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Please contact Scott Sargent at 360-705-7753 or sargenw@wsdot.wa.gov with comments, questions, or suggestions for improvement to the manual.

For updating printed manuals, page numbers indicating portions of the manual that are to be removed and replaced are shown below.

Chapter	Remove Pages	Insert Pages
Title Page	i – ii	i – ii
Foreword	iii – iv	iii – iv
Contents	v – xxii	v – xxii
Chapter 1 General Information	1-i – 1-ii	1-i – 1-ii
1.3 Quality Control/Quality Assurance Procedure	1.3-11 – 1.3-12	1.3-11 – 1.3-12
Chapter 3 Loads	3-i – 3-ii	3-i – 3-ii
3.4 Limit States	3.4-1 – 3.4-2	3.4-1 – 3.4-2
3.5 Load Factors and Load Combinations	3.5-1 – 3.5-2	3.5-1 – 3.5-2
3.9 Live Loads	3.9-1 – 3.9-4	3.9-1 – 3.9-4
3.12 Noise Barriers	3.12-1 – 3.12-2	3.12-1 – 3.12-2
3.99 References	3.99-1 – 3.99-2	3.99-1 – 3.99-2
Chapter 5 Concrete Structures	5-i – 5-vi	5-i – 5-vi
5.0 General	5.0-1 – 5.0-2	5.0-1 – 5.0-2
5.1 Materials	5.1-1 – 5.1-16	5.1-1 – 5.1-18
5.2 Design Considerations	5.2-7 – 5.2-8	5.2-7 – 5.2-8
5.3 Reinforced Concrete Box Girder Bridges	5.3-15 – 5.3-16	5.3-15 – 5.3-16
5.6 Precast Prestressed Girder Superstructures	5.6-1 – 5.6-24	5.6-1 – 5.6-26
Chapter 6 Structure Steel	6-i – 6-ii	6-i – 6-ii
6.1 Design Considerations	6.1-1 – 6.1-6	6.1-1 – 6.1-6
6.3 Design of I-Girders	6.3-1 – 6.3-2	6.3-1 – 6.3-2
Plan Sheets	6.4-A3 – 6.4-A4	6.4-A3 – 6.4-A4
	6.4-A7 – 6.4-A8	6.4-A7 – 6.4-A8

Chapter	Remove Pages	Insert Pages
Chapter 7 Substructure Design	7-i – 7-ii	7-i – 7-ii
7.1 General Substructure Considerations	7.1-1 – 7.1-2	7.1-1 – 7.1-2
7.2 Foundation Modeling for Seismic Loads	7.2-1 – 7.2-14	7.2-1 – 7.2-14
7.3 Column Design	7.3-1 – 7.3-4	7.3-1 – 7.3-6
7.4 Column Reinforcement	7.4-1 – 7.4-14	7.4-1 – 7.4-14
7.5 Abutment Design and Details	7.5-1 – 7.5-12	7.5-1 – 7.5-14
7.7 Footing Design	7.7-1 – 7.7-10	7.7-1 – 7.7-10
7.8 Drilled Shafts	7.8-1 – 7.8-12	7.8-1 – 7.8-12
7.9 Piles and Piling	7.9-1 – 7.9-6	7.9-1 – 7.9-6
Plan Sheets		7.3-A1-1
Chapter 8 Walls and Buried Structures	8-i – 8-ii	8-i – 8-ii
8.2 Miscellaneous Underground Structures	8.2-1 – 8.2-4	8.2-1 – 8.2-4
Chapter 10 Signs, Barriers, Approach Slabs, and Utilities	10-i – 10-iv	10-i – 10-iv
10.1 Sign and Luminaire Supports	10.1-1 – 10.1-10	10.1-1 – 10.1-10
10.2 Bridge Traffic Barriers	10.2-1 – 10.2-8	10.2-1 – 10.2-8
10.3 At Grade Traffic Barriers	10.3-1 – 10.3-4	10.3-1 – 10.3-6
10.4 Bridge Traffic Barrier Rehabilitation	10.4-1 – 10.4-2	10.4-1 – 10.4-2
10.5 Bride Railing	10.5-1 – 10.5-2	10.5-1 – 10.5-2
10.6 Bridge Approach Slabs	10.6-1 – 10.6-6	10.6-1 – 10.6-6
Plan Sheets	10.1-A0-1	10.1-A0-1
	10.1-A1-1 – 10.1-A1-2	10.1-A1-1 – 10.1-A1-2
	10.1-A2-1 – 10.1-A2-2	10.1-A2-1 – 10.1-A2-2
	10.1-A3-1 – 10.1-A3-2	10.1-A3-1 – 10.1-A3-2
	10.2-A1-1 – 10.2-A1-3	10.2-A1-1 – 10.2-A1-3
	10.2-A2-1 – 10.2-A2-3	10.2-A2-1 – 10.2-A2-3
	10.2-A3-1 – 10.2-A3-3	10.2-A3-1 – 10.2-A3-3
	10.2-A4-1 – 10.2-A4-3	10.2-A4-1 – 10.2-A4-3
	10.2-A5-1A – 10.2-A5-1B	10.2-A5-1A – 10.2-A5-1B
	10.2-A5-2A – 10.2-A5-2B	10.2-A5-2A – 10.2-A5-2B
	10.2-A5-3	10.2-A5-3
	10.2-A6-1A – 10.2-A6-1B	10.2-A6-1A – 10.2-A6-1B
	10.2-A6-2A – 10.2-A6-2B	10.2-A6-2A – 10.2-A6-2B
	10.2-A6-3	10.2-A6-3
	10.2-A7-1 – 10.2-A7-3	10.2-A7-1 – 10.2-A7-3
	10.5-A5-1 – 10.5-A5-4	10.5-A5-1 – 10.5-A5-4
	10.6-A1-1 – 10.1-A1-3	10.6-A1-1 – 10.1-A1-3
	10.6-A2-1 – 10.1-A2-2	10.6-A2-1 – 10.1-A2-2
	10.8-A1-1 – 10.8-A1-2	10.8-A1-1 – 10.8-A1-2
Chapter 13 Bridge Load Rating	13-i – 13-ii	13-i – 13-ii
13.1 General	13.1-1 – 13.1-2	13.1-1 – 13.1-2
	13.1-5 – 13.1-6	13.1-5 – 13.1-6
13.4 Load Rating Reports	13.4-1 – 13.4-2	13.4-1 – 13.4-2

To get the latest information, please sign up for email updates for individual manuals at www.wsdot.wa.gov/publications/manuals.

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Approved By Signature

Washington State
Department of Transportation

Bridge Design Manual (LRFD)

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Bridge and Structures Office

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Foreword

This manual has been prepared to provide Washington State Department of Transportation (WSDOT) bridge design engineers with a guide to the design criteria, analysis methods, and detailing procedures for the preparation of highway bridge and structure construction plans, specifications, and estimates.

It is not intended to be a textbook on structural engineering. It is a guide to acceptable WSDOT practice. This manual does not cover all conceivable problems that may arise, but is intended to be sufficiently comprehensive to, along with sound engineering judgment, provide a safe guide for bridge engineering.

A thorough knowledge of the contents of this manual is essential for a high degree of efficiency in the engineering of WSDOT highway structures.

This loose leaf form of this manual facilitates modifications and additions.
New provisions and revisions will be issued from time to time to keep this guide current. Suggestions for improvement and updating the manual are always welcome.

All manual modifications must be approved by the Bridge Design Engineer.

The electronic version of this document is available at:
www.wsdot.wa.gov/publications/manuals/m23-50.htm

/s/ Jugesh Kapur

Jugesh Kapur, P.E., S.E.
Bridge and Structures Engineer

Contents

Page

Chapter 1 General Information

Page

Chapter 2 Preliminary Design

Chapter 3 Loads

Page

Page

Page

Chapter 6 Structural Steel

Page

Chapter 8 Walls and Buried Structures

Chapter 9 Bearings and Expansion Joints

Page

Chapter 10 Signs, Barriers, Approach Slabs, and Utilities

Chapter 1 General Information — Contents

Output from commercial software shall be integrated into design calculations with a cover sheet that includes the WSDOT logo along with project name, S.R. number, designer and checker's name, date, supervising engineer, and sheet numbers.

Consultant submitted design calculations shall comply with the above requirements.

Design calculations prepared by the Bridge Design Office or Consultants need not be sealed and signed. Design calculations are considered part of the process that develops contract plans which are the final documents.

See Appendix 1.5-A4-1 for examples of Excel template for computer-generated design calculations. Code and other references used in developing calculations shall be specified. In general, when using Excel spreadsheet, enough information and equations shall be provided/ shown in the spreadsheet so that an independent checker can follow the calculations.

3. **Special Design Features –** Brief narrative of major design decisions or revisions and the reasons for them.
4. **Construction Problems or Revisions –** Not all construction problems can be anticipated during the design of the structure; therefore, construction problems arise during construction, which will require revisions. Calculations for revisions made during construction should be included in the design/check calculation file when construction is completed.

D. **File Exclusions –** The following items should not be included in the file:

1. Geometric calculations.
2. Irrelevant computer information.
3. Prints of Office Standard Sheets.
4. Irrelevant sketches.
5. Voided sheets.
6. Preliminary design calculations and drawings unless used in the final design.
7. Test hole logs.
8. Quantity calculations.

1.3.4 PS&E Review Period

See Section 12.4.10 for PS&E Review Period and Turn-in for AD Copy activities.

1.3.5 Addenda

Plan or specification revisions during the advertising period require an addendum. The Specifications and Estimate Engineer will evaluate the need for the addendum after consultation with the HQ Construction – Bridge, Region, and the HQ or Region Plans Branch. The Bridge Design Engineer or the Unit Supervisor must initial all addenda.

For addenda to contract plans, obtain the original drawing from the Bridge Projects Unit. Use shading or clouding to mark all changes (except deletions) and place a revision note at the bottom of the sheet (Region and HQ Plans Branch jointly determine addendum date) and a description of the change. Return the 11″ by 17″ signed original and copy to the Specifications and Estimate Engineer who will submit the copy to the HQ Plans Branch for processing. See Chapter 12 for additional information.

For changes to specifications, submit a copy of the page with the change to the Specifications and Estimate Engineer for processing.

1.3.6 Shop Plans and Permanent Structure Construction Procedures

This section pertains to fabrication shop plans, weld procedures, electrical and mechanical items, geotechnical procedures, such as: drilled shafts and tieback walls, and other miscellaneous items related to permanent construction.

The following is a guide for checking shop plans and permanent structure construction procedures.

A. **Bridge Shop Plans and Procedures**

1. Mark one copy of each sheet with the following, near the title block, in red pen or with a rubber stamp:

   Office Copy
   Contract (number)
   (Checker's initials) (Date)
   Approval Status (A, AAN, RFC or Structurally Acceptable)

2. On the Bridge Office copy, mark with red pen any errors or corrections. Yellow shall be used for highlighting the checked items. The red pen marks will be copied onto the other copies and returned to the Region Project Engineer. Comments made with red pen, especially for 8½″ by 11″ or 11″ by 17″ size sheets, shall be clear, neat, and conducive to being reproduced by Xerox. These comments should be "bubbled" so they stand out on a black and white Xerox copy. Use of large sheets should be discouraged because these require extra staff assistance and time to make these copies by hand.

3. Items to be checked are typically as follows: Check against Contract Plans and Addenda, Special Provisions, Previously Approved Changes and Standard Specifications.

   a. Material specifications (ASTM specifications, hardness, alloy and temper, etc.).

   b. Size of member and fasteners.

   c. Length dimensions, if shown on the Contract Plans.

   d. Finish (surface finish, galvanizing, anodizing, painting, etc.).

   e. Weld size and type and welding procedure if required.

   f. Strand or rebar placement, jacking procedure, stress calculations, elongations, etc.

   g. Fabrication — reaming, drilling, and assembly procedures.

   h. Adequacy of details.

   i. Erection procedures.

   For prestressed girders and post-tensioning shop plan review see Sections 5.6.3A and 5.8.6C respectively.

4. Items Not Requiring Check

   a. Quantities in bill of materials.

   b. Length dimensions not shown on Contract Plans except for spot checking and is emphasized by stamping the plans: *Geometry Not Reviewed by the Bridge and Structures Office.*

5. Project Engineer's Copy

   Do not use the Project Engineer's copy (comments or corrections are in green) as the office copy. Transfer the Project Engineer's corrections, if pertinent, to the office copy using red pen. The Project Engineer's comments may also be received by e-mail.

Chapter 3 Loads — Contents

3.4 Limit States

The basic limit state equation is as follows:

$$\Sigma \eta_i \gamma_i \, Q_i \leq \phi R_n \tag{3.4-1}$$

Where:

η_i = Limit State load modifier factor for ductility, redundancy, and importance of structure
γ_i = Load factor
Q_i = Load (i.e., dead load, live load, seismic load)
ϕ = Resistance factor
R_n = Nominal or ultimate resistance

This equation states that the force effects are multiplied by factors to account for uncertainty in loading, structural ductility, operational importance, and redundancy, must be less than or equal to the available resistance multiplied by factors to account for variability and uncertainty in the materials and construction.

Use a value of 1.0 for η_i except for the design of columns when a minimum value of γ_i is appropriate. In such a case, use $\eta_i = 0.95$. Compression members in seismic designs are proportioned and detailed to ensure the development of significant and visible inelastic deformations at the extreme event limit states before failure.

Strength IV load combination shall not be used for foundation design.

3.5 Load Factors and Load Combinations

The limit states load combinations, and load factors (γ_i) used for structural design are in accordance with the AASHTO *LRFD Specifications*, Table 3.4.1-1. For foundation design, loads are factored after distribution through structural analysis or modeling.

The live load factor for Extreme Event-I Limit State load combination, γ_{EQ} as specified in the AASHTO *LRFD Specifications* Table 3.4.1-1 for all WSDOT bridges shall be taken equal to 0.50. The γ_{EQ} factor applies to the live load force effect obtained from the bridge live load analysis. Associated mass of live load need not be included in the dynamic analysis.

The AASHTO *LRFD Specifications* allow the live load factor in Extreme Event-I load combination, γ_{EQ}, be determined on a project specific basis. The commentary indicates that the possibility of partial live load, i.e., $\gamma_{EQ} < 1.0$, with earthquakes should be considered. The application of Turkstra's rule for combining uncorrelated loads indicates that $\gamma_{EQ} = 0.50$ is reasonable for a wide range of values of average daily truck traffic (ADTT). The NCHRP Report 489 recommends live load factor for Extreme Event-I Limit State, γ_{EQ} equal to 0.25 for all bridges. This factor shall be increased to γ_{EQ} equal to 0.50 for bridges located in main state routes and congested roads.

Since the determination of live load factor, γ_{EQ} based on ADTT or based on bridges located in congested roads could be confusing and questionable, it is decided that live load factor of γ_{EQ} equal to 0.50 to be used for all WSDOT bridges regardless the bridge location or congestion.

The base construction temperature may be taken as 64°F for the determination of Temperature Load.

The load factors γ_{TG} and γ_{SE} are to be determined on a project specific basis in accordance with Articles 3.4.1 and 3.12 of the *LRFD Specifications*. Load Factors for Permanent Loads, γ_p are provided in AASHTO *LRFD Specifications* Table 3.4.1-2.

The load factor for down drag loads shall be as specified in the AASHTO Specifications Table 3.4.1-2. The Geotechnical Report will provide the down drag force (*DD*). The down drag force (*DD*) is a load applied to the pile/shaft with the load factor specified in the Geotechnical Report. Generally, live loads (*LL*) are less than the down drag force and should be omitted when considering down drag forces.

The Load Factors for Superimposed Deformations, γ_p are provided in Table 3.5-3.

	PS	CR, SH
Superstructure	1.0	1.0
Fixed (bottom) substructure supporting Superstructure (using I_g only)	0.5	0.5
All other substructure supporting Superstructure (using I_g or $I_{effective}$)	1.0	1.0

Load Factors for Superimposed Deformations
Table 3.5-3

3.5.1 Load Factors for Substructure

Table 3.5-4 provides general guidelines for when to use the maximum or minimum shaft/pile/column permanent load factors for axial capacity, uplift, and lateral loading.

In general, substructure design should use unfactored loads to obtain force distribution in the structure, and then factor the resulting moment and shear for final structural design. All forces and load factors are as defined previously.

Axial Capacity	Uplift	Lateral Loading
DC_{max}, DW_{max}	DC_{min}, DW_{min}	DC_{max}, DW_{max}
DC_{max}, DW_{max} for causing shear	DC_{max}, DW_{max} for causing shear	DC_{max}, DW_{max} causing shear
DC_{min}, DW_{min} for resisting shear	DC_{min}, DW_{min} *for resisting shear*	DC_{min}, DW_{min} resisting shear
DC_{max}, DW_{max} for causing moments	DC_{max}, DW_{max} for causing moments	DC_{max}, DW_{max} for causing moments
DC_{min}, DW_{min} for resisting moments	DC_{min}, DW_{min} for resisting moments	DC_{min}, DW_{min} for resisting moments
EV_{max}	EV_{min}	EV_{max}
DD = varies	DD = varies	DD = varies
EH_{max}	EH_{max} if causes uplift	EH_{max}

Minimum/Maximum Substructure Load Factors for Strength Limit State
Table 3.5-4

In the table above, “causing moment” and “causing shear” are taken to be the moment and shear causing axial, uplift, and lateral loading respectively. “Resisting” is taking to mean those force effects that are diminishing axial capacity, uplift, and lateral loading.

3.9 Live Loads

3.9.1 Live Load Designation

Live load design criteria are specified in the lower right corner of the bridge preliminary plan sheet. The Bridge Projects Unit determines the criteria using the following guideline:

- New bridges and Bridge widening with addition of substructure – HL-93
- Bridge superstructure widening with no addition of substructure – Live load criteria of the original design
- Detour and other temporary bridges – 75 percent of HL-93

3.9.2 Live Load Analysis of Continuous Bridges

The HL-93 live load model defined in the *LRFD Specifications* includes a dual truck train for negative moments and reactions and interior piers. The application of the dual truck train is somewhat unclear as specified in LRFD Article 3.6.1.3.1. WSDOT interprets that article as follows:

For negative moment between the points of contraflexure under a uniform load on all spans and reactions at interior piers only, 90 percent of the effect of two design trucks spaced a minimum of 50.0 feet between the rear axle of the lead truck and the lead axle of the rear truck, combined with 90 percent of the effect of the design lane load. The distance between the 32.0-kip axles of each truck shall be taken as 14.0 feet. The two design trucks shall be placed in adjacent spans in such position to produce maximum force effect.

Negative moment <u>between the points of contraflexure under a uniform load on all spans</u> and reactions at interior supports shall be investigated a dual design tandem spaced from 26.0 feet to 40.0 feet apart, combined with the design lane load specified in LRFD Article C3.6.1.3.1. For the purpose of this article, the pairs of the design tandem shall be placed in adjacent spans in such position to produce maximum force effect.

3.9.3 Loading for Live Load Deflection Evaluation

The loading for live load deflection criteria is defined in LRFD Article 3.6.1.3.2. Live load deflections for the Service I limit state shall satisfy the requirements of LRFD 2.5.2.6.2.

3.9.4 Distribution to Superstructure

A. **Multi Girder Superstructure** – The live load distribution factor for exterior girder of multi girder bridges shall be as follows:

   - For exterior girder design with slab cantilever length equal or less than <u>40 percent</u> of the adjacent interior girder spacing, use the live load distribution factor for interior girder. The slab cantilever length is defined as the distance from the centerline of the exterior girder to the edge of the slab.
   - For exterior girder design with slab cantilever length exceeding <u>40 percent</u> of the adjacent interior girder spacing, use the lever rule with the multiple presence factor of 1.0 for single lane to determine the live load distribution. The live load used to design the exterior girder shall not be less than the live load used for the adjacent interior girder.
   - The special analysis based on the conventional approximation of loads on piles as described in LRFD Article C4.6.2.2.2d shall not be used unless the effectiveness of diaphragms on the lateral distribution of truck load is investigated.

B. **Concrete Box Girders** – The load distribution factor for multi-cell cast in place concrete box girders shall be per *LRFD Specifications* for interior girders from Table 4.6.2.2.2b-1 for bending moment, and Table 4.6.2.2.3a-1 for shear. The live load distribution factor for interior girders shall then be multiplied by the number of webs to obtain the design live load for the entire superstructure. The correction factor for live load distribution for skewed support as specified in Tables 4.6.2.2.2e-1 for bending moment and 4.6.2.2.3c-1 for shear shall be considered.

$DF = N_b \times Df_i$ Live load distribution factor for multi-cell box girder (3.9.4-1)

Where:

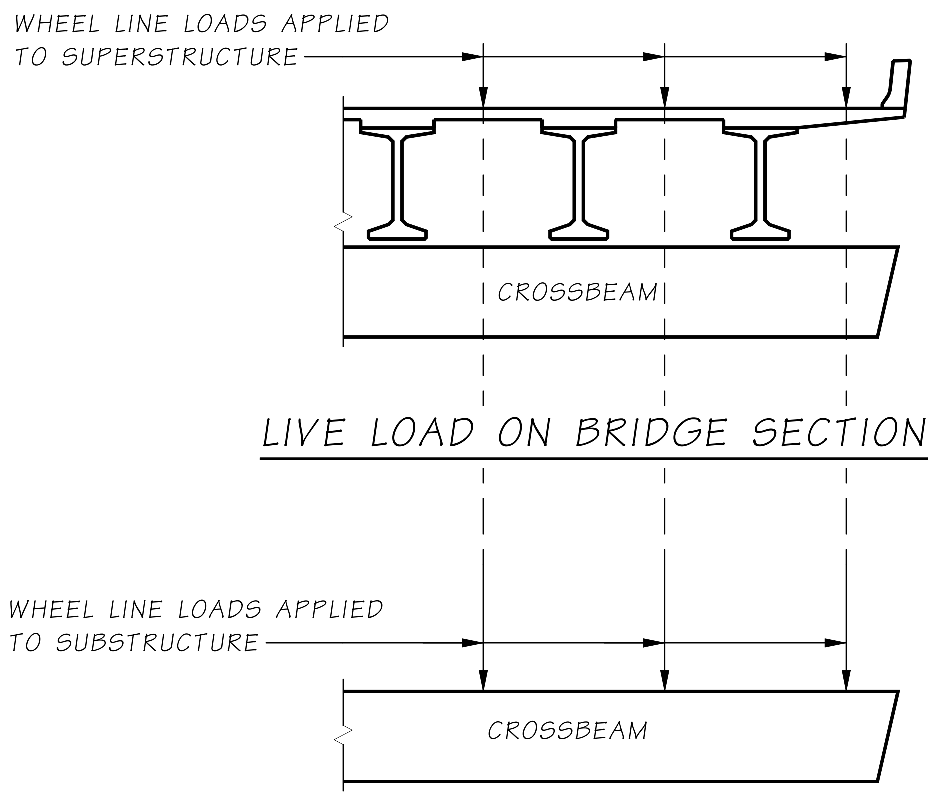
Df_i = Live load distribution factor for interior web

N_b = Number of webs

C. **Multiple Presence Factors** – A reduction factor will be applied in the substructure design for multiple loadings in accordance with AASHTO.

D. **Distribution to Substructure** – The number of traffic lanes to be used in the substructure design shall be determined by dividing the entire roadway slab width by 12. No fractional lanes shall be used. Roadway slab widths of less than 24 feet shall have a maximum of two design lanes.

E. **Distribution to Crossbeam** – The HL-93 loading is distributed to the substructure by placing wheel line reactions in a lane configuration that generates the maximum stress in the substructure. A wheel line reaction is ½ of the HL-93 reaction. Live loads are considered to act directly on the substructure without further distribution through the superstructure as illustrated in Figure 3.9-1. Normally, substructure design will not consider live load torsion or lateral distribution. Sidesway effects may be accounted for and are generally included in computer generated frame analysis results.



Live Load Distribution to Substructure

Figure 3.9-1

For steel and prestressed concrete superstructure where the live load is transferred to substructure through bearings, cross frames or diaphragms, the girder reaction may be used for substructure design. Live load placement is dependent on the member under design. Some examples of live load placement are as follows. The exterior vehicle wheel is placed 2 feet from the curb for maximum crossbeam cantilever moment or maximum eccentric foundation moment.

For crossbeam design between supports, the HL-93 lanes are placed to obtain the maximum positive moment in the member; then re-located to obtain the maximum shear or negative moment in the member.

For column design, the design lanes are placed to obtain the maximum transverse moment at the top of the column; then re-located to obtain the maximum axial force of the column.

3.9.5 Bridge Load Rating

Bridge designers are responsible for the bridges inventory and load rating of new bridges in accordance with the NBIS and the AASHTO Manual for Condition Evaluation of Bridge, the latest edition. See BDM Chapter 13.

3.12 Noise Barriers

The design requirement for noise barrier wall on bridges and walls are as follows:

1. The total height of noise barrier wall on bridges, from top of slab to top of noise barrier wall, shall be limited to 8′-0″.
2. The total height of noise barrier wall on retaining walls, from top of roadway to top of noise barrier wall, shall be limited to 14′-0″.
3. Noise barrier wall thickness shall be 7″ minimum with two layers of reinforcing bars in the cross section, with 1½″ minimum concrete cover on both faces. Self-consolidating concrete (SCC) shall be specified for cast-in-place (CIP) concrete noise walls. If conventional concrete is used for CIP noise walls, the minimum wall thickness shall be increased to 8″ with 1½″ minimum concrete cover on both faces and 2½″ minimum opening between two layers of reinforcing bars. The minimum wall thickness of 7″ with 1½″ minimum concrete cover on both faces, as shown in the attached detail, is adequate for precast noise walls (Figure 3.12-1).
4. All noise barriers which will be mounted on existing structures, supported by existing structures, or constructed as part of a new structure, shall be evaluated by the Bridge and Structures Office and the Geotechnical Office.
5. Wind load shall be based on Section 3.11 of this manual.
6. The vehicular collision force shall be based on the AASHTO LRFD Table A13.2-1 for design forces for traffic railing. The transverse force shall be applied horizontally at 3′-6″ height above deck.
7. Seismic load shall be as follows:

$$\text{Seismic Dead Load} = A \times f \times D \qquad (3.12\text{-}1)$$

Where:

A = Acceleration coefficient from the Geotechnical Report
D = Dead load of the wall
f = Dead load coefficient

Dead Load Coefficient, *f*	
Dead load coefficient, except on bridges – monolithic connection	1.0
Dead load coefficient, on bridges – monolithic connection	2.5
Dead load coefficient, for connection of precast wall to bridge barrier	8.0
Dead load coefficient, for connection of precast walls to retaining wall or moment slab barriers	5.0

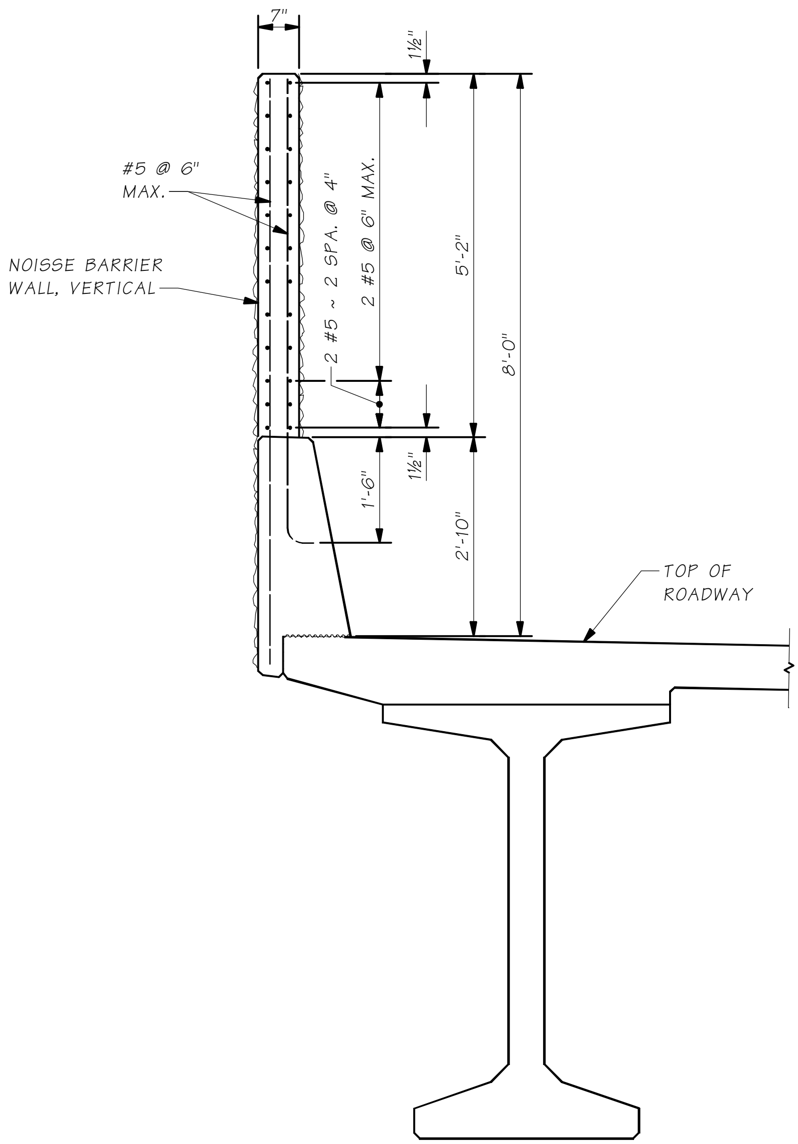
The product of A and f shall not be taken less than 0.10.

8. AASHTO LRFD Bridge design specifications shall be used for the structural design of noise barrier walls.

3.12.1 Standard Plan Noise Barrier Walls

This memorandum provides guidelines for the use of WSDOT Standard Plan Noise Barrier Walls.

The Standard Plan Noise Barrier Walls shall not be used for WSDOT projects where the seismic acceleration exceeds 0.3g. Noise barrier walls in projects where seismic acceleration exceeds 0.3g are considered special designs and shall be redesigned on a case-by-case basis.



NOISE BARRIER WALL ON BRIDGE.

TRAFFIC BARRIER REINFORCEMENT NOT SHOWN FOR CLARITY.

Noise Barrier Wall on Bridge
Figure 3.12-1

3.99 References

1. AASHTO, *LRFD Bridge Design Specifications for Design of Highway Bridges*, 6th Edition 2012.

Chapter 5 Concrete Structures — Contents

Chapter 5 Concrete Structures

5.0 General

The provisions in this section apply to the design of cast-in-place (CIP) and precast concrete structures.

Design of concrete structures shall be based on the requirements and guidance cited herein and in the current *AASHTO LRFD Bridge Design Specifications*, AASHTO Guide Specifications for LRFD Seismic Bridge Design, WSDOT General and Bridge Special Provisions and the WSDOT *Standard Specifications for Road, Bridge, and Municipal Construction* M 41-10.

5.1 Materials

5.1.1 Concrete

A. **Strength of Concrete** – Pacific NW aggregates have consistently resulted in excellent concrete strengths, which may exceed 10,000 psi in 28 days. Specified concrete strengths should be rounded to the next highest 100 psi.

1. **CIP Concrete Bridges** – Since conditions for placing and curing concrete for CIP components are not as controlled as they are for precast bridge components, Class 4000 concrete is typically used. Where significant economy can be gained or structural requirements dictate, Class 5000 concrete may be used with the approval of the Bridge Design Engineer, Bridge Construction Office, and Materials Lab.

2. **Precast Girders** – Nominal 28-day concrete strength (f'_c) for precast girders is 7,000 psi. Where higher strengths would eliminate a line of girders, a maximum of 10,000 psi can be specified.

   The minimum concrete compressive strength at release (f'_{ci}) for each prestressed girder shall be shown in the plans. For high strength concrete, the compressive strength at release shall be limited to 7,500 psi. Release strengths of up to 8,500 psi can be achieved with extended curing for special circumstances.

B. **Classes of Concrete**

1. **Class 3000** – Used in large sections with light to nominal reinforcement, mass pours, sidewalks, curbs, gutters, and nonstructural concrete guardrail anchors, luminaire bases.
2. **Class 4000** – Used in CIP post-tensioned or conventionally reinforced concrete box girders, slabs, traffic and pedestrian barriers, approach slabs, footings, box culverts, wing walls, curtain walls, retaining walls, columns, and crossbeams.
3. **Class 4000A** – Used for bridge approach slabs.
4. **Class 4000D** – Used for all CIP bridge decks unless otherwise approved by the WSDOT Bridge Design Engineer.
5. **Class 4000P** – Used for CIP pile and shaft.
6. **Class 4000W** – Used underwater in seals.
7. **Class 5000 or Higher** – Used in CIP post-tensioned concrete box girder construction or in other special structural applications if significant economy can be gained or structural requirements dictate. Class 5000 concrete is available within a 30-mile radius of Seattle, Spokane, and Vancouver. Outside this 30-mile radius, concrete suppliers may not have the quality control procedures and expertise to supply Class 5000 concrete.

The 28-day compressive design strengths (f'_c) are shown in Table 5.1.1-1.

Classes of Concrete	f'_c (psi)
COMMERCIAL	2300
3000	3000
4000, 4000A, 4000D	4000
4000W	2400*
4000P	3400**
5000	5000
6000	6000

*40 percent reduction from Class 4000.

**15 percent reduction from Class 4000 for piles and shafts.

28-Day Compressive Design Strength
Table 5.1.1-1

C. **Relative Compressive Concrete Strength**

1. During design or construction of a bridge, it is necessary to determine the strength of concrete at various stages of construction. For instance, Section 6-02.3(17)J of the WSDOT *Standard Specifications* discusses the time at which falsework and forms can be removed to various percentages of the concrete design strength. Occasionally, construction problems will arise which require a knowledge of the relative strengths of concrete at various ages. Table 5.1.1-2 shows the approximate values of the minimum compressive strengths of different classes of concrete at various ages. If the concrete has been cured under continuous moist curing at an average temperature, it can be assumed that these values have been developed.
2. Curing of the concrete (especially in the first 24 hours) has a very important influence on the strength development of concrete at all ages. Temperature affects the rate at which the chemical reaction between cement and water takes place. Loss of moisture can seriously impair the concrete strength.
3. If test strength is above or below that shown in Table 5.1.1-2, the age at which the design strength will be reached can be determined by direct proportion.

   For example, if the relative strength at 10 days is 64 percent instead of the minimum 70 percent shown in Table 5.1.1-2, the time it takes to reach the design strength can be determined as follows:

   Let x = relative strength to determine the age at which the concrete will reach the design strength

$$\frac{x}{70} = \frac{100}{64} \quad Therefore, x = 110\% \tag{5.1.1-1}$$

   From Table 5.1.1-2, the design strength should be reached in 40 days.

Age	Relative Strength	Class 5000	Class 4000	Class 3000		Age	Relative Strength	Class 5000	Class 4000	Class 3000
Days	%	ksi	ksi	ksi		Days	%	ksi	ksi	ksi
3	35	1.75	1.40	1.05		20	91	4.55	3.64	2.73
4	43	2.15	1.72	1.29		21	93	4.65	3.72	2.79
5	50	2.50	2.00	1.50		22	94	4.70	3.76	2.82
6	55	2.75	2.20	1.65		23	95	4.75	3.80	2.85
7	59	2.95	2.36	1.77		24	96	4.80	3.84	2.88
8	63	3.15	2.52	1.89		25	97	4.85	3.88	2.91
9	67	3.35	2.68	2.01		26	98	4.90	3.92	2.94
10	70	3.5	2.80	2.10		27	99	4.95	3.96	2.97
11	73	3.65	2.92	2.19		28	100	5.00	4.00	3.00
12	75	3.75	3.00	2.25		30	102	5.10	4.08	3.06
13	77	3.85	3.08	2.31		40	110	5.50	4.40	3.30
14	79	3.95	3.16	2.37		50	115	5.75	4.60	3.45
15	81	4.05	3.24	2.43		60	120	6.00	4.80	3.60
16	83	4.15	3.32	2.49		70	125	6.25	5.00	3.75
17	85	4.25	3.34	2.55		80	129	6.45	5.16	3.87
18	87	4.35	3.48	2.61		90	131	6.55	5.24	3.93
19	89	4.45	3.56	2.67		100	133	6.70	5.40	4.00

Relative and Compressive Strength of Concrete
Table 5.1.1-2

D. **Modulus of Elasticity** – The modulus of elasticity shall be determined as specified in AASHTO LRFD 5.4.2.4. For calculation of the modulus of elasticity, the unit weight of plain concrete (w_c) shall be taken as 0.155 kcf for precast pretensioned or post-tensioned spliced girders and 0.150 kcf for normal-weight concrete. The correction factor (K_1) shall normally be taken as 1.0.

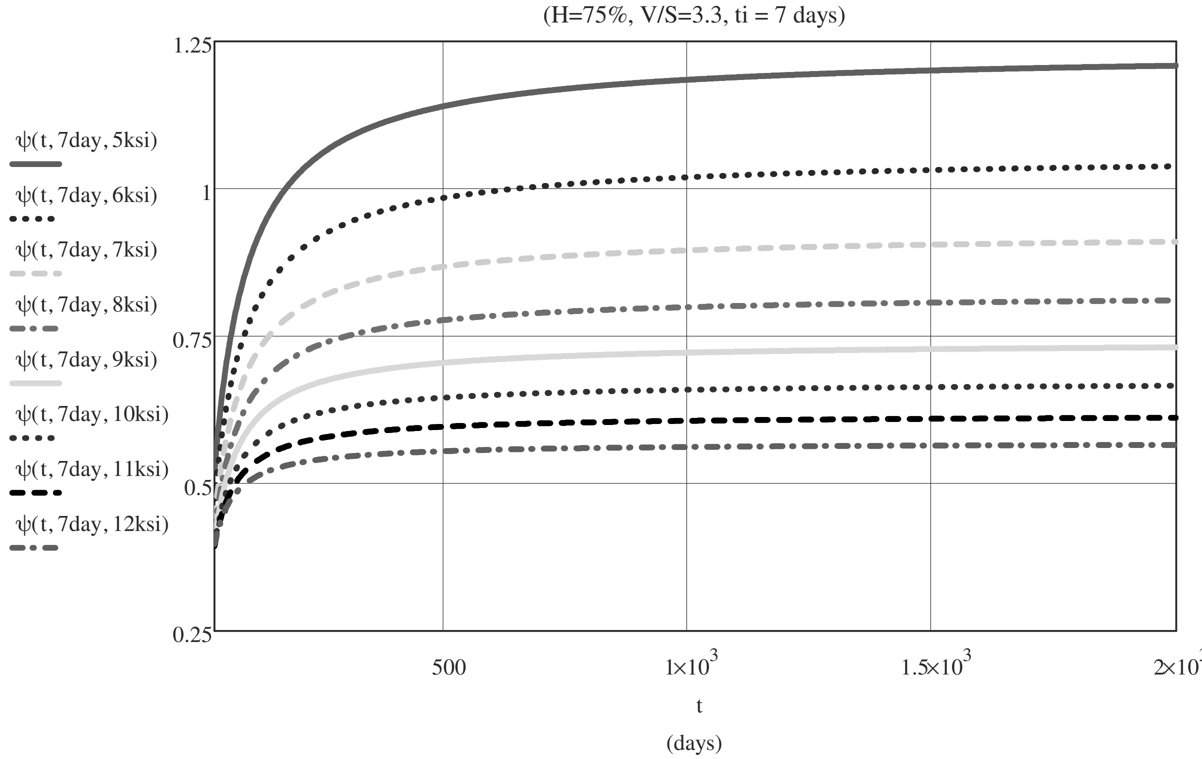
E. **Creep** – The creep coefficient shall be calculated per AASHTO LRFD 5.4.2.3.2. The relative humidity, *H*, may be taken as 75 percent for standard conditions. The maturity of concrete, *t*, may be taken as 2,000 days for standard conditions. The volume-to-surface ratio, *V/S*, is given in Table 5.6.1-1 for standard WSDOT girders.

In determining the maturity of concrete at initial loading, t_i, one day of accelerated curing by steam or radiant heat may be taken as equal to seven days of normal curing.

