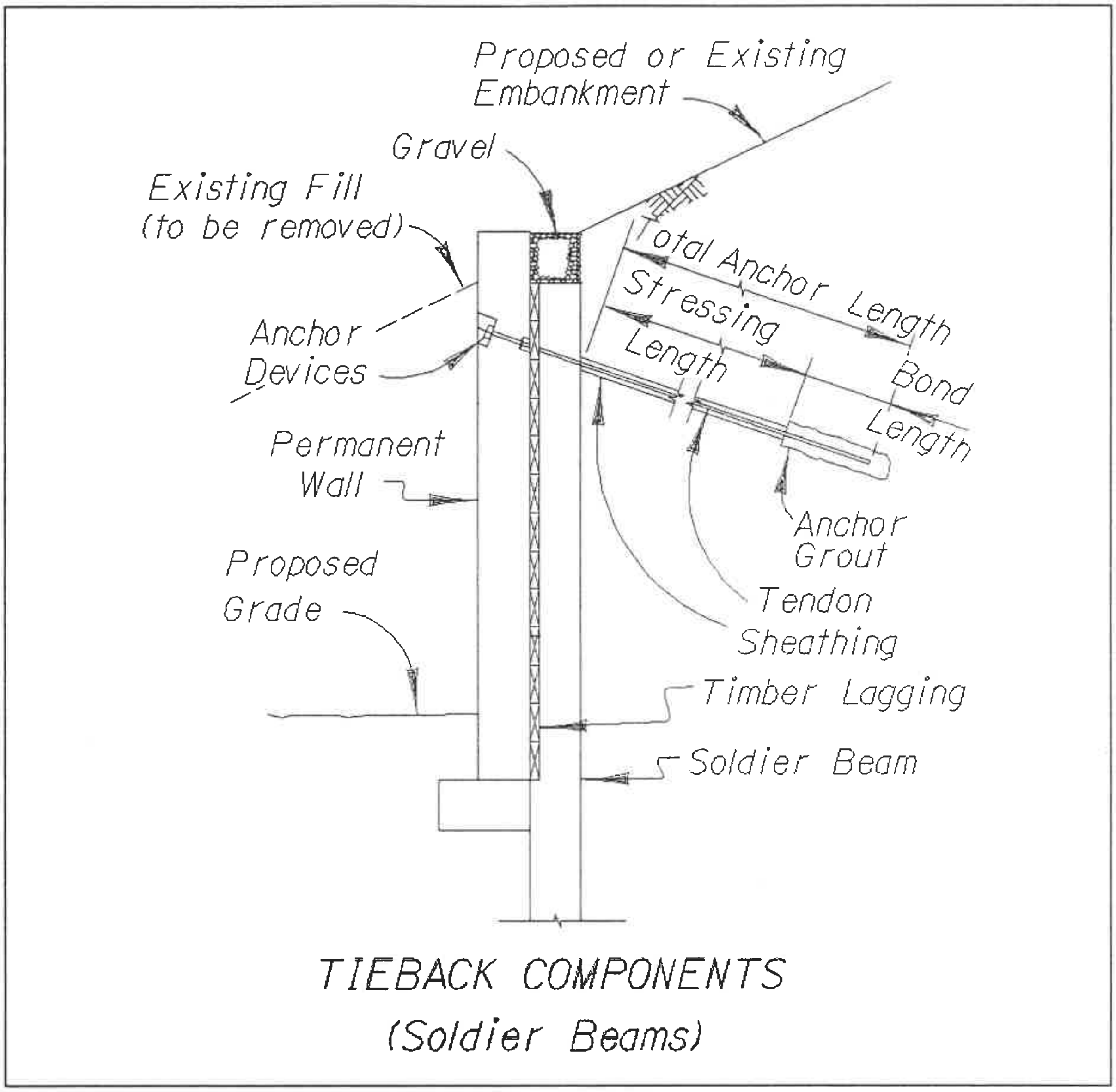
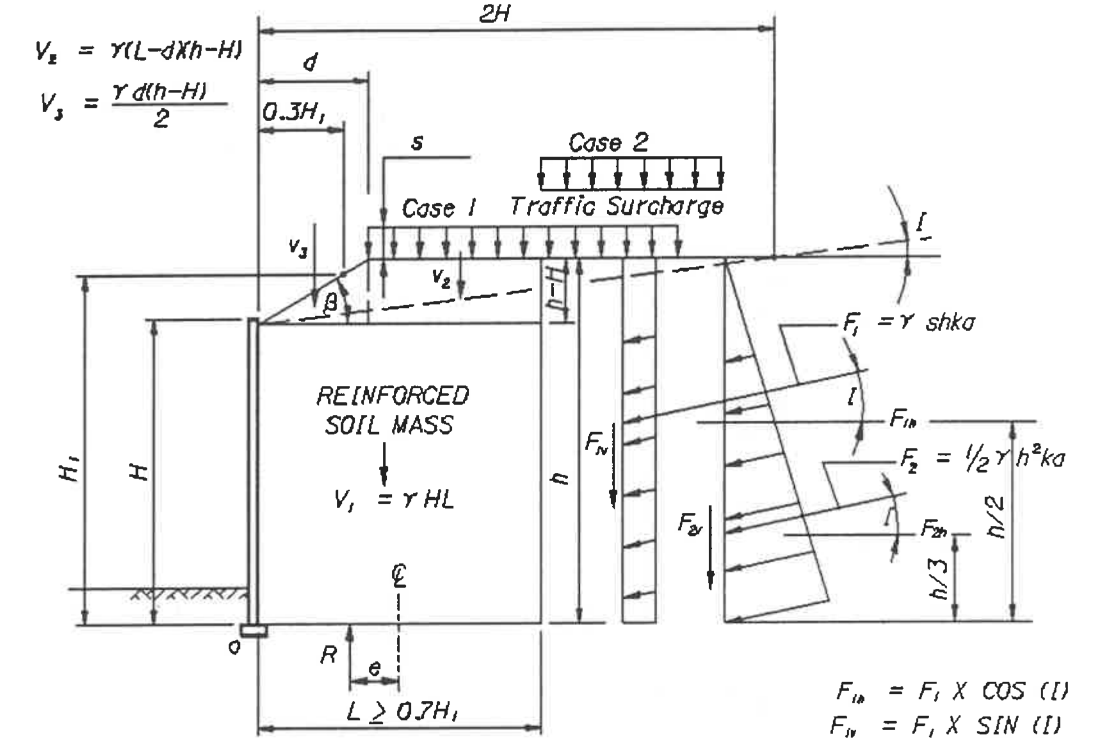

Proposed or Existing
Embankment
Gravel
Existing Fill
(to be removed)
Anchor
Devices
Permanent
Wall
Proposed
Grade
Total Anchor Length
Stressing
Length
Bond
Length
Anchor
Grout
Tendon
Sheathing
Timber Lagging
Soldier Beam
TIEBACK COMPONENTS
(Soldier Beams)

Figure 18-27

BROKEN BACK BACKFILL CASE WITH TRAFFIC SURCHARGE



$F_{1h} = F_1 \times COS\ (I)$
$F_{1v} = F_1 \times SIN\ (I)$
$F_{2h} = F_2 \times COS\ (I)$
$F_{2v} = F_2 \times SIN\ (I)$

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

$$H_1 = H + \frac{Tan\,\beta \times 0.3H}{(1-0.3\ Tan\,\beta)} \quad \text{(MECHANICAL HEIGHT)}$$

K_a FOR RANDOM FILL : $K_a = COS\ I \left[\frac{COS\ I - \sqrt{COS^2 I - COS^2 \phi}}{COS\ I + \sqrt{COS^2 I - COS^2 \phi}}\right]$

SAFETY FACTOR AGAINST OVERTURNING (MOMENTS ABOUT POINT O):

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

SAFETY FACTOR AGAINST SLIDING:

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

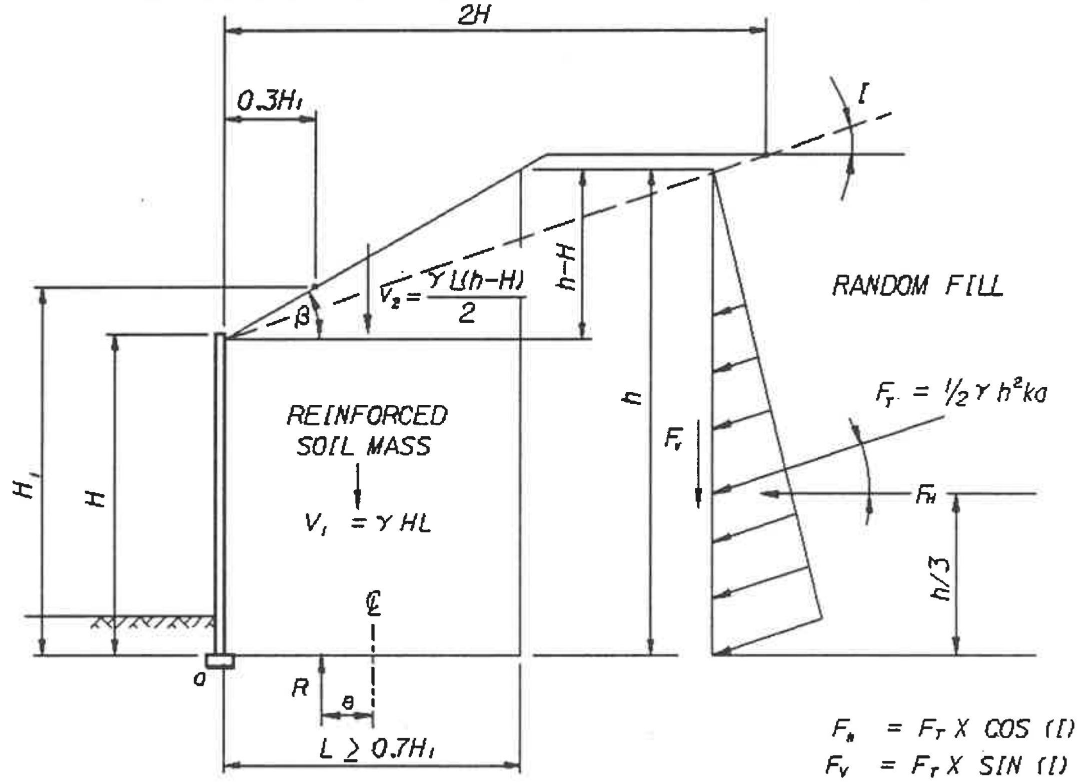
ϕ = Friction Angle of Backfill or Foundation, Whichever is lowest.