The final deflection is a combination of the elastic deflection and the creep effect associated with given loads shown by the equation below.

$$\Delta_{total} = \Delta_{elastic}[1 + \psi(t, t_i)] \tag{5.1.1-2}$$

Figure 5.1.1-1 provides creep coefficients for a range of typical initial concrete strength values, f'_{ci}, as a function of time from initial seven day steam cure (t_i = 7 days). The figure uses a volume-to-surface, *V/S*, ratio of 3.3 as an average for girders and relative humidity, *H*, equal to 75 percent.



Creep Coefficient for Standard Conditions as Function of Initial Concrete Strength
Figure 5.1.1-1

F. **Shrinkage** – Concrete shrinkage strain, ε_{sh}, shall be calculated per AASHTO LRFD.

G. **Grout** – Grout is usually a prepackaged cement based grout or nonshrink grout that is mixed, placed, and cured as recommended by the manufacturer. It is used under steel base plates for both bridge bearings and luminaries or sign bridge bases. Should the grout pad thickness exceed 4″, steel reinforcement shall be used. For design purposes, the strength of the grout, if properly cured, can be assumed to be equal to or greater than that of the adjacent concrete but not greater than 4000 psi. Nonshrink grout is used in keyways between precast prestressed tri-beams, double-tees, and deck bulb tees (see *Standard Specifications* Section 6-02.3(25)O for deck bulb tee exception).

H. **Mass Concrete** – Mass concrete is any volume of concrete with dimensions large enough to require that measures be taken to cope with the generation of heat from hydration of the cement and attendant volume change to minimize cracking. Temperature-related cracking may be experienced in thick-section concrete structures, including spread footings, pile caps, bridge piers, crossbeams, thick walls, and other structures as applicable.

Concrete placements with least dimension greater than 6 feet should be considered mass concrete, although smaller placements with least dimension greater than 3 feet may also have problems with heat generation effects. Shafts need not be considered mass concrete.

The temperature of mass concrete shall not exceed 160°F. The temperature difference between the geometric center of the concrete and the center of nearby exterior surfaces shall not exceed 35°F.

Designers could mitigate heat generation effects by specifying construction joints and placement intervals. Designers should consider requiring the Contractor to submit a thermal control plan, which may include such things as:

1. Temperature monitors and equipment.
2. Insulation.

3. Concrete cooling before placement.
4. Concrete cooling after placement, such as by means of internal cooling pipes.
5. Use of smaller, less frequent placements.
6. Other methods proposed by the Contractor and approved by the Engineer.

Concrete mix design optimization, such as using low-heat cement, fly ash or slag cement, low-water/cement ratio, low cementitious materials content, larger aggregate, etc. is acceptable as long as the concrete mix meets the requirements of the *Standard Specifications* for the specified concrete class.

The ACI Manual of Concrete Practice Publication 207 and specifications used for the Tacoma Narrows Bridge Project suspension cable anchorages (2003-2006) can be used as references.

I. **Self-Consolidating Concrete (SCC)** – Self-consolidating concrete (SCC) shall not be used in structural members. SCC may be used for other applications such as precast noise wall panels, barriers, three-sided structures, etc. as described in *Standard Specifications* 6-02.3(27).

Designers shall consider potential effects on mechanical and visco-elastic properties including lower modulus of elasticity, higher creep coefficient, higher shrinkage strain, longer bond transfer and development lengths of strands, flexural and shear strengths, etc.[25]

J. **Shotcrete** – Shotcrete could be used as specified in WSDOT Standard Plans. Shotcrete may not be suitable for some critical applications unless approved by the Engineer of Record.

Substitution of CIP conventional concrete in the contract document with shotcrete needs the approval of the Engineer of Record.

Some of the shortfalls of shotcrete as compared to conventional CIP concrete include:

- **Durability** – Conventional concrete is placed in forms and vibrated for consolidation. Shotcrete, whether placed by wet or dry material feed, is pneumatically applied to the surface and is not consolidated as conventional concrete. Due to the difference in consolidation, permeability can be affected. If the permeability is not low enough, the service life of the shotcrete will be affected and may not meet the minimum of 75 years specified for conventional concretes.

  Observation of some projects indicates the inadequate performance of shotcrete to properly hold back water. This results in leaking and potential freezing, seemingly at a higher rate than conventional concrete. Due to the method of placement of shotcrete, air entrainment is difficult to control. This leads to less resistance of freeze/thaw cycles.
- **Cracking** – There is more cracking observed in shotcrete surfaces compared to conventional concrete. Excessive cracking in shotcrete could be attributed to its higher shrinkage, method of curing, and lesser resistance to freeze/thaw cycles. The shotcrete cracking is more evident when structure is subjected to differential shrinkage.
- **Corrosion Protection** – The higher permeability of shotcrete places the steel reinforcement (whether mesh or bars) at a higher risk of corrosion than conventional concrete applications. Consideration for corrosion protection may be necessary for some critical shotcrete applications.
- **Safety** – Carved shotcrete and shotcrete that needs a high degree of relief to accent architectural features lead to areas of 4″-6″ of unreinforced shotcrete. These areas can be prone to an accelerated rate of deterioration. This, in turn, places pedestrians, bicyclists, and traffic next to the wall at risk of falling debris.
- **Visual Quality and Corridor Continuity** – As shotcrete is finished by hand, standard architectural design, as defined in the WSDOT *Design Manual* M 22-01, typically cannot be met. This can create conflicts with the architectural guidelines developed for the corridor. Many times the guidelines are developed with public input. If the guidelines are not met, the public develops a distrust of the process. In other cases, the use of faux rock finishes, more commonly used by the private sector, can create the perception of the misuse of public funds.

K. **Lightweight Aggregate Concrete** – Lightweight aggregate concrete may be used for precast and CIP members upon approval of the WSDOT Bridge Design Engineer.

5.1.2 Reinforcing Steel

A. **Grades** – Reinforcing bars shall be deformed and shall conform to Section 9-07.2 of the *Standard Specifications*. ASTM A706 Grade 60 reinforcement is preferred for WSDOT bridges and structures.

1. **Grade 80 Reinforcement** – Reinforcement conforming to ASTM A706 Grade 80 may be used in Seismic Design Category (SDC) A for all components. For SDCs B, C and D, ASTM A706 Grade 80 reinforcing steel shall not be used for elements and connections that are proportioned and detailed to ensure the development of significant inelastic deformations for which moment curvature analysis is required to determine the plastic moment capacity of ductile concrete members and expected nominal moment capacity of capacity protected members.

   ASTM A706 Grade 80 reinforcing steel may be used for capacity-protected members such as footings, bent caps, oversized shafts, joints, and integral superstructure elements that are adjacent to the plastic hinge locations if the expected nominal moment capacity is determined by strength design based on the expected concrete compressive strength with a maximum usable strain of 0.003 and a reinforcing steel yield strength of 80 ksi with a maximum usable strain of 0.090 for #10 bars and smaller, 0.060 for #11 bars and larger. The resistance factors for seismic related calculations shall be taken as 0.90 for shear and 1.0 for bending.

   ASTM A706 Grade 80 reinforcing steel shall not be used for oversized shafts where in-ground plastic hinging is considered as a part of the Earthquake-Resisting System (ERS).

   ASTM A706 Grade 80 reinforcing steel shall not be used for transverse and confinement reinforcement.

   For seismic hooks, f_y shall not be taken greater than 75 ksi.

   a. **Modifications to Resistance Factors for Conventional Construction** (AASHTO *LRFD Bridge Design Specifications* 5.5.4.2.1)

      For sections in which the net tensile strain in the extreme tension steel at nominal resistance is between the limits for compression-controlled and tension-controlled sections, φ may be linearly increased from 0.75 to that for tension-controlled sections as the net tensile strain in the extreme tension steel increases from the compression controlled strain limit, ε_{cl}, to the tension-controlled strain limit, ε_{tl}.

      This variation φ may be computed for prestressed members such that:

$$0.75 \le \varphi = 0.75 + \frac{0.25(\varepsilon_t - \varepsilon_{cl})}{(\varepsilon_{tl} - \varepsilon_{cl})} \le 1.0$$

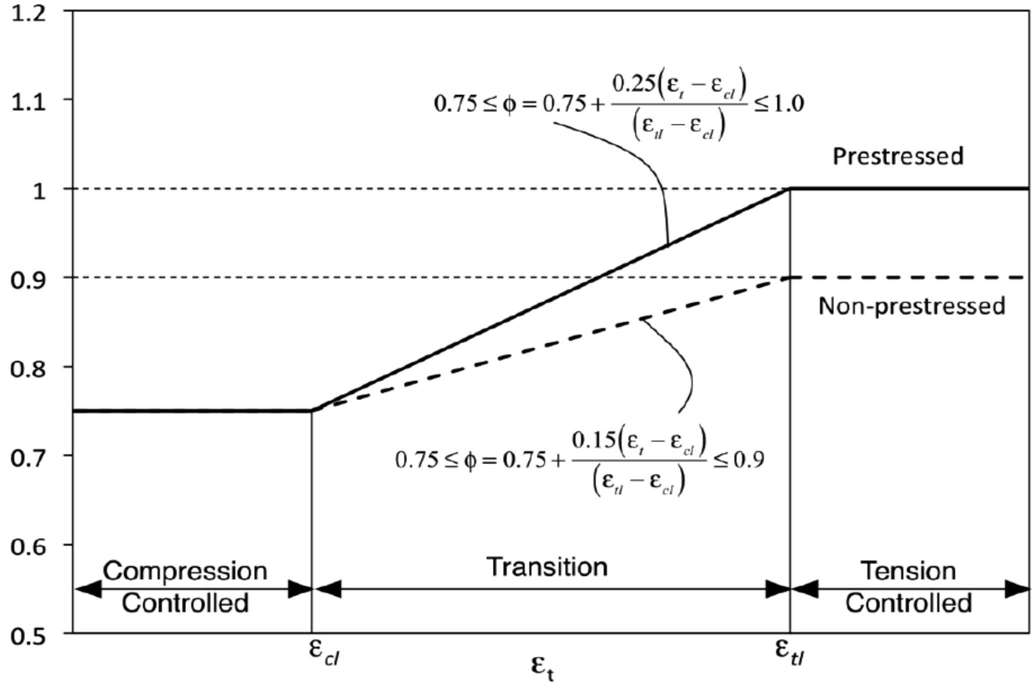
      and for nonprestressed members such that:

$$0.75 \le \varphi = 0.75 + \frac{0.15(\varepsilon_t - \varepsilon_{cl})}{(\varepsilon_{tl} - \varepsilon_{cl})} \le 0.9$$

      Where:

      ε_t = net tensile strain in the extreme tension steel at nominal resistance
      ε_{cl} = compression-controlled strain limit in the extreme tension steel (in./in.)
      ε_{tl} = tension-controlled strain limit in the extreme tension steel (in./in.)

      For sections subjected to axial load with flexure, factored resistances are determined by multiplying both P_n and M_n by the appropriate single value of φ. Compression-controlled and tension-controlled sections are defined as those that have net tensile strain in the extreme tension steel at nominal strength less than or equal to the compression-controlled strain limit, and equal to or greater than the tension-controlled strain limit, respectively. For sections with net tensile strain ε_t in the extreme tension steel at nominal strength between the above limits, the value of φ may be determined by linear interpolation, as shown in Figure 5.1.2-1.



Variation of φ with Net Tensile Strain ε_t
Figure 5.1.2-1

b. **Modifications to General Assumptions for Strength and Extreme Event Limit States** (*AASHTO LRFD Bridge Design Specifications* 5.7.2.1)

Sections are compression-controlled when the net tensile strain in the extreme tension steel is equal to or less than the compression-controlled strain limit, ε_{cl}, at the time the concrete in compression reaches its assumed strain limit of 0.003. The compression-controlled strain limit is the net tensile strain in the reinforcement at balanced strain conditions. For Grade 60 reinforcement, and for all prestressed reinforcement, the compression-controlled strain limit may be set equal to $\varepsilon_{cl} = 0.002$. For nonprestressed reinforcing steel with a specified minimum yield strength of 80.0 ksi, the compression-controlled strain limit may be taken as $\varepsilon_{cl} = 0.003$. For nonprestressed reinforcing steel with a specified minimum yield strength between 60.0 and 80.0 ksi, the compression controlled strain limit may be determined by linear interpolation based on specified minimum yield strength.

Sections are tension-controlled when the net tensile strain in the extreme tension steel is equal to or greater than the tension-controlled strain limit, $\varepsilon_{tl,}$ just as the concrete in compression reaches its assumed strain limit of 0.003. Sections with net tensile strain in the extreme tension steel between the compression-controlled strain limit and the tension-controlled strain limit constitute a transition region between compression-controlled and tension-controlled sections. The tension-controlled strain limit, $\varepsilon_{tl,}$ shall be taken as 0.0056 for nonprestressed reinforcing steel with a specified minimum yield strength, $f_y = 80.0$ ksi.

In the approximate flexural resistance equations f_y and f'_y may replace f_s and f'_s, respectively, subject to the following conditions:

- f_y may replace f_s when, using f_y in the calculation, the resulting ratio c/d_s does not exceed:

$$\frac{c}{d_s} \leq \frac{0.003}{0.003 + \varepsilon_{cl}}$$

  Where:

  c = distance from the extreme compression fiber to the neutral axis (in.)
  d_s = distance from extreme compression fiber to the centroid of the nonprestressed tensile reinforcement (in.)
  ε_{cl} = compression-controlled strain limit as defined above.

  If c/d exceeds this limit, strain compatibility shall be used to determine the stress in the mild steel tension reinforcement.

- f'_y may replace f'_s when, using f'_y in the calculation, if $c \geq 3d'_s$, and $f_y \leq 60.0$ ksi. If $c < 3d'_s$, or $f_y > 60.0$ ksi, strain compatibility shall be used to determine the stress in the mild steel compression reinforcement. Alternatively, the compression reinforcement may be conservatively ignored, i.e., $A'_s = 0$.

When using strain compatibility, the calculated stress in the nonprestressed reinforcing steel may not be taken as greater than the specified minimum yield strength.

When using the approximate flexural resistance equations it is important to assure that both the tension and compression mild steel reinforcement are yielding to obtain accurate results. The current limit on c/d_s assures that the mild tension steel will be at or near yield. The ratio $c \geq 3d'_s$ assures that mild compression steel with $f_y \leq 60.0$ ksi will yield. For yield strengths above 60.0 ksi, the yield strain is close to or exceeds 0.003, so the compression steel may not yield. It is conservative to ignore the compression steel when calculating flexural resistance. In cases where either the tension or compression steel does not yield, it is more accurate to use a method based on the conditions of equilibrium and strain compatibility to determine the flexural resistance. For Grade 40 reinforcement the compression-controlled strain limit may be set equal to $\varepsilon_{cl} = 0.0014$.

Values of the compression- and tension-controlled strain limits are given in Table 5.1.2-1 for common values of specified minimum yield strengths.

Specified Minimum Yield Strength, ksi	Compression Control, ε_{cl}	Tension Control, ε_{tl}
40	0.0014	0.005
60	0.002	0.005
75	0.0026	0.0054
80	0.0028	0.0056

Compression and Tension Controlled Strain Limits
Table 5.1.2-1

c. **Modifications to Development of Reinforcement** (*AASHTO LRFD Bridge Design Specifications* 5.11.2)

Development lengths shall be calculated using the specified minimum yield strength of the reinforcing steel. Reinforcing steel with a specified minimum yield strength up to 80 ksi is permitted.

For straight bars having a specified minimum yield strength greater than 75 ksi, transverse reinforcement satisfying the requirements of *AASHTO LRFD Bridge Design Specifications* 5.8.2.5 for beams and 5.10.6.3 for columns shall be provided over the required development length. Confining reinforcement is not required for slabs or decks.

For hooks in reinforcing bars having a specified minimum yield strength greater than 60 ksi, ties satisfying the requirements of *AASHTO LRFD Bridge Design Specifications* 5.11.2.4.3 shall be provided. For hooks not located at the discontinuous end of a member, the modification factors of *AASHTO LRFD Bridge Design Specifications* 5.11.2.4.2 may be applied.

d. **Modifications to Splices of Bar Reinforcement** (*AASHTO LRFD Bridge Design Specifications* 5.11.5)

For lap spliced bars having a specified minimum yield strength greater than 75 ksi, transverse reinforcement satisfying the requirements of *AASHTO LRFD Bridge Design Specifications* 5.8.2.5 for beams and 5.10.6.3 for columns shall be provided over the required splice length. Confining reinforcement is not required for slabs or decks.

B. **Sizes** – Reinforcing bars are referred to in the contract plans and specifications by number and vary in size from #3 to #18. For bars up to and including #8, the number of the bar coincides with the bar diameter in eighths of an inch. The #9, #10, and #11 bars have diameters that provide areas equal to 1″ × 1″ square bars, 1⅛″ × 1⅛″ square bars and 1¼″ × 1¼″ square bars respectively. Similarly, the #14 and #18 bars correspond to 1½″ × 1½″ and 2″ × 2″ square bars, respectively. Appendix 5.1-A3 shows the sizes, number, and various properties of the types of bars used in Washington State.

C. **Development**

1. **Tension Development Length** – Development length or anchorage of reinforcement is required on both sides of a point of maximum stress at any section of a reinforced concrete member. Development of reinforcement in tension shall be per AASHTO LRFD 5.11.2.1.

   Appendix 5.1-A4 shows the tension development length for both uncoated and epoxy coated Grade 60 bars for normal weight concrete with specified strengths of 3,000 to 6,000 psi.

2. **Compression Development Length** – Development of reinforcement in compression shall be per AASHTO LRFD 5.11.2.2. The basic development lengths for deformed bars in compression are shown in Appendix 5.1-A5. These values may be modified as described in AASHTO. However, the minimum development length shall be 1′-0″.

3. **Tension Development Length of Standard Hooks** – Standard hooks are used to develop bars in tension where space limitations restrict the use of straight bars. Tension development length of 90° & 180° standard hooks are shown in Appendix 5.1-A6.

D. **Splices** – Three methods are used to splice reinforcing bars: lap splices, mechanical splices, and welded splices. The Contract Plans shall clearly show the locations and lengths of splices. Splices shall be per AASHTO LRFD 5.11.5.

Lap splicing of reinforcing bars is the most common method. No lap splices, for either tension or compression bars, shall be less than 2′-0″.

1. **Tension Lap Splices** – Many of the same factors which affect development length affect splices. Consequently, tension lap splices are a function of the bar's development length, l_d. There are three classes of tension lap splices: Class A, B, and C. Designers are encouraged to splice bars at points of minimum stress and to stagger lap splices along the length of the bars.

   Appendix 5.1-A7 shows tension lap splices for both uncoated and epoxy coated Grade 60 bars for normal weight concrete with specified strengths of 3,000 to 6,000 psi.

2. **Compression Lap Splices** – The compression lap splices shown in Appendix 5.1-A5 are for concrete strengths greater than 3,000 psi. If the concrete strength is less than 3,000 psi, the compression lap splices shall be increased by one third. Note that when two bars of different diameters are lap spliced, the length of the lap splice shall be the larger of the lap splice for the smaller bar or the development length of the larger bar.

3. **Mechanical Splices** – Mechanical splices are proprietary splicing mechanisms. The requirements for mechanical splices are found in AASHTO LRFD 5.5.3.4 and 5.11.5.2.2.

4. **Welded Splices** – ASHTO LRFD 5.11.5.2.3 describes the requirements for welded splices. On modifications to existing structures, welding of reinforcing bars may not be possible because of the non-weldability of some steels.

E. **Hooks and Bends** – For hook and bend requirements, see AASHTO LRFD 5.10.2. Standard hooks and bend radii are shown in Appendix 5.1-A1.

F. **Fabrication Lengths** – Reinforcing bars are available in standard mill lengths of 40′ for bar sizes #3 and #4 and 60′ for bar sizes of #5 and greater. Designers shall limit reinforcing bar lengths to the standard mill lengths. Because of placement considerations, designers should consider limiting the overall lengths of bar size #3 to 30′ and bar size #5 to 40′.

Spirals of bar sizes #4 through #6 are available on 5,000 lb coils. Spirals should be limited to a maximum bar size of #6.

G. **Placement** – Placement of reinforcing bars can be a problem during construction. Sometimes it may be necessary to make a large scale drawing of reinforcement to look for interference and placement problems in confined areas. If interference is expected, additional details are required in the contract plans showing how to handle the interference and placement problems. Appendix 5.1-A2 shows the minimum clearance and spacing of reinforcement for beams and columns.

H. **Joint and Corner Details**

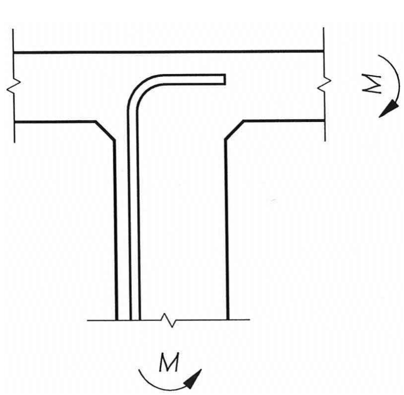
1. **T-Joint** – The forces form a tension crack at 45° in the joint. Reinforcement as shown in Figure 5.1.2-2 is more than twice as effective in developing the strength of the corner than if the reinforcement was turned 180°.

2. **"Normal" Right Corners** – Corners subjected to bending as shown in Figure 5.1.2-3 will crack radially in the corner outside of the main reinforcing steel. Smaller size reinforcing steel shall be provided in the corner to distribute the radial cracking.

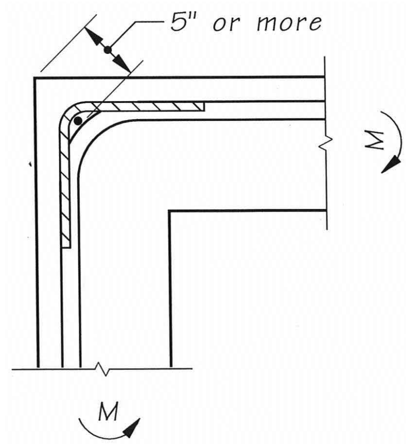
3. **Right or Obtuse Angle Corners** – Corners subjected to bending as shown in Figure 5.1.2-4 tend to crack at the reentrant corner and fail in tension across the corner. If not properly reinforced, the resisting corner moment may be less than the applied moment.

   Reinforced as shown in Figure 5.1.2-4, but without the diagonal reinforcing steel across the corner, the section will develop 85 percent of the ultimate moment capacity of the wall. If the bends were rotated 180°, only 30 percent of the wall capacity would be developed.

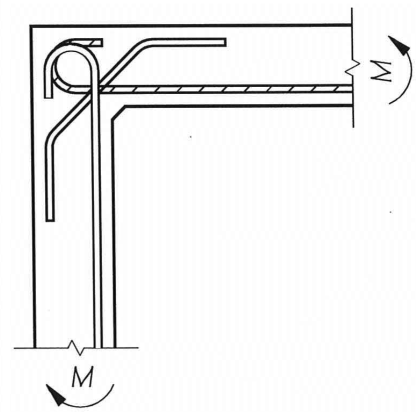
   Adding diagonal reinforcing steel across the corner, approximately equal to 50 percent of the main reinforcing steel, will develop the corner strength to fully resist the applied moment. Extend the diagonal reinforcement past the corner each direction for anchorage. Since this bar arrangement will fully develop the resisting moment, a fillet in the corner is normally unnecessary.



T-Joint Reinforcing Details
Figure 5.1.2-2



"Normal" Right Corner Reinforcing Details
Figure 5.1.2-3



Right or Obtuse Angle Corner Reinforcing Details
Figure 5.1.2-4

I. **Welded Wire Reinforcement in Precast Prestressed Girders** – Welded wire reinforcement can be used to replace mild steel reinforcement in precast prestressed girders. Welded wire reinforcement shall meet all AASHTO requirements (see AASHTO LRFD 5.4.3, 5.8.2.6, 5.8.2.8, C.5.8.2.8, 5.10.6.3, 5.10.7, 5.10.8, 5.11.2.6.3, etc.).

The yield strength shall be greater than or equal to 60 ksi. The design yield strength shall be 60 ksi. Welded wire reinforcement shall be deformed. Welded wire reinforcement shall have the same area and spacing as the mild steel reinforcement that it replaces.

Shear stirrup longitudinal wires (tack welds) shall be excluded from the web of the girder and are limited to the flange areas as described in AASHTO LRFD 5.8.2.8. Longitudinal wires for anchorage of welded wire reinforcement shall have an area of 40 percent or more of the area of the wire being anchored as described in ASTM A497 but shall not be less than D4.

5.1.3 Prestressing Steel

A. **General** – Three types of high-tensile steel used for prestressing steel are:

1. **Strands** – AASHTO M 203 Grade 270, low relaxation or stress relieved
2. **Bars** – AASHTO M 275 Type II
3. **Parallel Wires** – AASHTO M 204 Type WA

All WSDOT designs are based on low relaxation strands using either 0.5″ or 0.6″ diameter strands for girders, and ⅜″ or 7⁄16″ diameter strands for stay-in-place precast deck panels. Properties of uncoated and epoxy-coated prestressing stands are shown in Appendix 5.1-A8. 0.62″ and 0.7″ diameter strands may be used for top temporary strands in precast girders.

B. **Allowable Stresses** – Allowable stresses for prestressing steel are as listed in AASHTO LRFD 5.9.3.

C. **Prestressing Strands** – Standard strand patterns for all types of WSDOT prestressed girders are shown throughout Appendix 5.6-A and Appendix 5.9-A.

1. **Straight Strands** – The position of the straight strands in the bottom flange is standardized for each girder type.
2. **Harped Strands** – The harped strands are bundled between the harping points (the 0.4 and 0.6 points of the girder length). The girder fabricator shall select a bundle configuration that meets plan centroid requirements.

There are practical limitations to how close the centroid of harped strands can be to the bottom of a girder. The minimum design value for this shall be determined using the following guide: Up to 12 harped strands are placed in a single bundle with the centroid 4″ above the bottom of the girder. Additional strands are placed in twelve-strand bundles with centroids at 2″ spacing vertically upwards.

At the girder ends, the strands are splayed to a normal pattern. The centroid of strands at both the girder end and the harping point may be varied to suit girder stress requirements.

The slope of any individual harped strands shall not be steeper than 8 horizontal to 1 vertical for 0.6″ diameter strands, and 6 horizontal to 1 vertical for 0.5″ diameter strands.

The harped strand exit location at the girder ends shall be held as low as possible while maintaining the concrete stresses within allowable limits.

3. **Temporary Strands** – Temporary strands in the top flanges of girders may be required for shipping (see Section 5.6.3). These strands may be pretensioned and bonded only for the end 10 feet of the girder, or may be post-tensioned prior to lifting the girder from the form. These strands can be considered in design to reduce the required transfer strength, to provide stability during shipping, and to reduce the "A" dimension. These strands must be cut before the CIP intermediate diaphragms are placed.

D. **Development of Prestressing Strand** –

1. **General** – Development of prestressing strand shall be as described in AASHTO LRFD 5.11.4.

   The development length of bonded uncoated & coated prestressing strands are shown in Appendix 5.1-A8.

2. **Partially Debonded Strands** – Where it is necessary to prevent a strand from actively supplying prestress force near the end of a girder, it shall be debonded. This can be accomplished by taping a close fitting PVC tube to the stressed strand from the end of the girder to some point where the strand can be allowed to develop its load. Since this is not a common procedure, it shall be carefully detailed on the plans. It is important when this method is used in construction that the taping of the tube is done in such a manner that concrete cannot leak into the tube and provide an undesirable bond of the strand.

   Partially debonded strands shall meet the requirements of AASHTO LRFD 5.11.4.3.

3. **Strand Development Outside of Girder** – Extended bottom prestress strands are used to connect the ends of girders with diaphragms and resist loads from creep effects, shrinkage effects, and positive moments.

   Extended strands must be developed in the short distance within the diaphragm (between two girder ends at intermediate piers). This is normally accomplished by requiring strand chucks and anchors as shown in Figure 5.1.3-1. Strand anchors are normally installed at 1′-9″ from the girder ends. The number of extended strands shall not exceed one-half of the total number of straight strands in the girder and shall not be less than four.

   The designer shall calculate the number of extended straight strands needed to develop the required capacity at the end of the girder.

For fixed intermediate piers at the Extreme Event I limit state, the total number of extended strands for each girder end shall not be less than:

$$N_{ps} = 12[M_{sei} \cdot K - M_{SIDL}] \cdot \frac{1}{0.9\phi A_{ps} f_{py} d} \quad (5.1.3\text{-}1)$$

Where:

M_{sei}	=	Moment due to overstrength plastic moment capacity of the column and associated overstrength plastic shear, either within or outside the effective width, per girder, kip-ft
M_{SIDL}	=	Moment due to superimposed dead loads (traffic barrier, sidewalk, etc.) per girder, kip-ft
K	=	Span moment distribution factor as shown in Figure 5.1.3-2 (use maximum of K1 and K2)
A_{ps}	=	Area of each extended strand, in^2
f_{py}	=	Yield strength of prestressing steel specified in AASHTO LRFD Table 5.4.4.1-1, ksi
d	=	Distance from top of deck slab to c.g. of extended strands, in
φ	=	Flexural resistance factor, 1.0

The plastic hinging moment at the c.g. of the superstructure is calculated using the following:

$$M_{po}^{CG} = M_{po}^{top} + \frac{\left(M_{po}^{top} + M_{po}^{base}\right)}{L_c} h \quad (5.1.3\text{-}2)$$

Where:

M_{po}^{top}	=	Plastic overstrength moment at top of column, kip-ft
M_{po}^{base}	=	Plastic overstrength moment at base of column, kip-ft
h	=	Distance from top of column to c.g. of superstructure, ft
L_c	=	Column clear height used to determine overstrength shear associated with the overstrength moments, ft

For precast, prestressed girders with cast-in-place deck slabs, two-thirds of the plastic hinging moment at the c.g. of the superstructure shall be resisted by girders within the effective width. The remaining one-third shall be resisted by girders outside the effective width. The plastic hinging moment per girder is calculated using the following:

$$M_{sei}^{Int} = \frac{2M_{po}^{CG}}{3N_g^{int}} \text{ For girders within the effective width} \quad (5.1.3\text{-}3)$$

$$M_{sei}^{Ext} = \frac{M_{po}^{CG}}{3N_g^{ext}} \text{ For girders outside the effective width} \quad (5.1.3\text{-}4)$$

$$\text{If } M_{sei}^{Int} \geq M_{sei}^{Ext} \text{ then } M_{sei} = M_{sei}^{Int} \quad (5.1.3\text{-}5)$$

$$\text{If } M_{sei}^{Int} < M_{sei}^{Ext} \text{ then } M_{sei} = \frac{M_{po}^{CG}}{N_g^{int} + N_g^{ext}} \quad (5.1.3\text{-}6)$$

Where:

N_g^{int}	=	Number of girders encompassed by the effective width
N_g^{ext}	=	Number of girders outside the effective width

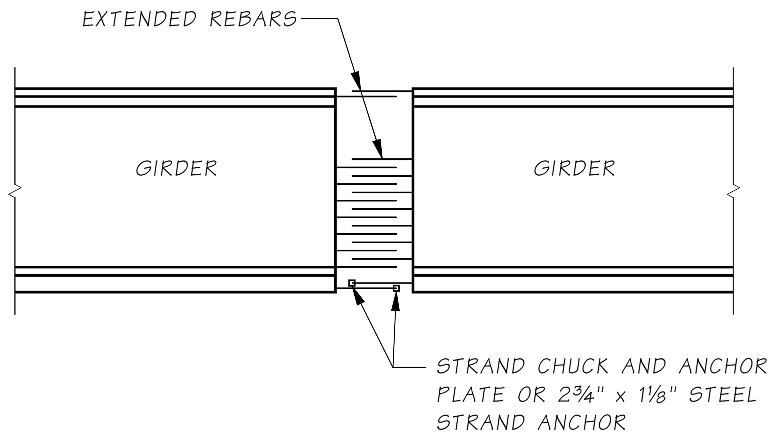
The effective width for the extended strand calculation shall be taken as:

$$B_{eff} = D_c + D_s \tag{5.1.3-7}$$

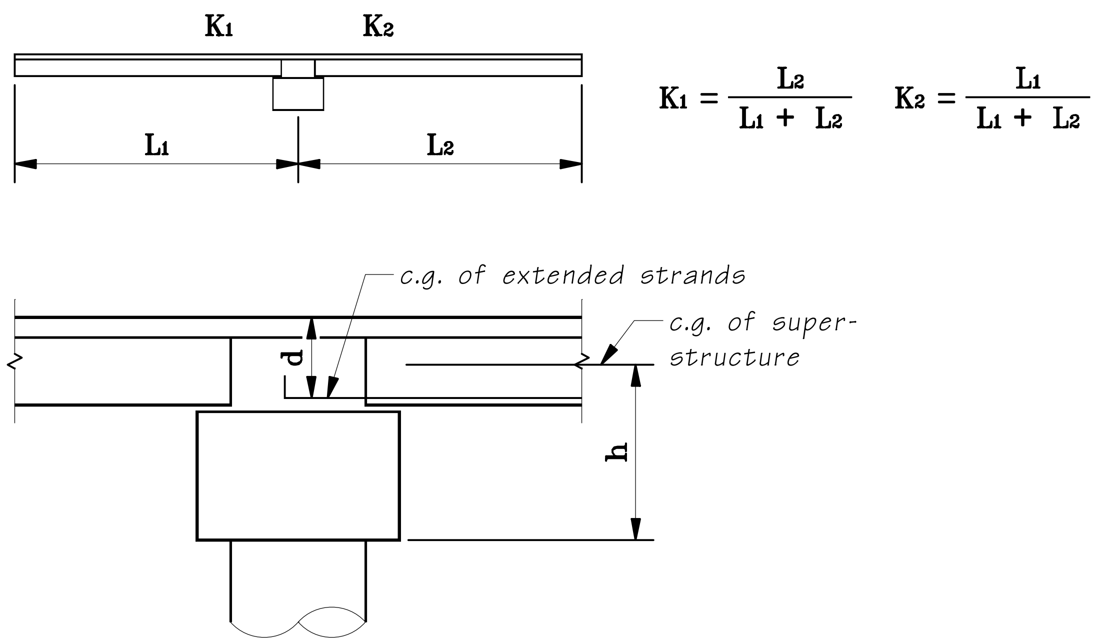
Where:

D_c = Diameter or width of column, see Figure 5.1.3-3
D_s = Depth of superstructure from top of column to top of deck slab, see Figure 5.1.3-3

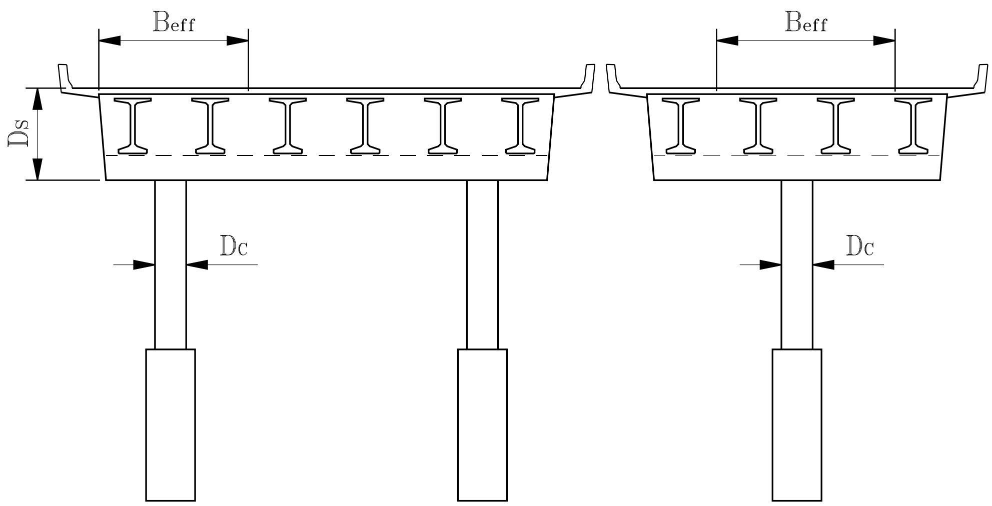
See Appendix 5-B10 for a design example.



Strand Development
Figure 5.1.3-1



Extended Strand Design
Figure 5.1.3-2



Effective Superstructure Width for Extended Strand Design
Figure 5.1.3-3

5.1.4 Prestress Losses

AASHTO LRFD Specifications outline the method of predicting prestress losses for usual prestressed concrete bridges that shall be used in design except as noted below.

A. **Instantaneous Losses**

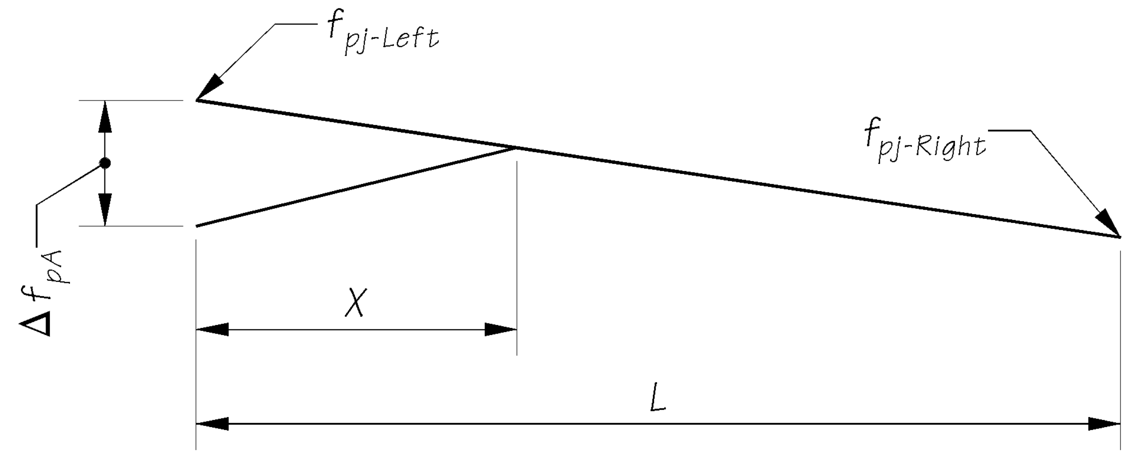
1. **Elastic Shortening of Concrete** – Transfer of prestress forces into the girder ends results in an instantaneous elastic loss. The prestress loss due to elastic shortening shall be added to the time dependent losses to determine the total losses. The loss due to elastic shortening shall be taken as per AASHTO LRFD 5.9.5.2.3.

   For pretensioned member and low-relaxation strands, f_{cgp} may be calculated based on $0.7f_{pu}$. For post-tensioned members with bonded tendons, f_{cgp} may be calculated based on prestressing force after jacking at the section of maximum moment.

2. **Anchorage Set Loss** – The anchor set loss shall be based on ⅜″ slippage for design purposes. Anchor set loss and the length affected by anchor set loss is shown in Figure 5.1.4-1.

$$x = \sqrt{\frac{\Delta_{set} A_{PT} E_p L}{P_{j\text{-}left} - P_{j\text{-}right}}} \qquad (5.1.4\text{-}1)$$

$$\Delta f_{pA} = \frac{2x\left(P_{j\text{-}left} - P_{j\text{-}right}\right)}{A_{PT} L} \qquad (5.1.4\text{-}2)$$



Anchorage Set Loss

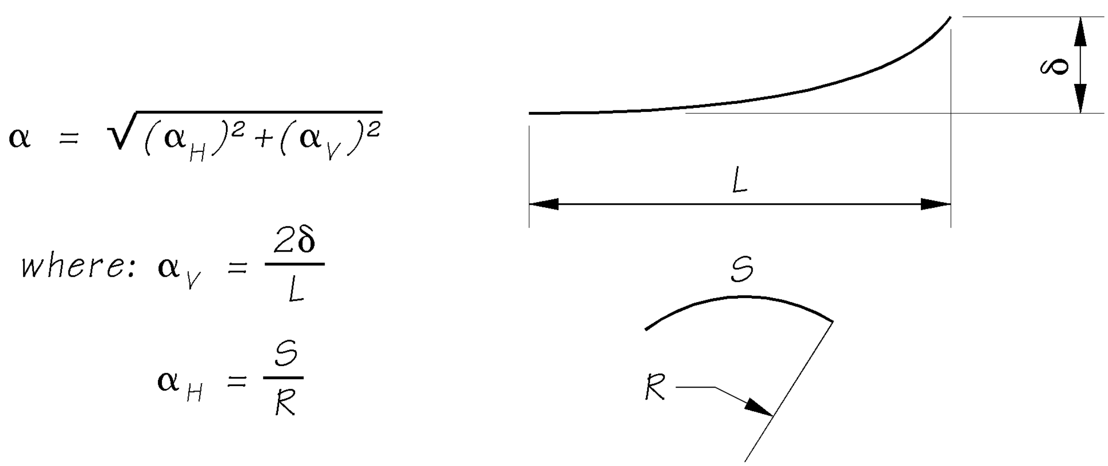
Figure 5.1.4-1

3. **Friction Losses** – Friction losses occurring during jacking and prior to anchoring depend on the system and materials used. For a rigid spiral galvanized ferrous metal duct system, μ shall be 0.20 and $K = 0.0002$. For plastic ducts, the designer shall use the values shown in AASHTO LRFD Table 5.9.5.2.2b.