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

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

Figure 18-28

BROKEN BACK BACKFILL CASE WITHOUT TRAFFIC SURCHARGE



$F_H = F_T \times \cos(I)$

$F_V = F_T \times \sin(I)$

FOR INFINITE SLOPE $I = \beta$

$$H_1 = H + \frac{\tan\beta \times 0.3H}{(1 - 0.3\tan\beta)} \quad \text{(MECHANICAL HEIGHT)}$$

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:

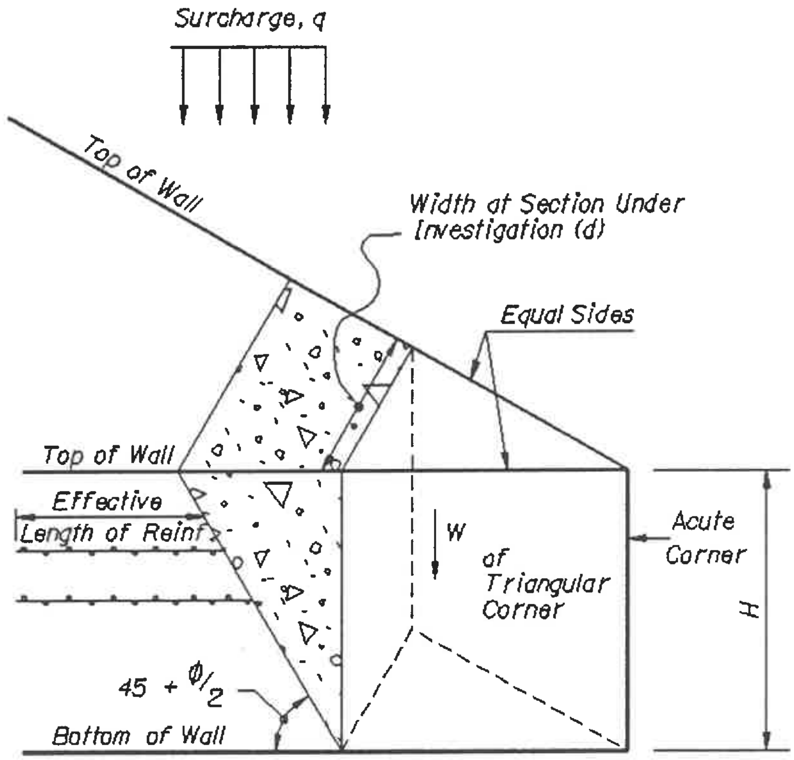
$$S.F.(S) = \frac{\Sigma \text{ Horizontal Resisting Force } (s)}{\Sigma \text{ Horizontal Driving Force } (s)} = \frac{R \tan\phi}{F_H} \geq 1.5$$

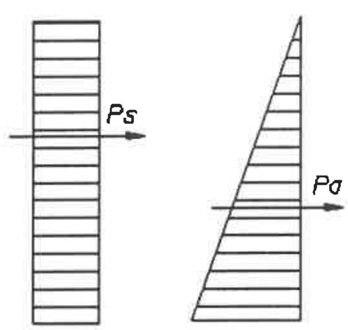
ϕ = Friction Angle of Backfill or Foundation, Whichever is lowest.

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

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

Figure 18-29





$Pa = \frac{1}{2} \gamma H^2 \; Ka \times$ Width at Section Under Investigation

$Ps = q \times H \times Ka \times$ Width at Section Under Investigation

F.S. Sliding (Acute Corner) $= \dfrac{W \times TAN\ \phi + R}{Pa + Ps} \geq 1.5$ 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.

Figure 18-30

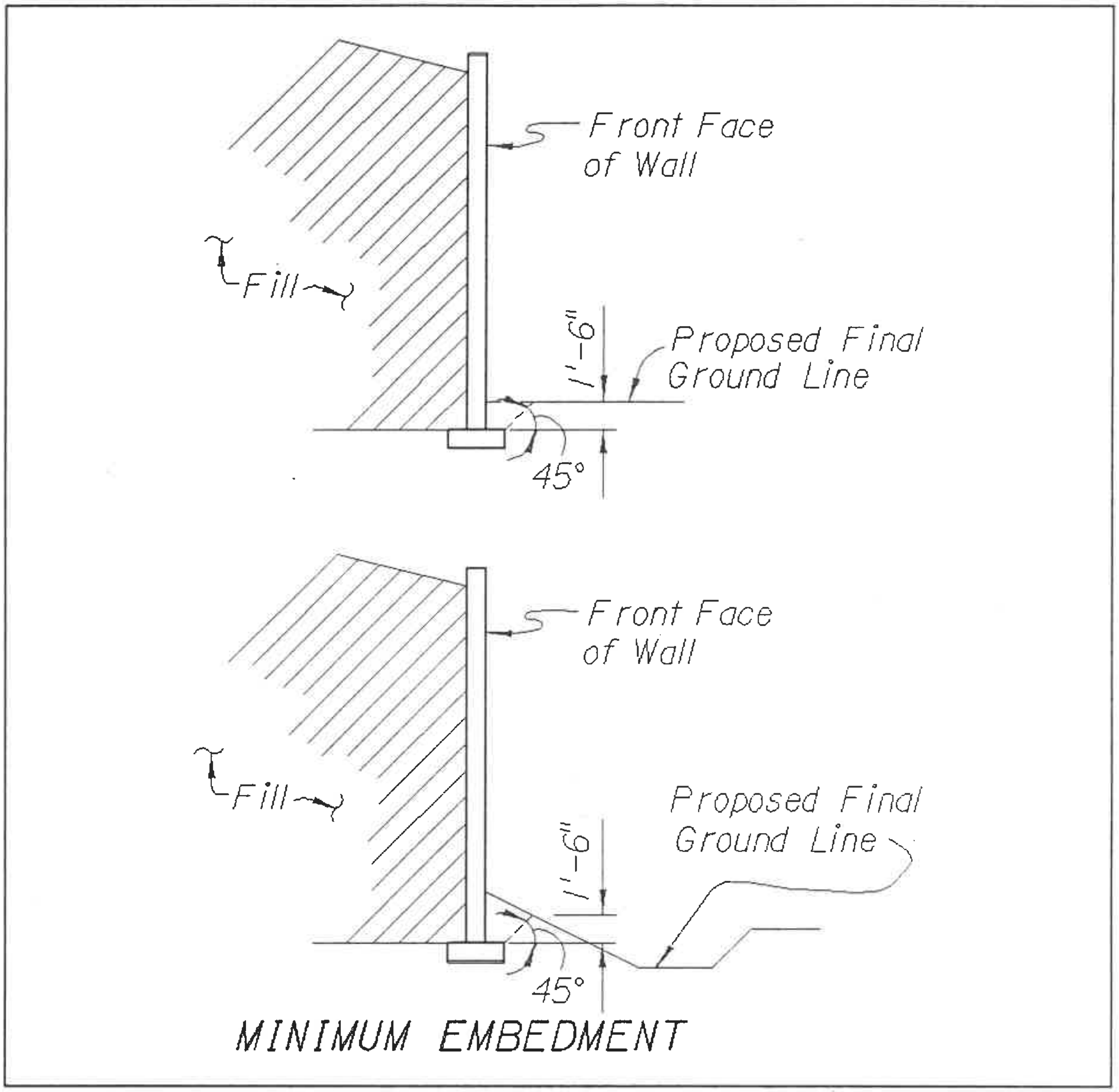

Front Face
of Wall
Fill
1'-6"
Proposed Final
Ground Line
45°
Front Face
of Wall
Fill
1'-6"
Proposed Final
Ground Line
45°
MINIMUM EMBEDMENT

Figure 18-31

Approved: Jerry F. Potter

Effective: November 2, 1992
Sequence No.: 0160
Responsible Office: Structures Design
Topic No.: 625-020-119-h

Chapter 19

SHOP DRAWINGS AND ERECTION DRAWINGS

19.1 Introduction

Shop Drawings include all drawings, diagrams, illustrations, schedules, catalog data, material certifications, fabrication procedures, storage and/or transportation procedures, test results, design calculations, etc., submitted by the Contractor to define some portion of the project work. While the Contract Plans and Specifications (including Supplemental and Special Provisions) define the overall nature of the project with many specific requirements, Shop Drawings provide a method for the Contractor to propose, under specification guides and for the Department to approve or reject, a particular material, product or system of accomplishing the work. Each Contractor knows best what materials and equipment can be provided more expeditiously and economically and, if an item proposed meets the contract requirements, the submission of Shop Drawings is the accepted method of approving an element of the structure while allowing flexibility in the Contractor's choice of materials and construction techniques.

It is mandatory, however, that Shop Drawings not be used to modify the construction contract time, the contract amount, the design intent nor in any way reduce the structural integrity or load-carrying capacity of the structure or its components. Such modifications can only be administered by revised plan sheets or specifications.

Erection Drawings include all drawings, diagrams, design calculations, procedure manuals and other data required to depict in detail the proposed assembly and methods of installation of components into the project work. The work of construction is the expertise of the Contractor, who should be allowed some latitude in the use of construction means, methods, techniques, sequences and procedures as are compatible with and will result in the project being completed in accordance with the requirements of the Contract Plans and Specifications. Shop Drawings for items such as steel girders, precast/prestressed beams, miscellaneous steel, etc., usually include plan views and/or elevation views denoting the correct placement of a component in the structure. Additional Erection Drawings are required for major structures for items such as special precasting, handling and erection equipment, or the erection of concrete segmental bridges. The Engineer of Record must ensure that the Contract Plans and Special Provisions for the project clearly define all requirements for submittal of Erection Drawings.

The following are definitions used herein:

- Engineer: As defined in FDOT Specifications, Section 1.
- Engineer of Record: As defined in the FDOT Specifications, Section 1.
- Specialty Engineer: As defined in the FDOT Specifications, Section 1.
- Consultant: As defined in the FDOT Specifications, Section 1.
- Resident Engineer: The Department's local area representative who reports directly to the District Construction Engineer and may be either a Departmental employee of the District or an employee of an engineering firm which is also serving as the Department's CEI (Construction Engineering and Inspection) Group. The Resident Engineer is the principal representative of the Department for a project at the District level. It shall be noted that neither the Resident Engineer nor the CEI Group is involved with the Shop/Erection Drawing review process but are recipients, only, of approved Shop/Erection Drawings.
- Architect of Record: As defined in the FDOT Specifications, Section 1.
- "Ballooning": The contractor's use of minimum 1/16" wide lines to "balloon" or "cloud" (encircle) notes or details on drawings, design calculations, etc., in order to explicitly and prominently call out any deviations from the Contract Plans or Specifications. The Engineer of Record may also use "ballooning" to make note of any limitations to their submittal review and disposition of shop and erection drawings.

- o **Record Shop Drawings:** The Department's official record copy of all Shop drawings, Erection Drawings, calculations, manuals, correspondence/ transmittal files and submittal activity record (logbook).
- o **Review Office:** The office or other Department entity responsible for performing the Department's review, record keeping, disposition and distribution of Shop and Erection Drawings.

19.2 Drawing Submittals Required

Generally, Shop Drawings shall be required for items which require fabrication at a location other than the project job site.

Unless otherwise noted in the Special Provisions for the project, Shop Drawings are not required for reinforcing steel for cast-in-place concrete which is completely detailed and listed on the Contract Plans or on the Department's Standard Index Drawings. A copy of reinforcing bar lists shall be forwarded by the Contractor to the Department's Resident Engineer for record purposes.

Components such as traffic signal equipment, steel or aluminum light poles, concrete strain poles and high mast lighting may not require submittal of Shop Drawings due to having prior certification by the Department. The Contractor may contact the Department's Resident Engineer or the appropriate Department Review Office for clarification of any item.

Material certifications are typically submitted by the Contractor directly to the State Materials Engineer in Gainesville.

Except as otherwise stipulated in the Specifications, prestressed concrete items not constructed from standard drawings require the submission of Shop Drawings which shall include detensioning schedules and a description of the fabricator's proposed method for restraining the prestressing strands from displacing vertically or horizontally during casting operations.

Shop Drawing submittals for structural steel shall include complete shop and field details including a bill of materials, all dimensions, bolt and hole sizes, camber diagrams, web cutting diagrams, weld symbols, surface preparation and shop paint. Welding procedures and welder qualifications shall be submitted in conjunction with the drawings.

In general, drawing submittals for any item shall follow industry standards in regard to the quantity and quality of information contained. As a minimum, the information shown on approved shop drawings should be complete enough to allow for fabrication of the item without referencing any other document. The Department shall expect submittals to meet or exceed the quality level of previously approved submittals of a similar nature.

During component fabrication and construction phases of the project, the Contractor may elect to submit to the Engineer, for consideration or approval, repair procedures or disposition requests due to errors or omissions in the work. The information required and the procedure to be followed by the Contractor in initiating such requests shall be in accordance with the FDOT Specifications or as determined by the Engineer.

19.3 Contractor Information Required

All Shop Drawings and Erection Drawings shall contain the following items as a minimum requirement: the complete State Project Number (#00000-0000), the drawing number, drawing title, a title block showing the name of the fabricator or producer and the Contractor for which the work is being done, the initials of the person(s) responsible for the drawing, and the date on which the work was performed.

The drawing shall also contain, adjacent to the title block, information which describes the location of the item(s) within the project. This information may consist of the Contract Drawing number, the station at which the item is positioned (as may be the case for sign structures or handrails), or the Site at which it is to be installed.

Before submission of each drawing, the Contractor shall have determined and verified all quantities, dimensions, specified performance criteria, installation requirements, materials, catalog numbers and similar data with respect thereto, and shall have reviewed and coordinated each drawing with other Shop Drawings and with the requirements of the Contract Plans and Specifications. The Contractor shall have stamped and initialed each sheet giving specific written indication of compliance with the above described specific responsibilities with respect to review of the submission.

The Contractor's approval signifies that the submittal meets the requirements of the Contract Plans and Specifications and conforms to field dimensions or other potential deviations from the established project documents. Drawing submittals received without stamping by the Contractor shall be returned for resubmittal.

At the time of each submission, the Contractor shall have given specific written notice (as in the transmittal letter) of each variation that the Shop/Erection Drawings may have from the requirements of the Contract Plans and Specifications. In addition, the drawings shall contain a specific notation which explicitly and prominently calls out any deviation. Approval of a Shop/Erection Drawing will not constitute nor be considered grounds for approval of a variation in which the project requirements are affected unless specifically so noted in the Department's approval comments as returned with the drawing submittal.

19.4 Submittals Requiring a Specialty Engineer

In general, and when so permitted in the Specifications,if a Shop/Erection Drawing submittal reflects any changes in the design and/or details of the Contract Plans, the Contractor shall have had a Specialty Engineer sign and seal one (1) print of each drawing affected as well as the cover sheet of one (1) copy of any design calculations required. The Contract Plans and Specifications (including Supplemental and Special Provisions) shall contain instructions regarding requirements of a Specialty Engineer for items such as concrete segmental bridge work, loads imposed on an existing structure, or certain construction procedures and/or equipment.

Submittals which introduce engineering input to the project, such as defining the configuration or structural capacity of prefabricated components or assemblies not contained in the Contract Plans, shall require the services of a Specialty Engineer. Drawings prepared solely as a guide for component fabrication/ installation and requiring no engineering input, such as reinforcing steel drawings and catalog information on standard products, do not require the use of a Specialty Engineer.

When required, the Specialty Engineer shall provide his impressed (not stamped) seal and his signature on one (1) record print of each drawing and on the cover sheet of one (1) record copy of calculations or computer printouts. Computer printouts are an acceptable substitute for manual computations provided they are accompanied by sufficient documentation of design assumptions and identified input and output information to permit their proper evaluation. Such information shall bear the impressed seal and signature of the Specialty Engineer as verification that he has accepted responsibility for the results.

It is emphasized that a Specialty Engineer may not affix his seal and signature to any item not prepared under his direct supervision and control. It is further emphasized that when a Specialty Engineer does affix his seal to a drawing, he is required to have both his signature and, below the signature, the date of that signature.

When a submittal requires a Specialty Engineer, the signed and sealed prints and calculations will be retained by the Department, as the official, record Shop Drawing. In this event only, when the Engineer of Record is a consultant to the Department, the consultant must print his own review copy and forward the reproducible prints and signed and sealed prints and calculations to the Department upon completion of his review. See also Section 19.6.c. hereinafter.

19.5 Scheduling of Submittals

Review of the submittal requirements and procedures at the outset of the construction contract relationship is of benefit to both the Contractor and the Department. Therefore, the Contractor may have been requested by the Department to provide a Working Schedule for Shop/Erection Drawing submittals.

The preparation of a Working Schedule will bring to the attention of the Contractor the number of submittals required and at times may denote items about which the Contractor may wish the Department's advice as to the manner in which the design is to be implemented. Adherence to the Working Schedule will make for a smoother working relationship between all parties involved in the project, and proper planning should reduce the possibility of a large number of submittals being forwarded for review concurrently.

The Contractor is generally required to schedule submissions such that a minimum of 45 calendar days is allowed for review by the Department for routine work of which the first 30 calendar days are alloted to prime review by the Engineer of Record. However, for most routine submittals, a time period of 14 to 21 calendar days should be adequate. For work of more complexity, the time for review by the Department may be adjusted proportionately to the complexity of the work. Allowance must also be made for potential resubmittals, and the Contractor normally is advised by the Department to consider a 75 calendar days to 90 calendar days total lead-time for submittals prior to the need for fabrication or construction work.