   To avoid the substantial friction loss caused by sharp tendon curvature in the end regions where the tendons flare out from a stacked arrangement towards the bearing plates, use 0.10 times the span length or 20 feet as the minimum flare zone length. The recommended minimum radius (horizontal or vertical) of flared tendons is 200 feet. In the special cases where sharp curvature cannot be avoided, extra horizontal and vertical ties shall be added along the concave side of the curve to resist the tendency to break through the web.

$$\Delta f_{pF} = f_{pj}\left(1 - e^{-(kx+\mu\alpha)}\right) \qquad (5.1.4\text{-}3)$$

   When summing the α angles for total friction loss along the structure, horizontal curvature of the tendons as well as horizontal and vertical roadway curvature shall be included in the summation. The α angles for horizontally and vertically curved tendons are shown in Figure 5.1.4-2.



The α Angles for Curved PT Tendons

Figure 5.1.4-2

B. **Approximate Estimate of Time-Dependent Losses** – The Approximate Estimate of Time-Dependent Losses of AASHTO LRFD 5.9.5.3 may be used for preliminary estimates of time-dependent losses for precast, prestressed girders with composite decks as long as the conditions set forth in AASHTO are satisfied.

C. **Refined Estimates of Time-Dependent Losses** – Final design calculations of time-dependent prestress losses shall be based on the Refined Estimates of Time-Dependent Losses of AASHTO LRFD 5.9.5.4.

D. **Total Prestress Loss** – For standard precast, pretensioned members with CIP deck subject to normal loading and environmental conditions and pretensioned with low relaxation strands, the total prestress loss may be estimated as:

$$\Delta f_{pT} = \Delta f_{pRO} + \Delta f_{pES} + \Delta f_{pED} + \Delta f_{pLT} \tag{5.1.4-4}$$

The first term relates to initial relaxation that occurs between the time of strand stressing and prestress transfer.

$$\Delta f_{pRO} = \frac{log(24t)}{40}\left(\frac{f_{pj}}{f_{py}} - 0.55\right) f_{pj} \tag{5.1.4-5}$$

Where:

t = Duration of time between strand stressing and prestress transfer, typically 1 day.
f_{pj} = Jacking stress
f_{py} = Yield strength of the strand

The second term, Δf_{pES}, accounts for elastic shortening and is in accordance with AASHTO LRFD 5.9.5.2.3a.

The elastic gain due to deck placement and superimposed dead loads is taken to be:

$$\Delta f_{pED} = \frac{E_p}{E_c}\left[-\frac{(M_{slab} + M_{diaphragms})e_{ps}}{I_g} - \frac{M_{sidl}(Y_{bc} - Y_{bg} + e_{ps})}{I_c}\right] \tag{5.1.4-6}$$

Where:

E_p = Modulus of elasticity of the prestressing strand
E_c = Modulus of elasticity of the concrete at the time of loading
M_{slab} = Moment caused by deck slab placement
$M_{diaphragms}$ = Moment caused by diaphragms and other external loads applied to the non-composite girder section
M_{sidl} = Moment caused by all superimposed dead loads including traffic barriers and overlays
e_{ps} = Eccentricity of the prestressing strand
I_g = Moment of inertia of the non-composite girder
I_c = Moment of inertia of the composite girder
Y_{bg} = Location of the centroid of the non-composite girder measured from the bottom of the girder
Y_{bc} = Location of the centroid of the composite girder measured from the bottom of the girder

Long term time dependent losses, Δf_{pLT}, are computed in accordance with the refined estimates of AASHTO LRFD 5.9.5.4 or a detailed time-step method.

E. **Temporary Losses** – For checking stresses during release, lifting, transportation, and erection of prestressed girders, the elastic and time-dependent losses may be computed based on the following assumptions.

1. **Lifting of Girders From Casting Beds** – For normal construction, forms are stripped and girders are lifted from the casting bed within one day.
2. **Transportation** – Girders are most difficult to transport at a young age. The hauling configuration causes reduced dead load moments in the girder and the potential for overstress between the harping points. Overstress may also occur at the support points depending on the prestressing and the trucking configuration. This is compounded by the magnitude of the prestress force not having been reduced by losses. For an aggressive construction schedule girders are typically transported to the job site around day 10.

   When losses are estimated by the Approximate Estimate of AASHTO LRFD 5.9.5.3, the losses at the time of hauling may be estimated by:

$$\Delta f_{pTH} = \Delta f_{pRO} + \Delta f_{pES} + \Delta f_{pH} \tag{5.1.4-7}$$

   Where:
   Δf_{pTH} = total loss at hauling
   Δf_{pH} = time dependent loss at time of hauling = $3\frac{f_{pi}A_{ps}}{A_g}\gamma_h\gamma_{st} + 3\gamma_h\gamma_{st} + 0.6$

3. **Erection** – During construction the non-composite girders must carry the full weight of the deck slab and interior diaphragms. This loading typically occurs around 120 days for a normal construction schedule.
4. **Final Configuration** – The composite slab and girder section must carry all conceivable loads including superimposed dead loads such as traffic barriers, overlays, and live loads. It is assumed that superimposed dead loads are placed at 120 days and final losses occur at 2,000 days.

5.1.5 Prestressing Anchorage Systems

There are numerous prestressing systems. Most systems combine a method of prestressing the strands with a method of anchoring it to concrete.

WSDOT requires approval of all multi-strand and/or bar anchorages used in prestressed concrete bridges as described in *Standard Specifications* 6-02.3(26).

5.1.6 Post-Tensioning Ducts

Post-tensioning ducts shall meet the requirements of *Standard Specifications* 6-02.3(26)E.

Ducts for longitudinal post-tensioning tendons in precast spliced I-girders shall be made of rigid galvanized spiral ferrous metal to maintain standard girder concrete cover requirements.

The radius of curvature of tendon ducts shall not be less than 20 feet except in anchorage areas where 12 feet may be permitted.

5.2.3 Strut-and-Tie Model

Strut-and-tie models shall be used near regions of discontinuity or where beam theory is not applicable. Design and detailing considerations for strut-and-tie modeling is covered in AASHTO LRFD 5.6.3. See Appendix 5-B for a strut-and-tie design example for a pier cap.

5.2.4 Deflection and Camber

A. **General** – Flexural members are designed to have adequate stiffness to limit deflections or any deformations which may adversely affect the strength or serviceability of the structure at service load plus impact. The minimum superstructure depths are specified in AASHTO LRFD Table 2.5.2.6.3-1 and deflections shall be computed in accordance with AASHTO LRFD 5.7.3.6.2.

   Accurate predictions of deflections are difficult to determine, since modulus of elasticity of concrete, E_c, varies with stress and age of concrete. Also, the effects of creep on deflections are difficult to estimate. For practical purposes, an accuracy of 10 to 20 percent is often sufficient. Prestressing can be used advantageously to control deflections; however, there are cases where excessive camber due to prestress has caused problems.

B. **Preliminary Estimate for Precast Prestressed Members** – For preliminary design, the long term deflection and camber of precast prestressed members may be estimated using the procedure given in the PCI Design Handbook10 4.8.4.

C. **Deflection Calculation for Precast Prestressed Girders –** The "*D*" dimension is the computed girder deflection at midspan (positive upward) immediately prior to deck slab placement.

   Standard Specification 6-02.3(25)K defines two levels of girder camber at the time the deck concrete is placed, denoted D @ 40 Days and D @ 120 Days. They shall be shown in the plans to provide the contractor with lower and upper bounds of camber that can be anticipated in the field.

   PGSuper calculates estimated cambers at 40 days (D_{40}) and 120 days (D_{120}). Due to variations in observed camber, these estimated cambers are generally considered to be upper bounds at their respective times. This is based on measured girder cambers of prestressed precast concrete girders compared with the estimated cambers from PGSuper.

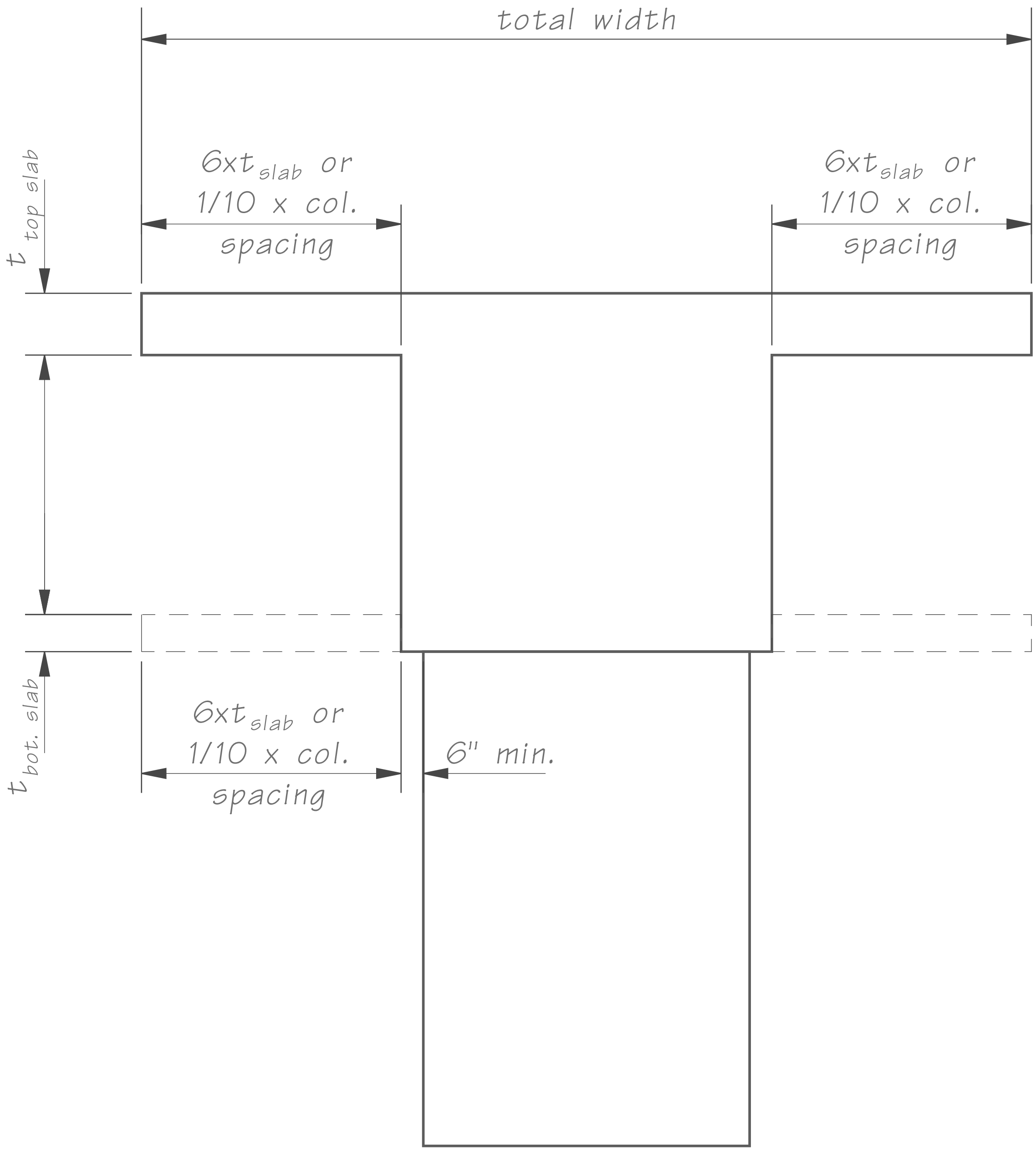
   D @ 120 Days is the upper bound of expected camber range at a girder age of 120 days after the release of prestress and is primarily intended to mitigate interference between the top of the cambered girder and the placement of concrete deck reinforcement. It is also used to calculate the "A" dimension at the girder ends. The age of 120 days was chosen because data has shown that additional camber growth after this age is negligible. D @ 120 Days may be taken as D120, the estimated camber at 120 days reported by PGSuper.

   D @ 40 Days is the lower bound of expected camber range at a girder age of 40 days (30 days after the earliest allowable girder shipping age of 10 days). To match the profile grade, girders with too little camber require an increased volume of haunch concrete along the girder length. For girders with large flange widths, such as the WF series, this can add up to significant quantities of additional concrete for a large deck placement. Thus, the lower bound of camber allows the contractor to assess the risk of increased concrete quantities and mitigates claims for additional material. D @ 40 Days shall be taken as 50 percent of D40, the estimated camber at 40 days reported by PGSuper.

   Figure 5.2.4-1 shows a typical pattern of girder deflection with time at centerline span. Portions of this characteristic curve are described below. The subparagraph numbers correspond to circled numbers on the curve.

1. **Elastic Deflection Due to Release of Prestress** – The prestress force produces moments in the girder tending to bow the girder upward. Resisting these moments are girder section dead load moments. The result is a net upward deflection.
2. **Creep Deflection Before Cutting Temporary Strands** – The girder continues to deflect upward due to the effect of creep. This effect is computed using the equation stated in Section 5.1.1E.
3. **Deflection Due to Cutting of Temporary Strands** – Cutting of temporary strands results in an elastic upward deflection. The default time interval for creep calculations for release of top temporary strands is 90 days after the release of prestress during girder fabrication for D_{120} (10 days for D_{40}).
4. **Diaphragm Load Deflection** – The load of diaphragm is applied to the girder section resulting in an elastic downward deflection. The default time interval for creep calculations for placing diaphragms is 90 days after the release of prestress during girder fabrication for D_{120} (10 days for D_{40}).
5. **Creep Deflection After Casting Diaphragms** – The girder continues to deflect upward for any time delay between diaphragms and deck slab casting.
6. **Deck Slab Load Deflection** – The load of the deck slab is applied to the girder section resulting in an elastic downward deflection. The default time interval for creep calculations for placing the deck slab is 120 days after the release of prestress during girder fabrication for D_{120} (40 days for D_{40}).
7. **Superimposed Dead Load Deflection** – The load of the traffic barriers, sidewalk, overlay, etc. is applied to the composite girder section resulting in an elastic downward deflection.
8. **Final Camber** – It might be expected that the above deck slab dead load deflection would be accompanied by a continuing downward deflection due to creep. However, many measurements of actual structure deflections have shown that once the deck slab is poured, the girder tends to act as though it is locked in position. To obtain a smooth riding surface on the deck, the deflection indicated on Figure 5.2.4-1 as “Screed Camber” (known as “C”) is added to the profile grade elevation of the deck screeds. The “C” dimension and the “Screed Setting Dimensions” detail shall be given in the plans.

D. **Pre-camber –** Precast prestressed girders may be precambered to compensate for the natural camber and for the effect of the roadway geometry. Precambering is allowed upon approval of the WSDOT Bridge Design Engineer.



Effective Width of Crossbeam
Figure 5.3.3-3

Crossbeam is usually cast to the fillet below the top slab. To avoid cracking of concrete on top of the crossbeam, construction reinforcement shall be provided at approximately 3″ below the construction joint. The design moment for construction reinforcement shall be the factored negative dead load moment due to the weight of crossbeam and adjacent 10′ of superstructure each side. The total amount of construction reinforcement shall be adequate to develop an ultimate moment at the critical section at least 1.2 times the cracking moment M_{cr}.

Special attention should be given to the details to ensure that the column and crossbeam reinforcement will not interfere with each other. This can be a problem especially when round columns with a great number of vertical bars must be meshed with a considerable amount of positive crossbeam reinforcement passing over the columns.

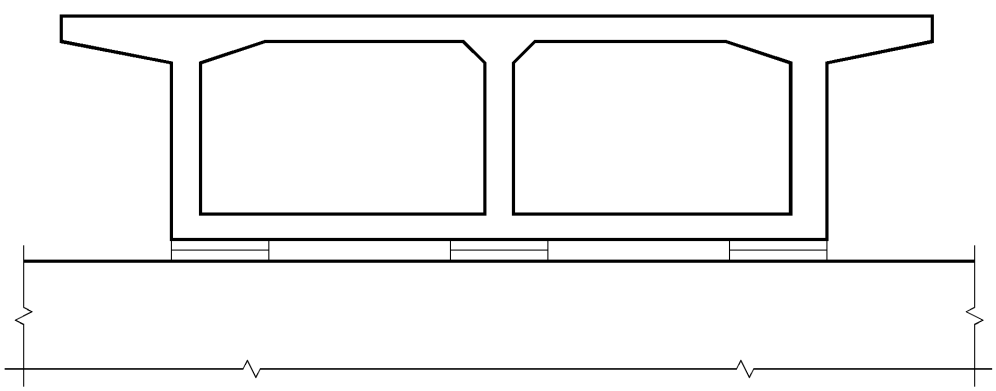
1. **Top Reinforcement –** The negative moment critical section shall be at the ¼ point of the square or equivalent square columns.
   a. **When Skew Angle ≤ 25° –** If the bridge is tangent or slightly skewed deck transverse reinforcement is normal or radial to centerline bridge, the negative cap reinforcement can be placed either in contact with top deck negative reinforcement (see Figure 5.3.3-1) or directly under the main deck reinforcement.
   b. **When Skew Angle > 25° –** When the structure is on a greater skew and the deck steel is normal or radial to the longitudinal centerline of the bridge, the negative cap reinforcement should be lowered to below the main deck reinforcement (see Figure 5.3.3-2).
   c. **To avoid cracking of concrete –** Interim reinforcement is required below the construction joint in crossbeams.
2. **Skin Reinforcement –** Longitudinal skin reinforcement shall be provided per AASHTO LRFD 5.7.3.4.

5.3.4 End Diaphragm

A. **Basic Geometry –** Bearings at the end diaphragms are usually located under the girder stems to transfer loads directly to the pier as shown in Figure 5.3.4-1. In this case, the diaphragm width shall be equal to or greater than bearing grout pads as shown Figure 5.3.4-2.

Designer shall provide access space for maintenance and inspection of bearings.

Allowance shall be provided to remove and replace the bearings. Lift point locations, jack capacity, number of jacks, and maximum permitted lift shall be shown in the plan details.



Bearing Locations at End Diaphragm
Figure 5.3.4-1

5.6 Precast Prestressed Girder Superstructures

The precast prestressed girder bridge is an economical and rapid type of bridge construction and often preferred for WSDOT bridges.

Precast sections are generally fabricated in plant or somewhere near the construction site and then erected. Precasting permits better material quality control and is often more economical than CIP concrete.

Pre-tensioning is accomplished by stressing strands to a predetermined tension and then placing concrete around the strands, while the stress is maintained. After the concrete has hardened, the strands are released and the concrete, which has become bonded to the tendon, is prestressed as a result of the strands attempting to relax to their original length. The strand stress is maintained during placing and curing of the concrete by anchoring the ends of strands to abutments that may be as much as 500′ apart. The abutments and appurtenances used in the prestressing procedure are referred to as a pre-tensioning bed or bench.

5.6.1 WSDOT Standard Girder Types

A girder type consists of a series of girder cross sections sharing a common shape. The numbers within girder series generally refer to the depth of the section in inches. Refer to Standard Specification 6-02.3(25) for a comprehensive list of Standard WSDOT girder types. Standard WSDOT girder types include:

Prestressed Concrete I Girders – Washington State Standard I Girders were adopted in the mid-1950s. The original series was graduated in 10foot increments from 30 feet to 100 feet. In 1990, revisions were made incorporating the results of the research done at Washington State University on girders without end blocks. The revisions included three major changes: a thicker web; end blocks were eliminated; and strand spacing was increased. The current Series of this type include W42G, W50G, W58G, and W74G.

Prestressed Concrete Wide Flange I Girders and Spliced Prestressed Concrete Girders – In 1999, deeper girders, commonly called “Supergirders” were added to the WSDOT standard concrete girders. These new supergirders may be pretensioned or post-tensioned. The pretensioned Series are designated as WF74G, WF83G and WF95G and the post-tensioned (spliced) Series are designated as WF74PTG, WF83PTG and WF95PTG.

In 2004 Series WF42G, WF50G, and WF58G were added to the prestressed girder standards. In 2008, Series WF66G, WF100G, and WF100PTG were added to the prestressed girder standards. In 2009, Series WF36G was added to the prestressed girder standards.

Bulb Tee Girders – In 2004 Series W32BTG, W38BTG and W62BTG were added to the prestressed girder standards.

Deck Bulb Tee Girders – This type of girder has a top flange designed to support traffic loads. They include Series W35DG, W41DG, W53DG and W65DG.

Prestressed Concrete Tub Girders – In 2004 prestressed concrete tub girders were added as standard girders.

All WSDOT prestressed girders are high performance high strength concrete girders. They generally rely on high strength concrete to be effective for the spans expected as a single piece. The approximate ranges of maximum span lengths are as shown in Table 5.6.1-1 and Appendix 5.6-A1.

Standard drawings for WSDOT prestressed girders are shown in Appendix 5.6-A and 5.9-A.

Type	Depth (in)	Area (in^2)	Iz (in^4)	Yb (in)	Wt (k/ft)	Volume to Surface Ratio (in)	Max. Span Capability (ft)	Max. Length (252 kips Limit) (ft)
W42G	42.00	373.25	76092	18.94	0.428	2.77	85	-
W50G	50.00	525.5	164958	22.81	0.602	3.12	115	-
W58G	58.00	603.5	264609	28.00	0.692	3.11	130	-
W74G	73.50	746.7	546110	38.08	0.856	2.90	150	-
WF36G	36.00	690.8	124772	17.54	0.792	3.24	105	-
WF42G	42.00	727.5	183642	20.36	0.834	3.23	120	-
WF50G	50.00	776.5	282559	24.15	0.890	3.22	140	-
WF58G	58.00	825.5	406266	27.97	0.946	3.21	155	-
WF66G	66.00	874.5	556339	31.80	1.002	3.20	165	-
WF74G	74.00	923.5	734356	35.66	1.058	3.19	175	-
WF83G	82.63	976.4	959393	39.83	1.119	3.19	190	-
WF95G	94.50	1049.1	1328995	45.60	1.202	3.18	190	-
WF100G	100.00	1082.8	1524912	48.27	1.241	3.17	205	203
W32BTG	32.00	537.0	73730	17.91	0.615	2.89	80	-
W38BTG	38.00	573.0	114108	21.11	0.657	2.90	95	-
W62BTG	62.00	717.0	384881	33.73	0.822	2.92	135	-
12″ Slab	12.00	556.0	6557	5.86	0.637	4.74	33	-
18″ Slab	18.00	653.1	21334	8.79	0.748	3.80	50	-
26″ Slab	26.00	920.1	64049	12.76	1.054	4.86	72	-
30″ Slab	30.00	1031.1	103241	14.75	1.181	4.73	83	-
36″ Slab	36.00	1379.7	205085	17.77	1.581	5.34	100	-
U54G4	54.00	1038.8	292423	20.97	1.190	3.51	130	-
U54G5	54.00	1110.8	314382	19.81	1.273	3.47	130	-
U66G4	66.00	1208.5	516677	26.45	1.385	3.51	150	-
U66G5	66.00	1280.5	554262	25.13	1.467	3.47	150	-
U78G4	78.00	1378.2	827453	32.06	1.579	3.51	170	160
U78G5	78.00	1450.2	885451	30.62	1.662	3.48	170	152
UF60G4	60.00	1207.7	483298	26.03	1.384	3.48	150	-
UF60G5	60.00	1279.7	519561	24.74	1.466	3.45	150	-
UF72G4	72.00	1377.4	787605	31.69	1.578	3.48	160	160
UF72G5	72.00	1449.4	844135	30.26	1.661	3.45	170	152
UF84G4	84.00	1547.1	1190828	37.42	1.773	3.48	180	142
UF84G5	84.00	1619.1	1272553	35.89	1.855	3.46	180	136

Section Properties of WSDOT Standard Precast Prestressed Girders
Table 5.6.1-1

5.6.2 Design Criteria

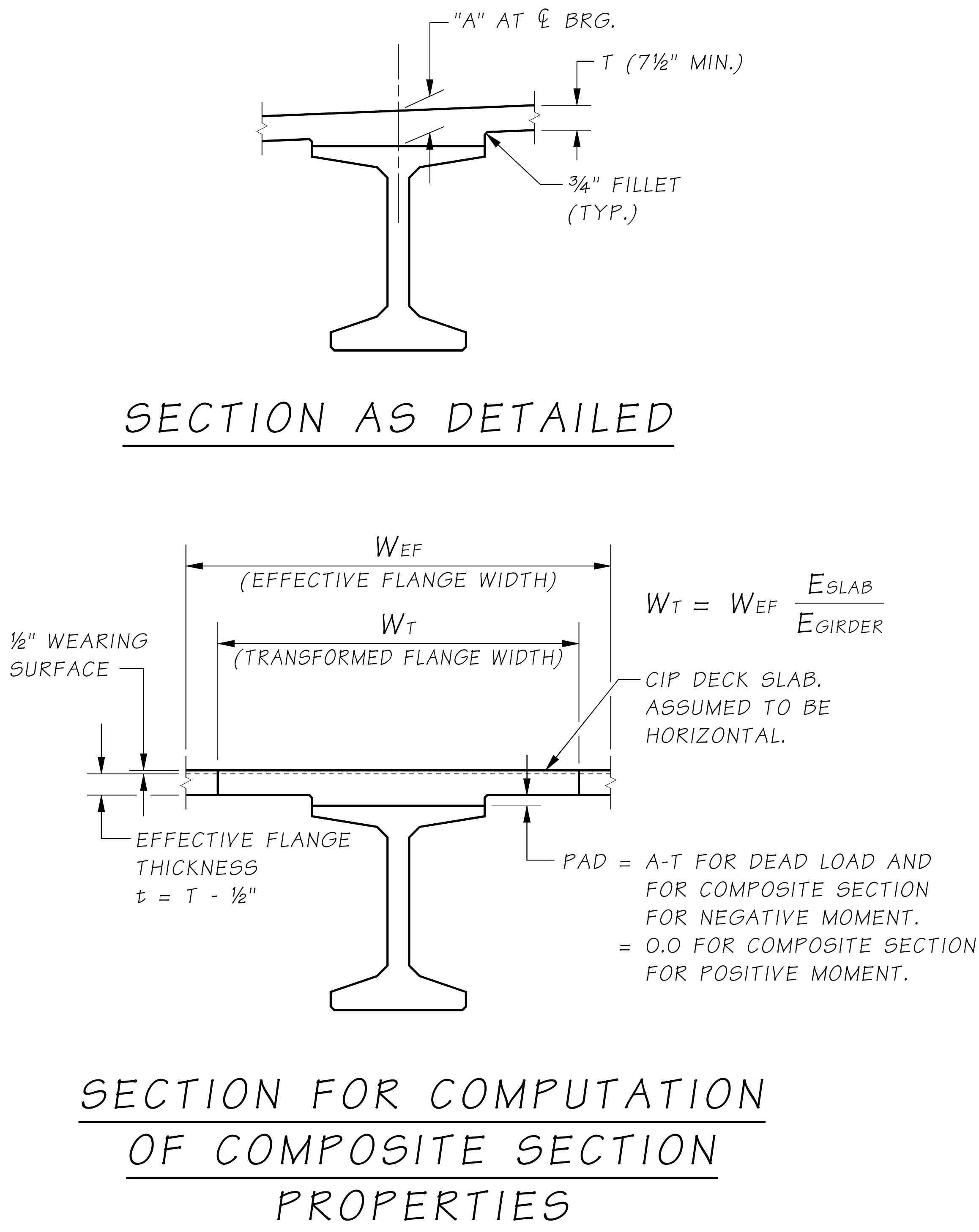
WSDOT design criteria for precast prestressed girder superstructures are given in Table 5.6.2-1.

AASHTO LRFD 5.14.1.4 "Bridges Composed of Simple Span Precast Girders Made Continuous" allows for some degree of continuity for loads applied on the bridge after the continuity diaphragms have been cast and cured. This assumption is based on the age of the girder when continuity is established, and degree of continuity at various limit states. Both degree of continuity and time of continuity diaphragm casting may result in contractual and design issues. Designing these types of bridges for the envelope of simple span and continuous spans for applicable permanent and transient loads is the approach used by WSDOT as it has yielded good results.

Design Specifications	AASHTO LRFD Specifications and WSDOT *Bridge Design Manual* M 23-50
Design Method	Precast, prestressed members shall be designed for service limit state for allowable stresses and checked for strength limit state for ultimate capacity.
Superstructure Continuity	Precast, prestressed girder superstructures shall be designed for the envelope of simple span and continuous span loadings for all permanent and transient loads. Loads applied before establishing continuity (typically before placement of continuity diaphragms) need only be applied as a simple span loading. Continuity reinforcement shall be provided at supports for loads applied after establishing continuity.
Loads and Load Factors	Service, Strength, Fatigue, and Extreme Event Limit State loads and load combinations shall be per AASHTO LRFD Specifications
Allowable Stresses	WSDOT *Bridge Design Manual* M 23-50 Table 5.2.1-1
Prestress Losses	WSDOT *Bridge Design Manual* M 23-50 Section 5.1.4
Shear Design	AASHTO LRFD 5.8 and WSDOT *Bridge Design Manual* M 23-50 Section 5.2.2.B
Shipping and Handling	WSDOT Bridge Design Manual M 23-50 Section 5.6.3
Continuous Structure Configuration	Girder types and spacing shall be identical in adjacent spans. Girder types and spacing may be changed at expansion joints.
Girder End Support Skew Angle	Girder end support skew angles shall be limited to 45° for all precast prestressed girders. Skew angles for precast slabs, deck bulb-tees and trapezoidal tubs shall be limited to 30°.
Intermediate Diaphragms	CIP concrete intermediate diaphragms shall be provided for all prestressed girder bridges (except slabs) as shown below: • ⅕ points of span for span length > 160′-0″. • ¼ points of span for 120′-0″ < span length ≤ 160′-0″. • ⅓ points of span for 80′-0″ < span length ≤ 120′-0″. • Midpoint of span for 40′-0″ < span length ≤ 80′-0″. • No diaphragm requirement for span length ≤ 40′-0″. Intermediate diaphragms shall be either partial or full depth as described in Section 5.6.4.C.4.

Design Criteria for Precast Prestressed Girders
Table 5.6.2-1

A. **Support Conditions** – The prestressed girders are assumed to be supported on rigid permanent simple supports. These supports can be either bearing seats or elastomeric pads. The design span length is the distance center to center of bearings for simple spans. For continuous spans erected on falsework (raised crossbeam), the effective point of support for girder design is assumed to be the face of the crossbeam. For continuous spans on crossbeams (dropped or semi-dropped crossbeam), the design span length is usually the distance center to center of temporary bearings.



Typical Section for Computation of Composite Section Properties
Figure 5.6.2-1

B. **Composite Action**

1. **General –** The sequence of construction and loading is extremely important in the design of prestressed girders. The composite section has a much larger capacity than the basic girder section but it cannot take loads until the deck slab has obtained adequate strength. Assumptions used in computing composite section properties are shown in Figure 5.6.2-1.

2. **Load Application –** The following sequence and method of applying loads is typically used in girder analysis:

   a. Girder dead load is applied to the girder section.

   b. Diaphragm dead load is applied to the girder section.

   c. Deck slab dead load is applied to the girder section.

   d. Superimposed dead loads (such as barriers, sidewalks and overlays) and live loads are applied to the composite section.

   The dead load of one traffic barrier may be divided among a maximum of three girders.

3. **Composite Section Properties –** Minimum deck slab thickness is 7½″, but may be thicker if girder spacing dictates. This slab forms the top flange of the composite girder in prestressed girder bridge construction.

   a. **Effective and Transformed Flange Width –** The effective flange width of a concrete deck slab for computing composite section properties shall be per AASHTO LRFD 4.6.2.6. The effective flange width shall be reduced by the ratio E_{slab}/E_{girder} to obtain the transformed flange width. The effective modulus of the composite section with the transformed flange width is then E_{girder}.

   b. **Effective Flange Thickness** – The effective flange thickness of a concrete deck slab for computing composite section properties shall be the deck slab thickness reduced by ½″ to account for wearing. Where a bridge will have an overlay applied prior to traffic being allowed on the bridge, the full deck slab thickness may be used as effective slab thickness.

   c. **Flange Position –** An increased dimension from top of girder to top of deck slab at centerline of bearing at centerline of girder shall be shown in the Plans. This is called the “A” dimension. It accounts for the effects of girder camber, vertical curve, deck slab cross slope, etc. See Appendix 5-B1 for method of computing.

   For purposes of calculating composite section properties for negative moments, the pad/haunch height between bottom of deck slab and top of girder shall be taken as the “A” dimension minus the flange thickness “T”at intermediate pier supports and shall be reduced by girder camber a appropriate at other locations.

   For purposes of calculating composite section properties for positive moments, the bottom of the deck slab shall be assumed to be directly on the top of the girder. This assumption may prove to be true at center of span where excess girder camber occurs.

d. **Section Dead Load –** The deck slab dead load to be applied to the girder shall be based on the full deck slab thickness. The full effective pad/haunch weight shall be added to that load over the full length of the girder. The full effective pad or haunch height is typically the “A” dimension minus the flange thickness “T”, but may be higher at midspan for a crest vertical curve.

C. **Design Procedure**

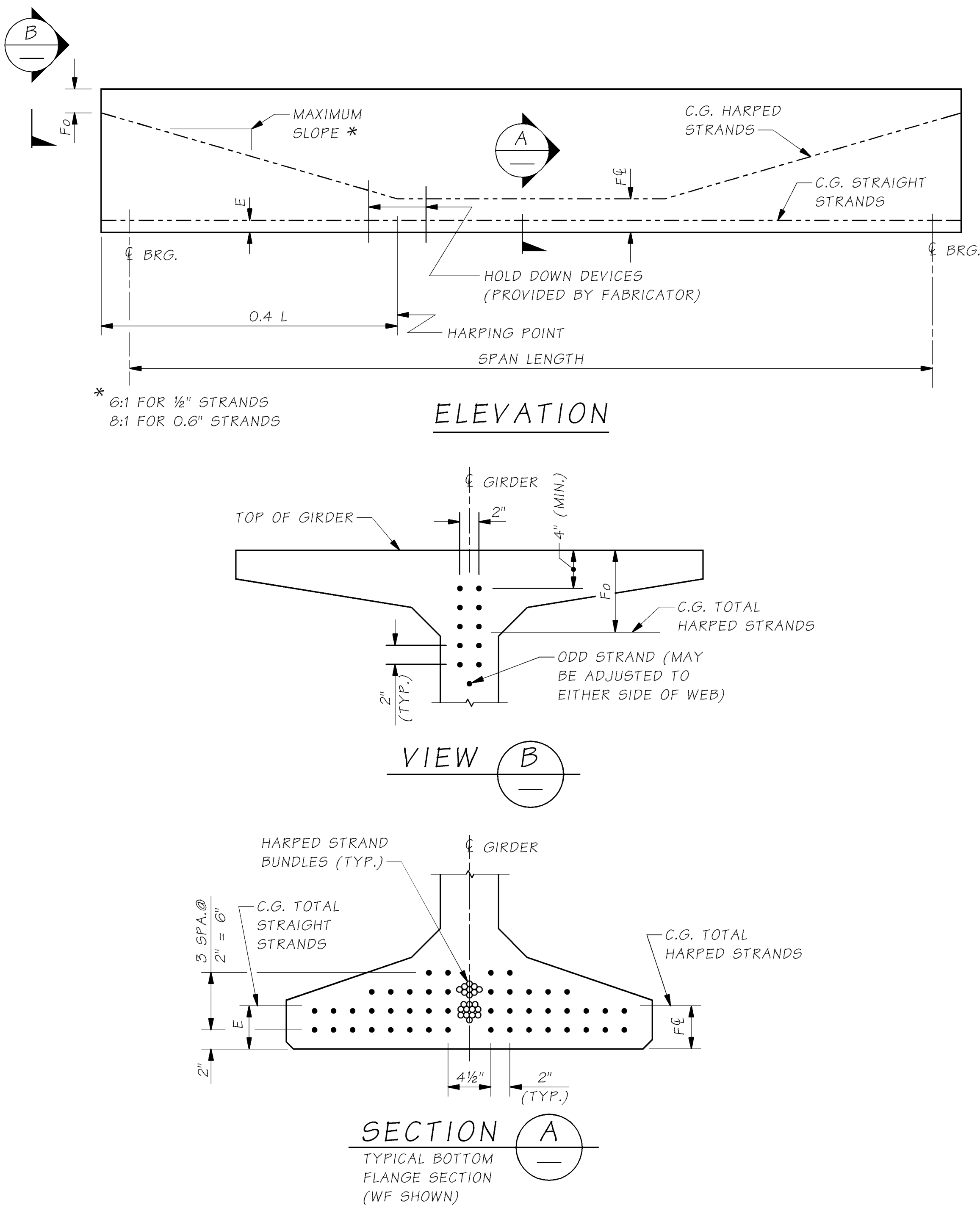
1. **General –** The WSDOT Prestressed Girder Design computer program PGSuper is the preferred method for final design.

2. **Stress Conditions –** The designer shall ensure that the stress limits as described in Table 5.2.1-1 are not exceeded for prestressed girders. Each condition is the result of the summation of stresses with each load acting on its appropriate section (such as girder only or composite section).

   Dead load impact need not be considered during lifting.

   During shipping, girder stresses shall be checked using two load cases. The first load case consists of a plumb girder with dead load impact of 20% acting either up or down. The second load case consists of an inclined girder with no dead load impact. The angle of inclination shall be the equilibrium tilt angle computed for lateral stability (see BDM 5.6.3.D.6 and equation (12) in reference[12]) with a roadway superelevation of 6%.

D. **Standard Strand Locations –** Standard strand locations of typical prestressed girders are shown in Figure 5.6.2-2 and Appendices 5.6-A and 5.9-A.