The Contractor must make submittals for approval with such promptness as to cause no delay in his fabrication and construction schedules. Only in emergency cases should special consideration be requested. If a submittal requires resubmission, an approximate additional 30 calendar days should have been scheduled by the contractor for approval of the resubmittal of which the first 15 calendar days are allotted to prime review by the Enginer of Record.

19.6 Transmittal of Submittals

Submittal of Shop/Erection Drawings shall be made to the Department or Consultant, as applicable, only by the Contractor for the project. In that the Department's legal contracts and documents are with the Contractor, submittals shall not be accepted directly from a subcontractor or fabricator. Situations may occur when a subcontractor or fabricator is allowed to make an advance submittal for review; however, the actual submittal to be stamped and approved must follow from the Contractor with the Contractor's stamp. Subcontractors and fabricators are encouraged to contact the appropriate Department Review Office for guidance or advice at any time. Figure 19.1 shows the flow of submittals during the review process. All transmittals of submittals between parties shall be accomplished by OVERNIGHT DELIVERY.

The Special Provisions for the project may denote the amount of drawings, etc. to be submitted and the procedure to be followed. Furthermore, the office to which the Contractor shall transmit his submittal and the procedure to be followed may also be defined during the pre-construction conference for the project. In the absence of such instructions, the following shall apply:

A. On projects where the Engineer of Record is a Consultant to the Department and unless otherwise directed at the project's pre-construction conference, the Contractor shall have submitted one (1) set of mylar or xerographic reproducibles and one (1) set of prints of all drawings directly to the consulting Engineer of Record. On projects where the Department is the Engineer of Record, the Contractor shall have submitted one (1) set of mylar or xerographic reproducibles and one (1) set of prints directly to the appropriate Department Review Office. For design calculations, four (4) complete sets, including computer printouts, shall be submitted with the drawings. All drawings shall be on sheets not larger than 24" x 36". The Contractor's letter of transmittal should always accompany the drawings and a copy should always have been sent to the Department's Resident Engineer. On those projects where the Engineer of Record is a Consultant to the Department, a second copy of the Contractor's letter should also have been sent to the Department's Review Office.

B. On projects where the Engineer of Record is Department in-house staff, submittals shall have been transmitted to the appropriate FDOT Review Office as directed at the project's pre-construction conference. The Department's Review Office is the principal contact group and "clearing house" for all construction submittals and information desired by the Contractor regarding structural, mechanical or electrical items.

C. On projects where the Engineer of Record is a Consultant to the Department and has been retained by the Department to review construction items, submittals (unless otherwise noted below) shall have been transmitted by the Contractor directly to the Consultant. When one (1) set of mylar or xerographic reproducibles and one (1) set of prints are received, the Consultant shall perform his review utilizing the prints, transfer his review comments to the reproducible sheets, indicate his disposition by stamping the reproducible sheets as described hereinafter, retain the prints for his files and, finally, transmit the reproducible sheets to the Department's Review Office for review and distribution. When submittals require a Specialty Engineer as described, the Consultant shall first make a copy of the reproducible prints for his review and files. The signed and sealed prints and calculations, along with the reproducible prints, form the official, record Shop Drawing submittal and must be retained by the Department. Upon completion of his review, the Consultant shall transfer his comments to both the reproducible prints and the signed and sealed prints, indicate his disposition on both and transmit both to the Department as described above.

D. On projects where the Engineer of Record is a Consultant to the Department but has <u>not</u> been retained by the Department to review construction items, submittals (unless otherwise noted below) shall have been transmitted by the Contractor directly to the Department's Review Office as directed at the project's pre-construction conference.

E. Submittals related to Architectural or Building Structures, such as Rest Area Pavilions and Maintenance Warehouses, shall have been made according to the requirements of the Special Facilities Section, Office of Design, Florida Department of Transportation, 605 Suwannee Street, MS 48, Tallahassee, FL 32399-0450, Phone (904)487-4273.

F. All submittals related to roadway plans such as lighting, attenuators, retained earth systems, etc. (except bridge items such as poles and bracket arms, or as noted below) shall be distributed in accordance with the "Roadway Plans Preparation Manual" for the component involved or as otherwise directed at the project's preconstruction conference. Submittals related to bridge items shall have been transmitted to the Department as previously described in this Section.

G. Submittals concerning overhead sign structures shall have been transmitted in accordance with Article 9.6.A thru D above.

H. For items not specified above or for which questions may arise as to submittal requirements, the Contractor should be advised to contact the appropriate Department Review Office.

I. For submittals of any type, the Contractor shall always have transmitted a copy of the letter of transmittal to the Resident Engineer.

19.7 Disposition of Submittals

The approval or disapproval of submittals by the Department shall be indicated by one of the following designations: "APPROVED" (no further action required), "APPROVED AS NOTED" (make corrections noted - no further submittal required), "RESUBMIT" (make corrections noted and resubmit for approval), or "NOT APPROVED" (rejected - do not resubmit the concept or component as submitted).

The disposition designation shall be indicated on each and every drawing sheet, or on the cover sheet of calculations, by the use of a red ink stamp. Stamps shall identify the approving groups, such as the Engineer of Record - Consultant, the Department's assigned commercial inspection agency and/or Department personnel, and the date; however, only the FDOT red ink stamp constitutes an authoritative response to a submittal. All notations or corrections made on the approval prints shall be consistently marked on all drawings.

All Consultants reviewing submittals shall stamp and initial each item as noted above with the firm's appropriate stamp. Consultants must declare any limitations to the extent of their review and approval by the terminology of their standard stamp and/or by additional written and "ballooned" notes on the submittal items. When the Engineer of Record is a Consultant and when he retains a Sub-Consultant to assist in the submittal review, the Engineer of Record shall signify disposition of the submittal as noted above with his firm's appropriate stamp prior to transmitting it by overnight delivery to the Department. In this event it is the Engineer of Record's prerogative to also require a disposition stamp by his Sub-Consultant.

When a submittal contains deviations from the Contract Plans and Specifications, the Consultant and the Department shall determine as to whether or not a Supplemental Agreement or Value Engineering (VECP) proposal is required. If either procedure is required to be initiated, the submittal shall not be reviewed until a decision is finalized.

When the Engineer of Record receives a submittal that is not in accordance with the requirements of this Chapter, the Contractor shall be advised to resubmit immediately with the corrections or additions necessary.

Review and approval by the Engineer of Record (Consultant and/or Department) shall be for conformance with the design concept of the project and for compliance with the information given in the Contract Plans and Specifications (including Supplemental and Special Provisions). The review and approval shall not extend to means, methods, techniques, sequences or procedures of construction (except where a specific means, method, technique, sequence or procedure of construction is indicated in or required by the Contract Plans and Specifications) or programs incident thereto. The review and approval of a separate item as such will not indicate approval of the assembly in which the item functions.

Disposition of Shop Drawing submittals by the Engineer of Record for construction and erection equipment including beams and winches, launch gantry, erection trusses, forms, falsework, midspan and/or longitudinal closures, lifting devices, temporary bearing fixity devices, cranes, form travelers, segment carrying equipment and stability devices shall be either "NOT APPROVED" if deemed to be unacceptable or, if acceptable, shall be "APPROVED AS NOTED" with the following note included on the submittal drawings:

> "Drawings are acceptable for coordination with, relationship to, and effects upon the permanent bridge; but have not been reviewed for self-adequacy. Adequacy and intended function remain the sole responsibility of the Contractor."

Unless otherwise specifically designated in a Consultant's Scope of Services or required by the Department, the Engineer of Record is <u>not</u> responsible for accepting or reviewing calculations or drawings pertaining to construction formwork. These documents should normally have been submitted to the Engineer or, in the event they are erroneously transmitted to the Engineer of Record, should be immediately re-routed to the EOR.

When the Engineer of Record is a Consultant to the Department and when the Department receives the Consultant's transmittal of a shop drawing submittal reviewed and stamped for disposition as noted above, the Department will perform a second, confirmation review of the submittal. The primary purposes of the Department's review include: conformance with FDOT policy, standards, etc.; uniformity of disposition with similar submittals; accuracy and completeness of the Consultant's review; and attention to specific details, areas of work, etc. that have experienced recurring problems during fabrication and/or construction.

When the Department concurs with the Consultant's review and disposition of the submittal, the Department will stamp and distribute the submittal including a record copy for the Consultant. Should the Department's review and/or disposition of the submittal differ from that of the Consultant, the final disposition of the submittal will be resolved in accordance with the following procedures:

A. Minor Modifications: The submittal will be processed when notations not involving design decisions are added, modified or deleted and when the disposition of the submittal remains unchanged or changed only in accordance with the following schedule:

From	To
Approved	Approved as Noted
Approved as Noted	Approved
Resubmit	Not Approved
Not Approved	Resubmit

In this event, the Department will notify the Consultant of the modifications, document the notification in the project's shop drawing file, process and distribute the submittal and furnish the Consultant with a record copy.

B. Major Modifications: The submittal will be returned to the Consultant for re-review when notations involving significant design decisions must be added, deleted or modified, when the submittal's review is deemed by the

Department to be incomplete or require significantly more work or when the disposition of the submittal requires one of the following changes:

From	To
Approved or Approved as Noted	Not Approved or Resubmit
Not Approved or Resubmit	Approved or Approved as Noted

Again, in this event, the Department will notify the Consultant and document the notification. The submittal will be returned to the Consultant for re-review and return to the Department.

When the Specialty Engineer is required by the Contract Plans and specifications to perform a portion of the design of the project, the Engineer of Record shall confirm that:

1. The Specialty Engineer is qualified to design and prepare the submittal.

2. The specified number of submittals have been furnished.

3. A minimum of one (1) set of Shop Drawings and the cover sheet of one (1) set of calculations have been correctly signed, sealed and dated by the Specialty Engineer.

4. The Specialty Engineer has understood the intent of the design and has used the correct specified criteria.

5. The configuration set forth in the submittal is consistent with that of the Contract Documents.

6. The Specialty Engineer's methods, assumptions and approach to the design is in keeping with accepted engineering practices.

7. The Specialty Engineer's design does not contain any gross inadequacies that would jeopardize or threaten public safety.

A detailed review of design calculations is not required, and a detailed review of dimensions (other than at interface areas with other work) is not required.

When a submittal has been returned as "RESUBMIT", the Contractor shall have made corrections as required and shall have returned the required number of corrected copies for review. All revisions to a drawing, etc., shall have been noted with a symbol consisting of the revision number within a triangle located next to revised area. The Contractor must have directed specific attention in writing to revisions other than the corrections called for by the Department on previous submittals.

Figure 19.1 shows the submittal and distributional flow of a shop drawing transmittal.

19.8 Segmental Bridges - Shop Drawing Check List

The following list is for guidance only. There may be occasions when particular details and needs are more or less than this list:

A. Construction Methods and Sequence (Overall Scheme)

This should be the first submittal as it lays out the Contractor's philosophy and overall approach to the project. It should cover:

1. Overall construction schedule (program) for the duration of the contract. Milestone dates should be clearly shown - for example, the need to open a structure by a certain time for traffic operations.

2. Overall construction sequence. The order in which each of the structures is to be built and the sequence in which individual spans or cantilevers are constructed.

3. The general location of any physical obstacles to construction that might impose restraints to the sequence and an outline of how the Contractor intends to avoid or handle such obstacles as he builds the structure. Obstacles might include road and rail clearances, temporary diversions, transmission lines, pipelines, local property rights and so on.

4. The general location of any temporary construction obstacles and how these are to be handled. Such might include excavation or cofferdams for an adjacent structure, piling rig or other plant clearances, temporary haul road clearances, etc.

5. The appropriate location of any temporary stability towers or other falsework.

6. The approximate location of any special lifting equipment in relation to the structure including clearances required for operation of that equipment. (For example, crane positions and operating radii).

7. The conceptual position of any special construction devices such as launching girders, support trusses, pier brackets, stability devices, beam and winch type equipment, etc. (with outline details only at this time) of how the Contractor intends to attach such equipment to the structure. (The precise details of such attachments would be covered under later detailed submittals).

8. Outline proposals for the lifting, handling and storage of segments. (Again, precise details and any extra reinforcement provisions, etc. would be covered under later detailed submittals.)

9. Any other information pertinent to the Contractor's scheme at this time.

The above information should be in as concise form as possible on one or two drawings. The intent is to provide an overall integrated picture of the Contractor's intentions. As such, these drawings are for information only and it should be made quite clear that the delivery and receipt of such drawings does not constitute approval to the details implied therein. They are to be accepted for information only and not approved. However, the Contractor's subsequent detailed submittals should comply with the overall concepts.

B. Casting Curves and Geometry Control

Casting curves contain the superstructure geometry and compensations for deflections arising as a result of the construction sequence, methods, temporary loads, temporary supports, creep and shrinkage, etc. Camber diagrams are only the deflection compensation portions of the casting curves. Casting curves and camber diagrams may be presented in numerically tabular or graphic forms. The format is not critical, but the information given should be clear and concise, leaving no room for doubt or misinterpretation. Examples and illustrations should be shown to help clarify the data presented. Casting curves and camber should be generated according to the Contractor's proposed methods, sequence, schedule and equipment of the overall scheme. Changes to his overall scheme might require recomputation and submittal of new casting curves and camber.

Geometry control is the process of making field observations and measurements in the casting cell and combining these with the theoretical casting curve data to produce the required structural shape, segment by segment. It involves accurate instrument work and geometry calculations using graphical or computerized methods.

It is normal practice for the geometry control system to be explained in a manual prepared by or on behalf of the Contractor.

C. Post-Tensioning System and Computation

Contractors usually sublet this work to specialty suppliers. There are some differences of detail between suppliers but, by and large, these are not significant. Usually the differences are only in the shape and size of anchorage devices and jacks for a given tendon size and load.

The Post-Tensioning proposals should show and be checked for:

1. Dimensions and details of anchorage devices.

2. Jack sizes and required clearances.

3. Special jack handling devices and insets or fixture needed in the segments for same.

4. Proposals for threading of tendons (i.e., use of steel wire pulling socks, or welded pulling eyes, etc.).

5. Proposals for cutting off strand which has been affected by any heat from welding for pulling.

6. Proposals for cutting of surplus strand prior to and after stressing.

7. Information on the jacking equipment, pumps and dial gauges, etc.

8. The storage of materials and protection from corrosion.

9. Assumptions for the stressing operation, coefficient of friction, wobble factor, elastic modulus or stress - strain curve, anchorage draw-in (wedge set), etc.

10. A summary of the jacking loads, tendon forces and extensions before and after seating the wedges.

11. A stressing sequence and schedule for groups of tendons.

12. Post-tensioning duct profiles and geometric layout used in the computations.

13. Proposed recording sheets.

14. Details, sequence, schedule, operations and stressing forces for any temporary post-tensioning.

15. Any special requirements for bursting rebar or extra rebar to restrain radial forces if the profiles are different from those shown in the contract plans.

16. Details for the means of securing the anchorage hardware in position until the concrete has been cast.

17. Details for the splicing of ducts to ensure that a smooth profile is maintained and that any connections are grout-tight.

18. Details of any special bar or tendon couplers such as those to show adequate clearance for couplers when the tendon elongates with stressing, etc.

19. Details for post-tensioning duct supports with regard to strength and frequency to maintain a good profile during concreting.

20. Details of grout joints such as the locations at all high points and at sufficiently close spacing to ensure a good grouting operation.

21. Information on proposed grouting procedures such as the grout mix including admixtures, the grout pump and delivery system, the sequence of grouting (work "uphill" in one direction along a tendon), back-up facilities, grouting pressures, etc.

The post-tensioning supplier might not be responsible for all of the above information. Some of it, particularly that relating to rebar, hardware, ducts, vents, etc. should be covered on the segment detail shop drawings. Also, the grouting operation (Item 21) might be by a separate subcontractor. Nevertheless, the Contractor is responsible for coordinating all this activity and for making sure that all the information and details are integrated. It should be noted that Items 4 thru 8 are more for the benefit of field personnel than part of the shop drawing review.

D. Segment Shop Drawings

The main purpose of these drawings is to bring all the information together in a format from which the parts can be easily assembled. This involves the integration of diverse details from many areas. Typically the following should be checked:

1. Segment number and direction of erection.

2. All dimensions including widths, lengths, thicknesses, tapers, fillets, radii, working points, post-tensioning duct locations and profiles, clearances, rebar spacings, blockouts, positions of embedded items, holes, grout, vents, anchorage positions and orientations.

3. All reinforcement including bar sizes, shapes, locations, spacings, covers, clearances for the largest sized aggregate, clearances for cumulative tolerances on bending and fixing dimensions, avoidance of conflicts with Post-tensioning ducts, anchorages and hardware including any special lifting or equipment connections. As a general rule, rebar should be adjusted to avoid Post-tensioning and other important embedments.

4. Clearances for post-tensioning jacks, including temporary post-tensioning bar jacks. Make sure there is enough room to thread a jack onto a post-tensioning tendon remembering that most center hole jacks require abut 3 - 4 ft. of strand projecting out beyond the anchorage. Likewise with bar tendons, especially in blockouts, there has to be room for the jack to be placed over and threaded onto the extended section of bar beyond its anchorage.

5. Clearances for lifting devices. Check that there is room to place anchor plates and nuts on the bottom side of any bars connecting through the slabs to a lifting device, etc.

6. Anchorage and Buttress Detail - check that there is adequate rebar in these zones for any bursting and local radial forces. This should be covered on the design drawings but might have to be modified as a result of the Contractor's choice of post-tensioning system. The rebar should not cause congestion and there should be adequate spacing for concrete placement and compaction.

7. Casting of blockouts with regards to the material to be used, reinforcing needed to extend out of the concrete and time of casting in relation to erection stressing, etc.

E. Erection Equipment

These drawings should be reviewed for procedure and structural effect on the structure, and checked to see that the end result is obtainable. The shop and erection drawings shall be prepared by the Contractor's Specialty Engineer and will be reviewed as described in Articles 19.4 and 19.7 of this chapter.

19.9 Distribution of Submittals

If the initial review and approval of a submittal is performed by a Consultant to the Department, the Consultant shall retain one (1) set of materials for his files and transmit the reproducible set of prints (or other sets of calculations or multiple sets of prints) to the Department's Review Office.

Subsequent to the review and approval of a submittal by the Department, final distribution by overnight delivery is made in accordance with the following schedule:

	Engineer of Record	
Distribution	FDOT	Consultant
Department Office File	1 Set Repros & 1 Set Calcs	1 Set Repros & 1 Set Calcs
Engineer of Record		1 Set Prints & 1 Set Calcs
Department Resident Engineer	2 Sets Prints	2 Sets Prints
Prime Contractor	3 Sets Prints (Minimum) & 1 Set Calcs	3 Sets Prints (Minimum) & 1 Set Calcs

When precast/prestressed concrete components are involved, the Department's District Prestress Engineer is furnished two (2) sets and the Department's Materials and Research Office (Gainesville) is furnished one (1) set of prints. When structural steel components are involved, the Department's Assigned Commercial Inspection Agency is furnished two (2) sets.