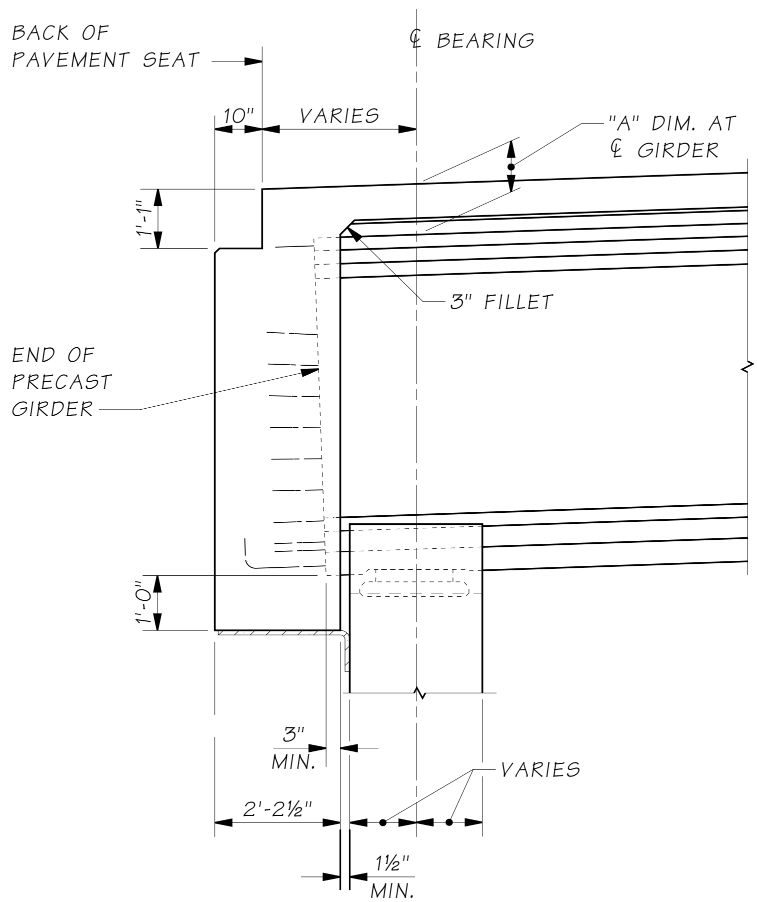
Typical Prestressed Girder Configuration
Figure 5.6.2-2

E. **Girder End Types –** There are four end types shown on the standard girder sheets. Due to the extreme depth of the WF83G, WF95G, and WF100G girders, and possible end of girder tilt at the piers for profile grades, the designer will need to pay particular attention to details to assure the girders will fit and perform as intended. The four end types are shown as follows:

1. **End Type A –** End Type A as shown in Figure 5.6.2-3 is for cantilever end piers with an end diaphragm cast on the end of the girders. End Type A has a recess at the bottom of the girder near the end for an elastomeric bearing pad. See Appendix 5.6-A7-9 and 5.6-A9-12 for bearing pad details. The recess at the centerline of bearing is 0.5″ deep. This recess is to be used for profile grades up to and including 4 percent. The recess is to be replaced by an embedded steel plate flush with the bottom of the girder for grades over 4 percent. A tapered bearing plate, with stops at the edges to contain the elastomeric pad, can be welded or bolted to the embedded plate to provide a level bearing surface.

   Reinforcing bars and pretensioned strands project from the end of the girder. The designer shall assure that these bars and strands fit into the end diaphragm. Embedment of the girder end into the end diaphragm shall be a minimum of 3″ and a maximum of 6″. For girder ends where the tilt would exceed 6″ of embedment, the girder ends shall be tilted to attain a plumb surface when the girder is erected to the profile grade.

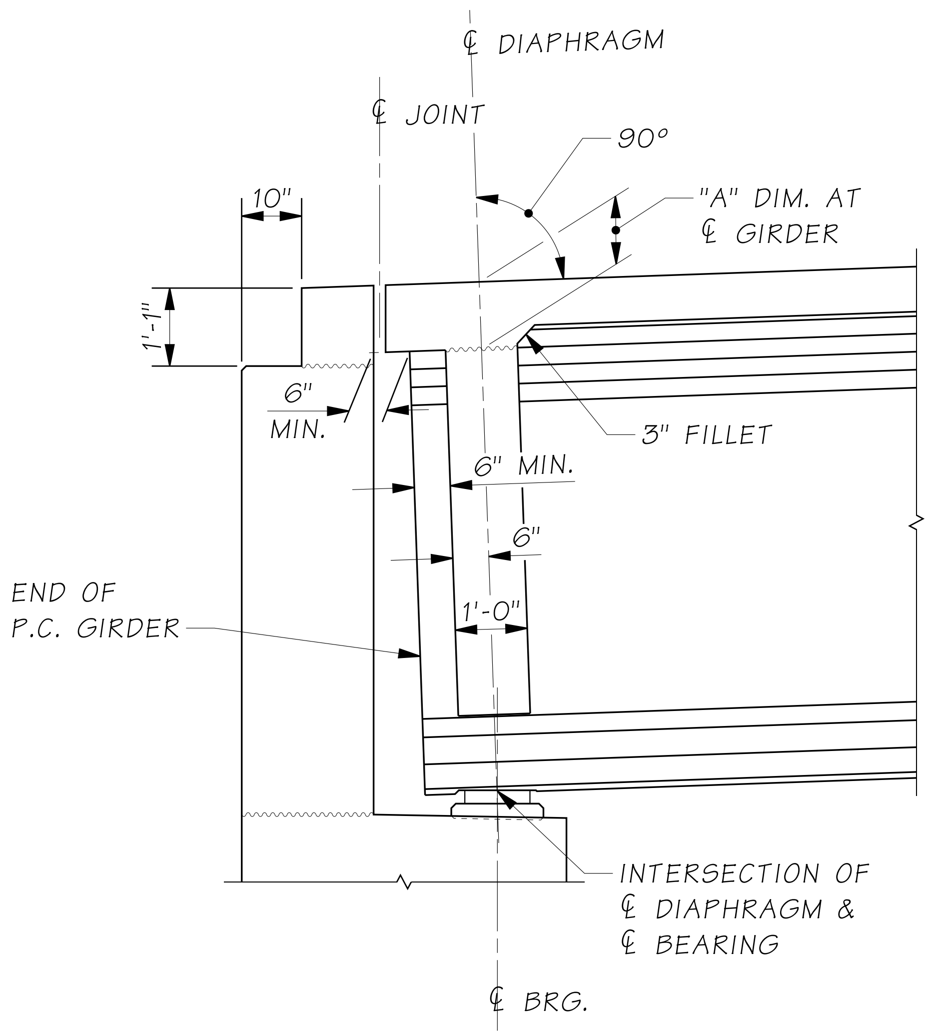
   The gap between the end diaphragm and the stem wall shall be a minimum of 1½″ or ½″ greater than required for longitudinal bridge movement.



End Type A (End Diaphragm on Girder)
Figure 5.6.2-3

2. **End Type B –** End Type B as shown in Figure 5.6.2-4 is for "L" type abutments. End Type B also has a recess at the bottom of the girder for an elastomeric bearing pad. Notes regarding the bearing recess on End Type A also apply to End Type B. End Type B is the only end type that does not have reinforcing or strand projecting from the girder end.

   The centerline of the diaphragm is normal to the roadway surface. The centerline of the bearing is coincident with the centerline of the diaphragm at the top of the elastomeric pad.



End Type B (L-Shape End Pier)
Figure 5.6.2-4

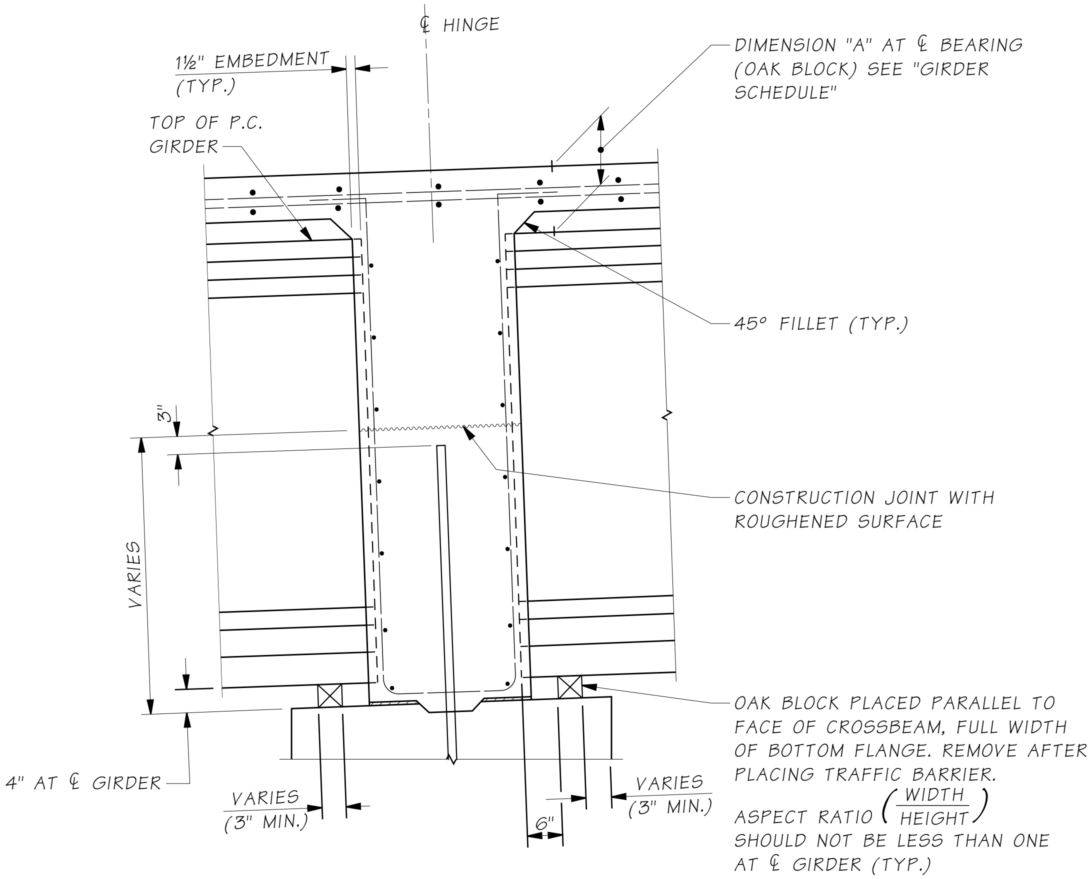
3. **End Type C –** End Type C as shown in Figure 5.6.2-5 is for continuous spans and an intermediate hinge diaphragm at an intermediate pier. There is no bearing recess and the girder is temporarily supported on oak blocks. This detail is generally used only in low seismic areas such as east of the Cascade Mountains.

   The designer shall check the edge distance and provide a dimension that prevents edge failure, or spalling, at the top corner of the supporting cross beam for load from the oak block including dead loads from girder, deck slab, and construction loads.

   For prestressed girders with intermediate hinge diaphragms, designers shall:

   a. Check size and minimum embedment in crossbeam and diaphragm for hinge bars.

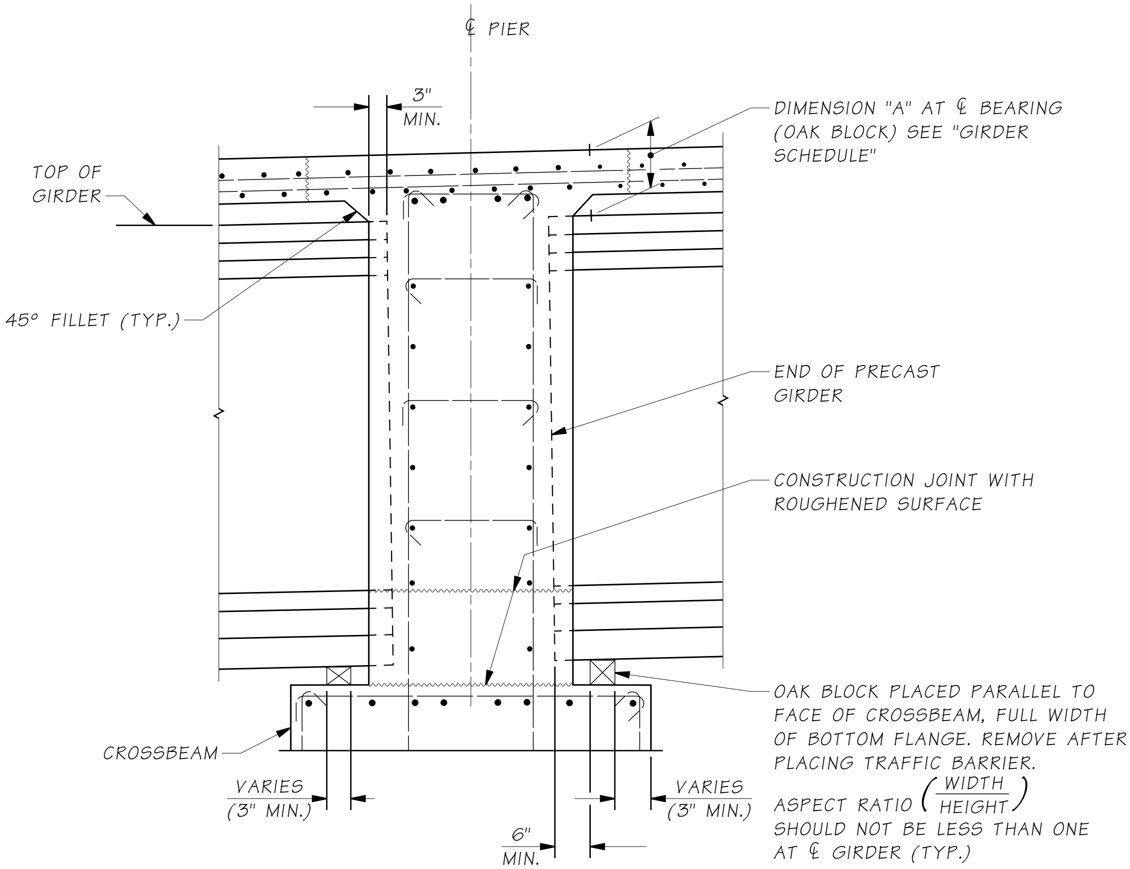
   b. Check interface shear friction at girder end (see Section 5.2.2.C.2).



End Type C (Intermediate Hinge Diaphragm)
Figure 5.6.2-5

4. **End Type D –** End Type D as shown in Figure 5.6.2-6 is for continuous spans fully fixed to columns at intermediate piers. There is no bearing recess and the girder is temporarily supported on oak blocks.

   The designer shall check the edge distance and provide a dimension that prevents edge failure, or spalling, at the top corner of the supporting cross beam for load from the oak block including dead loads from girder, deck slab, and construction loads. The designer shall check interface shear friction at the girder end (see Section 5.2.2.C.2).



End Type D
Figure 5.6.2-6

F. **Splitting Resistance in End Regions of Prestressed Girders –** The splitting resistance of pretensioned anchorage zones shall be as described in AASHTO LRFD 5.10.10.1. For pretensioned I-girders or bulb tees, the end vertical reinforcement shall not be larger than #5 bars and spacing shall not be less than 2½″. The remaining splitting reinforcement not fitting within the h/4 zone may be placed beyond the h/4 zone at a spacing of 2½″.

G. **Confinement Reinforcement in End Regions of Prestressed Girders –** Confinement reinforcement per AASHTO LRFD 5.10.10.2 shall be provided.

H. **Girder Stirrups** – Girder stirrups shall be field bent over the top mat of reinforcement in the deck slab.

Girder stirrups may be prebent, but the extended hook shall be within the core of the slab (the inside edge of the hook shall terminate above the bottom mat deck slab bars).

I. **Transformed Section Properties –** Transformed section properties shall not be used for design of prestressed girders. Use of gross section properties remains WSDOT’s standard methodology for design of prestressed girders including prestress losses, camber and flexural capacity.

In special cases, transformed section properties may be used for the design of prestressed girders with the approval of the WSDOT Bridge Design Engineer. The live load factor at the Service III load combination shall be as follows:

- γ_{LL} = 0.8 when gross section properties are used
- γ_{LL} = 1.0 when transformed section properties are used

5.6.3 Fabrication and Handling

A. **Shop Plans –** Fabricators of prestressed girders are required to submit shop plans which show specific details for each girder. These shop plans are checked and approved by the Project Engineer's office for conformance with the Contract Plans and specifications.

B. **Special Problems for Fabricators**

1. **Strand Tensioning –** The method selected for strand tensioning may affect the design of the girders. The strand arrangements shown in the office standard plans and included in the PGSuper computer program are satisfactory for tensioning methods used by fabricators in this state. Harped strands are normally tensioned by pulling them as straight strands to a partial tension. The strands are then deflected vertically as necessary to give the required harping angle and strand stress. In order to avoid overtensioning the harped strands by this procedure, the slope of the strands is limited to a maximum of 6:1 for 0.5″ ϕ strands and 8:1 for 0.6″ϕ strands. The straight strands are tensioned by straight jacking.

2. **Hold Down Forces –** Forces on the hold-down units are developed as the harped strands are raised. The hold-down device provided by the fabricator must be able to hold the vertical component of the harping forces. Normally a two or more hold-down unit is required. Standard commercial hold-down units have been preapproved for use with particular strand groups.

3. **Numbers of Strands –** Since the prestressing beds used by the girder fabricators can carry several girders in a line, it is desirable that girders have the same number of strands where practical. This allows several girders to be set up and cast at one time.

   For pretensioned concrete girders, the number of permanent prestressing strands (straight and harped) shall be limited to 100 total 0.6″ϕ strands.

C. **Handling of Prestressed Girders**

1. **In-Plant Handling** – The maximum weight that can be handled by precasting plants in the Pacific Northwest is 252 kips. Pretensioning lines are normally long enough so that the weight of a girder governs capacity, rather than its length. Headroom is also not generally a concern for the deeper sections.

2. **Lateral Stability during Handling** – The designer shall specify the lifting embedment locations (3′ minimum from ends - see Standard Specification 6-02.3(25)L) and the corresponding concrete strength at release that provides an adequate factor of safety for lateral stability. The calculations shall conform to methods as described in references [2, 11, 12, 13]. Recommended factors of safety of 1.0 against cracking, and 1.5 against failure shall be used.

   Lateral stability can be a concern when handling long, slender girders. Lateral bending failures are sudden, catastrophic, costly, pose a serious threat to workers and surroundings, and therefore shall be considered by designers. When the girder forms are stripped from the girder, the prestressing level is higher and the concrete strength is lower than at any other point in the life of the member. Lifting embedment/support misalignment, horizontal girder sweep and other girder imperfections can cause the girder to roll when handling, causing a component of the girder weight to be resisted by the weak axis.

   Lateral stability may be improved using the following methods:

   a. Move the lifting embedments away from the ends. This may increase the required concrete release strength, because decreasing the distance between lifting devices increases the concrete stresses at the harp point. Stresses at the support may also govern, depending on the exit location of the harped strands.

   b. Select a girder section that is relatively wide and stiff about its vertical (weak) axis.

c. Add temporary prestressing in the top flange.

d. Brace the girder.

e. Raise the roll axis of the girder with a rigid yoke.

For stability analysis of prestressed girders during in-plant handling, in absence of more accurate information, the following parameters shall be used:

1. Height of pick point above top of girder = 0.0″
2. Lifting embedment transverse placement tolerance = 0.25″
3. Maximum girder sweep tolerance at midspan = 0.000521 in/in of total girder length

D. **Shipping Prestressed Girders**

1. **General –** The ability to ship girders can be influenced by a large number of variables, including mode of transportation, weight, length, height, and lateral stability. The ability to ship girders is also strongly site-dependent. For large or heavy girders, routes to the site shall be investigated during the preliminary design phase. To this end, on projects using large or heavy girders, WSDOT can place an advisory in their special provisions including shipping routes, estimated permit fees, escort vehicle requirements, Washington State Patrol requirements, and permit approval time.

2. **Mode of Transportation –** Three modes of transportation are commonly used in the industry: truck, rail, and barge. In Washington State, an overwhelming percentage of girders are transported by truck, so discussion in subsequent sections will be confined to this mode. However, on specific projects, it may be appropriate to consider rail or barge transportation.

   Standard rail cars can usually accommodate larger loads than a standard truck. Rail cars range in capacity from approximately 120 to 200 kips. However, unless the rail system runs directly from the precasting plant to the jobsite, members must be trucked for at least some of the route, and weight may be restricted by the trucking limitations.

   For a project where a large number of girders are required, barge transportation is usually the most economical. Product weights and dimensions are generally not limited by barge delivery, but by the handling equipment on either end. In most cases, if a product can be made and handled in the plant, it can be shipped by barge.

3. **Weight Limitations –** The net weight limitation with trucking equipment currently available in Washington State is approximately 190 kips, if a reasonable delivery rate (number of pieces per day) is to be maintained. Product weights of up to 252 kips can be hauled with currently available equipment at a limited rate.

   Long span prestressed concrete girders may bear increased costs due to difficulties encountered during fabrication, shipping, and erection. Generally, costs will be less if a girder can be shipped to the project site in one piece. However, providing an alternate spliced-girder design to long span one-piece pretensioned girders may reduce the cost through competitive bidding.

   When a spliced prestressed concrete girder alternative is presented in the Plans, the substructure shall be designed and detailed for the maximum force effect case only (no alternative design for substructure).

   Local carriers should be consulted on the feasibility of shipping large or heavy girders on specific projects.

4. **Support Locations –** The designer shall provide shipping support locations in the plans to ensure adequate girder stability. Shipping support locations shall be no closer than the girder depth to the ends of the girder at the girder centerline. The overhangs at the leading and trailing ends of the girders should be minimized and equal if possible. Generally, the leading end overhang should not exceed 15′ to avoid interference with trucking equipment. Local carriers should be consulted if a larger leading end overhang is required. Shipping support locations shall maintain the concrete stresses within allowable limits.

   Length between shipping support locations may be governed by turning radii on the route to the jobsite. Potential problems can be circumvented by moving the support points closer together (away from the ends of the girder), or by selecting alternate routes. Up to 130′ between supports is typically acceptable for most projects.

5. **Height Limitations –** The height of a deep girder section sitting on a jeep and steerable trailer is of concern when considering overhead obstructions on the route to the jobsite. The height of the support is approximately 6′ above the roadway surface. When adding the depth of the girder, including camber, the overall height from the roadway surface to the top of concrete can rapidly approach 14′. Overhead obstructions along the route should be investigated for adequate clearance in the preliminary design phase. Obstructions without adequate clearance must be bypassed by selecting alternate routes.

   Expectations are that, in some cases, overhead clearance will not accommodate the vertical stirrup projection on deeper WSDOT standard girder sections. Alternate stirrup configurations can be used to attain adequate clearance, depending on the route from the plant to the jobsite.

6. **Lateral Stability during Shipping –** The designer shall specify support locations in the Plans that provide an adequate factor of safety for lateral stability during shipping. The calculations shall conform to methods as described in references [2, 11, 12, 13]. Recommended factors of safety of 1.0 against cracking, and 1.5 against failure (rollover of the truck) shall be used. See the discussion above on lateral stability during handling of prestressed girders for suggestions on improving stability.

   For lateral stability analysis of prestressed girders during shipping, in absence of more accurate information, the following parameters shall be used:

   a. Roll stiffness of entire truck/trailer system:

$$K_\theta = the\ maximum\ of \begin{cases} 28{,}000\dfrac{kip \cdot in}{rad} \\ \left(4{,}000\dfrac{kip \cdot in}{rad \cdot axle}\right) \cdot N \end{cases}$$

   Where:
   N = required number of axles = W_g/W_a, *rounded up to the nearest integer*
   W_g = total girder weight (kip)
   W_a = 18 (kip/axle)

   b. Height of girder bottom above roadway = 72″

   c. Height of truck roll center above road = 24″

   d. Center to center distance between truck tires = 72″

   e. Maximum expected roadway superelevation = 0.06

   f. Maximum girder sweep tolerance at midspan = 0.001042 in/in of total girder length

   g. Support placement lateral tolerance = ±1″

   h. Increase girder C.G. height over roadway by 2% for camber

E. **Erection –** A variety of methods are used to erect precast concrete girders, depending on the weight, length, available crane capacity, and site access. Lifting girders during erection is not as critical as when they are stripped from the forms, particularly when the same lifting devices are used for both. However, if a separate set of erection devices are used, the girder shall be checked for stresses and lateral stability. In addition, once the girder is set in place, the free span between supports is usually increased. Wind can also pose a problem. Consequently, when girders are erected, they shall immediately be braced. The temporary bracing of the girders is the contractor's responsibility.

F. **Construction Sequence for Multi-Span Prestressed Girder Bridges –** For multi-span prestressed girder bridges, the sequence and timing of the superstructure construction has a significant impact on the performance and durability of the bridge. In order to maximize the performance and durability, the "construction sequence" details shown in Appendix 5.6-A2 shall be followed for all new WSDOT multi-span prestressed girder bridges. Particular attention shall be paid to the timing of casting the lower portion of the pier diaphragms/crossbeams (30 days minimum after girder fabrication) and the upper portion of the diaphragms/crossbeams (10 days minimum after placement of the deck slab). The requirements apply to multi-span prestressed girder bridges with monolithic and hinge diaphragms/crossbeams.

5.6.4 Superstructure Optimization

A. **Girder Selection –** Cost of the girders is a major portion of the cost of prestressed girder bridges. Much care is therefore warranted in the selection of girders and in optimizing their position within the structure. The following general guidelines should be considered.

  1. **Girder Series Selection –** All girders in a bridge shall be of the same series unless approved otherwise by the Bridge and Structures Engineer. If vertical clearance is no problem, a larger girder series, utilizing fewer girder lines, may be a desirable solution.

     Fewer girder lines may result in extra reinforcement and concrete but less forming cost. These items must also be considered.

  2. **Girder Concrete Strength –** Higher girder concrete strengths should be specified where that strength can be effectively used to reduce the number of girder lines, see Section 5.1.1.A.2. When the bridge consists of a large number of spans, consideration should be given to using a more exact analysis than the usual design program in an attempt to reduce the number of girder lines. This analysis shall take into account actual live load, creep, and shrinkage stresses in the girders.

  3. **Girder Spacing –** Consideration must be given to the deck slab cantilever length to determine the most economical girder spacing. This matter is discussed in Section 5.6.4.B. The deck slab cantilever length should be made a maximum if a line of girders can be saved. It is recommended that the overhang length, from edge of slab to center line of exterior girder, be less than 40% of girder spacing; then the exterior girder can use the same design as that of the interior girder. The following guidance is suggested.

     a. **Tapered Spans –** On tapered roadways, the minimum number of girder lines should be determined as if all girder spaces were to be equally flared. As many girders as possible, within the limitations of girder capacity should be placed. Deck slab thickness may have to be increased in some locations in order to accomplish this.

     b. **Curved Spans –** On curved roadways, normally all girders will be parallel to each other. It is critical that the exterior girders are positioned properly in this case, as described in Section 5.6.4.B.

     c. **Geometrically Complex Spans –** Spans which are combinations of taper and curves will require especially careful consideration in order to develop the most effective and economical girder arrangement. Where possible, girder lengths and numbers of straight and harped strands should be made the same for as many girders as possible in each span.

d. **Number of Girders in a Span –** Usually all spans will have the same number of girders. Where aesthetics of the underside of the bridge is not a factor and where a girder can be saved in a short side span, consideration should be given to using unequal numbers of girders. It should be noted that this will complicate crossbeam design by introducing torsion effects and that additional reinforcement will be required in the crossbeam.

B. **Deck Slab Cantilevers –** The exterior girder location is established by setting the dimension from centerline of the exterior girder to the adjacent curb line. For straight bridges this dimension will normally be no less than 2′-6″ for W42G, W50G, and W58G; 3′-0″ for W74G; and 3′-6″ for WF74G, WF83G, WF95G and WF100G. Some considerations which affect this are noted below.

1. **Appearance –** Normally, for best appearance, the largest deck slab overhang which is practical should be used.
2. **Economy –** Fortunately, the condition tending toward best appearance is also that which will normally give maximum economy. Larger curb distances may mean that a line of girders can be eliminated, especially when combined with higher girder concrete strengths.
3. **Deck Slab Strength –** It must be noted that for larger overhangs, the deck slab section between the exterior and the first interior girder may be critical and may require thickening.
4. **Drainage –** Where drainage for the bridge is required, water from bridge drains is normally piped across the top of the girder and dropped inside of the exterior girder line. A large deck slab cantilever length may severely affect this arrangement and it must be considered when determining exterior girder location.
5. **Bridge Curvature –** When straight prestressed girders are used to support curved roadways, the curb distance must vary. Normally, the maximum deck slab overhang at the centerline of the long span will be made approximately equal to the overhang at the piers on the inside of the curve. At the point of minimum curb distance, however, the edge of the girder top flange should be no closer than 1′-0″ from the deck slab edge. Where curvature is extreme, other types of bridges should be considered. Straight girder bridges on highly curved alignments have a poor appearance and also tend to become structurally less efficient.

C. **Diaphragm Requirements**

1. **General –** Diaphragms used with prestressed girder bridges serve two purposes. During the construction stage, the diaphragms help to provide girder stability for pouring the deck slab. During the life of the bridge, the diaphragms act as load distributing elements, and are particularly advantageous for distribution of large overloads. Diaphragms also improve the bridge resistance to over-height impact loads.

   Diaphragms for prestressed girder bridges shall be cast-in-place concrete. Standard diaphragms and diaphragm spacings are given in the office standards for prestressed girder bridges. For large girder spacings or other unusual conditions, special diaphragm designs shall be performed.
2. **Design –** Diaphragms shall be designed as transverse beam elements carrying both dead load and live load. Wheel loads for design shall be placed in positions so as to develop maximum moments and maximum shears.
3. **Geometry –** Diaphragms shall normally be oriented parallel to skew (as opposed to normal to girder centerlines). This procedure has the following advantages:

   a. The build-up of higher stresses at the obtuse corners of a skewed span is minimized. This build-up has often been ignored in design.

b. Skewed diaphragms are connected at points of approximately equal girder deflections and thus tend to distribute load to the girders in a manner that more closely meets design assumptions.

c. The diaphragms have more capacity as tension ties and compression struts are continuous. Relatively weak inserts are only required at the exterior girder.

On curved bridges, diaphragms shall normally be placed on radial lines.

4. **Full or Partial Depth Intermediate Diaphragms –** Prestressed concrete girder bridges are often damaged by over-height loads. The damage may range from spalling and minor cracking of the bottom flange or web of the prestressed concrete girder to loss of a major portion of a girder section.

   Based on research done by WSU (see reference 24), the use of intermediate diaphragms for I-shape and deck bulb tee prestressed concrete girder bridges shall be as follows:

   a. Full depth intermediate diaphragms as shown in the office standard plans shall be used for bridges crossing over roads of ADT > 50000.

   b. Either full depth or partial depth intermediate diaphragms as shown in the office standard plans may be used for all bridges not included in item 1.

   The use of full or partial depth intermediate diaphragms in bridge widenings shall be considered on a case-by-case basis depending on the width of the widening and number of added girders.

D. **Skew Effects –** Skew in prestressed girder bridges affects structural behavior and member analysis and complicates construction.

1. **Analysis** – Normally, the effect of skew on girder analysis is ignored. It is assumed that skew has little structural effect on normal spans and normal skews. For short, wide spans and for extreme skews (values over 45°), the effect of the skew on structural action shall be investigated. All trapezoidal tub, slab, tri-beam and deck bulb-tee girders have a skew restriction of 30°.

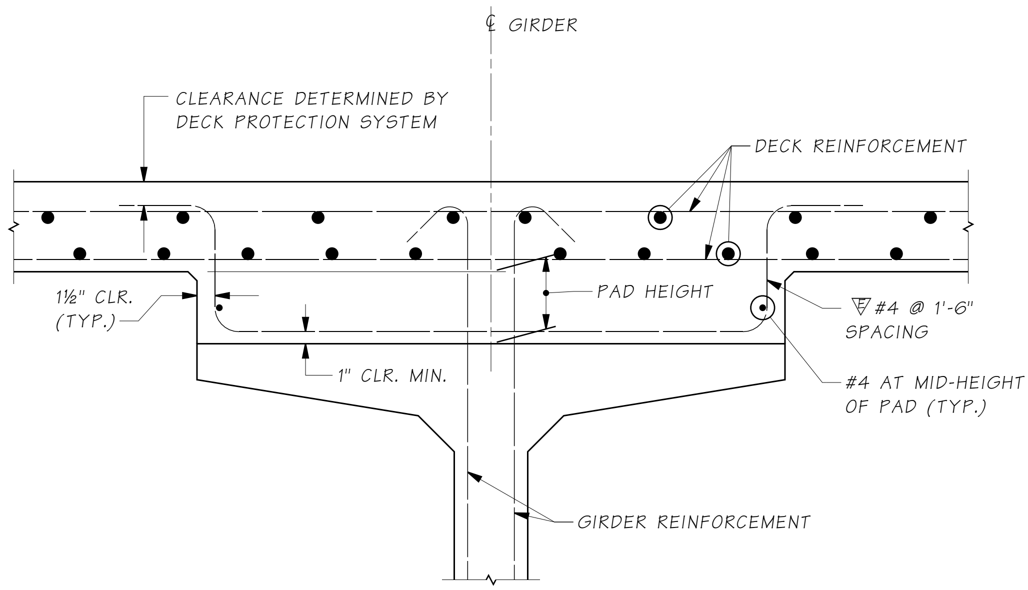
2. **Detailing** – To minimize labor costs and to avoid stress problems in prestressed girder construction, the ends of girders for continuous spans shall normally be made skewed. Skewed ends of prestressed girders shall always match the piers they rest on at either end.

E. **Grade and Cross Slope Effects –** Large cross slopes require an increased amount of the girder pad dimension ('A' dimension) necessary to ensure that the structure can be built. This effect is especially pronounced if the bridge is on a horizontal or vertical curve. Care must be taken that deck drainage details reflect the cross slope effect.

Girder lengths shall be modified for added length along grade slope.

F. **Curve Effect and Flare Effect –** Curves and tapered roadways each tend to complicate the design of straight girders. The designer must determine what girder spacing to use for dead load and live load design and whether or not a refined analysis, that considers actual load application, is warranted. Normally, the girder spacing at centerline of span can be used for girder design, especially in view of the conservative assumptions made for the design of continuous girders.

G. **Girder Pad Reinforcement –** Girders with a large "A" dimension may require a deep pad between the top of the girder and the bottom of the deck. When the depth of the pad at the centerline of the girder exceeds 6″, reinforcement shall be provided in the pad as shown in Figure 5.6.4-1.



Girder Pad Reinforcement
Figure 5.6.4-1

5.6.5 Repair of Damaged Girders at Fabrication

When girders suffer defects during fabrication or damage before becoming part of a final structure, the girder repairs shall be addressed with pre-approved repair procedures from the current Annual Plant Approval document for the fabricator (see Standard Specification 6-02.3(25)A). If the repairs cannot be addressed by this document, the fabricator will initiate the Fax Resolution process from the current Annual Plant Approval document to address contract specific repairs with the Project Office and HQ Bridge Construction. Normally, no designer action is required. When evaluating repairs for unusual situations not covered, the designer must ensure that the required strength and appearance of the girder can be maintained. If stressing will occur after the repair is made, normally no test loading is required; however, such a test should be considered. See reference [14] for guidance.

5.6.6 Repair of Damaged Girders in Existing Bridges

A. **General –** This section is intended to cover repair of damaged girders on existing bridges. For repair of newly constructed girders, see Section 5.6.5. Overheight loads are a fairly common source of damage to prestressed girder bridges. The damage may range from spalling and minor cracking of the lower flange of the girder to loss of a major portion of a girder section. Occasionally, one or more strands may be broken. The damage is most often inflicted on the exterior or first interior girder.

B. **Repair Procedure –** The determination of the degree of damage to a prestressed girder is largely a matter of judgment. Where the flange area has been reduced or strands lost, calculations can aid in making this judgment decision. The following are general categories of damage and suggested repair procedures [15, 16].

   1. **Minor Damage –** If the damage is slight and concerns only spalling of small areas of the outside surface of the concrete, repair may be accomplished by replacing damaged concrete areas with concrete grout. The area where new concrete is to be applied shall first be thoroughly cleaned of loose material, dried, and then coated with epoxy.

2. **Moderate Damage –** If damage is moderate, consisting of loss of a substantial portion of the flange and possibly loss of one or more strands, a repair procedure must be developed using the following guidelines. It is probable that some prestress will have been lost in the damaged area due to reduction in section and consequent strand shortening or through loss of strands. The following repair procedure is recommended to assure that as much of the original girder strength as possible is retained:

   a. **Determine Condition –** Sketch the remaining cross section of the girder and compute its reduced section properties. Determine the stress in the damaged girder due to the remaining prestress and loads in the damaged state. If severe overstresses are found, action must be taken to restrict loads on the structure until the repair has been completed. If the strand loss is so great that AASHTO prestress requirements cannot be met with the remaining strands, consideration should be given to replacing the girder.

   b. **Restore Prestress If Needed –** If it is determined that prestress must be restored, determine the stress in the bottom fiber of the girder as originally designed due to $DL + LL + I +$ Prestress. (This will normally be about zero psi). Determine the additional load (P) that, when applied to the damaged girder in its existing condition, will result in this same stress. Take into account the reduced girder section, the effective composite section, and any reduced prestress due to strand loss. Should the damage occur outside of the middle one-third of the span length, the shear stress with the load (P) applied should also be computed. Where strands are broken, consideration should be given to coupling and jacking them to restore their prestress.

   c. **Prepare a Repair Plan –** Draw a sketch to show how the above load is to be applied and specify that the damaged area is to be thoroughly prepared, coated with epoxy, and repaired with grout equal in strength to the original concrete. Specify that this load is to remain in place until the grout has obtained sufficient strength. The effect of this load is to restore lost prestress to the strands which have been exposed.

   d. **Test Load –** Consideration should be given to testing the repaired girder with a load equivalent to $1.0DL + 1.5(LL+IM)$. The *LL* Live Load for test load is HL-93.

3. **Severe Damage –** Where the damage to the girder is considered to be irreparable due to loss of many strands, extreme cracking, etc., the girder may need to be replaced. This has been done several times, but involves some care in determining a proper repair sequence.

   In general, the procedure consists of cutting through the existing deck slab and diaphragms and removing the damaged girder. Adequate exposed reinforcement steel must remain to allow splicing of the new bars. The new girder and new reinforcement is placed and previously cut concrete surfaces are cleaned and coated with epoxy. New deck slab and diaphragm portions are then poured.

   It is important that the camber of the new girder be matched with that in the old girders. Excessive camber in the new girder can result in inadequate deck slab thickness. Girder camber can be controlled by prestress, curing time, or dimensional changes.

   Pouring the new deck slab and diaphragms simultaneously in order to avoid overloading the existing girders in the structure should be considered. Extra bracing of the girder at the time of deck slab pour shall be required.

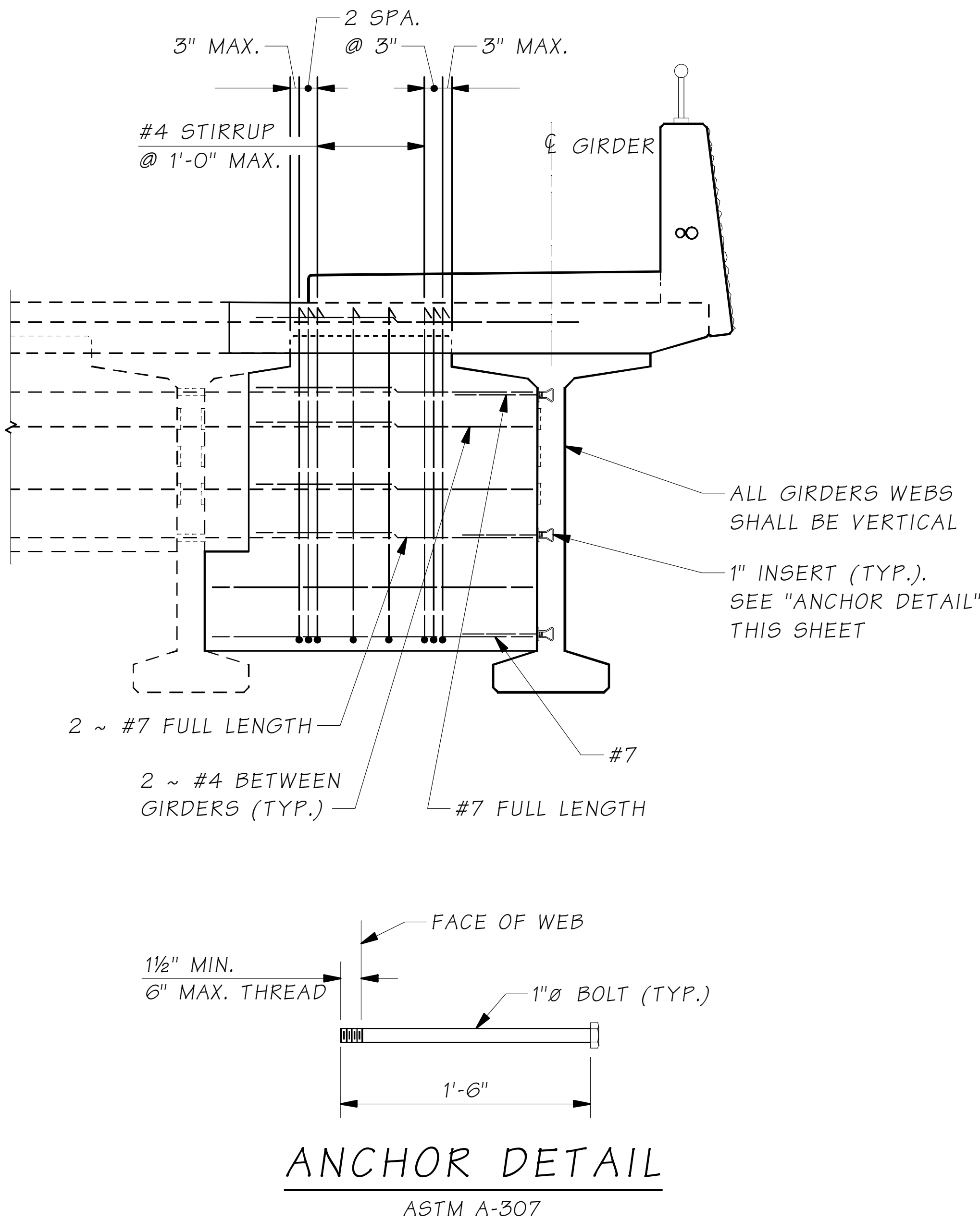
   Methods of construction shall be specified in the plans that will minimize inconvenience and dangers to the public while achieving a satisfactory structural result. High early strength grouts and concretes should be considered.

In case of replacement of a damaged girder, the intermediate diaphragms adjacent to the damaged girder shall be replaced with full depth diaphragms as shown in Figure 5.6.6-1.

In case of replacement of a damaged girder, the replacement girder shall preferably be the same type as the original damaged girder.

In case of repair of a damaged girder with broken or damaged prestressing strands, the original damaged strands shall be replaced with similar diameter strands. Restoration of the prestress force as outlined in BDM 5.6.6 B-2b shall be considered.

Existing bridges with pigmented sealer shall have replacement girders sealed. Those existing bridges without pigmented sealer need not be sealed.



Full Depth Intermediate Diaphragm Replacement

Figure 5.6.6-1

4. **Repair vs. Replacement of Damaged Girder –** Several factors need to be considered when evaluating whether to repair or to replace a damaged girder. Among them are the level of concrete damage, number of broken strands, location and magnitude of web damage, permanent offset of the original girder alignment, and overall structural integrity. Other considerations include fresh damage to previously damaged girders, damage to adjacent girders, and cost of repair versus replacement. Ultimately, the evaluation hinges on whether the girder can be restored to its original capacity and whether the girder can be repaired sufficiently to carry its share of the original load.

   The following guidelines describe damaged girder conditions which require replacement:

   - **Strand Damage** – More than 25% of prestressing strands are damaged/severed. If over 25% of the strands have been severed, replacement is required. Splicing is routinely done to repair severed strands. However, there are practical limits as to the number of couplers that can be installed in the damaged area.
   - **Girder Displacements** – The bottom flange is displaced from the horizontal position more than ½″ per 10′ of girder length. If the alignment of the girder has been permanently altered by the impact, replacement is required. Examples of non-repairable girder displacement include cracks at the web/flange interface that remain open. Abrupt lateral offsets may indicate that stirrups have yielded. A girder that is permanently offset may not be restorable to its original geometric tolerance by practical and cost-effective means.
   - **Concrete Damage at Harping Point** – Concrete damage at harping point resulting in permanent loss of prestress. Extreme cracking or major loss of concrete near the harping point may indicate a change in strand geometry and loss in prestress force. Such loss of prestress force in the existing damaged girder cannot be restored by practical and cost effective means, and requires girder replacement.
   - **Concrete Damage at Girder Ends** – Severe concrete damage at girder ends resulting in permanent loss of prestress or loss of shear capacity. Extreme cracking or major loss of concrete near the end of a girder may indicate unbonding of strands and loss in prestress force or a loss of shear capacity. Such loss of prestress force or shear capacity in the existing damaged girder cannot be restored by practical and cost-effective means, and requires girder replacement.

   There are other situations as listed below which do not automatically trigger replacement, but require further consideration and analysis.

   - **Significant Concrete Loss** – For girder damage involving significant loss of concrete from the bottom flange, consideration should be given to verifying the level of stress remaining in the exposed prestressing strands. Residual strand stress values will be required for any subsequent repair procedures.
   - **Adjacent Girders** – Capacity of adjacent undamaged girders. Consideration must be given as to whether dead load from the damaged girder has been shed to the adjacent girders and whether the adjacent girders can accommodate the additional load.
   - **Previously Damaged Girders** – Damage to a previously damaged girder. An impact to a girder that has been previously repaired may not be able to be restored to sufficient capacity.
   - **Cost** – Cost of repair versus replacement. Replacement may be warranted if the cost of repair reaches 70% of the replacement project cost.

C. **Miscellaneous References –** The girder replacement contracts and similar jobs listed in Table 5.6.6-1 should be used for guidance:

Contract	Project Name	Bridge Number	Total Bridge Length (ft)	Year work planned	Work Description
C-7425	I-5 Bridge 005/518 Girder Replacement	5/518	322	2008	Replace damaged PCG
C-7637	SR 520/ W Lake Sammamish Pkwy To SR 202 HOV And SR	11/1	287	2009	Replace damaged PCG in one span
C-7095	SR 14, Lieser Road Bridge Repair	14/12	208	2006	Replace damaged PCG
C-7451	I-90 Bridge No. 90/121-Replace Portion Of Damaged	90/121	250	2007	Replace damaged PCG
C-7567	Us395 Col Dr Br & Court St Br - Bridge Repair	395/103	114	2008	Replace damaged PCG
C-7774	SR 509, Puyallup River Bridge Special Repairs	509/11	3584	2010	Replace fire damage PCG span
C-9593	Columbia Center IC Br. 12/432(Simple Span)				Repair
C-9593	16th Avenue IC Br. 12/344 (Continuous Span)				Repair
C-9446	Mae Valley U Xing (Simple Span)				
KD-2488	13th Street O Xing 5/220 (Northwest Region)				
KD-2488	SR 506 U Xing 506/108 (Northwest Region)				
C-5328	Bridge 5/411 NCD (Continuous Span)				
KD-2976	Chamber of Commerce Way Bridge 5/227				
KD-20080	Golden Givens Road Bridge 512/10				
KD-2154	Anderson Hill Road Bridge 3/130W				

Girder Replacement Contracts
Table 5.6.6-1

5.6.7 Short Span Precast Prestressed Bridges

A. **General –** The term “deck girder” refers to a girder whose top flange or surface is the driving surface, with or without an overlay. They include slab, double-tee, ribbed and deck bulb-tee girders.

Deck girders without a composite CIP deck slab shall have a minimum concrete cover of 2″ over the top mat. The top mat of reinforcement in the deck (top flange) shall be epoxy-coated.

B. **Slab Girders –** Slab girder lengths shall be limited to the girder depth divided by 0.03 due to unexpected variations from traditional beam camber calculations. The following are maximum girder lengths using this criteria:

12″ deep slab = maximum girder length of 33′
18″ deep slab = maximum girder length of 50′
26″ deep slab = maximum girder length of 72′
30″ deep slab = maximum girder length of 83′
36″ deep slab = maximum girder length of 100′

The standard width of slab girders is shown in the girder standard plans. The width of slab girders can be increased but generally should not exceed 8′-0″.

A minimum 5″ composite CIP deck slab shall be placed over slab girders. The CIP concrete deck slab shall at a minimum be Class 4000D concrete with one layer of #4 epoxy coated reinforcement in both the transverse and longitudinal directions spaced at 1′-0″ maximum. Welded ties are still required.

The AASHTO LRFD 2.5.2.6.2 deflection criteria shall be satisfied for slab girders.

Temporary top strands are not required for the lateral stability of slab girders. Temporary top strands can be used if required to control concrete stresses due to plant handling, shipping and erection. These strands shall be bonded for 10′ at both ends of the girder, and unbonded for the remainder of the girder length. Temporary strands shall be cut prior to placing the CIP deck slab.

The specified design compressive strength ($f'c$) of slab girders should be kept less than or equal to 8 ksi to allow more fabricators to bid.

C. **Double-Tee and Ribbed Deck Girders –** Double-tee and ribbed deck girders shall be limited to widening existing similar structures. An HMA overlay with membrane shall be specified. These sections are capable of spanning up to 60′.

D. **Deck Bulb-Tee Girders –** Deck bulb-tee girders have standard girder depths of 35, 41, 53, and 65 inches. The top flange/deck may vary from 4-feet 1-inch to 6-feet wide. They are capable of spanning up to 135 feet.

Deck bulb-tee girders with an HMA overlay shall be limited to pedestrian bridges and to widening existing similar structures with an HMA overlay. A waterproofing membrane shall be provided. This is not a preferred option for WSDOT bridges, but is often used by local agencies.

Deck bulb-tee girders may be used with a minimum 5″ composite CIP deck slab as described above for slab girders. Welded ties and grouted keys at flange edges shall still be provided.

Thin flange deck bulb-tee girders (3″ top flange instead of 6″) with a minimum 7½″ composite CIP deck slab and two mats of epoxy-coated reinforcement are an alternative to deck bulb-tee girders. Thin flange deck bulb tee sections can be up to 8 feet wide. This is a preferred option for WSDOT bridges. It does not require welded ties and grouted keys.

5.6.8 Precast Prestressed Concrete Tub Girders

A. **General –** Precast prestressed concrete tub girders (U and UF sections) are an option for moderate bridge spans.

The standard tub girders (U sections) have 4′-0″ or 5′-0″ bottom flange widths and are 4′-6″, 5′-6″ or 6′-6″ deep. A 6″ deep top flange can be added to tub girders (UF sections) to improve structural efficiency and to accommodate placement of stay-in-place precast deck panels.

B. **Curved Precast Tub Girders –** Curved precast tub girders may be considered for bridges with moderate horizontal radiuses. Precast I-girders may not be curved.

Curved precast tub girders can either be designed in one piece or in segments depending on span configurations and shipping limitations. Curved precast tub girders are post-tensioned at the fabrication plant and shipped to the jobsite. Additional jobsite post-tensioning may be required if segment assembly is necessary, or if continuity over intermediate piers is desired. Closure joints at segment splices shall meet the requirements of Section 5.9.4.C.

The following limitations shall be considered for curved precast tub girders:

1. The overall width of precast curved segments for shipment shall not exceed 16 feet.
2. The location of the shipping supports shall be carefully studied so that the precast segment is stable during shipping. The difference in dead load reactions of the shipping supports within the same axle shall not exceed 5 percent.
3. The maximum shipping weight of precast segments may be different depending on the size of precast segments. The shipping weight shall meet the legal axle load limits set by the RCW, but in no case shall the maximum shipping weight exceed 275 kips.
4. The minimum web thickness shall be 10″. Other cross-sectional dimensions of WSDOT standard tub girders are applicable to curved precast tub girders.
5. Effects of curved tendons shall be considered per Section 5.8.1.F.
6. The clear spacing between ducts shall be 2″ min. The duct diameter shall not exceed 4½″.

5.6.9 Prestressed Girder Checking Requirement

A. Shear reinforcing size and spacing shall be determined by the designer.

B. Determine lifting location and required concrete strength at release to provide adequate stability during handling. Generally temporary strands provide additional stability for lifting and transportation, and reduce the camber. Less camber allows for less "A" dimension and concrete pad dead weight on the structure. Temporary strands are cut after the girders are erected and braced and before the intermediate diaphragms are cast.

C. Due to the extreme depth of the WF83G, WF95G, and WF100G girders, and possible tilt at the piers for profile grades, the designer will need to pay particular attention to details to assure the girders will fit and perform as intended.

D. Check edge distance of supporting cross beam.

5.6.10 Review of Shop Plans for Pretensioned Girders

Pretensioning shop drawings shall be reviewed by the designer. Shop drawings, after review by the designer, shall be stamped with the official seal and returned to the bridge construction support office. The review must include:

A. All prestressing strands shall be of ½″ or 0.6″ diameter grade 270 low relaxation uncoated strands.

B. Number of strands per girder.

C. Jacking stresses of strands shall not exceed $0.75f_{pu}$.

D. Strand placement patterns and harping points.

E. Temporary strand pattern, bonded length, location and size of blockouts for cutting strands.

F. Procedure for cutting temporary strands and patching the blockouts shall be specified.

G. Number and length of extended strands and rebars at girder ends.

H. Locations of holes and shear keys for intermediate and end diaphragms.

I. Location and size of bearing recesses.

J. Saw tooth at girder ends.

K. Location and size of lifting loops or lifting bars.

L. All horizontal and vertical reinforcement.

M. Girder length and end skew.

Chapter 6 Structural Steel — Contents

6.1 Design Considerations

6.1.1 Codes, Specification, and Standards

Steel highway bridges shall be designed to the following codes and specifications:

- AASHTO LRFD *Bridge Design Specifications,* Sixth Edition
- AASHTO *Guide Specifications for Horizontally Curved Steel Girder Highway Bridges, 2003* (Retained for reference, the body of curved girder specifications has been incorporated in the main text of the LRFD code with the 2005 interims)
- AASHTO/AWS D1.5M/D1.5: 2010 *Bridge Welding Code*
- ANSI/AWS A2.4-98 *Standard Symbols for Welding, Brazing, and Nondestructive Examination*

The following codes and specifications shall govern steel bridge construction:

- WSDOT *Standard Specifications for Road, Bridge, and Municipal Construction,* latest edition
- AASHTO/AWS D1.5M/D1.5: 2008 *Bridge Welding Code*
- AASHTO *Guide Specifications for Highway Bridge Fabrication with HPS70W Steel*, latest edition

The following AASHTO/NSBA Steel Bridge Collaboration publications are available to aid in the design and fabrication of steel bridges. These publications can be downloaded from the AASHTO website or a copy can be obtained from the Steel Specialist:

- *Design Drawing Presentation Guidelines*
- *Guidelines for Design for Constructibility*
- *Shop Detail Drawing Presentation Guidelines*
- *Steel Bridge Fabrication Guide Specification*
- *Steel Bridge Fabrication QC/QA Guide Specification*
- *Guide Specification for Coating Systems with Inorganic Zinc-Rich Primer*
- *Guidelines for Steel Girder Bridge Analysis*
- *Steel Bridge Bearing Design and Detailing Guidelines*
- *Steel Bridge Erection Guide Specification*
- *Guidelines for Design Details*
- *Shop Detail Drawing Review/Approval Guidelines*

For checking the capacity of gusset plates in fracture critical trusses:

- *Load Rating Guidance and Examples for Bolted and Riveted Gusset Plates in Truss Bridges, Publication No. FHWA-if-09-014*

6.1.2 Preferred Practice

Unshored, composite construction is used for most plate girder bridges. Shear connectors are placed throughout positive and negative moment regions, for full composite behavior. One percent longitudinal deck steel, placed in accordance with AASHTO LRFD Article 6.10.1.7, ensures adequate deck performance in negative moment regions. For service level stiffness analysis, such as calculating live load moment envelopes, the slab may be considered composite and uncracked for the entire bridge length, provided the above methods are used. See AASHTO LRFD Articles 6.6.1.2.1 and 6.10.1.5. For negative moment at strength limit states, the slab is ignored while reinforcing steel is included for stress and section property calculations. Where span arrangement is not well balanced, these assumptions may not apply.

Plastic design may be utilized for simple span and positive moment regions of medium to long span plate girder bridges. In negative moment regions, plastic design is only economical for short span beams.

Currently, economical design requires simplified fabrication with less emphasis on weight reduction. The number of plate thicknesses and splices should be minimized. Also, the use of fewer girder lines, spaced at a maximum of about 14 feet, saves on fabrication, shipping, painting, and future inspection. Widely spaced girders will have heavier flanges, hence, greater stability during construction. Normally, eliminating a girder line will not require thickening remaining webs or increasing girder depth. The increased shear requirement can be met with a modest addition of web stiffeners or slightly thicker webs at interior piers.

For moderate to long spans, partially stiffened web design is the most economical. This method is a compromise between slender webs requiring transverse stiffening throughout and thicker, unstiffened webs. Stiffeners used to connect crossframes shall be welded to top and bottom flanges. Jacking stiffeners shall be used adjacent to bearing stiffeners, on girder or diaphragm webs, in order to accommodate future bearing replacement. Coordinate jack placement in substructure and girder details.

Steel framing should consist of main girders and crossframes. Bottom lateral systems should only be used when temporary bracing is not practical. Where lateral systems are needed, they should be detailed carefully for adequate fatigue life.

Standard corrosion protection for steel bridges is a three-coat paint system, west of the Cascades and where paint is required for appearance. Weathering steel should be considered for dry, eastern regions. When weathering steel is used and appearance is not critical, a single shop coat of inorganic zinc-rich primer may be considered in coastal regions.

WSDOT does not currently allow the use of steel stay-in-place deck forms.

6.1.3 Preliminary Girder Proportioning

The superstructure depth is initially determined during preliminary plan development and is based upon the span/depth ratios provided in Chapter 2 of this manual. The depth may be reduced to gain vertical clearance, but the designer should verify live load deflection requirements are met. See AASHTO LRFD Table 2.5.2.6.3-1. It is office practice to limit live load deflections in accordance with the optional criteria of AASHTO LRFD Articles 2.5.2.6.2 and 3.6.1.3.2.

The superstructure depth is typically shown as the distance from the top of the concrete roadway slab to the bottom of the web. Web depths are generally detailed in multiples of 6 inches.

On straight bridges, interior and exterior girders should be detailed as equal. Spacing should be such that the distribution of wheel loads on the exterior girder is close to that of the interior girder. The number of girder lines should be minimized, with a maximum spacing of 14 feet. Three or more girders lines are considered redundant. If a non-redundant bridge is proposed, approval must be obtained from the Bridge Design Engineer.

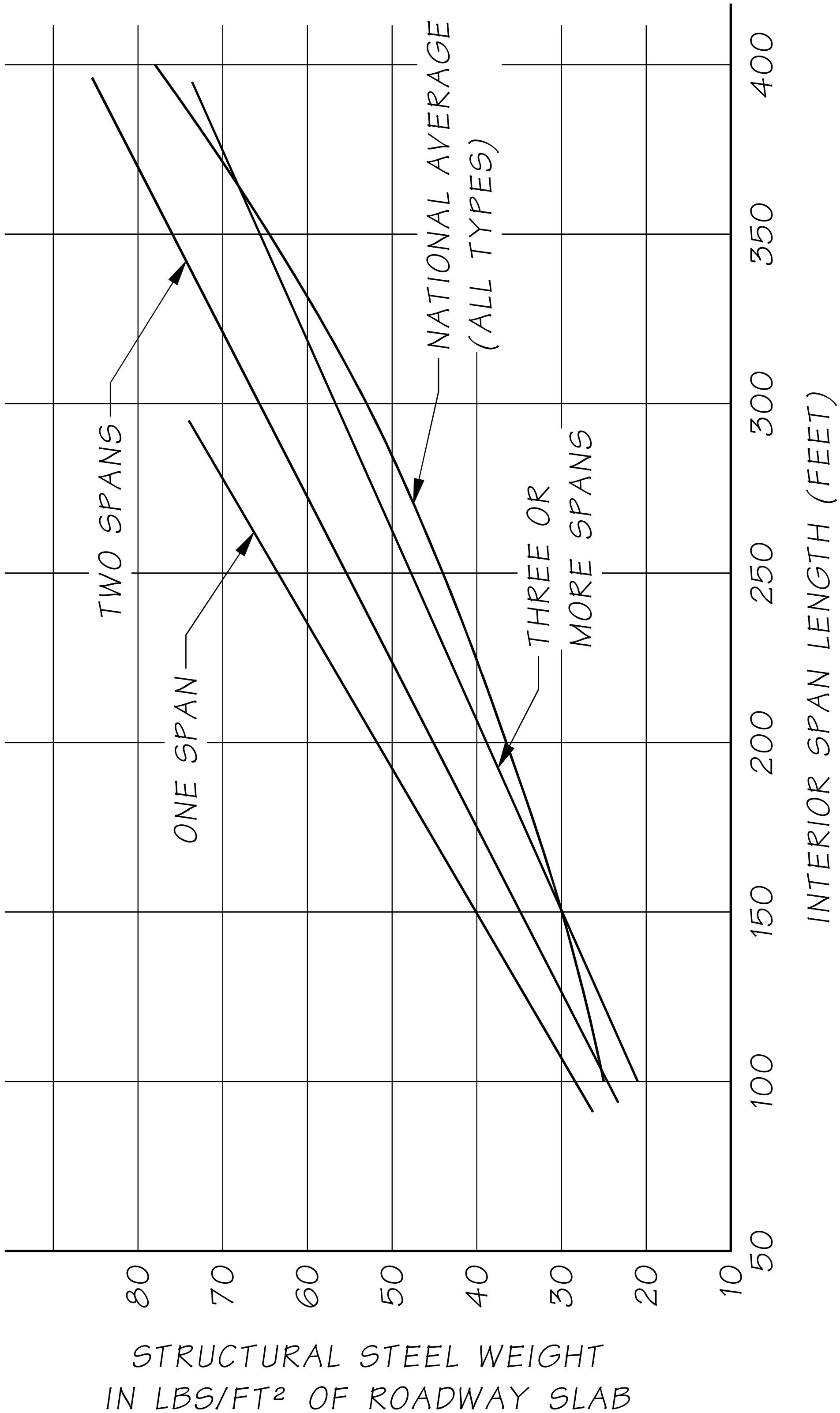
6.1.4 Estimating Structural Steel Weights

For the preliminary quantities or preliminary girder design, an estimate of steel weights for built-up plate composite “I” girders can be obtained from Figure 6.1.4-1 This figure is based upon previous designs with AASHTO HS-20 live loads with no distinction between service load designs and load factor designs. This chart also provides a good double check on final quantities.

The weights shown include webs, flanges, and all secondary members (web stiffeners, diaphragms, crossframe, lateral systems, gusset plates) plus a small allowance for weld metal, bolts, and shear connectors.

Both straight and curved box girder quantities may be estimated with the chart, using a 10 to 20 percent increase.

The chart should only be used for a lower bound estimate of curved I-girder weight. Roadway width and curvature greatly influence girder weight, including cross frames.



Composite Welded Steel Plate "I" Girder

Figure 6.1.4-1.

6.1.5 Bridge Steels

The most common types of steel used for bridges are now grouped in ASTM A 709 or AASHTO M 270 specifications. The following table shows equivalent designations. Grades of steel are based on minimum yield point.

ASTM	ASTM A 709/ AASHTO M 270
A 36	Grade 36
A 572 gr 50	Grade 50
A 992 (W shapes)	Grade 50S
A 588	Grade 50W
	Grade HPS50W**
A 852	Grade 70W
---	Grade HPS70W
A 514*	Grade 100*
	Grade I00W*

*Minimum yield strength is 90 ksi for plate thickness greater than 2½″.
**Avoid unless project has a large order with long lead times available.

Plates and rolled sections are available in these specifications and grades. Rolled sections include beams (W, S, and M shapes), H-piles, tees, channels, and angles. These materials are prequalified under the Bridge Welding Code. The common specification for wide flange beams is now ASTM A 992.

Use AASHTO M 270 grades 50 or 50W for plate girders. The fabricated costs of structural carbon and structural low alloy steel plate girders are about equal. AASHTO now recommends grade HPS70W instead of grade 70W for bridge use. HPS70W can be economical if used selectively in hybrid design. For moderate spans consider HPS70W for the bottom flanges throughout and top flanges near interior piers. The use of M 270 grade 100 or 100W requires approval by the Bridge Design Engineer, and should not be used until grade HPS100W is available.

All main load-carrying members or components subject to tensile stress shall be identified in the plans and be required to meet the minimum Charpy V-notch (CVN) fracture toughness values as specified in AASHTO LRFD Table 6.6.2-2, temperature zone 2. Fracture critical members or components shall also be designated in the plans.

Availability of weathering steel can be a problem for some sections. For example, steel suppliers do not stock angles or channels in weathering steel. Weathering steel wide flange and tee sections are difficult to locate or require a mill order. ASTM A 709 and AASHTO M 270 bridge steels are not stocked by local service centers. The use of bridge steel should be restricted to large quantities such as found in typical plate girder projects. The older ASTM specification steels, such as A 36, should be specified when a fabricator would be expected to purchase from local service centers. The older AASHTO designations, such as M183, have been dropped.

Structural tubes and pipes are covered by other specifications. See the latest edition of the AISC *Manual of Steel Construction* for selection and availability. These materials are not considered prequalified under the Bridge Welding Code. They are covered under the Structural Welding Code AWS D1.1. Structural tubing ASTM A 500 is not recommended for dynamic loading applications unless minimum CVN requirements are specified.

6.1.6 Available Plate Sizes

Readily available lengths and thicknesses of steel plates should be used to minimize costs. Tables of standard plate sizes have been published by various steel mills and should be used for guidance. These tables are available through the steel specialist, or online.

In general, an individual plate should not exceed 12′-6″ feet in width, including camber requirements, or a length of about 60 feet. If either or both of these dimensions are exceeded, a butt splice is required and should be shown or specified on the plans. Some plates may be available in lengths over 90 feet, so web splice locations should be considered optional. Quenched and tempered plates are limited to 50 feet, based on oven size.

Plate thicknesses of less than 5/16 inches should not be used for bridge applications.

When metric units are used, all steel dimensions, including thickness, should be hard converted. For example, specify 25 mm, not 25.4 mm plate.

Preferred plate thicknesses, English units, are a follows:

- 5/16″ to ⅞″ in 1/16″ increments
- ⅞″ to 1 ¼″ in ⅛″ increments
- 1¼″ to 4″ in ¼″ increments

6.1.7 Girder Segment Sizes

Locate bolted field splices so that individual girder segments can be handled, shipped, and erected without imposing unreasonable requirements on the contractor. Crane limitations need to be considered in congested areas near traffic or buildings. Transportation route options between the girder fabricator and the bridge site can affect the size and weight of girder sections allowed. Underpasses with restricted vertical clearance in sag vertical curves can be obstructions to long, tall segments shipped upright. The region should help determine the possible routes, and the restrictions they impose, during preliminary planning or early in the design phase.

Segment lengths should be limited to 150 feet, depending upon cross section. Long, slender segments can be difficult to handle and ship due to their flexibility. Horizontal curvature of girder segments may increase handling and shipping concerns. Out-to-out width of curved segments, especially box girders, should not exceed 14 feet without additional travel permits and requirements. Weight is seldom a controlling factor for I-girders. However, 40 tons is a practical limit for some fabricators. Limit weight to a maximum of 100 tons if delivery by truck is anticipated.

Consider the structure's span length and the above factors when determining girder segment lengths. In general, field splices should be located at dead load inflection points. When spans are short enough, some field splices can be designated optional if resulting segment lengths and weights meet the shipping criteria.

6.1.8 Computer Programs

The designer should consult the steel specialist to determine the computer program best suited for a particular bridge type.

Office practice and good engineering principles require that the results of any computer program or analysis be independently verified for accuracy. Also, programs with built-in code checks must be checked for default settings. Default settings may reflect old code or office practice may supersede the code that the program was written for.

6.1.9 Fasteners

All bolted connections shall be friction type (slip-critical). Assume Class B faying surfaces where inorganic zinc primer is used. If steel will be given a full paint system in the shop, the primed faying surfaces need to be masked to maintain the Class B surface.

Properties of High-Strength Bolts

Material	Bolt Diameter	Tensile Strength ksi	Yield Strength ksi
AASHTO M 164	⅝ - 1 inc	120	92
(ASTM A325)	1⅛ - 1½ inc.	105	81
	Over 1½		Not Available
ASTM A 449	¼ - 1 inc	120	92
(No AASHTO	1⅛ - 1½ inc.	105	81
equivalent)	1¾ -3 inc.	90	58
	Over 3		Not Available
AASHTO			
M 314	¼ - 3 inc	125-150	105
(ASTM F 1554)			
Grade 105			
AASHTO			
M253	⅝ - 1½ inc.	150-170	130
(ASTM A 490)			
	Over 1½		Not Available
ASTM A 354	½ - 2½ inc.	150	130
Grade BD			
(No AASHTO	3 - 4 inc.	140	115
equivalent)	Over 4		Not Available

General Guidelines for Steel Bolts

A. **M 164 (A325**) – High strength steel, headed bolts for use in structural joints. These bolts may be hot-dip galvanized. Do not specify for anchor bolts.

B. **A449 –** High strength steel bolts and studs for general applications including anchor bolts. Recommended for use as anchor bolts where strengths equivalent to A325 bolts are desired. These bolts may be hot-dip galvanized. Do not use these anchor bolts for seismic applications due to low CVN impact toughness.

C. **M 314 (F1554) - Grade 105** – Higher strength anchor bolts to be used for larger sizes (1½″ to 3″). When used in seismic applications, ASTM F 1554 shall be specified, since AASHTO M 314 lacks the CVN supplemental requirements. Specify supplemental CVN requirement S5 when these are used in seismic applications (most bridge bearings that resist lateral loads). Lower grades may also be suitable for sign structure foundations. This specification should also be considered for seismic restrainer rods, and may be galvanized.

D. **M 253 (A490) –** High strength alloy steel, headed bolts for use in structural joints. These bolts should not be galvanized, because of the high susceptibility to hydrogen embrittlement. In lieu of galvanizing, the application of an approved zinc rich paint may be specified. Do not specify for anchor bolts.

E. **A354 - Grade BD** – high strength alloy steel bolts and studs. These are suitable for anchor bolts where strengths equal to A490 bolts are desired. These bolts should not be galvanized. If used in seismic applications, specify minimum CVN toughness of 25 ft-lb at 40°F.

6.3 Design of I-Girders

6.3.1 Limit States for AASHTO LRFD

Structural components shall be proportioned to satisfy the requirements of strength,extreme event, service, and fatigue limit states as outlined in AASHTO LRFD Articles1.3.2 and 6.5.

Service limit states are included in Service I and Service II load combinations. Service I load combination is used to check the live load deflection limitations of AASHTO LRFD Article 2.5.2.6. Service II load combination is the AASHTO LRFD equivalent of the LFD overload provisions. Service II places limits on permanent deflection, no yielding, slenderness of the web in compression, and slip of bolted connections.

The fatigue live load specified in AASHTO LRFD Article 3.6.1.4 shall be used for checking girder details per Article 6.6. A single fatigue truck, without lane loading or variable axle spacing, is placed for maximum and minimum effect to a detail under investigation. The impact is 15%, regardless of span length. The load factor is 1.5. It is generally possible to meet the constant amplitude fatigue limit (CAFL) requirement for details with good fatigue performance. Limiting the calculated fatigue range to the CAFL ensures infinite fatigue life. Webs shall be checked for fatigue loading in accordance with AASHTO LRFD Article 6.10.5.3, using the calculated fatigue stress range for flexure or shear. Shear connector spacing shall be according to AASHTO LRFD Article 6.10.10. Generally, the fatigue resistance (Zr) for ⅞″ diameter shear connectors can be taken as 4.2 kips for an infinite number of cycles (CAFL = 4.2 kips).

Flanges and webs must meet strength limit state requirements for both construction and final phases. Constructibility requirements for flanges and webs are covered in AASHTO LRFD Article 6.10.3. Flexure resistance is specified in AASHTO LRFD Articles 6.10.7 and 6.10.8; shear resistance is specified in AASHTO LRFD Article 6.10.9

Pier crossframes shall be designed for seismic loading, extreme event load combination. Bolts are treated as bearing type connections with AASHTO LRFD Article 6.5.4.2 resistance factors. The resistance factor for all other members is 1.0 at extreme limit state.

6.3.2 Composite Section

Live load plus impact is applied to the transformed composite section using Es/Ec, commonly denoted n. Long-term loading (dead load of barriers, signs, luminaries, overlays, etc.) is applied to the transformed composite section using 3n. Positive moments are applied to these composite sections accordingly, both for service and strength limit states. The slab may be considered effective in negative moment regions provided tensile stresses in the deck are below the modulus of rupture. This is generally possible for Service I load combination and fatigue analysis. For strength limit state loadings, the composite section includes longitudinal reinforcing while the deck is ignored.

6.3.3 Flanges

Flange thickness is limited to 4″ maximum in typical bridge plate, but the desirable maximum is 3″. Structural Steel Notes on contract plans shall require all plates for flange material shall be purchased such that the ratio of reduction of thickness from a slab to plate shall be at least 3.0:1. This requirement helps ensure the plate material has limited inclusions and micro-porosity that can create problems during cutting and welding. Recent inquires with major domestic steel mills found that the 3.0:1 reduction requirement can be obtained up to 4″ thick plate. The number of plate thicknesses used for a given project should be kept to a minimum. Generally, the bottom flange should be wider than the top flange. Flange width changes should be made at bolted field splices. Thickness transitions are best done at welded splices. AASHTO LRFD Article 6.13.6.1.5 requires fill plates at bolted splices to be developed, if thicker than ¼″. Since this requires a significant increase in the number of bolts for thick fill plates, keeping the thickness transition ¼″ or less by widening pier segment flanges can be a better solution. Between field splices, flange width should be kept constant.

6.3.4 Webs

Maintain constant web thickness throughout the structure. If different web thickness is needed, the transition should be at a welded splice. Horizontal web splices are not needed unless web height exceeds 12′-6″. Vertical web splices for girders should be shown on the plans in an elevation view with additional splices made optional to the fabricator. All welded web splices on exterior faces of exterior girders and in tension zones elsewhere shall be ground smooth. Web splices of interior girders need not be ground in compression zones.

6.3.5 Transverse Stiffeners

These stiffeners shall be used in pairs at crossframe locations on interior girders and on the inside of webs of exterior girders. They shall be welded to the top flange, bottom flange and web at these locations. This detail is considered fatigue category C' for longitudinal flange stress. Stiffeners used between crossframes shall be located on one side of the web, welded to the compression flange, and cut short of the tension flange. Stiffeners located between crossframes in regions of stress reversal shall be welded to one side of the web and cut short of both flanges. Alternatively, they may be welded to both flanges if fatigue Category C' is checked. Transverse stiffeners may be dropped when not needed for strength. If crossframe spacing is less than 3 times the web depth, additional stiffeners may only be necessary near piers.

Stiffened webs require end panels to anchor the first tension field. The jacking stiffener to bearing stiffeners space shall not be used as the anchor panel. The first transverse stiffener is to be placed at no greater spacing than 1.5 times the web depth from the bearing or jacking stiffener.

Transverse stiffeners must be designed and detailed to meet AASHTO LRFD Article 6.10.11.1. Where they are used to connect crossframes, they should be a minimum width of 8″ to accommodate two bolt rows.

6.3.6 Longitudinal Stiffeners

On long spans where web depths exceed 10 feet, comparative cost evaluations shall be made to determine whether the use of longitudinal stiffeners will be economical. The use of longitudinal stiffeners may be economical on webs 10 feet deep or greater. Weld terminations for longitudinal stiffeners are fatigue prone details. Longitudinal stiffener plates should be continuous, splices being made with full penetration welds before being attached to webs. Transverse stiffeners should be pieced to allow passage of longitudinal stiffeners.

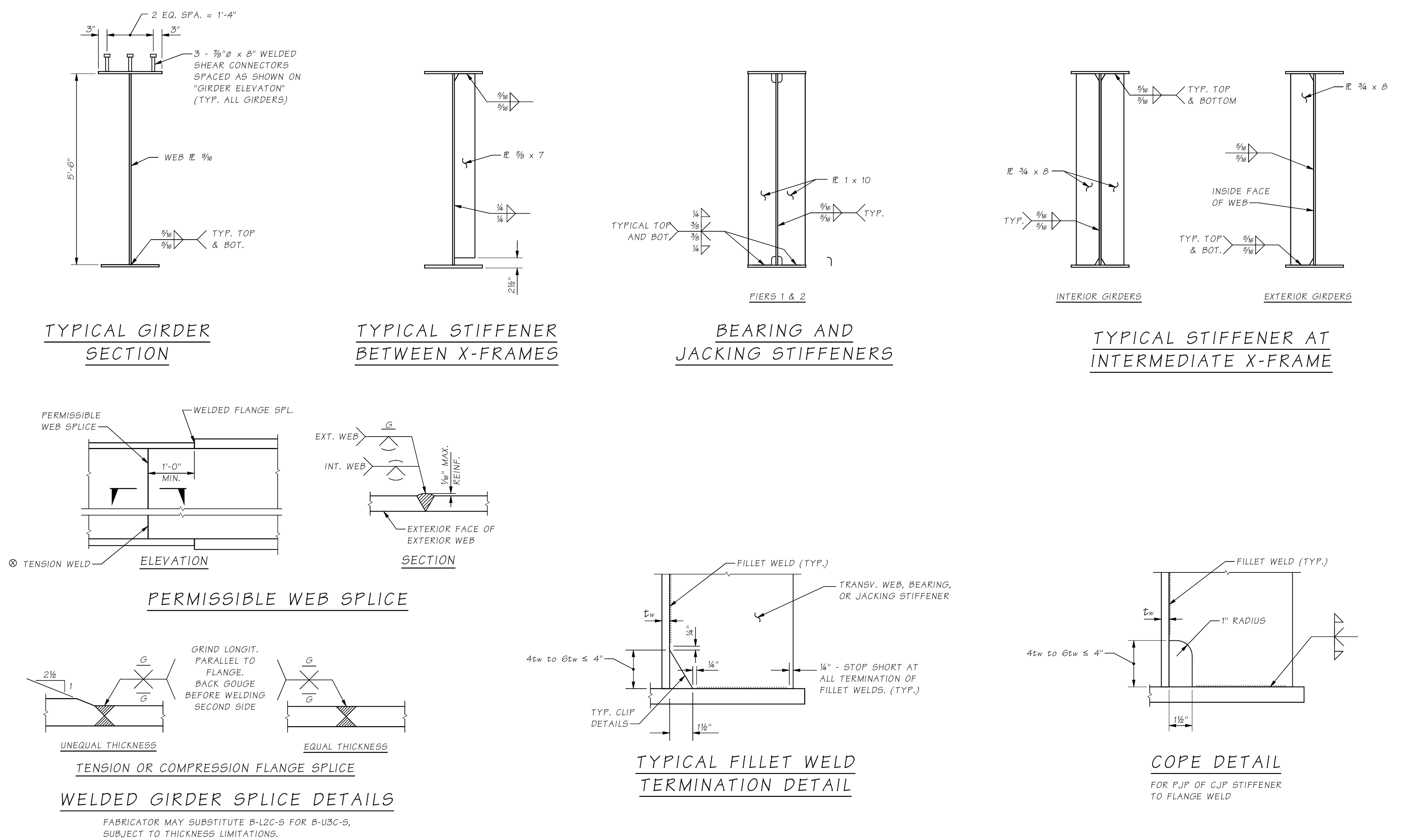
Design of longitudinal stiffeners is covered by AASHTO LRFD Article 6.10.11.3.

6.3.7 Bearing Stiffeners

Stiffeners are required at all bearings to enable the reaction to be transmitted from the web to the bearing. These stiffeners are designated as columns, therefore, must be vertical under total dead load. The connection of the bearing stiffener to flanges consists of partial penetration groove welds, of sufficient size to transmit design loads.

Pier crossframes may transfer large seismic lateral loads through top and bottom connections. Weld size must be designed to ensure adequate load path from deck and crossframes into bearings.

Design of bearing stiffeners is covered by AASHTO LRFD Article 6.10.11.2.