The Contractor shall be responsible for transmitting a copy of the returned submittal to the appropriate subcontractor or fabricator.

When approval of a submittal is denied ("RESUBMIT" or "NOT APPROVED"), distribution of the submittal shall be made to the Department Review Office's File and the Prime Contractor only, with a copy of the transmittal letter to the Department's Resident Engineer.

19.10 Review of Prequalified Joint Welding Procedures

The shop drawing review process of all prequalified joint welding procedures will be a dual role of responsibility between the Engineer of Record and the Department's Assigned Commercial Inspection Agency (ACIA). The FDOT has now consulted with an ACIA to assist in the review of all welding procedures. It is the intent that all Engineers of Record understand their role in the review process, the role of the ACIA and the correct transmittal process of the welding procedures.

Upon receiving a submittal consisting of prequalified joint welding procedures from the Contractor, the Engineer of Record shall review the procedures in conjunction with the drawings to which they pertain. The review shall determine whether or not the Fabricator's welding procedures conform with the concept of the original design described within the contract plans. A comparison shall be made of the plate sizes, types of welds, weld designations, weld sizes, grades of materials, etc. as described and illustrated in the Fabricator's prequalified joint welding procedures to those described in the contract plans. Determining whether or not these elements parallel those of the design intentions are the interests and responsibility of the Engineer of Record during his review.

Upon the completion of his review, the Engineer of Record shall indicate his disposition of procedures in accordance with Section 19.7 and, if the procedures are acceptable, transmit them by overnight delivery directly to the ACIA with a copy of the transmittal sent to the Department's Review Office. If the stamp signifies the procedures as unacceptable, the Engineer of Record will send them back to the Department's Review Office for the Contractor's resubmittal and the same transmittal procedure will begin again.

When the Department is the Engineer of Record, approved procedures are transmitted to the ACIA for review and unacceptable procedures are returned to the Contractor for resubmittal.

Upon receiving a submittal of procedures approved by the Engineer of Record, the ACIA will perform a review of the proposed shop welding fabrication of the structural steel for general compliance with the 1988 AASHTO/AWS Bridge Welding Code as modified by the AASHTO/AWS 01-5-88. The ACIA's responsibility during their review will be to determine whether or not the Fabricator has provided the correct information needed to perform the weld called for on the procedures. Upon this determination, the ACIA will stamp the procedures accordingly and transmit them by overnight delivery back to the Department's Review Office. If the ACIA determines the procedures to be unacceptable, the FDOT will transmit the submittal back to the Contractor by overnight delivery for resubmittal and the same transmittal procedures will begin again.

It is extremely important that the Engineer of Record complete his review of the procedures as quickly as possible and promptly transmit them by overnight delivery to the ACIA. Similarly, the ACIA shall complete its review of the procedures as quickly as possible and promptly transmit them by overnight delivery to the FDOT.

19.11 Submittal Activity Record (Logbook)

The Department's Review Office is responsible for maintaining a Submittal Activity Record (Logbook) on each project reviewed by the office. The logbook shall be updated each day that any Shop Drawing submittal activity occurs.

The Logbook consists of a microcomputer database program developed and maintained by the Structures Design Office and furnished to each Departmental Review Office. The following minimum data shall be entered in the Logbook for each submittal:

A. State Project Number.

B. Submittal Number.

C. Description of Submittal.

D. Number of Sheets in the Submittal.

E. Number of Pages of Calculations, in Reports, in Manuals, etc.

F. Date Transmitted by Contractor to the Engineer of Record.

G. Date Transmitted by Engineer of Record (when EOR is not the Review Office) to the Review Office.

H. Date Distributed by the Review Office to the Contractor.

I. Date Transmitted by the Department's Assigned Commercial Inspection Agency to the Review Office.

J. Disposition as either "A" (Approved), "AN" (Approved as Noted), "R" (Resubmit) or "NA" (Not Approved).

The Logbook is an historical record of the activity devoted to an individual submittal as well as that for the project as a whole. It can serve as a verification of review time, to respond to inquiries of a particular submittal's status and as a record of manpower effort to aid in estimating and allocating future workload.

When requested by the Structures Design Office (usually on an annual basis), each Review Office will furnish an up-to-date copy of its Logbook in the required diskette format. This information is used by the Structures Design Office to build a statewide, historical, Shop Drawing file.

19.12 Archiving Record Shop Drawings

Upon completion and acceptance of a construction project by the Department (usually by receipt of a written Notice of Acceptance), the Review Office, within thirty (30) days, shall transmit the Record Shop Drawings to the District Structures and Facilities Engineer (DSFE) in the District in which the project is located. The Record Shop Drawings may include some or all of the following documents:

1. Shop Drawings
2. Erection Drawings
3. Calculations
4. Manuals
5. Project Files of Shop Drawing transmittal letters, etc.
6. Submittal Activity Record (Logbook printout)

The Review Office shall complete the Record Shop Drawing Transmittal (see Figure 19.2), Form No. 625-020-119-f, in triplicate, retaining one (1) copy and transmitting two (2) copies, along with the Record Shop Drawings described above, to the DSFE. The Record Shop Drawing Transmittal describes all the Record Shop Drawing documents transmitted to the DSFE.

The Submittal Activity Record (logbook) is intended to serve as the listing of all Shop and Erection Drawings transmitted. Other transmitted material such as project files, samples, etc. should be listed individually on the Transmittal (Form No. 625-020-119-f) shown in Figure 19-2.

Upon receipt of the Record Shop Drawings, the DSFE shall verify the documents, material, etc. transmitted, sign and date both copies of the Record Shop Drawing Transmittal, retain one (1) copy for his files and return the second signed copy to the Review Office.

The Review Office shall maintain a file of Record Shop Drawing Transmittals (Form No. 625-020-119-f) for future reference and use. Once the DSFE's signed copy is received, the Review Office's initially retained Record Shop Drawing Transmittal may be discarded.

It should be noted that for Shop Drawing submittals requiring a Specialty Engineer, the Record Shop Drawing submittal normally will consist of both reproducible prints and signed and sealed prints.