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GIRDER DETAILS

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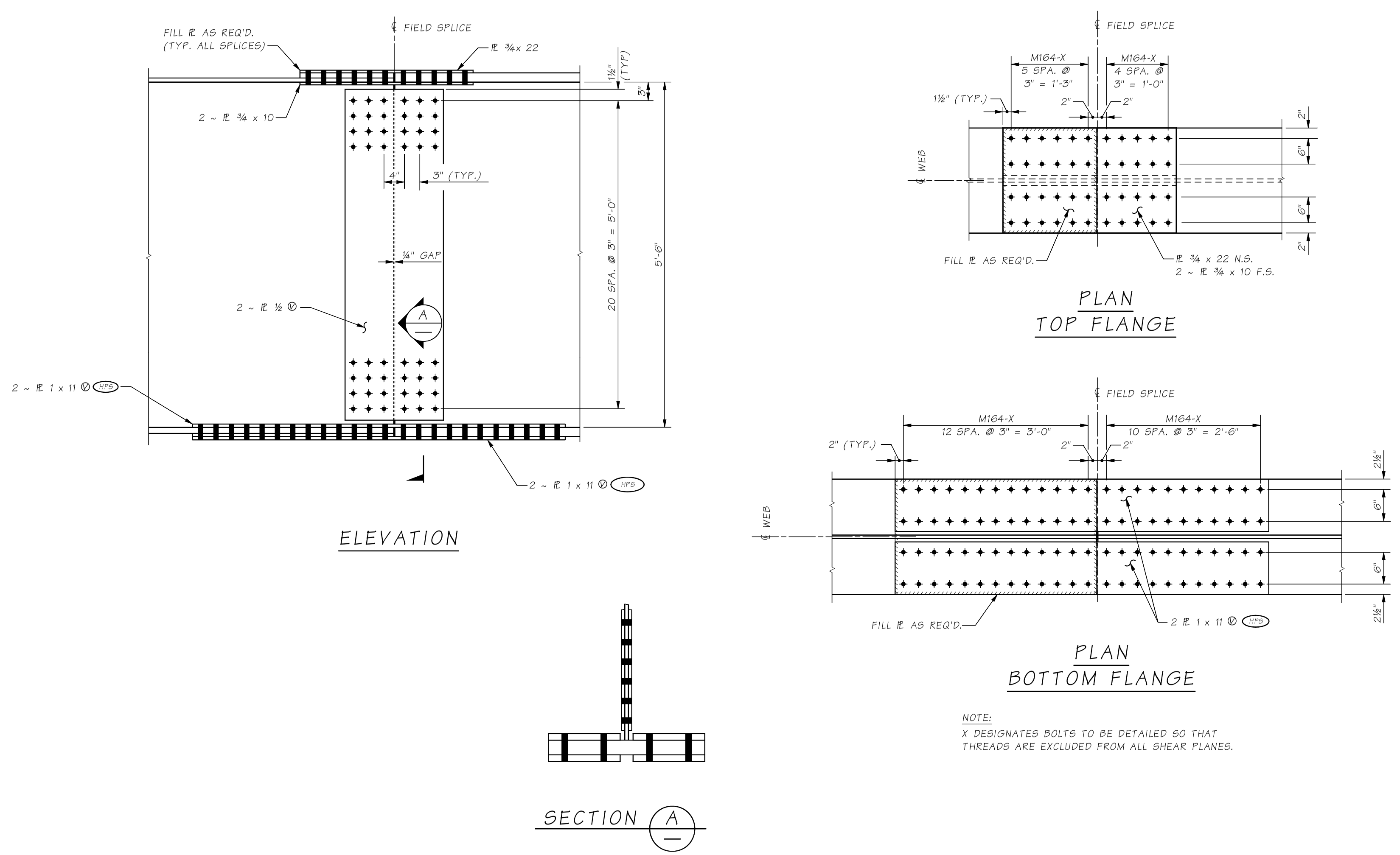

NOTE:
X DESIGNATES BOLTS TO BE DETAILED SO THAT THREADS ARE EXCLUDED FROM ALL SHEAR PLANES.

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STEEL PLATE GIRDER
FIELD SPLICE

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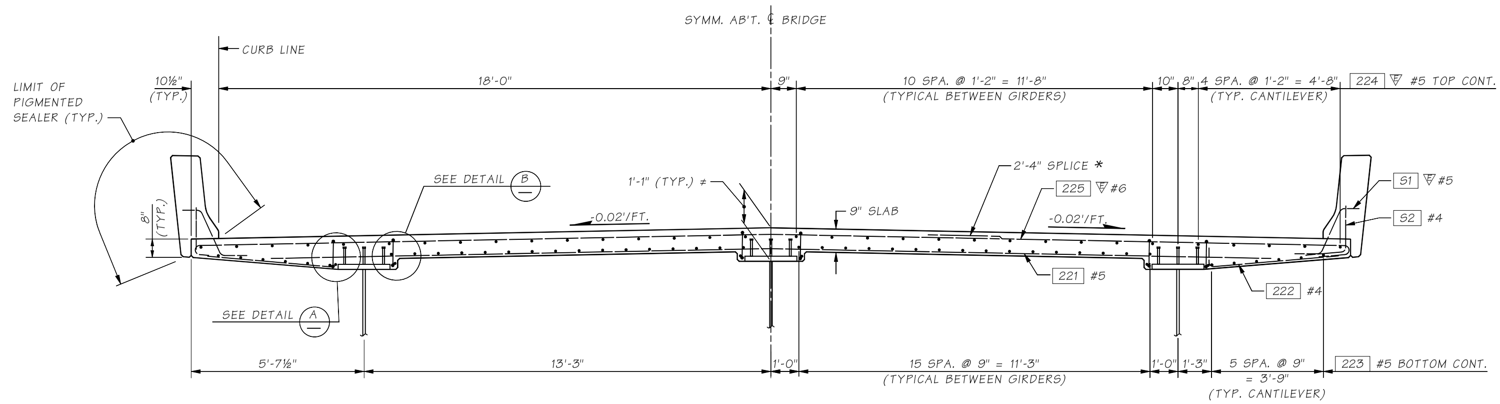
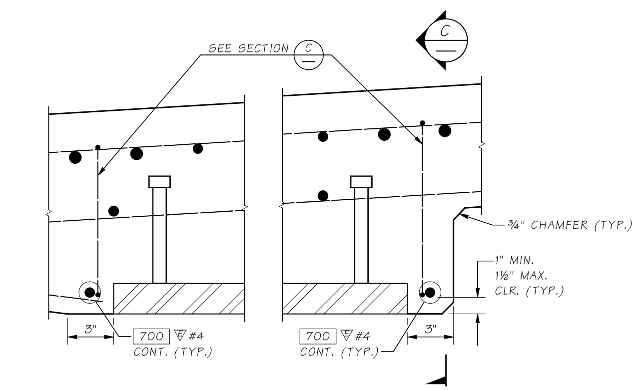


TYPICAL ROADWAY SECTION

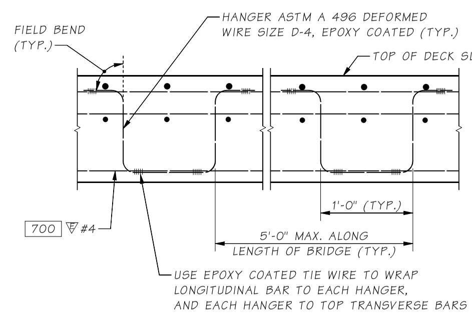
* - POSITION SPLICE MIDWAY BETWEEN GIRDERS. STAGGER SPLICES ABOUT ℄ BRIDGE, EVERY OTHER BAR. ROTATE HOOKS AS REQUIRED TO PROVIDE MINIMUM CONCRETE COVER.

≠ - DIMENSION TO BE ADJUSTED AFTER A SURVEY OF GIRDER ELEVATIONS, REFER TO STD. SPEC. 6-03.3(39).



DETAIL A
TYPICAL EACH OVERHANG

DETAIL B
TYPICAL EACH FILLET



SECTION C
HANGER SIDE VIEW

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BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

STEEL PLATE GIRDER ROADWAY SECTION

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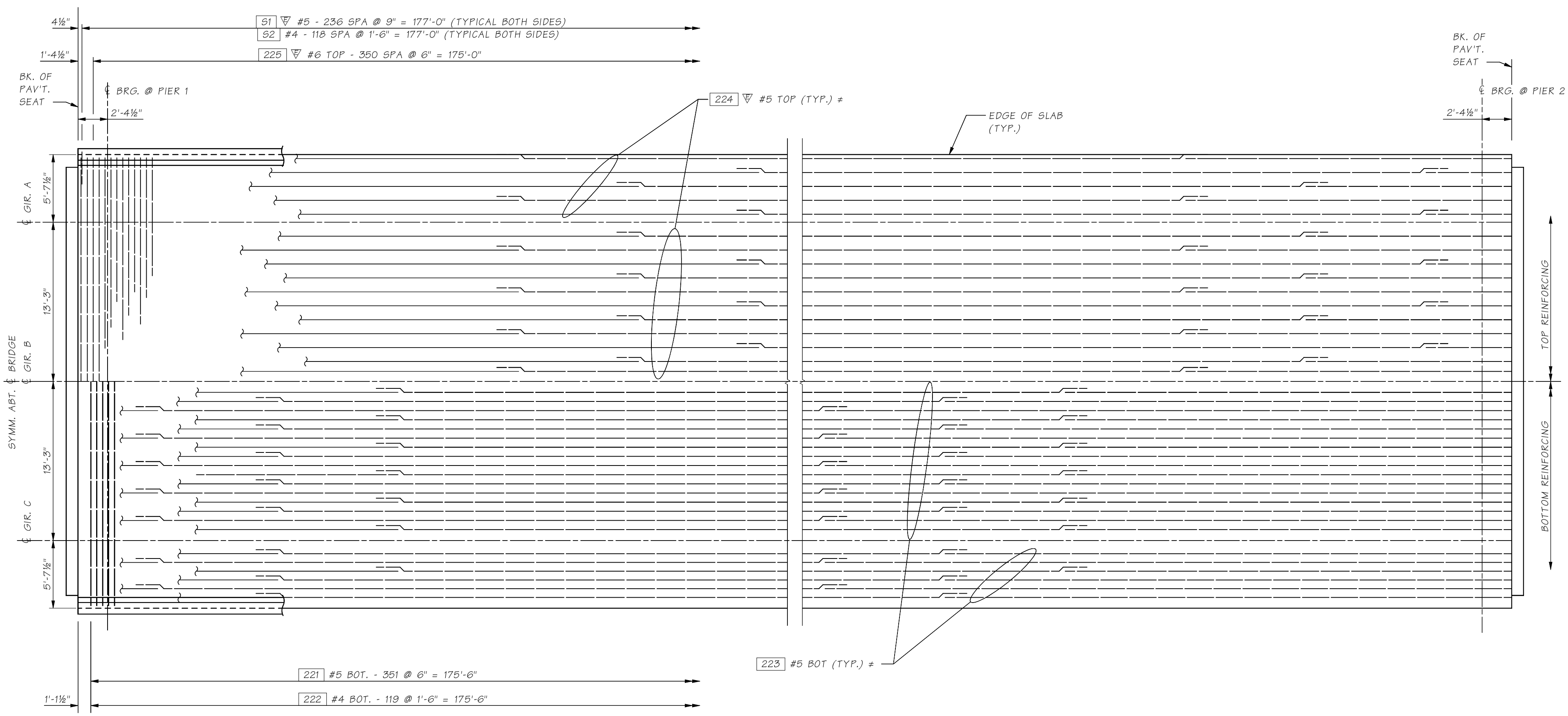

ROADWAY SLAB REINFORCING

≠ LOCATION OF REBAR SPLICES AT CONTRACTORS OPTION, EXCEPT ALL SPLICES SHALL BE ALTERNATED, SO NO MORE THAN 50% OF REBAR IS SPLICED AT THE SAME LOCATION, NORMAL TO ℄ OF BRIDGE.

NOTE:
2'-0" MINIMUM REBAR LAP SPLICE LENGTH FOR ALL LONGITUDINAL BARS.

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STEEL PLATE GIRDER
SLAB PLAN

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Chapter 7 Substructure Design — Contents

7.1 General Substructure Considerations

Note that in the following guidelines where reference is made to AASHTO LRFD, the item can be found in the current *AASHTO LRFD Bridge Design Specifications*. And for any reference to AASHTO Seismic, the item can be found in the current *AASHTO Guide Specifications for LRFD Seismic Bridge Design*.

7.1.1 Foundation Design Process

A flowchart is provided in Figure 7.1.1-1 which illustrates the overall design process utilized by the WSDOT Bridge and Structures Office to accomplish an LRFD foundation design. Note this process is also outlined in the WSDOT *Geotechnical Design Manual* M 46-03 (GDM) in Section 8.2. The Bridge and Structures Office (BO), the Geotechnical Branch (GB) and the Hydraulics Branch (HB) have been abbreviated. The steps in the flowchart are defined as follows:

A. **Scoping Level Design** – This phase of the design process involves the region requesting initial bridge options and costs for a future project. Depending on the complexity of the project, this phase could include a Type, Size and Location (TS&L) Report.

   This design step may result in informal communication between the BO and the GB and/or HB with the request for preliminary information and recommendations. The level of communication will depend on the available information provided by the region and the complexity of the project. The type of information that may be received from the GB and HB are as follows:

   - Anticipated soil site conditions.
   - Maximum embankment slopes.
   - Possible foundation types and geotechnical hazards such as liquefaction.
   - Scour potential for piers if a water crossing.
   - Potential for future migration of a stream or river crossing.

   In general, these recommendations rely on existing site data. Site borings may not be available and test holes are drilled later. The GB provides enough information to select potential foundation types for an initial scoping level or TS&L level plan and estimate.

B. **Develop Site Data and Preliminary Bridge Plan** – In the second phase, the BO obtains site data from the region (see Section 2.2) and develops the preliminary bridge plan. The preliminary pier locations determine soil boring locations at this time. The GB and/or the HB may require the following information to continue their preliminary design.

   - Structure type and magnitude of settlement the structure can tolerate (both total and differential).
   - At abutments – Approximate maximum top of foundation elevation.
   - At interior piers – The initial size, shape and number of columns and how they are configured with the foundation (e.g., whether a single foundation element supports each column, or one foundation element supports multiple columns)
   - At water crossings – Pier scour depth, if known, and any potential for migration of the water crossing in the future. Typically, the GB and the BO should coordinate pursuing this information with the HB.
   - Any known structural constraints that affect the foundation type, size, or location.
   - Any known constraints that affect the soil resistance (utilities, construction staging, excavation, shoring, and falsework).

C. **Preliminary Foundation Design** – The third phase is a request by the BO for a preliminary foundation memorandum. The GB memo will provide preliminary soil data required for structural analysis and modeling. This includes any subsurface conditions and the preliminary subsurface profile.

The concurrent geotechnical work at this stage includes:

- Completion of detailed boring logs and laboratory test data.
- Development of foundation type, soil capacity, and foundation depth.
- Development of static/seismic soil properties and ground acceleration.
- Recommendations for constructability issues.

The BO may also request the HB to provide preliminary scour design recommendations if the structure is located over a water crossing.

D. **Structural Analysis and Modeling** – In the fourth phase, the BO performs a structural analysis of the superstructure and substructure using a bridge model and preliminary soil parameters. Through this modeling, the designer determines loads and sizes for the foundation based on the controlling LRFD limit states. Structural and geotechnical design continues to investigate constructability and construction staging issues during this phase.

In order to produce a final geotechnical report, the BO provides the following structural feedback to the geotechnical engineer:

- Foundation loads for service, strength, and extreme limit states.
- Foundation size/diameter and depth required to meet structural design.
- Foundation details that could affect the geotechnical design of the foundations.
- Foundation layout plan.
- Assumed scour depths for each limit state (if applicable)

For water crossings, the BO also provides the information listed above to the hydraulics engineer to verify initial scour and hydraulics recommendations are still suitable for the site.

(See Chapter 2 for examples of pile design data sheets that shall be filled out and submitted to the geotechnical engineer at the early stage of design.)

E. **Final Foundation Design** – The last phase completes the geotechnical report and allows the final structural design to begin. The preliminary geotechnical assumptions are checked and recommendations are modified, if necessary. The final report is complete to a PS&E format since the project contract will contain referenced information in the geotechnical report, such as:

- All geotechnical data obtained at the site (boring logs, subsurface profiles, and laboratory test data).
- All final foundation recommendations.
- Final constructability and staging recommendations.

The designer reviews the final report for new information and confirms the preliminary assumptions. With the foundation design complete, the final bridge structural design and detailing process continues to prepare the bridge plans. Following final structural design, the structural designer shall follow up with the geotechnical designer to ensure that the design is within the limits of the geotechnical report.

7.2 Foundation Modeling for Seismic Loads

7.2.1 General

Bridge modeling for seismic events shall be in accordance with requirements of the AASHTO Seismic Section 5, "Analytical Models and Procedures." The following guidance is for elastic dynamic analysis. Refer to AASHTO Seismic 5.4 for other dynamic analysis procedures.

The following sections were originally developed for a force-based seismic design as required in previous versions of the AASHTO LRFD Specifications. Modifications have been made to the following sections to incorporate the provisions of the new AASHTO Seismic Specifications. It is anticipated that this section will continue to be revised as more experience is gained through the application of the AASHTO Seismic Specifications.

7.2.2 Substructure Elastic Dynamic Analysis Procedure

The following is a general description of the iterative process used in an elastic dynamic analysis. ***Note:*** An elastic dynamic analysis is needed to determine the displacement demand, $\Delta_{D.}$ The substructure elements are first designed using Strength, Service or Extreme II limit state load cases prior to performing the dynamic analysis.

1. Build a Finite Element Model (FEM) to determine initial structure response ($EQ+DL$). Assume that foundation springs are located at the bottom of the column.

   A good initial assumption for fixity conditions of deep foundations (shafts or piles) is to add 10′ to the column length in stiff soils and 15′ to the column in soft soils.

   Use multi-mode response spectrum analysis to generate initial displacements.

2. Determine foundation springs using results from the seismic analysis in the longitudinal and transverse directions. ***Note:*** The load combinations specified in AASHTO Seismic 4.4 shall NOT be used in this analysis.

3. For spread footing foundations, the FEM will include foundation springs calculated based on the footing size as calculated in Section 7.2.7 of this manual. No iteration is required unless the footing size changes. ***Note:*** For Site Classes A and B the AASHTO Seismic Specification allows spread footings to be modeled as rigid or fixed.

4. For deep foundation analysis, the FEM and the soil response program must agree or converge on soil/structure lateral response. In other words, the moment, shear, deflection, and rotation of the two programs should be within 10 percent. More iteration will provide convergence much less than 1 percent. The iteration process to converge is as follows:

   a. Apply the initial FEM loads (moment and shear) to a soil response program such as DFSAP. DFSAP is a program that models Short, Intermediate or Long shafts or piles using the Strain Wedge Theory. See discussion below for options and applicability of DFSAP and Lpile soil response programs.

   b. Calculate foundation spring values for the FEM. ***Note:*** The load combinations specified in AASHTO Seismic 4.4 shall not be used to determine foundation springs.

   c. Re-run the seismic analysis using the foundation springs calculated from the soil response program. The structural response will change. Check to insure the FEM results (M, V, Δ, θ, and spring values) in the transverse and longitudinal direction are within 10 percent of the previous run. This check verifies the linear spring, or soil response (calculated by the FEM) is close to the predicted nonlinear soil behavior (calculated by the soil response program). If the results of the FEM and the soil response program differ by more than 10 percent, recalculate springs and repeat steps (a) thru (c) until the two programs converge to within 10 percent.

Special note for single column/single shaft configuration: The seismic design philosophy requires a plastic hinge in the substructure elements above ground (preferably in the columns). Designers should note the magnitude of shear and moment at the top of the shaft, if the column “zero” moment is close to a shaft head foundation spring, the FEM and soil response program will not converge and plastic hinging might be below grade.

Throughout the iteration process it is important to note that any set of springs developed are only applicable to the loading that was used to develop them (due to the inelastic behavior of the soil in the foundation program). This can be a problem when the forces used to develop the springs are from a seismic analysis that combines modal forces using a method such as the Complete Quadratic Combination (CQC) or other method. The forces that result from this combination are typically dominated by a single mode (in each direction as shown by mass participation). This results in the development of springs and forces that are relatively accurate for that structure. If the force combination (CQC or otherwise) is not dominated by one mode shape (in the same direction), the springs and forces that are developed during the above iteration process may not be accurate.

Guidelines for the use of DFSAP and Lpile programs:

- The DFSAP Program may be used for pile and shaft foundations for static soil structural analysis cases.
- The DFSAP Program may be used for pile and shaft foundations for liquefied soil structural analysis case of a shaft or pile foundation with static soil properties reduced by the Geotechnical Branch to account for effects of liquefaction. The Liquefaction option in either Lpile or DFSAP programs shall not be used (the liquefaction option shall be disabled). The Liquefied Sand soil type shall not be used in Lpile
- The Lpile Program may be used for a pile supported foundation group. Pile or shaft foundation group effect efficiency shall be taken as recommended in the project geotechnical report.

7.2.3 Bridge Model Section Properties

In general, gross section properties may be assumed for all FEM members, except concrete columns.

A. **Cracked Properties for Columns –** Effective section properties shall be in accordance with the AASHTO Seismic Section 5.6.

B. **Shaft Properties –** The shaft concrete strength and construction methods lead to significant variation in shaft stiffness described as follows.

   For a stiff substructure response:

   1. Use 1.5 f'_c to calculate the modulus of elasticity. Since aged concrete will generally reach a compressive strength of at least 6 ksi when using a design strength of 4 ksi, the factor of 1.5 is a reasonable estimate for an increase in stiffness.
   2. Use I_g based on the maximum oversized shaft diameter allowed by Section 6-19 of the *Standard Specifications*.
   3. When permanent casing is specified, increase shaft I_g using the transformed area of a ¾″ thick casing. Since the contractor will determine the thickness of the casing, ¾″ is a conservative estimate for design.

   For a soft substructure response:

   1. Use 0.85 f'_c to calculate the modulus of elasticity. Since the quality of shaft concrete can be suspect when placed in water, the factor of 0.85 is an estimate for a decrease in stiffness.
   2. Use I_g based on the nominal shaft diameter. Alternatively, I_e may be used when it is reflective of the actual load effects in the shaft.

3. When permanent casing is specified, increase I using the transformed area of a ⅜″ thick casing.

   Since the contractor will determine the thickness of the casing, ⅜″ is a minimum estimated thickness for design.

C. **Cast-in-Place Pile Properties** – For a stiff substructure response:

1. Use 1.5 f'_c to calculate the modulus of elasticity. Since aged concrete will generally reach a compressive strength of at least 6 ksi when using a design strength of 4 ksi, the factor of 1.5 is a reasonable estimate for an increase in stiffness.
2. Use the pile I_g plus the transformed casing moment of inertia.
   Note: If DFSAP is used for analysis, the reinforcing and shell properties are input and the moment of inertia is computed internally.

$$I_{pile} = I_g + (n)(I_{shell}) + (n-1)(I_{reinf}) \qquad (7.2.3\text{-}1)$$

Where:
$n = E_s/E_c$

Use a steel casing thickness of ¼″ for piles less than 14″ in diameter, ⅜″ for piles 14″ to 18″ in diameter, and ½″ for larger piles.
Note: These casing thicknesses are to be used for analysis only, the contractor is responsible for selecting the casing thickness required to drive the piles.

For a soft substructure response:

1. Use 1.0 f'_c to calculate the modulus of elasticity.
2. Use pile I_g, neglecting casing properties.

7.2.4 Bridge Model Verification

As with any FEM, the designer should review the foundation behavior to ensure the foundation springs correctly imitate the known boundary conditions and soil properties. Watch out for mismatch of units.

All finite element models must have dead load static reactions verified and boundary conditions checked for errors. The static dead loads (*DL*) must be compared with hand calculations or another program's results. For example, span member end moment at the supports can be released at the piers to determine simple span reactions. Then hand calculated simple span *DL* or PGsuper *DL* and *LL* is used to verify the model.

Crossbeam behavior must be checked to ensure the superstructure *DL* is correctly distributing to substructure elements. A 3D bridge line model concentrates the superstructure mass and stresses to a point in the crossbeam. Generally, interior columns will have a much higher loading than the exterior columns. To improve the model, crossbeam I_g should be increased to provide the statically correct column *DL* reactions. This may require increasing I_g by about 1000 times. Many times this is not visible graphically and should be verified by checking numerical output. Note that most finite element programs have the capability of assigning constraints to the crossbeam and superstructure to eliminate the need for increasing the I_g of the crossbeam.

Seismic analysis may also be verified by hand calculations. Hand calculated fundamental mode shape reactions will be approximate; but will ensure design forces are of the same magnitude.

Designers should note that additional mass might have to be added to the bridge FEM for seismic analysis. For example, traffic barrier mass and crossbeam mass beyond the last column at piers may contribute significant weight to a two-lane or ramp structure.

7.2.5 Deep Foundation Modeling Methods

A designer must assume a foundation support condition that best represents the foundation behavior. Deep foundation elements attempt to imitate the non-linear lateral behavior of several soil layers interacting with the deep foundation. The bridge FEM then uses the stiffness of the element to predict the seismic structural response. Models using linear elements that are not based on non-linear soil-structure interaction are generally considered inaccurate for soil response/element stress and are not acceptable. There are three methods used to model deep foundations (FHWA Report No. 1P-87-6). Of these three methods the Bridge and Structures Office prefers Method II for the majority of bridges.

A. **Method I – Equivalent Cantilever Column –** This method assumes a point of fixity some depth below the bottom of the column to model the stiffness of the foundation element. This shall only be used for a preliminary model of the substructure response in SDC C and D.

B. **Method II – Equivalent Base Springs –** This method models deep foundations by using a {6x6} matrix. There are two techniques used to generate the stiffness coefficients for the foundation matrix. The equivalent stiffness coefficients assessed are valid only at the given level of loading. Any changes of the shaft-head loads or conditions will require a new run for the program to determine the new values of the equivalent stiffness coefficients. These equivalent stiffness coefficients account for the nonlinear response of shaft materials and soil resistance.

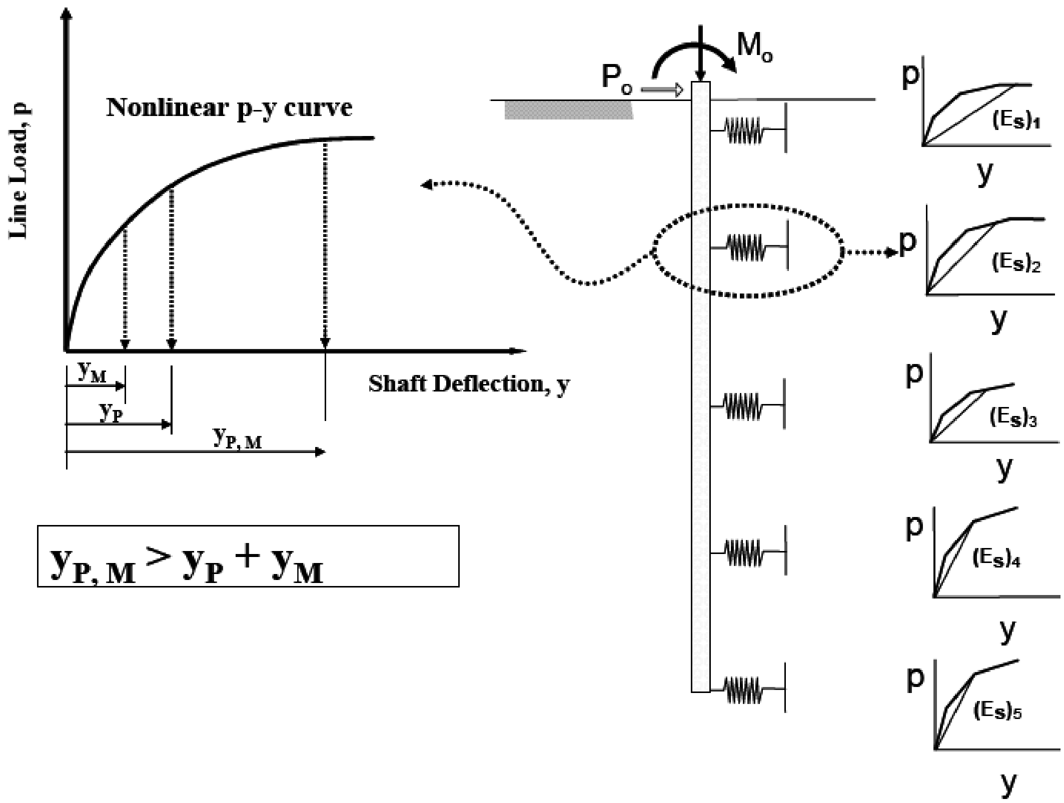
   Technique I – The matrix is generated, using superposition, to reproduce the non-linear behavior of the soil and foundation at the maximum loading. With Technique I, 10 terms are produced, 4 of these terms are "cross couples." Soil response programs, such as Lpile or DFSAP, analyze the non-linear soil response. The results are then used to determine the equivalent base springs. See Appendix 7-B1 for more information.

   Technique II – The equivalent stiffness matrix generated using this technique uses only the diagonal elements (no cross coupling stiffnesses). The DFSAP program shall be used to develop the equivalent stiffness matrix. This technique is recommended to construct the foundation stiffness matrix (equivalent base springs).

   In Technique II the "cross couple" effects are internally accounted for as each stiffness element and displacement is a function of the given Lateral load (*P*) and Moment (*M*). Technique II uses the total response ($\Delta_{t(P,M)}$ $\theta_{t(P,M)}$) to determine displacement and equivalent soil stiffness, maintaining a nonlinear analysis. Technique I requires superposition by adding the individual responses due to the lateral load and moment to determine displacement and soil stiffness. Using superposition to combine two nonlinear responses results in errors in displacement and stiffness for the total response as seen in the Figure 7.2.5-1. As illustrated, the total response due to lateral load (*P*) and moment (*M*) does not necessarily equal the sum of the individual responses. For more details on the equivalent stiffness matrix, see the DFSAP reference manual.

C. **Method III – Non-Linear Soil Springs –** This method attaches non-linear springs along the length of deep foundation members in a FEM model. See Appendix 7-B2 for more information. This method has the advantage of solving the superstructure and substructure seismic response simultaneously. The soil springs must be nonlinear PY curves and represent the soil/structure interaction. This cannot be done during response spectrum analysis with some FEM programs.

D. **Spring Location (Method II) –** The preferred location for a foundation spring is at the bottom of the column. This includes the column mass in the seismic analysis. For design, the column forces are provided by the FEM and the soil response program provides the foundation forces. Springs may be located at the top of the column. However, the seismic analysis will not include the mass of the columns. The advantage of this location is the soil/structure analysis includes both the column and foundation design forces.



Limitations on the Technique I (Superposition Technique)
Figure 7.2.5-1

Designers should be careful to match the geometry of the FEM and soil response program. If the location of the foundation springs (or node) in the FEM does not match the location input to the soil response program, the two programs will not converge correctly.

E. **Boundary Conditions (Method II) –** To calculate spring coefficients, the designer must first identify the predicted shape, or direction of loading, of the foundation member where the spring is located in the bridge model. This will determine if one or a combination of two boundary conditions apply for the transverse and longitudinal directions of a support.

A fixed head boundary condition occurs when the foundation element is in double curvature where translation without rotation is the dominant behavior. Stated in other terms, the shear causes deflection in the opposite direction of applied moment. This is a common assumption applied to both directions of a rectangular pile group in a pile supported footing.

A free head boundary condition is when the foundation element is in single curvature where translation and rotation is the dominant behavior. Stated in other terms, the shear causes deflection in the same direction as the applied moment. Most large diameter shaft designs will have a single curvature below ground line and require a free head assumption. The classic example of single curvature is a single column on a single shaft. In the transverse direction, this will act like a flagpole in the wind, or free head. What is not so obvious is the same shaft will also have single curvature in the longitudinal direction (below the ground line), even though the column exhibits some double curvature behavior. Likewise, in the transverse direction of multi-column piers, the columns will have double curvature (frame action). The shafts will generally have single curvature below grade and the free head boundary condition applies. The boundary condition for large shafts with springs placed at the ground line will be free head in most cases.

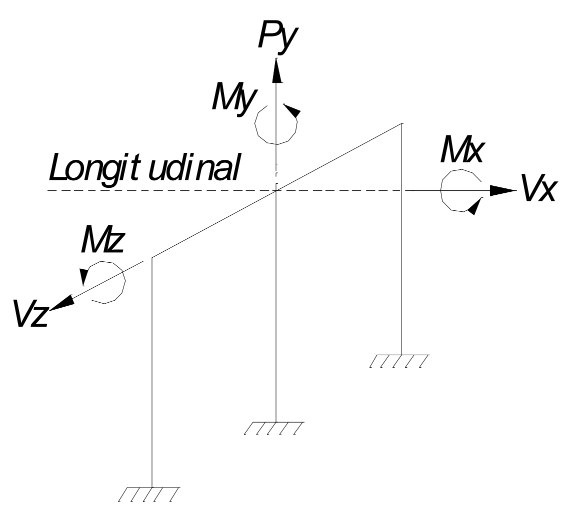
The key to determine the correct boundary condition is to resolve the correct sign of the moment and shear at the top of the shaft (or point of interest for the spring location). Since multi-mode results are always positive (CQC), this can be worked out by observing the seismic moment and shear diagrams for the structure. If the sign convention is still unclear, apply a unit load in a separate static FEM run to establish sign convention at the point of interest.

The correct boundary condition is critical to the seismic response analysis. For any type of soil and a given foundation loading, a fixed boundary condition will generally provide soil springs four to five times stiffer than a free head boundary condition.

F. **Spring Calculation (Method II) –** The first step to calculate a foundation spring is to determine the shear and moment in the structural member where the spring is to be applied in the FEM. Foundation spring coefficients should be based on the maximum shear and moment from the applied longitudinal OR transverse seismic loading. The combined load case (1.0*L* and 0.3*T*) shall be assumed for the design of structural members, and NOT applied to determine foundation response. For the simple case of a bridge with no skew, the longitudinal shear and moment are the result of the seismic longitudinal load, and the transverse components are ignored. This is somewhat unclear for highly skewed piers or curved structures with rotated springs, but the principle remains the same.

G. **Matrix Coordinate Systems (Method II) –** The Global coordinate systems used to demonstrate matrix theory are usually similar to the system defined for substructure loads in Section 7.1.3 of this manual, and is shown in Figure 7.2.5-2. This is also the default Global coordinate system of GTStrudl. This coordinate system applies to this Section to establish the sign convention for matrix terms. Note vertical axial load is labeled as *P*, and horizontal shear load is labeled as *V*.

Also note the default Global coordinate system in SAP 2000 uses *Z* as the vertical axis (gravity axis). When imputing spring values in SAP2000 the coefficients in the stiffness matrix will need to be adjusted accordingly. SAP2000 allows you to assign spring stiffness values to support joints. By default, only the diagonal terms of the stiffness matrix can be assigned, but when selecting the advanced option, terms to a symmetrical {6x6} matrix can be assigned.



Global Coordinate System
Figure 7.2.5-2

H. **Matrix Coefficient Definitions (Method II) –** The stiffness matrix containing the spring values and using the standard coordinate system is shown in Figure 7.2.5-3. (Note that cross-couple terms generated using Technique I are omitted). For a description of the matrix generated using Technique I see Appendix 7-B1. The coefficients in the stiffness matrix are generally referred to using several different terms. Coefficients, spring or spring value are equivalent terms. Lateral springs are springs that resist lateral forces. Vertical springs resist vertical forces.

$$\left\{\begin{array}{ccccccc} & Vx & Py & Vz & Mx & My & Mz \\ Vx & K11 & 0 & 0 & 0 & 0 & 0 \\ Py & 0 & K22 & 0 & 0 & 0 & 0 \\ Vz & 0 & 0 & K33 & 0 & 0 & 0 \\ Mx & 0 & 0 & 0 & K44 & 0 & 0 \\ My & 0 & 0 & 0 & 0 & K55 & 0 \\ Mz & 0 & 0 & 0 & 0 & 0 & K66 \end{array}\right\} \times \left\{\begin{array}{c} \text{Disp.} \\ \Delta x \\ \Delta y \\ \Delta z \\ \theta x \\ \theta y \\ \theta z \end{array}\right\} = \left\{\begin{array}{c} \text{Force} \\ Vx \\ Py \\ Vz \\ Mx \\ My \\ Mz \end{array}\right\}$$

Standard Global Matrix
Figure 7.2.5-3

Where the linear spring constants or K values are defined as follows, using the Global Coordinates:

K11 = Longitudinal Lateral Stiffness (kip/in)
K22 = Vertical or Axial Stiffness (kip/in)
K33 = Transverse Lateral Stiffness (kip/in)
K44 = Transverse Bending or Moment Stiffness (kip-in/rad)
K55 = Torsional Stiffness (kip-in/rad)
K66 = Longitudinal Bending or Moment Stiffness (kip-in/rad)

The linear lateral spring constants along the diagonal represent a point on a non-linear soil/structure response curve. The springs are only accurate for the applied loading and less accurate for other loadings. This is considered acceptable for Strength and Extreme Event design. For calculation of spring constants for Technique I see Appendix 7-B1. For calculation of spring constants for Technique II see the DFSAP reference manual.

I. **Group Effects** – When a foundation analysis uses Lpile or an analysis using PY relationships, group effects will require the geotechnical properties to be reduced before the spring values are calculated. The geotechnical report will provide transverse and longitudinal multipliers that are applied to the PY curves. This will reduce the pile resistance in a linear fashion. The reduction factors for lateral resistance due to the interaction of deep foundation members is provided in the WSDOT *Geotechnical Design Manual* M 46-03, Section 8.12.2.5.

   Group effect multipliers are not valid when the DFSAP program is used. Group effects are calculated internally using Strain Wedge Theory.

J. **Shaft Caps and Pile Footings –** Where pile supported footings or shaft caps are entirely below grade, their passive resistance should be utilized. In areas prone to scour or lateral spreading, their passive resistance should be neglected. DFSAP has the capability to account for passive resistance of footings and caps below ground.

7.2.6 Lateral Analysis of Piles and Shafts

7.2.6.1 Determination of Tip Elevations

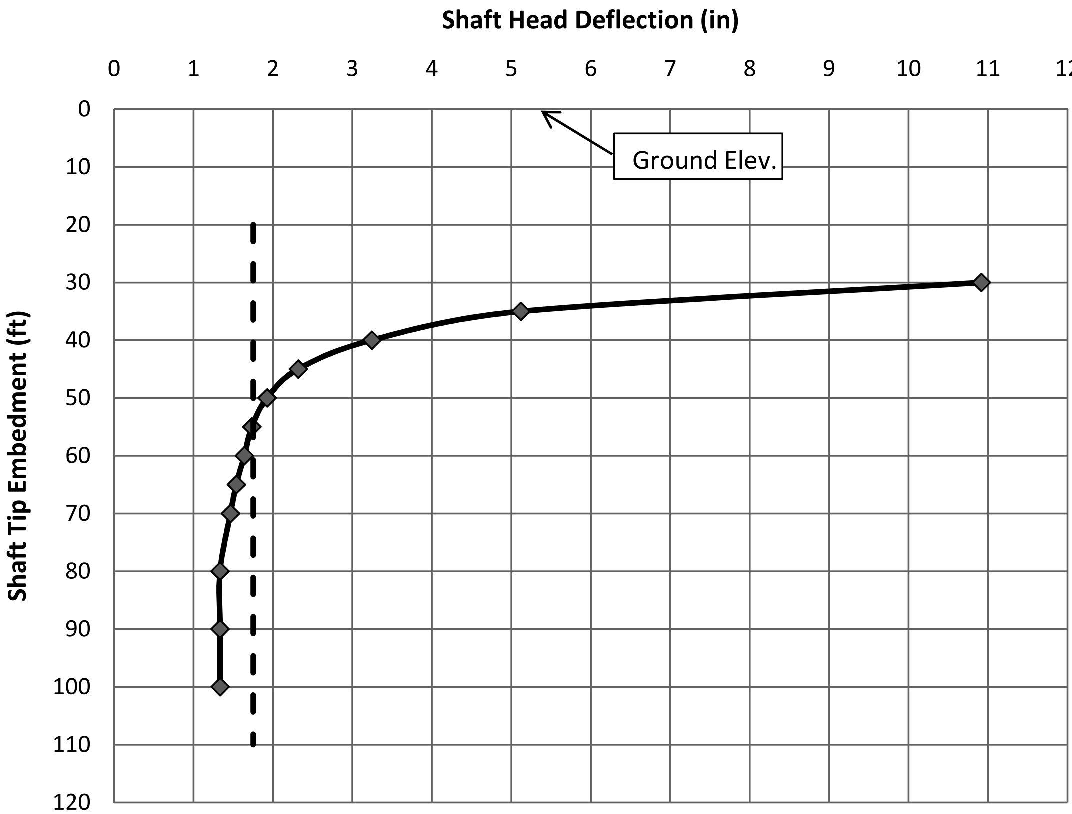
Lateral analysis of piles and shafts involves determination of a shaft or pile tip location sufficient to resist lateral loads in both orthogonal directions. In many cases, the shaft or pile tip depth required to resist lateral loads may be deeper than that required for bearing or uplift. However, a good starting point for a tip elevation is the depth required for bearing or uplift. Another good "rule-of-thumb" starting point for shaft tips is an embedment depth of 6 diameters (6*D*) to 8 diameters (8*D*). Refer also to the geotechnical report minimum tip elevations provided by the geotechnical engineer.