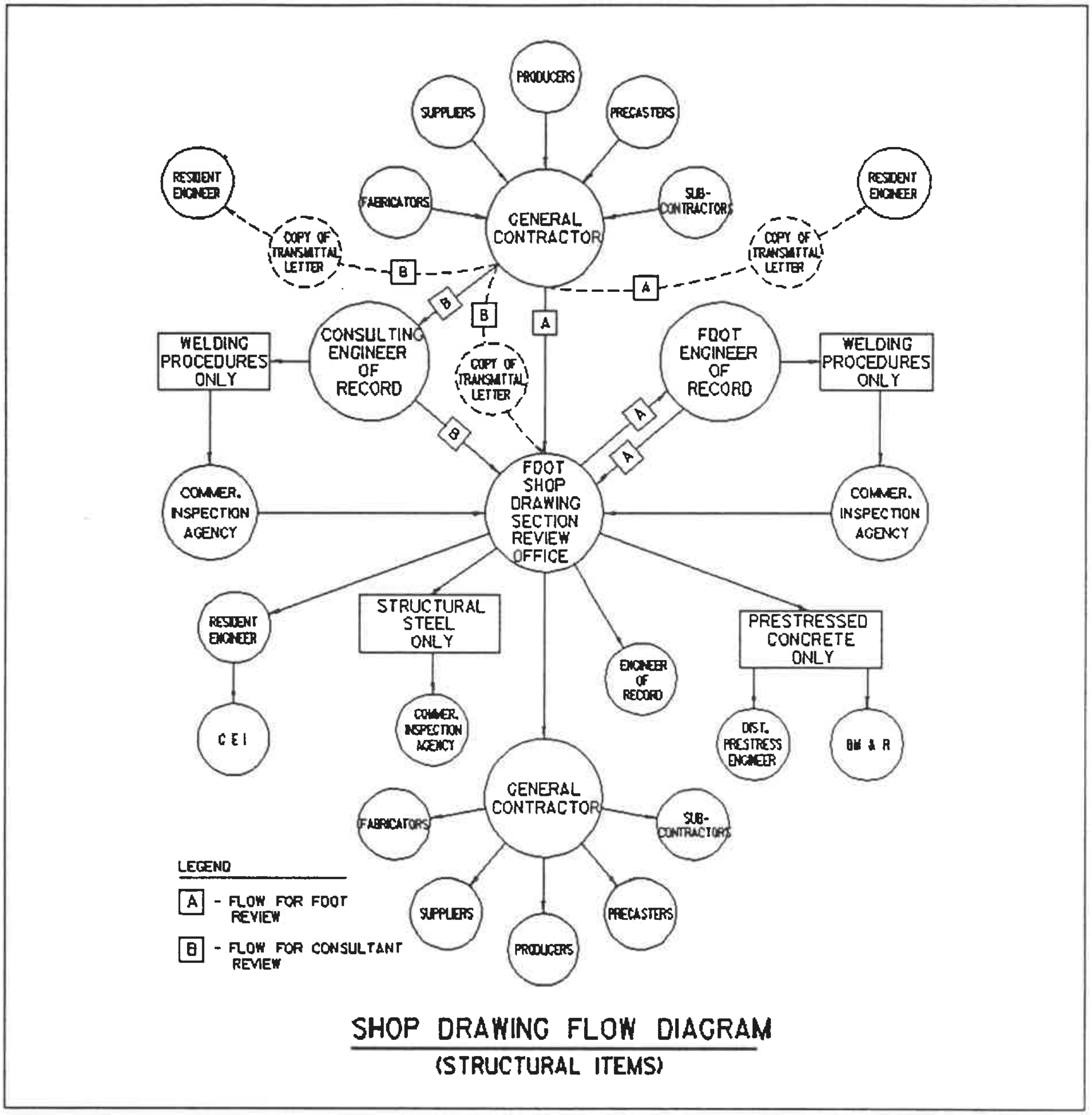

PRODUCERS
SUPPLIERS
PRECASTERS
RESIDENT ENGINEER
FABRICATORS
GENERAL CONTRACTOR
SUB-CONTRACTORS
RESIDENT ENGINEER
COPY OF TRANSMITTAL LETTER
B
COPY OF TRANSMITTAL LETTER
A
B
B
A
WELDING PROCEDURES ONLY
CONSULTING ENGINEER OF RECORD
COPY OF TRANSMITTAL LETTER
FDOT ENGINEER OF RECORD
WELDING PROCEDURES ONLY
A
B
A
COMMER. INSPECTION AGENCY
FDOT SHOP DRAWING SECTION REVIEW OFFICE
COMMER. INSPECTION AGENCY
RESIDENT ENGINEER
STRUCTURAL STEEL ONLY
PRESTRESSED CONCRETE ONLY
ENGINEER OF RECORD
CEI
COMMER. INSPECTION AGENCY
DIST. PRESTRESS ENGINEER
BM & R
GENERAL CONTRACTOR
FABRICATORS
SUB-CONTRACTORS
LEGEND
A - FLOW FOR FDOT REVIEW
B - FLOW FOR CONSULTANT REVIEW
SUPPLIERS
PRECASTERS
PRODUCERS
SHOP DRAWING FLOW DIAGRAM
(STRUCTURAL ITEMS)

Figure 19-1

625-020-119-h

Florida Department of Transportation

RECORD SHOP DRAWING TRANSMITTAL

Date ____________________

TO: District _____ Structures and Facilities Engineer

FROM: ____
Review Office

PROJECT TITLE ____________________________

STATE PROJECT NO. ____________________________

WPI NUMBER ____________________________

FAP NUMBER ____________________________

BRIDGE NUMBER ____________________________

CONTRACTOR ____________________________

ENGINEER OF RECORD ____________________________

We are transmitting herewith the following Record Shop Drawings for archiving:

1. Shop and Erection Drawing Submittals per attached Logbook.
2. Submittal Activities Record (Logbook)
3. ____________________________
4. ____________________________
5. ____________________________
6. ____________________________

For the Review Office: ____
(Signature) (Date)

For the District Structures and Facilities Engineer:

____
(Signature) (Date)

Figure 19-2

Approved: Jerry F. Potter

Effective: November 2, 1992
Sequence No.: 0160
Responsible Office: Structures Design
Topic No.: 625-020-120-h

Chapter 20

SIGNING AND SEALING PLANS

20.1 General

All plans, specifications and other engineering documents for structures prepared for the Department must be signed and sealed in accordance with Chapter 19 of the FDOT Roadway Plans Preparation Manual, Volume 1, and, additionally, in accordance with the Structures Detailing Manual and the remainder of this chapter.

20.2 Older Plans

If the FDOT Project Manager or Office Director (for FDOT work) or a principal of the Consulting Engineering Firm certifies that the originating Professional Engineer in responsible charge of the work is not reasonably attainable, then another Professional Engineer may sign and seal the sheet(s) in question provided there is a note on the plans stating that the plans were not prepared under this person's supervision but were reviewed by the Professional Engineer who is signing and sealing for general conformance to the required design work and design intent.

If the persons responsible for design, checking, drawing, and approval are not available, their initials and the date should be printed or leroyed by the Professional Engineer sealing the plans. If the person or persons are unknown, then unknown and the date should be written in.

The "Approved by" must always be the Professional Engineer in responsible charge of the work.

20.3 Mini Plans

For mini plans (8 1/2" x 14") the appropriate signatures may be placed on the cover sheet in tabulated form in lieu of placement on each sheet. It will not be necessary to list each sheet separately if the same group of individuals were responsible for design, supervision and approval for a series of sheets, i.e., sheets 3 through 7 of 7, etc. The cover sheet will also be signed. The entire set of documents must be sealed.

20.4 Plans Done by Others

The procedures and guidelines as noted and referenced in this chapter also apply to projects the Department is going to let to contract for city and county governments and utility companies, which were either prepared by the government agency/company or their consultant.

20.5 Compliance

It shall be the responsibility of the State or District Structures Design Engineer (Consultant Plans Review Group for consultant plans) or the Design Engineer for internally prepared plans to ensure the plans are properly signed and sealed.

20.6 Revisions

All plan sheets which are revised after original signing and sealing shall be revised in accordance with Chapter 20 of the FDOT Roadway Plans Preparation Manual, Volume 1.

Approved: Jerry Potter

Effective: November 2, 1992
Sequence No.: 0160
Responsible Office: Structures Design
Topic No.: 625-020-121-h

Chapter 21

MISCELLANEOUS HIGHWAY RELATED STRUCTURES

21.1 Design of Overhead Sign Structures

Overhead sign structures are normally designed and detailed by the Engineer of Record as part of the design plans in accordance with Chapter 3, Section 3.6 of these guidelines; however, the design and details occasionally may be required to be provided as shop drawing submittals by the Contractor's Specialty Engineer. In either case the Engineer of Record is responsible for providing a schematic line drawing in elevation showing the location and size of the sign panels with the required clearances and span lengths.

Wind loads for overhead sign structures shall be determined from Figure 21-3.

A. Overhead Signs in Urban Areas

In addition to the signing shown on the original signing plans, span type overhead sign structures in urban areas shall be designed for a minimum sign area of 120 (12'W by 10'H) square feet per lane. This minimum sign area applies only to lanes not covered by a sign of the minimum size on the original plans. Also this minimum sign area pertains to just the lanes in one traffic direction unless the original signing design requires signs for both traffic directions on the sign structure. Cantilever overhead sign structures shall be designed for the actual sign area or an 80 square foot sign (8'W by 10'H) at the end of the cantilever, whichever produces the higher stress or load at the section under consideration.

B. Overhead Signs in Rural Areas

Overhead signs in rural areas should be designed for the actual sign area.

An example of how to apply this minimum sign area is demonstrated by Figures 21-1 and 21-2. The actual sign area for a typical set of Signing Plans is shown in Figure 21-1 and the required sign design area for this same structure is given in Figure 21-2. The 6'W by 15'H sign is replaced by a 12'W by 10'H sign centered in the middle lane and the innermost lane, originally without a sign, has an added 12'W by 10'H sign centered in the lane. The lanes for opposing traffic do not have any signs added since there are no actual signs planned for any of these lanes.

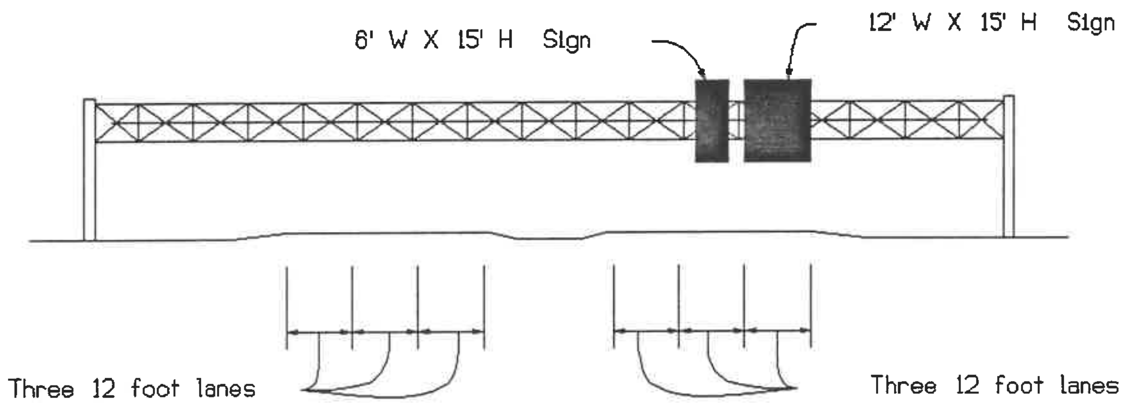


Figure 21-1

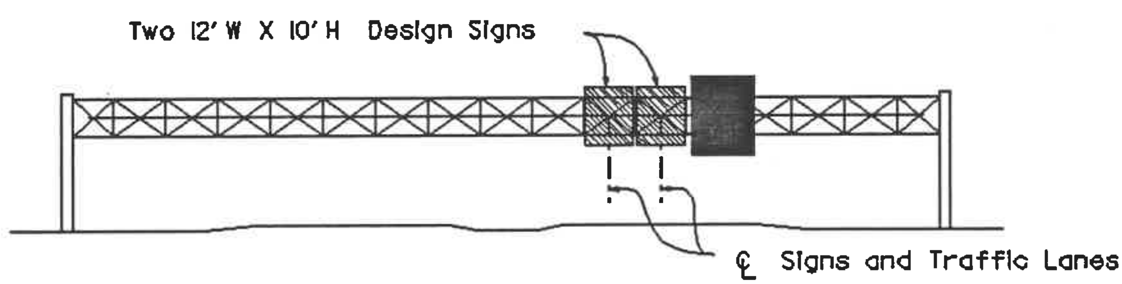


Figure 21-2

21.2 Design of High Mast Light Pole Foundations

Designs for high mast light poles are provided as shop drawings or are prequalified by the manufacturer; however, foundations for these structures are provided on the Contract Documents. Foundation designs should consider the following:

A. Except for unusual circumstances, foundations shall be drilled shafts 4 feet in diameter.

B. These shafts may be designed using either COM624 which is a program available on the FDOT IBM mainframe (see Chapter 17) or by using Broms procedure which is a hand calculation method. Both of these techniques are for the analysis of piles under the lateral load, the primary loading of concern in these cases.

C. A minimum safety factor of 1.5 against overturning must be provided regardless of which method is employed.

D. Loads for these drilled shafts may be determined by use of Figure 21-3 and the Wyoming Pole Program which is available on the FDOT IBM mainframe (see Chapter 17).

E. The contract documents should clearly state the loads these shafts are designed to resist.

When reviewing the shop drawings for these poles, the Engineer of Record should verify that the pole reactions are within the capacity of the foundations.

21.3 Design of Traffic Mast Arms

Contract documents may or may not include a design for traffic mast arms depending on the scope of services. The Engineer of Record should be aware of these items:

A. If the drawings do contain a structural design for these items, the drawings should clearly state if alternate designs will be allowed and under what criteria.

B. If a structural design for traffic mast arms is not included, the drawing should clearly state that the Contractor is responsible for the design of these items and under what conditions and criteria.

C. Responsibility for the design of these structures includes the design of the foundations.

Wind loads for the foundations for traffic mast arms shall be determined from Figure 21-3.

21.4 Design of Roadway Light Poles

The Engineer of Record typically has no responsibility for the structural design of roadway light poles. The structural integrity of these items is the responsibility of the Contractor's Specialty Engineer, and is verified by review of the shop drawings by the Engineer of Record.

21.5 Anchor Bolts for Overhead Sign, Signal, and Lighting Structures

Anchor bolts for these structures may conform to AASHTO specification M 314-90.

21.6 Design of Sound Barrier Walls

The AASHTO "Guide Specifications for Structural Design of Sound Barriers" shall be used for the design. However, the designers should also refer to the Structures Design Guidelines and to the Detailing Manual for further guidance in the preparation of the construction documents.

All sound barrier walls that are located on other structures such as bridges or retaining walls, shall be designed for the wind pressures shown in Table 1-2.1.2.C (Exposure B) of the above AASHTO guide. The wind velocity of 110 MPH shall be used for all designs in the State of Florida. Therefore, walls in the 0 thru 14 ft. height zone (as defined in the Guide) shall be designed for 37 psf wind pressure, and walls higher than 14 ft. but no higher than 29 ft. shall take into consideration the higher 47 psf wind pressure. If the wall is to be located on an embankment, the height zone shall be determined by using the elevation of adjoining ground as being the approximate elevation of the original ground surface prior to embankment construction. Table 1-2.1.2.C of the 1989 AASHTO Guide Specifications shows a minimum wind pressure of 37 psf for walls 14 thru 29 ft. and 110 mph. The 37 psf value shown for this height zone is erroneous, the correct value is 47 psf.

The design of the foundation and posts between panels shall take into consideration the different wind pressures of the applicable height zones. Therefore; a sound barrier wall with the centroid of the loaded area 29 ft. above adjacent ground (in the 14 ft. thru 29 ft. height zone), shall have posts and foundations designed for wind pressures of 37 psf up to a vertical distance above the ground of 28 ft., and 47 psf above this elevation.

Besides the structural integrity of the sound barrier wall, the structural engineer should also be concerned with aesthetics, maintainability, constructability, and of course, cost and durability.

Normally, the structural design should proceed as follows:

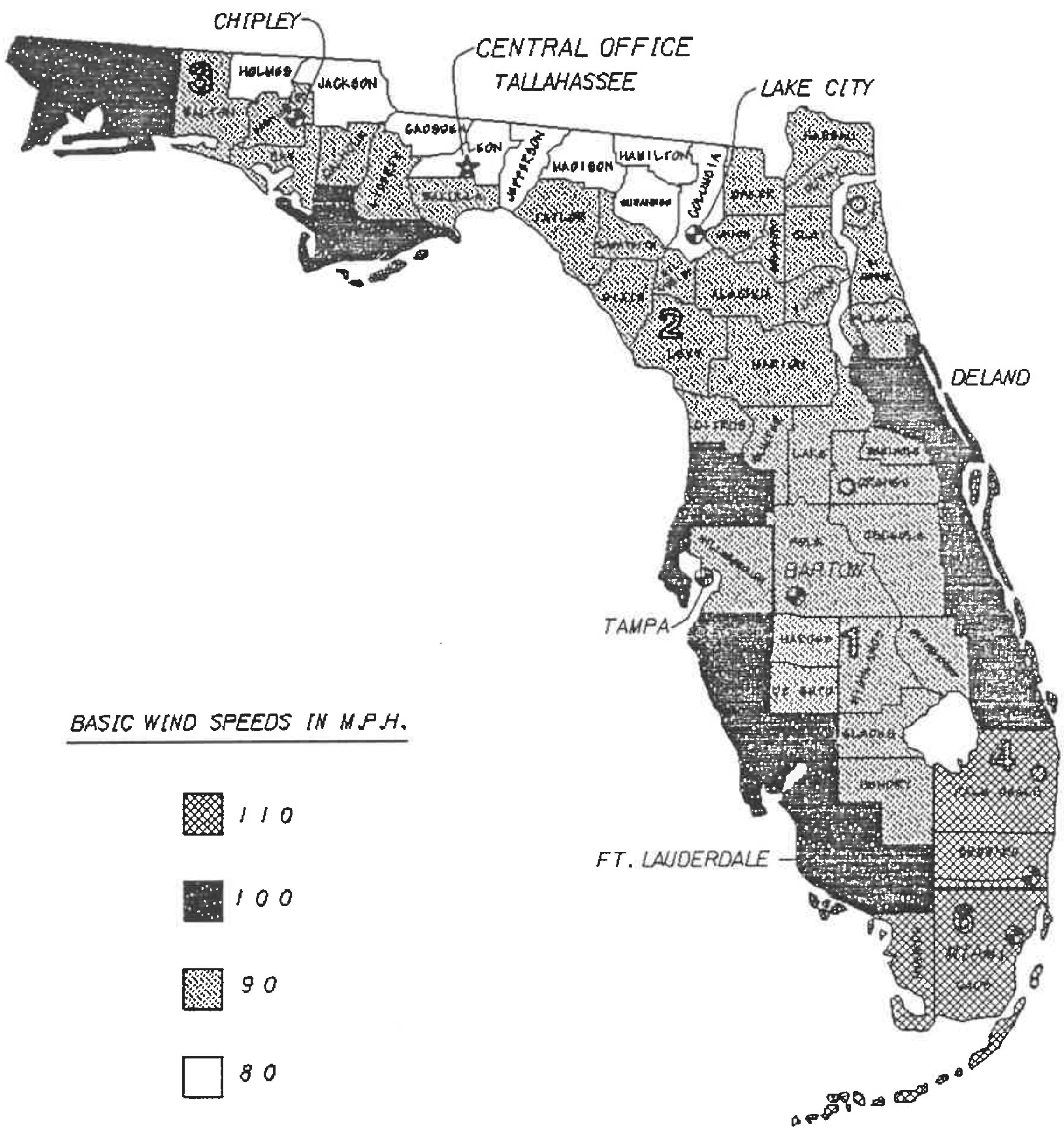
A. Once the wall location, alignments, height and minimum thickness are determined, the soil exploration should be undertaken. The geotechnical engineer should provide the criteria to be followed for the exploration; however, it may be assumed that as a minimum, the borings will be spaced at a maximum of 100 ft., their depth will be 1.5 times the wall height and the number of borings per wall will be no fewer than two (2).

B. The set of drawings to be prepared by the Engineer-of-Record shall consist to two basic groups of drawings. One basic group of drawings referred to as Control Drawings shall provide all controls parameters, such as, alignments, limits, notes, etc., and shall provide all the information which is common to all wall types including but nor necessarily limited to:

1. Wall alignments (horizontal and vertical)
2. Wall limits (beginning and ending)
3. Location of all existing utilities (overhead and/or underground in the vicinity of the proposed wall)
4. Location of Fire-access holes

5. Location of Drainage Holes
6. Sound Barrier Graphics details
7. General Notes
8. "Report of Core Borings" (Soil Information Data)
9. Quantities (wall area as described above for payment purposes only; the itemized quantities such as concrete volume, etc., shall be provided in the specific drawings)
10. All other information that may be construed to be of general nature.

C. The other group of drawings shall deal with the specific details required for the implementation of the selected wall type. All wall components such as: foundation, posts, panels, etc. shall be fully detailed for construction. If standard or conventional (non-proprietary) designs are to be implemented, then these drawings shall provide the specific information. Likewise, if proprietary designs are to be implemented, then the proprietary sound barrier wall drawings shall provide the specific information.



Design Wind Speeds
(50 Year Recurrence)

Figure 21-3