A parametric study or analysis should be performed to evaluate the sensitivity of the depth of the shaft or pile to the displacement of the structure (i.e. the displacement of the shaft or pile head) in order to determine the depth required for stable, proportionate lateral response of the structure. Determination of shaft or pile tip location requires engineering judgment, and consideration should be given to the type of soil, the confidence in the soil data (proximity of soil borings) and the potential variability in the soil profile. Arbitrarily deepening shaft or pile tips may be conservative but can also have significant impact on constructability and cost.

The following is a suggested approach for determining appropriate shaft or pile tip elevations that are located in soils. Other considerations will need to be considered when shaft or pile tips are located in rock, such as the strength of the rock. This approach is based on the displacement demand seismic design procedures specified in the AASHTO Seismic Specifications.

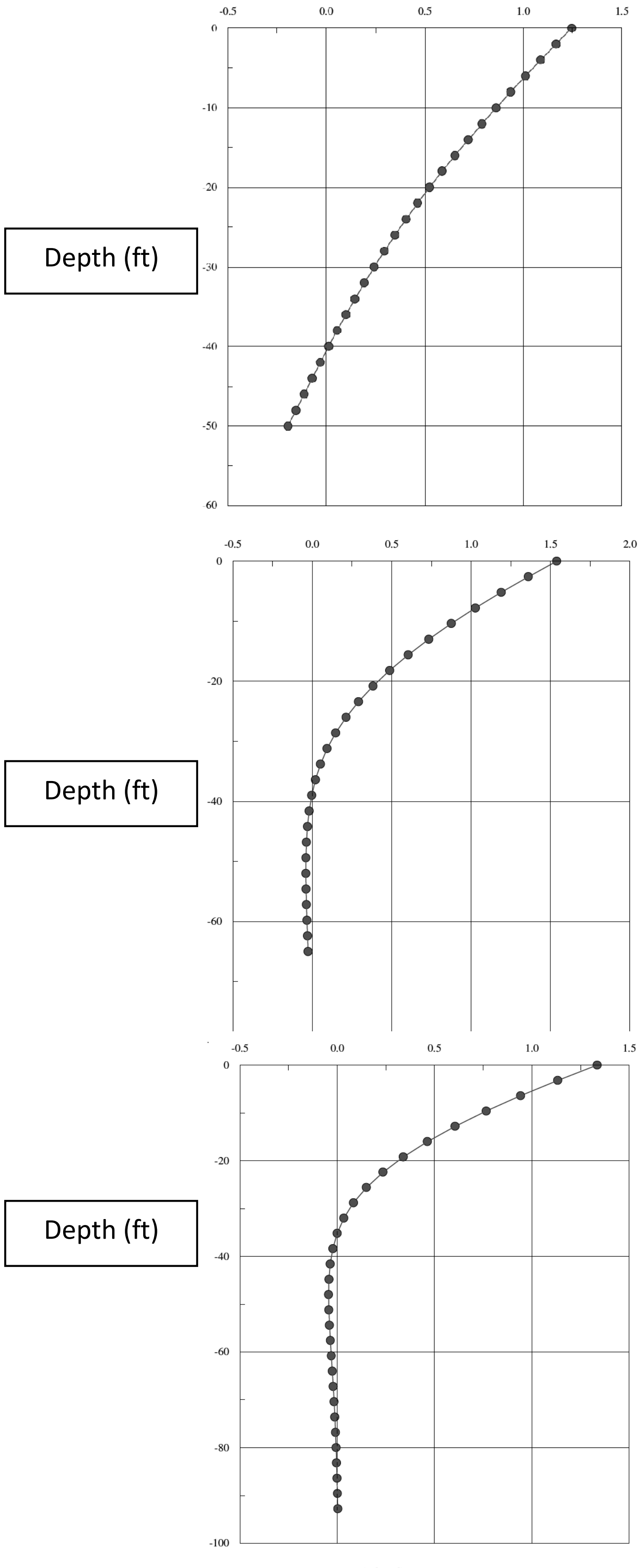
1. Size columns and determine column reinforcement requirements for Strength and Service load cases.
2. Determine the column plastic over-strength moment and shear at the base of the column using the axial dead load and expected column material properties. A program such as Xtract or SAP2000 may be used to help compute these capacities. The plastic moments and shears are good initial loads to apply to a soil response program (DFSAP or Lpile). In some cases, Strength or other Extreme event loads may be a more appropriate load to apply in the lateral analysis. For example, in eastern Washington seismic demands are relatively low and elastic seismic or Strength demands may control.
3. Perform lateral analysis using the appropriate soil data from the Geotechnical report for the given shaft or pile location. If final soil data is not yet available, consult with the Geotechnical engineer for preliminary values to use for the site.

   Note: Early in the lateral analysis it is wise to obtain moment and shear demands in the shaft or pile and check that reasonable reinforcing ratios can be used to resist the demands. If not, consider resizing the foundation elements and restart the lateral analysis.
4. Develop a plot of embedment depth of shaft or pile versus lateral deflection of the top of shaft or pile. The minimum depth, or starting point, shall be the depth required for bearing or uplift or as specified by the geotechnical report. An example plot of an 8′ diameter shaft is shown in Figure 7.2.6-1 and illustrates the sensitivity of the lateral deflections versus embedment depth. Notice that at tip depths of approximately 50′ (roughly 6*D*) the shaft head deflections begin to increase substantially with small reductions in embedment depth. The plot also clearly illustrates that tip embedment below 70′ has no impact on the shaft head lateral deflection.
5. From the plot of embedment depth versus lateral deflection, choose the appropriate tip elevation. In the example plot in Figure 7.2.6-1, the engineer should consider a tip elevation to the left of the dashed vertical line drawn in the Figure. The final tip elevation would depend on the confidence in the soil data and the tolerance of the structural design displacement. For example, if the site is prone to variability in soil layers, the engineer should consider deepening the tip; say 1 to 3 diameters, to ensure that embedment into the desired soil layer is achieved. The tip elevation would also depend on the acceptable lateral displacement of the structure. To assess the potential variability in the soil layers, the geotechnical engineer assigned to the project should be consulted.



Shaft Tip Elevation vs Shaft Head Deflection
Figure 7.2.6-1

6. With the selected tip elevation, review the deflected shape of the shaft or pile, which can be plotted in DFSAP or Lpile. Examples are shown in Figure 7.2.6-2. Depending on the size and stiffness of the shaft or pile and the soil properties, a variety of deflected shapes are possible, ranging from a rigid body (fence post) type shape to a long slender deflected shape with 2 or more inflection points. Review the tip deflections to ensure they are reasonable, particularly with rigid body type deflected shapes. Any of the shapes in the Figure may be acceptable, but again it will depend on the lateral deflection the structure can tolerate.



Various Shaft Deflected Shapes
Figure 7.2.6-2

The engineer will also need to consider whether liquefiable soils are present and/or if the shaft or pile is within a zone where significant scour can occur. In this case the analysis needs to be bracketed to envelope various scenarios. It is likely that a liquefiable or scour condition case may control deflection. In general, the WSDOT policy is to not include scour with Extreme Event I load combinations. In other words, full seismic demands or the plastic over-strength moment and shear, are generally not applied to the shaft or pile in a scoured condition. However, in some cases a portion of the anticipated scour will need to be included with the Extreme Event I load combination limit states. When scour is considered with the Extreme Event I limit state, the soil resistance up to a maximum of 25 percent of the scour depth for the design flood event (100 year) shall be deducted from the lateral analysis of the pile or shaft. In all cases where scour conditions are anticipated at the bridge site or specific pier locations, the geotechnical engineer and the Hydraulics Branch shall be consulted to help determine if scour conditions should be included with Extreme Event I limit states.

If liquefaction can occur, the bridge shall be analyzed using both the static and liquefied soil conditions. The analysis using the liquefied soils would typically yield the maximum bridge deflections and will likely control the required tip elevation, whereas the static soil conditions may control for strength design of the shaft or pile.

Lateral spreading is a special case of liquefied soils, in which lateral movement of the soil occurs adjacent to a shaft or pile located on or near a slope. Refer to the WSDOT *Geotechnical Design Manual* M 46-03 for discussion on lateral spreading. Lateral loads will need to be applied to the shaft or pile to account for lateral movement of the soil. There is much debate as to the timing of the lateral movement of the soil and whether horizontal loads from lateral spread should be combined with maximum seismic inertia loads from the structure. Most coupled analyses are 2*D*, and do not take credit for lateral flow around shafts, which can be quite conservative. The AASHTO Seismic Spec. permits these loads to be uncoupled; however, the geotechnical engineer shall be consulted for recommendations on the magnitude and combination of loads. See WSDOT *Geotechnical Design Manual* M 46-03 Sections 6.4.2.8 and 6.5.4.2 for additional guidance on combining loads when lateral spreading can occur.

7.2.6.2 Pile and Shaft Design for Lateral Loads

The previous section provides guidelines for establishing tip elevations for shafts and piles. Sensitivity analyses that incorporate both foundation and superstructure kinematics are often required to identify the soil conditions and loadings that will control the tip, especially if liquefied or scoured soil conditions are present. Several conditions will also need to be analyzed when designing the reinforcement for shafts and piles to ensure the controlling case is identified. All applicable strength, service and extreme load cases shall be applied to each condition. A list of these conditions includes, but is not limited to the following:

1. Static soil properties with both stiff and soft shaft or pile properties. Refer to Sections 7.2.3(B) and 7.2.3(C) for guidelines on computing stiff and soft shaft or pile properties.
2. Dynamic or degraded soil properties with both stiff and soft shaft or pile properties.
3. Liquefied soil properties with both stiff and soft shaft or pile properties.
    a. When lateral spreading is possible, an additional loading condition will need to be analyzed. The geotechnical engineer shall be consulted for guidance on the magnitude of seismic load to be applied in conjunction with lateral spreading loads. See WSDOT *Geotechnical Design Manual* M 46-03 Sections 6.4.2.8 and 6.5.4.2 for additional guidance on combining loads when lateral spreading can occur.
4. Scour condition with stiff and soft shaft or pile properties. The scour condition is typically not combined with Extreme Event I load combinations, however the designer shall consult with the Hydraulics Branch and geotechnical engineer for recommendations on load combinations. If scour is considered with the Extreme Event I limit state, the analysis should be conducted assuming that the soil in the upper 25 percent of the estimated scour depth for the design (100 year) scour event has been removed to determine the available soil resistance for the analysis of the pile or shaft.

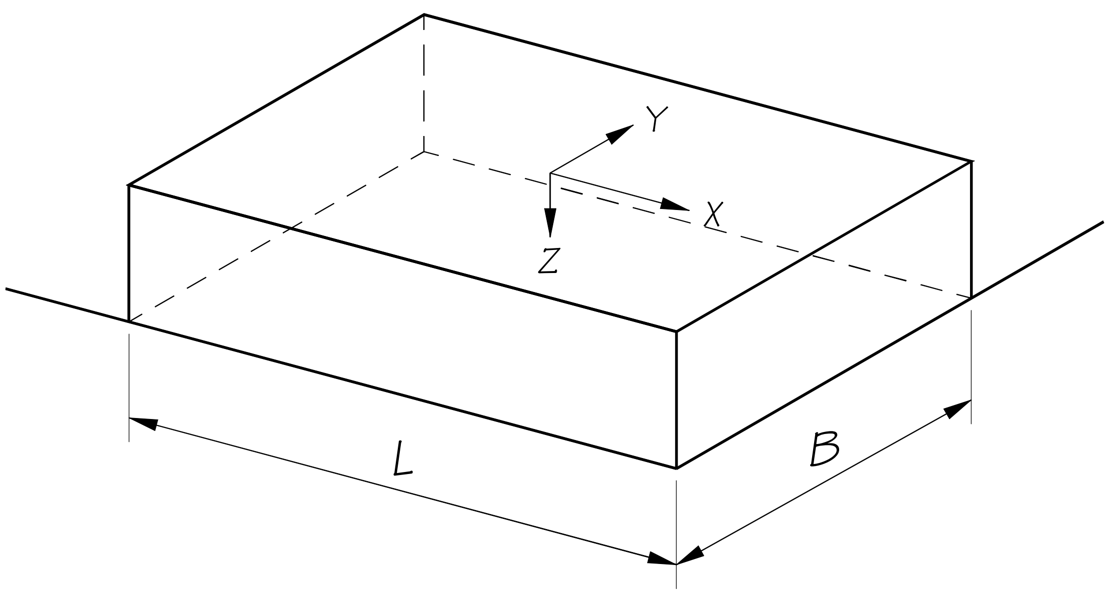
Note: Often, the highest acceleration the bridge sees is in the first cycles of the earthquake, and degradation and/or liquefaction of the soil tends to occur toward the middle or end of the earthquake. Therefore, early in the earthquake, loads are high, soil-structure stiffness is high, and deflections are low. Later in the earthquake, the soil-structure stiffness is lower and deflections higher. This phenomenon is normally addressed by bracketing the analyses as discussed above.

However, in some cases a site specific procedure may be required to develop a site specific design response spectrum. A site specific procedure may result in a reduced design response spectrum when compared to the general method specified in the AASHTO Seismic 3.4. Section 3.4 requires the use of spectral response parameters determined using USGA/AASHTO Seismic Hazard Maps. The AASHTO Seismic Spec. limits the reduced site specific response spectrum to two-thirds of what is produced using the general method. Refer to the WSDOT *Geotechnical Design Manual* M 46-03 Chapter 6 for further discussion and consult the geotechnical engineer for guidance.

Refer to Section 7.8 Drilled Shafts and Chapter 4 for additional guidance/requirements on design and detailing of drilled shafts and Section 7.9 Piles and Piling and Chapter 4 for additional guidance/requirements on design and detailing of piles.

7.2.7 Spread Footing Modeling

For a first trial footing configuration, Strength column moments or column plastic hinging moments may be applied to generate footing dimensions. Soil spring constants are developed using the footing plan area, thickness, embedment depth, Poisson's ratio ν, and shear modulus G. The Geotechnical Branch will provide the appropriate Poisson's ratio and shear modulus. Spring constants for shallow rectangular footings are obtained using the following equations developed for rectangular footings. This method for calculating footing springs is referenced in ASCE 41-06, Section 4.4.2.1.2. (***Note:*** ASCE 41-06 was developed from FEMA 356.)



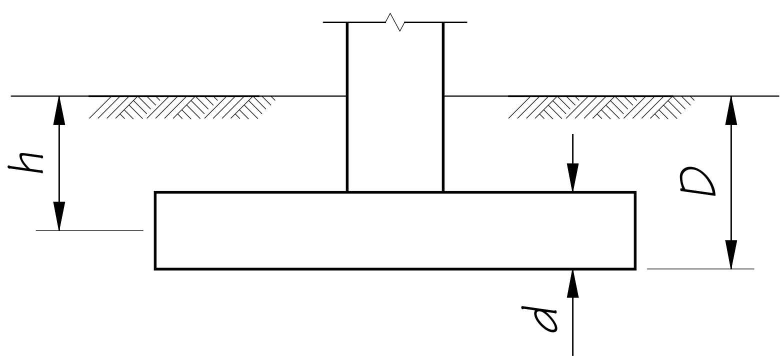
Orient axes such that $L > B$.
If $L = B$ use x-axis equations for both x-axis and y-axis.
Figure 7.2.7-1

Where:

K = βK_{sur}
K = Translation or rotational spring
K_{sur} = Stiffness of foundation at surface, see Table 7.2.7-1
β = Correction factor for embedment, see Table 7.2.7-2

Degree of Freedom	K_{sur}
Translation along x-axis	$\frac{GB}{2-v}\left[3.4\left(\frac{L}{B}\right)^{0.65}+1.2\right]$
Translation along y-axis	$\frac{GB}{2-v}\left[3.4\left(\frac{L}{B}\right)^{0.65}+0.4\frac{L}{B}+0.8\right]$
Translation along z-axis	$\frac{GB}{1-v}\left[1.55\left(\frac{L}{B}\right)^{0.75}+0.8\right]$
Rocking about x-axis	$\frac{GB^3}{1-v}\left[0.4\left(\frac{L}{B}\right)+0.1\right]$
Rocking about y-axis	$\frac{GB^3}{1-v}\left[0.47\left(\frac{L}{B}\right)^{2.4}+0.034\right]$
Torsion about z-axis	$GB^3\left[0.53\left(\frac{L}{B}\right)^{2.45}+0.51\right]$

Stiffness of Foundation at Surface
Table 7.2.7-1



Where:

d = Height of effective sidewall contact (may be less than total foundation height if the foundation is exposed).

h = Depth to centroid of effective sidewall contact.

Figure 7.2.7-2

Degree of Freedom	β
Translation along x-axis	$\left(1+0.21\sqrt{\frac{D}{B}}\right)\left[1+1.6\left(\frac{hd(B+L)}{BL^2}\right)^{0.4}\right]$
Translation along y-axis	$\left(1+0.21\sqrt{\frac{D}{L}}\right)\left[1+1.6\left(\frac{hd(B+L)}{LB^2}\right)^{0.4}\right]$
Translation along z-axis	$\left[1+\frac{1}{21}\frac{D}{B}\left(2+2.6\frac{B}{L}\right)\right]\cdot\left[1+0.32\left(\frac{d(B+L)}{BL}\right)^{\frac{2}{3}}\right]$
Rocking about x-axis	$1+2.5\frac{d}{B}\left[1+\frac{2d}{B}\left(\frac{d}{D}\right)^{-0.2}\sqrt{\frac{B}{L}}\right]$
Rocking about y-axis	$1+1.4\left(\frac{d}{L}\right)^{0.6}\left[1.5+3.7\left(\frac{d}{L}\right)^{1.9}\left(\frac{d}{D}\right)^{-0.6}\right]$
Torsion about z-axis	$1+2.6\left(1+\frac{B}{L}\right)\left(\frac{d}{B}\right)^{0.9}$

Correction Factor for Embedment

Table 7.2.7-2

7.3 Column Design

7.3.1 Preliminary Plan Stage

The preliminary plan stage determines the initial column size, column spacing, and bridge span length based on a preliminary analysis. Columns are spaced to give maximum structural benefit except where aesthetic considerations dictate otherwise. Piers normally are spaced to meet the geometric and aesthetic requirements of the site and to give maximum economy for the total structure. Good preliminary engineering judgment results in maximum economy for the total structure.

The designer may make changes after the preliminary plan stage. The design unit supervisor will need to review all changes, and if the changes are more than minor dimension adjustments, the Bridge Projects Engineer and the State Bridge and Structures Architect will also need to be involved in the review.

Tall piers spaced farther apart aesthetically justify longer spans. Difficult and expensive foundation conditions will also justify longer spans. Span lengths may change in the design stage if substantial structural improvement and/or cost savings can be realized. The designer should discuss the possibilities of span lengths or skew with the supervisor as soon as possible. Changes in pier spacing at this stage can have significant negative impacts to the geotechnical investigation.

Column spacing should minimize column dead load moments. Multiple columns are better suited for handling lateral loads due to wind and/or earthquake. The designer may alter column size or spacing for structural reasons or change from a single-column pier to a multicolumn pier.

7.3.2 General Column Criteria

Columns should be designed so that construction is as simple and repetitious as possible. The diameter of circular columns should be a multiple of one foot, however increments of 6 inches may be appropriate in some cases. Rectangular sections shall have lengths and widths that are multiples of 3 inches. Long rectangular columns are often tapered to reduce the amount of column reinforcement required for strength. Tapers should be linear for ease of construction.

Understanding the effects on long columns due to applied loads is fundamental in their design. Loads applied to the columns consist of reactions from loads applied to the superstructure and loads applied directly to the columns. For long columns, it may be advantageous to reduce the amount of reinforcement as the applied loads decrease along the column. In these cases, load combinations need to be generated at the locations where the reinforcement is reduced.

A. **Construction Joints –** Bridge plans shall show column construction joints at the top of footing or pedestal and at the bottom of crossbeam. Optional construction joints with roughened surfaces should be provided at approximately 30-foot vertical spacing.

B. **Modes of Failure –** A column subject to axial load and moment can fail in several modes. A “short” column can fail due to crushing of the concrete or to failure of the tensile reinforcement. A “long” column can fail due to elastic buckling even though, in the initial stages, stresses are well within the normal allowable range. Long column failure is normally a combination of stability and strength failure that might occur in the following sequence:

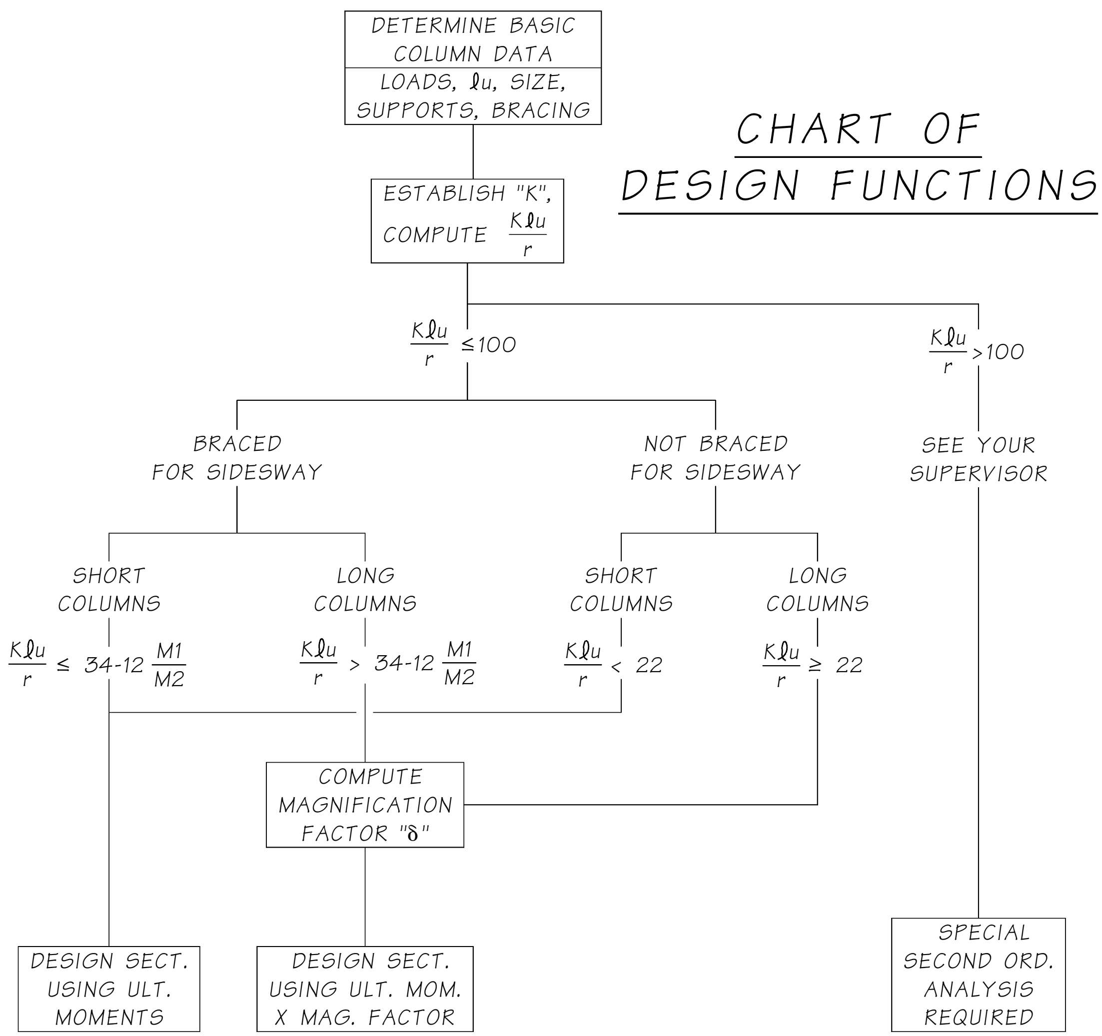
    1. Axial load is applied to the column.
    2. Bending moments are applied to the column, causing an eccentric deflection.
    3. Axial loads act eccentrically to the new column center line producing P-Δ moments which add directly to the applied moments.
    4. P-Δ moments increase the deflection of the column and lead to more eccentricity and moments.

5. The P-Δ analysis must prove the column loading and deflection converges to a state where column stresses are acceptable. Otherwise, the column is not stable and failure can be catastrophic. Refer to AASHTO Seismic 4.11.5 for discussion on P-Δ effects and when they shall be considered in the design. In most cases P-Δ effects can be neglected.

Unlike building columns, bridge columns are required to resist lateral loads through bending and shear. As a result, these columns may be required to resist relatively large applied moments while carrying nominal axial loads. In addition, columns are often shaped for appearance. This results in complicating the analysis problem with non-prismatic sections.

7.3.3 Column Design Flowchart – Evaluation of Slenderness Effects

Figure 7.3.3-1 illustrates the basic steps in the column design process for evaluating the effects of slenderness on columns and methods for computing magnification of moments on columns.



Column Design Flowchart for Non-Seismic Design
Figure 7.3.3-1

7.3.4 Slenderness Effects

This section supplements and clarifies AASHTO LRFD specifications. The goal of a slenderness analysis is to estimate the additional bending moments in the columns that are developed due to axial loads acting upon a deflected structure. Two primary analysis methods exist: the moment magnifier method and the second-order analysis. The designer must decide which method to use based upon the slenderness ratio (kL_u/r) of the column(s).

Method 1: Allowed if $kL_u/r < 100$. Section 7.3.5 of this manual discusses the approximate moment magnifier method that is generally more conservative and easier to apply.

Method 2: Recommended by AASHTO LRFD for all situations and is mandatory for $kL_u/r > 100$. Section 7.3.6 of this manual discusses a second-order structural analysis that accounts directly for the axial forces and can lead to significant economy in the final structure.

In general, tall thin columns and piles above ground (pile bents) are considered unbraced and larger short columns are considered braced.

A. **Braced or Unbraced Columns –** In a member with loads applied at the joints, any significant deflection "sideways" indicated the member is unbraced. The usual practice is to consider the pier columns as unbraced in the transverse direction. The superstructure engages girder stops at the abutment and resists lateral sidesway due to axial loads. However, pier lateral deflections are significant and are considered unbraced. Short spanned bridges may be an exception.

   Most bridge designs provide longitudinal expansion bearings at the end piers. Intermediate columns are considered unbraced because they must resist the longitudinal loading. The only time a column is braced in the longitudinal direction is when a framed bracing member does not let the column displace more than $L/1500$. L is the total column length. In this case, the bracing member must be designed to take all of the horizontal forces.

7.3.5 Moment Magnification Method

The moment magnification method is described in AASHTO LRFD 4.5.3.2.2. The following information is required.

- Column geometry and properties: E, I, L_u, and k.
- All Strength loads obtained from conventional elastic analyses using appropriate stiffness and fixity assumptions and column under strength factor (ϕ).

Computations of effective length factors, k, and buckling loads, Pc, are not required for a second-order analysis, though they may be helpful in establishing the need for such an analysis. In general, if magnification factors computed using the AASHTO LRFD specifications are found to exceed about 1.4, then a second-order analysis may yield substantial benefits.

7.3.6 Second-Order Analysis

A second-order analysis that includes the influence of axial loads on the deflected structure is required under certain circumstances, and may be advisable in others. It can lead to substantial economy in the final design of many structures. The designer should discuss the situation with the supervisor before proceeding with the analysis. The ACI Building Code (ACI 318-08), should be consulted when carrying out a second-order analysis.

For columns framed together, the entire frame should be analyzed as a unit. Analyzing individual columns result in overly conservative designs for some columns and non-conservative results for others. This is a result of redistribution of the lateral loads in response to the reduced stiffness of the compression members. For example, in a bridge with long, flexible columns and with short, stiff columns both integrally connected to a continuous superstructure, the stiff columns will tend to take a larger proportion of the lateral loading as additional sidesway under axial loads occurs.

A. **Design Methods for a Second-Order Analysis –** The preferred method for performing a second-order analysis of an entire frame or isolated single columns is to use a nonlinear finite element program, such as GTSTRUDL, with appropriate stiffness and restraint assumptions. The factored group loads are applied to the frame, including the self-weight of the columns. The model is then analyzed using the nonlinear option available in GTSTRUDL. The final design moments are obtained directly from the analysis.

PΔ moments are added to the applied moments using an iterative process until stability is reached. The deflections should converge within 5 percent of the total deflection. Analysis must include the effect of the column weight; therefore, the axial dead load must be adjusted as follows:

$$P_u = P_u + {}^{1}/_{3}\,(factored\ column\ weight) \qquad (7.3.6\text{-}1)$$

B. **Applying Factored Loads –** For a second-order analysis, loads are applied to the structure and the analysis results in member forces and deflections. It must be recognized that a second-order analysis is non-linear and the commonly assumed principle of superposition may not be applicable. The loads applied to the structure should be the entire set of factored loads for the load group under consideration. The analysis must be repeated for each group load of interest. The problem is complicated by the fact that it is often difficult to predict in advance which load groups will govern.

For certain loadings, column moments are sensitive to the stiffness assumptions used in the analysis. For example, loads developed as a result of thermal deformations within a structure may change significantly with changes in column, beam, and foundation stiffness. Accordingly, upper and lower bounds on the stiffness should be determined and the analysis repeated using both sets to verify the governing load has been identified.

C. **Member Properties –** As with a conventional linear elastic frame analysis, various assumptions and simplifications must be made concerning member stiffness, connectivity, and foundation restraint. Care must be taken to use conservative values for the slenderness analysis. Reinforcement, cracking, load duration, and their variation along the members are difficult to model while foundation restraint will be modeled using soil springs.

7.3.7 Shear Design

Shear design should follow the “Simplified Procedure for Nonprestressed Sections” in AASHTO LRFD 5.8.3.4.1.

7.3.8 Column Silos

Column silos are an acceptable technique to satisfy the balanced stiffness and frame geometry requirements of Section 4.2.7 and the AASHTO Seismic Specifications. Due to the construction and inspection complications of column silos, designers are encouraged to meet balanced stiffness and frame geometry requirements by the other methods suggested in Section 4.1.4 of the AASHTO Seismic Specifications prior to use of column silos.

A. **General Design and Detailing Requirements**

1. Column silo plans, specifications, and estimates shall be included in the Contract Documents.
2. Column silos shall be designed to resist lateral earth and hydrostatic pressure, including live load surcharge if applicable, for a 75-year minimum service life.
3. Column silos are not permitted for in-water locations such as in rivers and lakes.
4. Clearance between the column and the column silo shall be adequate for column lateral displacement demands, construction and post-earthquake inspection, but shall not be less than 1′-6″.

5. A 6″ minimum clearance shall be provided from the top of column silo to ground level.
6. Maximum depth of column silos shall not exceed 15 feet.
7. Column silos shall be watertight when located below the highest expected groundwater elevation. Silo covers need not be liquidtight.
8. Column silos shall be positively attached to the foundation element.

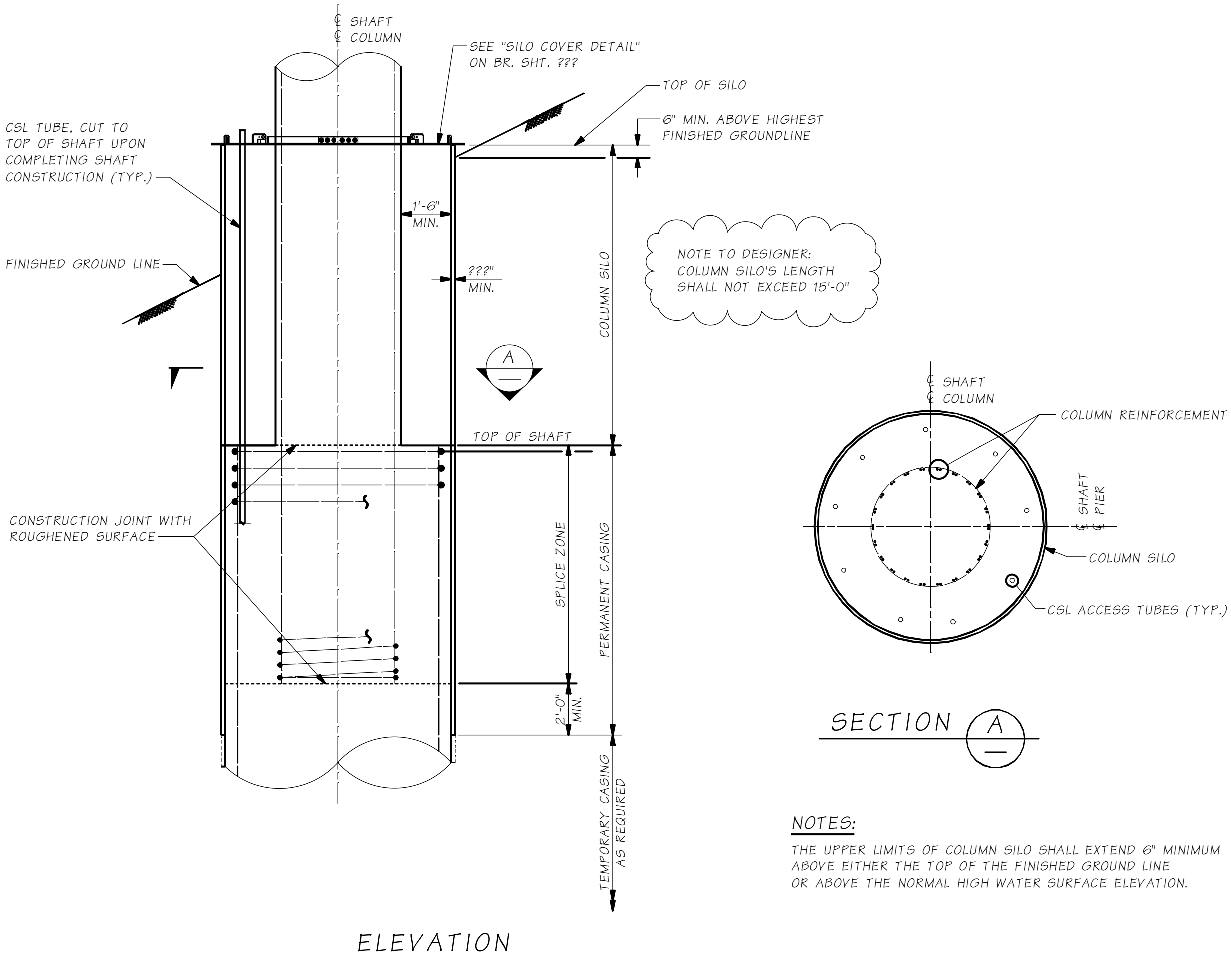
B. **Column Silos Formed From Extending Shaft Casing –** Designers shall determine a minimum steel casing thickness sufficient to resist lateral loads and shall provide it in the Contract Documents. This thickness shall include a sacrificial steel area as recommended in AASHTO LRFD Specification Section C10.7.5 for corrosion resistance. The actual steel casing size and materials shall be determined by the Contractor as delineated in WSDOT *Standard Specifications* Section 6-19 and 9-36. Appropriate detailing, as shown in Figure 7.3.8-1, shall be provided.

C. **Column Silos Formed by Other Methods –** Column silos formed by other methods, such as corrugated metal pipes, may be considered if the general requirements above are satisfied.

D. **Column Silo Covers and Access Hatches –** A column silo cover, including access hatches, shall be specified in the Contract Plans as shown in Appendix 7.3-A1-1. Column silo covers and access hatches shall be painted in accordance with WSDOT *Standard Specifications* Section 6-07.3(9).

Column silo covers shall be protected from vehicular loading. Column silo covers shall be capable of sliding on top of the column silo and shall not restrain column lateral displacement demands. Obstructions to the column silo cover sliding such as barriers or inclined slopes are not allowed adjacent to the column silo where they may interfere with column lateral displacement demands. Column silo covers and tops of column silos shall be level.

Sufficient access hatches shall be provided in the column silo cover so that all surfaces of the column and the column silo can be inspected. Access hatches shall include a minimum clear opening of 1′-0″ x 1′-0″ to accommodate the lowering of pumping and inspection equipment into the column silo. Access hatches for direct personnel access shall have a minimum clear opening of 2′-0″ square. Column silo covers shall be designed to be removable by maintenance and inspection personnel. Public access into the column silo shall be prevented.



Column Silo on Shaft Foundation
Figure 7.3.8-1

7.4 Column Reinforcement

7.4.1 Reinforcing Bar Material

Steel reinforcing bars for all bridge substructure elements (precast and cast-in-place) shall be in accordance with Section 5.1.2.

7.4.2 Longitudinal Reinforcement Ratio

The reinforcement ratio is the steel area divided by the gross area of the section (A_s/A_g). The maximum reinforcement ratio shall be 0.04 in SDCs A, B, C and D. The minimum reinforcement ratio shall be 0.007 for SDC A, B, and C and shall be 0.01 for SDC D.

For bridges in SDC A, if oversized columns are used for architectural reasons, the minimum reinforcement ratio of the gross section may be reduced to 0.005, provided all loads can be carried on a reduced section with similar shape and the reinforcement ratio of the reduced section is equal to or greater than 0.01 and $0.133f'_c/f_y$. The column dimensions are to be reduced by the same ratio to obtain the similar shape.

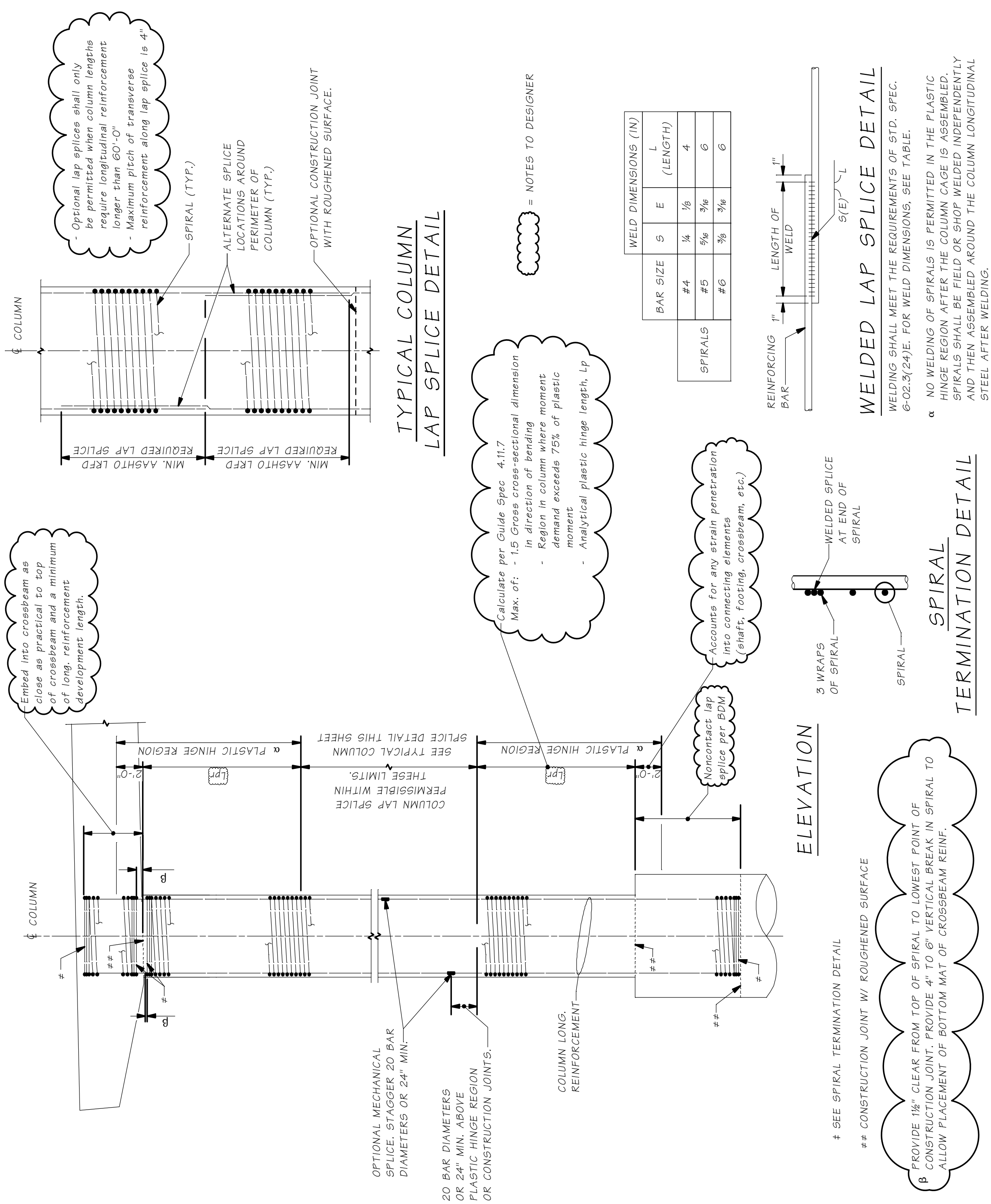
7.4.3 Longitudinal Splices

In general, column longitudinal reinforcement shall not be spliced at points of maximum moment, plastic hinge regions, or in columns less than 30 feet long between the top of footing, or shaft, and the bottom of crossbeam. The bridge plans must show lap splice location, length, and optional mechanical splice locations. *Standard Specifications* Section 6-02.3(24)F covers requirements for mechanical splices.

Column longitudinal reinforcement splices shall be staggered. For intermediate column construction joints, the shortest staggered lap bar shall project above the joint 60 bar diameters or minimum of 24″. For welded or mechanical splices, the bar shall project above the joint 20 bar diameters. Figure 7.4.3-1 shows the standard practice for staggered splice locations.

For bridges in SDCs A through D, splices of #11 and smaller bars may use lap slices. When space is limited, #11 and smaller bars can use welded splices, an approved mechanical butt splice, or the top bar can be bent inward (deformed by double bending) to lie inside and parallel to the bars below. When the bar size exceeds #11, a welded splice or an approved mechanical butt splice is required. The smaller bars in the splice determine the type of splice required.

Mechanical splices shall meet requirements of the *Standard Specifications* Section 6-02.3(24)F and “Ultimate Splice” strain requirements provided in AASHTO LRFD Table C5.10.11.41f-1. See the current Bridge Special Provision for “Ultimate Splice Couplers.”



Column Splice and Plastic Hinge Region Details
Figure 7.4.3-1

7.4.4 Longitudinal Development

A. **Crossbeams –** Development of longitudinal reinforcement shall be in accordance with AASHTO Seismic 8.8.4.

A detail showing horizontal lower crossbeam reinforcement and vertical column reinforcement is preferred but not required.

B. **Footings –** Longitudinal reinforcement at the bottom of a column should extend into the footing and rest on the bottom mat of footing reinforcement with standard 90° hooks. In addition, development of longitudinal reinforcement shall be in accordance with AASHTO Seismic 8.8.4 and AASHTO LRFD 5.11.2.1.

C. **Drilled Shafts –** Column longitudinal reinforcement in drilled shafts is typically straight. Embedment shall be a minimum length equal to $l_{ns} = l_s + s$ (per TRAC Report WA-RD 417.1 titled "Noncontact Lap Splices in Bridge Column-Shaft Connections").

Where:

l_s = the larger of $1.7 \times l_{ac}$ or $1.7 \times l_d$ (for Class C lap splice) where:

l_{ac} = development length from the Seismic Guide Spec. 8.8.4 for the column longitudinal reinforcement.

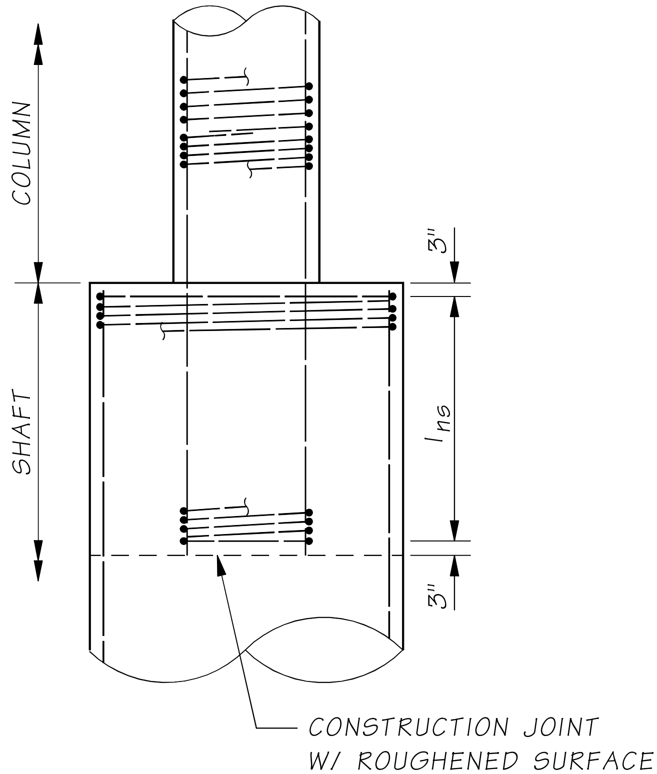
l_d = tension development length from AASHTO LRFD Section 5.11.2.1 for the column longitudinal reinforcement.

s = distance between the shaft and column longitudinal reinforcement

The requirements of the AASHTO Seismic 8.8.10 for development length of column bars extended into oversized pile shafts for SDC C and D shall not be used.

All applicable modification factors for development length, except one, in AASHTO LRFD 5.11.2 may be used when calculating l_d. The modification factor in 5.11.2.1.3 that allows l_d to be decreased by the ratio of (A_s required)/(A_s provided), shall not be used. Using this modification factor would imply that the reinforcement does not need to yield to carry the ultimate design load. This may be true in other areas. However, our shaft/column connections are designed to form a plastic hinge, and therefore the reinforcement shall have adequate development length to allow the bars to yield.

See Figure 7.4.4-1 for an example of longitudinal development into drilled shafts.



Longitudinal Development Into Drilled Shafts
Figure 7.4.4-1

7.4.5 Transverse Reinforcement

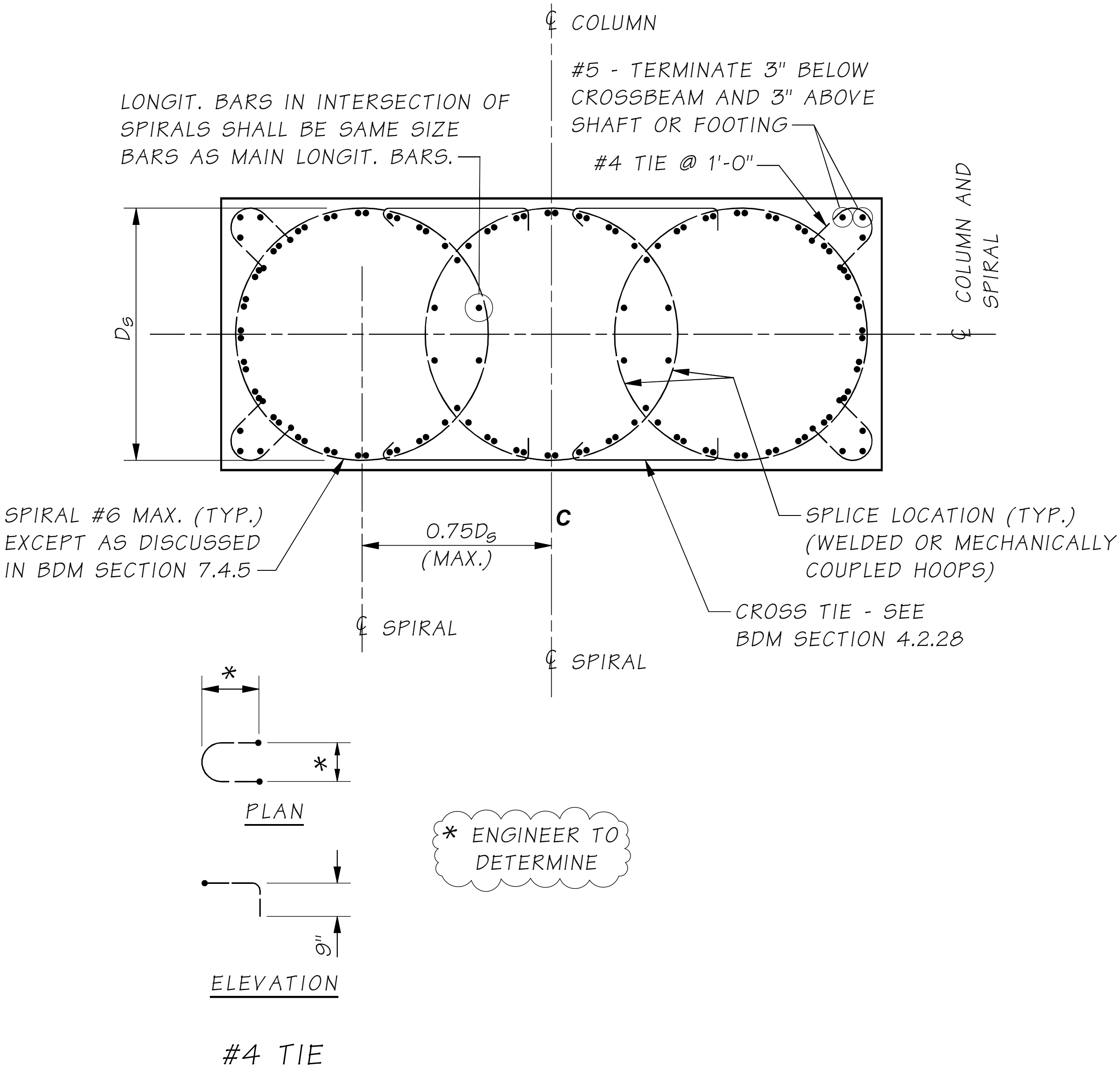
A. **General –** All transverse reinforcement in columns shall be deformed. Although allowed in the AASHTO LRFD Specification, plain bars or plain wire may not be used for transverse reinforcement.

Columns in SDC A may use spirals, circular hoops, or rectangular hoops and crossties.

Columns in SDC B, C, and D shall use spiral or circular hoop transverse confinement reinforcement where possible, although rectangular hoops with ties may be used when large, odd shaped column sections are required.

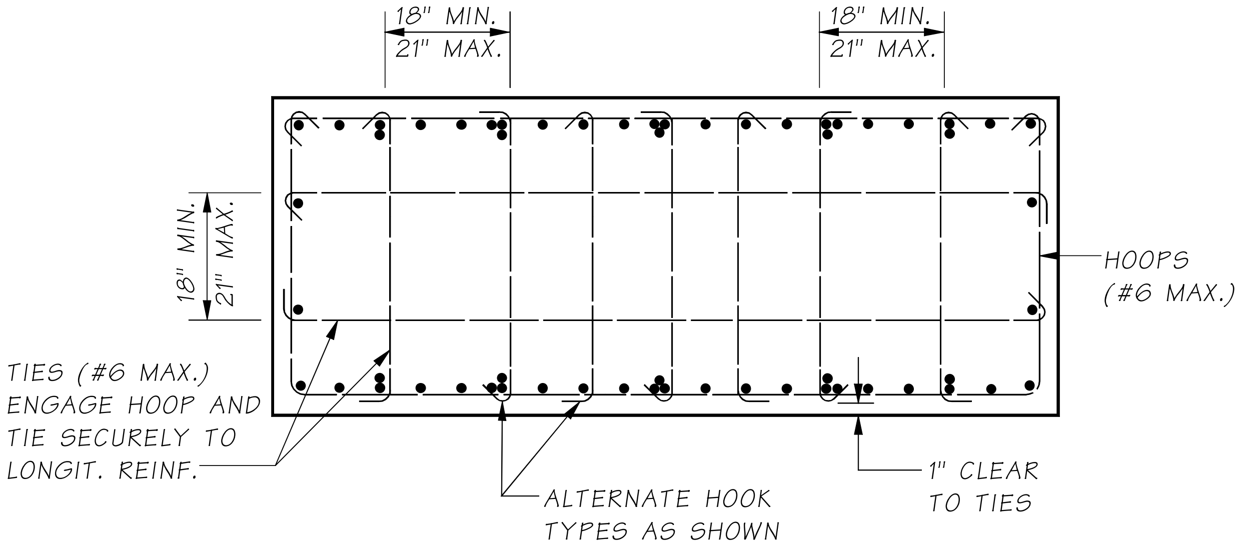
Spirals are the preferred confinement reinforcement and shall be used whenever a #6 spiral is sufficient to satisfy demands. When demands require reinforcement bars greater than #6, circular hoops of #7 through # 9 may be used. Bundled spirals shall not be used for columns or shafts. Also, mixing of spirals and hoops within the same column is not permitted by the AASHTO Seismic Specification. Figure 7.4.5-1 and 7.4.5-2 show transverse reinforcement details for rectangular columns in high and low seismic zones, respectively.

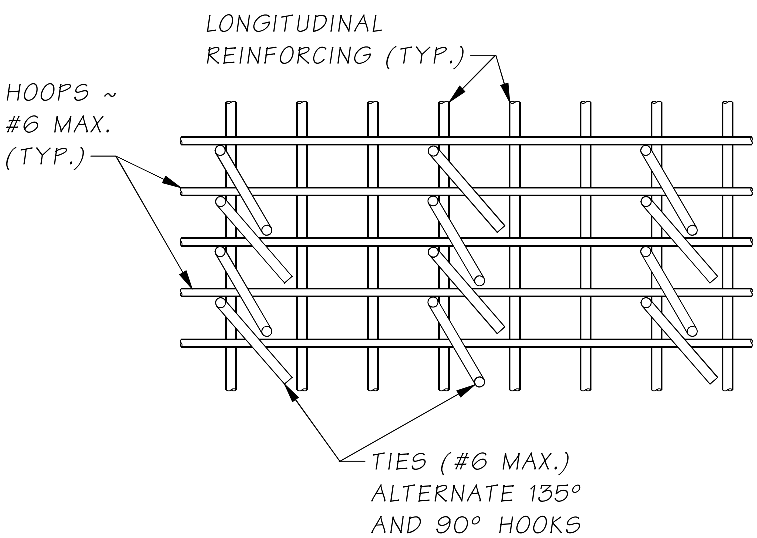
When rectangular hoops with ties are used, consideration shall be given to column constructability. Such considerations can include, but are not limited to a minimum of 2′-6″ by 3′-0″ open rectangle to allow access for the tremie tube and construction workers for concrete placement, in-form access hatches, and/or external vibrating.



Constant and Tapered Rectangular Column Section SDCs C and D

Figure 7.4.5-1





Constant and Tapered Rectangular Column Section SDCs A and B
Figure 7.4.5-2

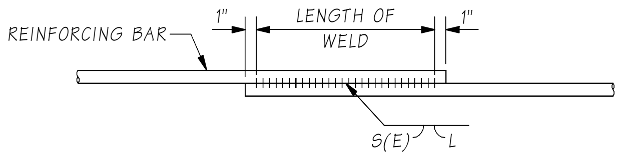
B. **Spiral Splices and Hoops –** Welded laps shall be used for splicing and terminating spirals and shall conform to the details shown in Figure 7.4.5-3. Only single sided welds shall be used, which is the preferred method in construction. Spirals or butt-welded hoops are required for plastic hinge zones of columns. Lap spliced hoops are not permitted in columns in any region.

Although hooked lap splices are structurally acceptable, and permissible by AASHTO LRFD Specification for spirals or circular hoops, they shall not be allowed due to construction challenges. While placing concrete, tremies get caught in the protruding hooks, making accessibility to all areas and its withdrawal cumbersome. It is also extremely difficult to bend the hooks through the column cage into the core of the column.

When welded hoops or mechanical couplers are used, the plans shall show a staggered pattern around the perimeter of the column so that no two adjacent welded splices or couplers are located at the same location. Also, where interlocking hoops are used in rectangular or non-circular columns, the splices shall be located in the column interior.

Circular hoops for columns shall be shop fabricated using a manual direct butt weld, resistance butt weld, or mechanical coupler. Currently, a Bridge Special Provision has been developed to cover the fabrication requirements of hoops for columns and shafts, which may eventually be included in the *Standard Specifications*. Manual direct butt welded hoops require radiographic nondestructive examination (RT), which may result in this option being cost prohibitive at large quantities. Resistance butt welded hoops are currently available from Caltrans approved fabricators in California and have costs that are comparable to welded lap splices. Fabricators in Washington State are currently evaluating resistance butt welding equipment. When mechanical couplers are used, cover and clearance requirements shall be accounted for in the column details.

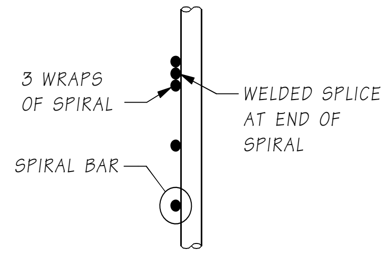
Field welded lap splices and termination welds of spirals of any size bar are not permitted in the plastic hinge region and should be clearly designated on the contract plans. If spirals are welded while in place around longitudinal steel reinforcement, there is a chance that an arc can occur between the spiral and longitudinal bar. The arc can create a notch that can act as a stress riser and may cause premature failure of the longitudinal bar when stressed beyond yield. ***Note:*** It would acceptable to field weld lap splices of spirals off to the side of the column and then slide into place over the longitudinal reinforcement.



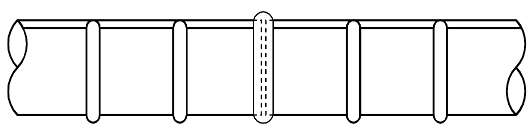
WELDED LAP SPLICE DETAIL

WELDED LAP SPLICE IS SUITABLE FOR SPIRALS IN COLUMNS AND SHAFTS UP TO BAR SIZE #6. LAP SPLICE FOR BAR SIZES #7 TO #9 ARE ONLY INTENDED FOR SHAFT HOOPS. WELDING SHALL MEET THE REQUIREMENTS OF STD. SPEC. 6-02.3(24)E. FOR WELD DIMENSIONS, SEE TABLE BELOW.

		WELD DIMENSIONS (IN)		
	BAR SIZE	S	E	L (LENGTH)
SPIRALS	#4	1/4	1/8	4
	#5	5/16	3/16	6
	#6	3/8	3/16	6
HOOPS FOR SHAFTS	#7	7/16	1/4	7
	#8	1/2	1/4	8
	#9	9/16	5/16	8

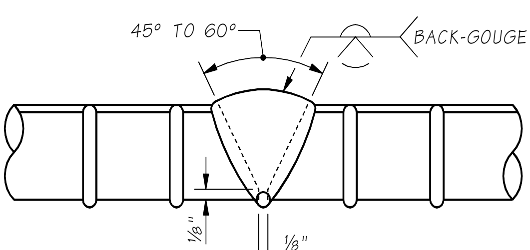


SPIRAL TERMINATION DETAIL



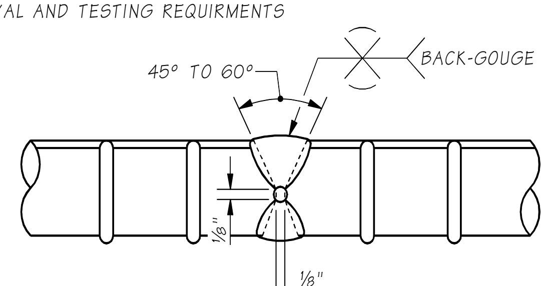
RESISTANCE BUTT JOINT DETAIL

SEE SPECIAL PROVISIONS FOR APPROVAL AND TESTING REQUIRMENTS



SINGLE V-GROOVE WELD

AWS D1.4 FIGURE 3.2(A)



DOUBLE V-GROOVE WELD

AWS D1.4 FIGURE 3.2(B)

MANUAL DIRECT BUTT JOINT DETAILS

ALL BACKING SHALL BE REMOVED.
SEE SPECIAL PROVISIONS FOR RT TESTING FREQUENCY

Welded Spiral Splice and Butt Splice Details
Figure 7.4.5-3

7.4.6 Column Hinges

Column hinges of the type shown in Figure 7.4.6-1 were built on past WSDOT bridges. Typically they were used above a crossbeam or wall pier. These types of hinges are suitable when widening an existing bridge crossbeam or wall pier with this type of detail.

The area of the hinge bars in square inches is as follows:

$$A_s = \frac{\frac{(P_u)}{2} + \left[P_u^2 + V_u^2\right]^{1/2}}{.85\, F_y \cos\theta} \tag{7.4.6-1}$$

Where:

P_u is the factored axial load
V_u is the factored shear load
F_y is the reinforcing yield strength (60 ksi)
θ is the angle of the hinge bar to the vertical

The development length required for the hinge bars is 1.25 l_d. All applicable modification factors for development length in AASHTO LRFD 5.11.2 may be used when calculating l_d. Tie and spiral spacing shall conform to AASHTO LRFD confinement and shear requirements. Ties and spirals shall not be spaced more than 12″ (6″ if longitudinal bars are bundled). Premolded joint filler should be used to assure the required rotational capacity. There should also be a shear key at the hinge bar location.

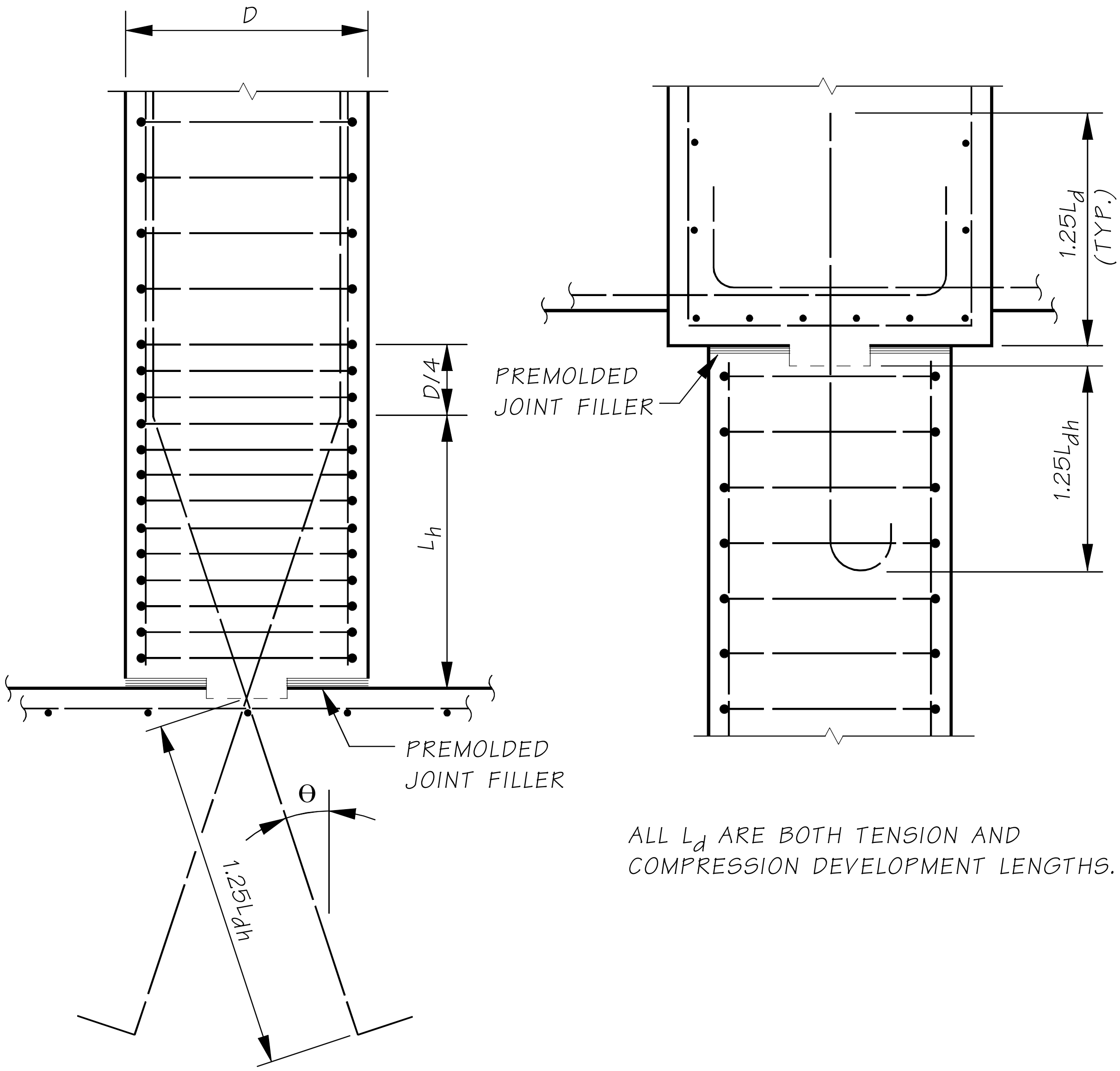
When the hinge reinforcement is bent, additional confinement reinforcing may be necessary to take the horizontal component from the bent hinge bars. The maximum spacing of confinement reinforcing for the hinge is the smaller of that required above and the following:

$$S_{max} = \frac{A_v F_y}{\left[\frac{P_u \tan\theta}{.85\, l_h} + \frac{V_s}{d}\right]} \tag{7.4.6-2}$$

Where:

A_v, V_s, and d are as defined in AASHTO Article "Notations"
l_h is the distance from the hinge to where the bend begins

Continue this spacing one-quarter of the column width (in the plane perpendicular to the hinge) past the bend in the hinge bars.



Hinge Details
Figure 7.4.6-1

7.4.7 Reduced Column Fixity

Reduced column fixity uses a reduced column section to decrease overstrength plastic demands into the foundation. The conceptual detail for reduced column base fixity is shown below for a spread footing foundation. This concept could be used for shaft and pile supported foundations also. Traditional column designs are preferred over this detail, but this may be used if it is determined that traditional details will not satisfy the design code requirements due to architectural, balanced stiffness, or other project specific requirements. The reduction at the base of the column shall be designed as described below and detailed as shown in Figure 7.4.7-1. Similar checks will be required if the reduced section were placed at the crossbeam, along with any additional checks required for those sections. One such additional check is joint shear in the crossbeam based on the overstrength plastic capacity of the reduced column section. The design and detail at the top of columns, for architectural flares, is similar.

A. **Inner Concrete Column**

1. **Longitudinal Reinforcement**

a. The longitudinal inner column reinforcement shall extend a distance of L_{ns} into the column and shall be set on top of bottom mat reinforcement of foundation with standard 90° hooks.

$$L_{ns} = L_s + sc + L_p \tag{7.4.7-1}$$

Where:

L_s = The larger of $1.7 \times L_{ac}$ or $1.7 \times L_d$ (for Class C lap splice)
L_{ac} = Development length of bar from the AASHTO Seismic 8.8.4.
L_d = Tension development length from AASHTO LRFD 5.11.2.1
(***Note:*** All applicable modification factors for L_d may be used except for the reduction specified in Section 5.11.2.2.2 for As required/As provided)
sc = Distance from longitudinal reinforcement of outer column to inner column.
L_p = Analytical Plastic Hinge Length defined in the AASHTO Seismic 4.11.6-3.

b. The longitudinal reinforcing in the inner column shall meet all the design checks in the AASHTO Seismic and AASHTO LRFD Specifications. Some specific checks of the inner column (inner core) will be addressed as follows:

(1) A shear friction check shall be met using the larger of the overstrength plastic shear (*Vpo*) or the ultimate shear demand from strength load cases at the hinge location. The area of longitudinal inner column reinforcement, A_{st}, in excess of that required in the tensile zone for flexural resistance (usually taken as ½ the total longitudinal bars) may be used for the required shear friction reinforcement, A_{vf}.

(2) The flexural capacity of the inner column shall be designed to resist the strength load cases and meet cracking criteria of the service load cases. Special consideration shall be given to construction staging load cases where the column stability depends on completion of portions of the superstructure.

(3) The axial capacity of the inner column shall meet the demands of strength load cases assuming the outer concrete has cracked and spalled off. The gross area, A_g, shall be the area contained inside the spiral reinforcement.

(4) The inner core shall be designed and detailed to meet all applicable requirements of AASHTO Seismic Section 8.

2. **Transverse Reinforcement**

   a. The portion of the transverse reinforcement for the inner core, inside the larger column dimension (above the foundation), shall meet all the requirements of the AASHTO Seismic and AASHTO LRFD Specification. The demand shall be based on the larger of the overstrength plastic shear demand (*Vpo*) of the inner column or the ultimate shear demand from strength load cases at the hinge location. The transverse reinforcement shall be extended to the top of the longitudinal reinforcement for the inner column (L_{ns}).

   b. The portion of transverse reinforcement for the inner core, in the foundation, shall meet the minimum requirements of the AASHTO Seismic 8.8.8, for compression members, based on the dimensions of the inner column. This reinforcement shall be extended to the bend radius of the of the longitudinal inner column reinforcement for footings or as required for column-shaft connections.

   c. A gap in the inner column transverse reinforcement shall be sized to allow the foundation top mat reinforcement and foundation concrete to be placed prior to setting the upper portion of the transverse inner column reinforcement. This gap shall be limited to 5″; a larger gap will require the WSDOT Bridge Design Engineer's approval. The spiral reinforcement above the footing shall be placed within 1″ of the top of footing to reduce the required gap size. The WSDOT Spiral termination details will be required at each end of this gap, the top of the upper transverse reinforcement, but not the bottom of the lower transverse reinforcement with spread footings.

3. **Analytical Plastic Hinge Region**

   a. The analytical plastic hinge length of the reduced column section shall be based on horizontally isolated flared reinforced concrete columns, using equation 4.11.6-3 of the AASHTO Seismic Specifications.

   b. The end of the column which does not have a reduced column section shall be based on equation 4.11.6-1 of the AASHTO Seismic Specifications.

B. **Outer Concrete Column**

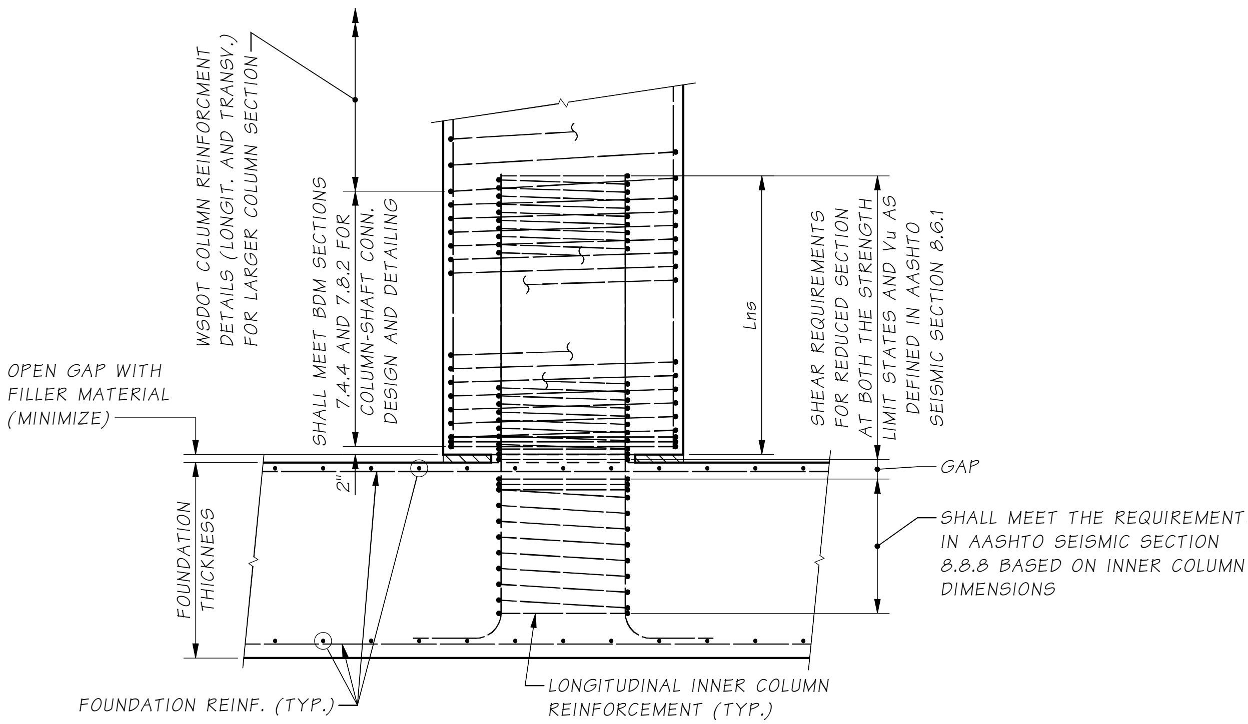
1. The WSDOT Bridge and Structures Office normal practices and procedures shall be met for the column design, with the following exceptions:

   a. The end with the reduced column shall be detailed to meet the seismic requirements of a plastic hinge region. This will ensure that if a plastic hinge mechanism is transferred into the large column shape, it will be detailed to develop such hinge. The plastic shear this section shall be required to resist shall be the same as that of the inner column section.

   b. The WSDOT spiral termination detail shall be placed in the large column at the reduced section end, in addition to other required locations.

   c. In addition to the plastic hinge region requirements at the reduced column end, the outer column spiral reinforcement shall meet the requirements of the WSDOT Noncontact Lap Splices in Bridge Column-Shaft Connections. The k factor shall be taken as 0.5 if the column axial load, after moment distribution, is greater than $0.10f'_cA_g$ and taken as 1.0 if the column axial load is in tension. A_g shall be taken as the larger column section. Linear interpolation may be used between these two values.

2. The column end without the reduced column section shall be designed with WSDOT practices for a traditional column, but shall account for the reduced overstrength plastic shear, applied over the length of the column, from the overstrength plastic capacities at each column end.



Pinned Column Base Reduced Column Fixity at Base

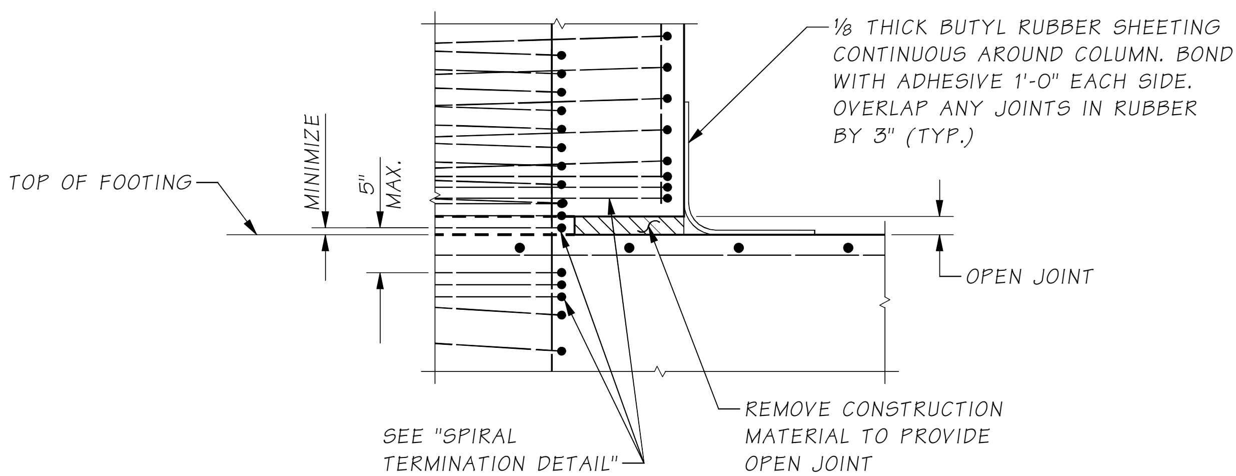
Figure 7.4.7-1

C. **Gap in Concrete at Reduced Column Section**

1. This gap shall be minimized, but not less than 2″. It shall also be designed to accommodate the larger of 1.5 times the calculated service, strength or extreme elastic rotation or the plastic rotation from a pushover analysis times the distance from the center of the column to the extreme edge of the column. The gap shall be constructed with a material sufficiently strong to support the wet concrete condition. The final material must also meet the requirements described below. If a material can meet both conditions, then it can be left in place after construction, otherwise the construction material must be removed and either cover the gap or fill the gap with a material that meets the following:

   a. The material in the gap must keep soil or debris out of the gap for the life of the structure, especially if the gap is to be buried under fill at the foundation and inspections will be difficult/impossible.

   b. The gap shall be sized to accommodate 1.5 times the rotations from service, strength and extreme load cases. In no loading condition shall the edge of the larger column section cause a compressive load on the footing. If a filler material is used in this gap which can transfer compressive forces once it has compressed a certain distance, then the gap shall be increased to account for this compressive distance of the filler material.



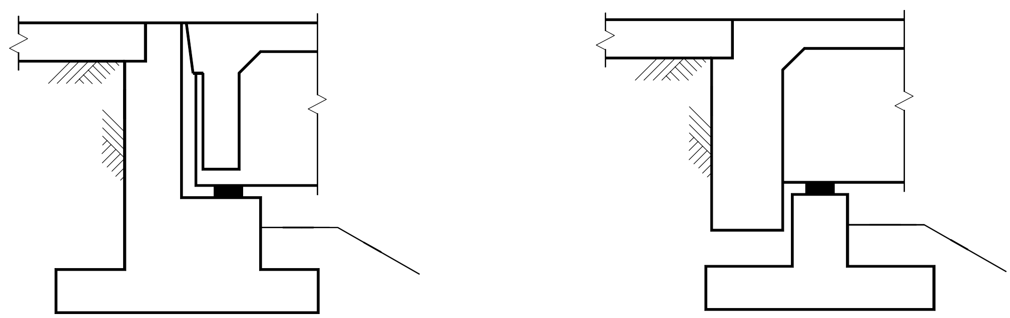
Open Gap Detail
Figure 7.4.7-2

7.5 Abutment Design and Details

7.5.1 General

A. **Abutment Types** – There are five abutment types described in the following section that have been used by the Bridge and Structures Office. Conventional stub and cantilever abutments on spread footings, piles, or shafts are the preferred abutment type for WSDOT bridges. The representative types are intended for guidance only and may be varied to suit the requirements of the bridge being designed.

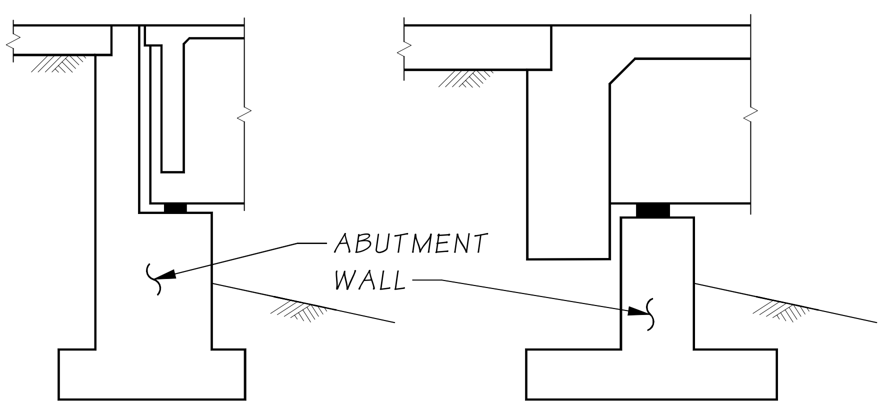
1. **Stub Abutments –** Stub abutments are short abutments where the distance from the girder seat to top of footing is less than approximately 4 feet, see Figure 7.5.1-1. The footing and wall can be considered as a continuous inverted T-beam. The analysis of this type abutment shall include investigation into both bending and shear stresses parallel to centerline of bearing. If the superstructure is relatively deep, earth pressure combined with longitudinal forces from the superstructure may become significant.



Stub "L" Abutment Stub Abutment

Figure 7.5.1-1

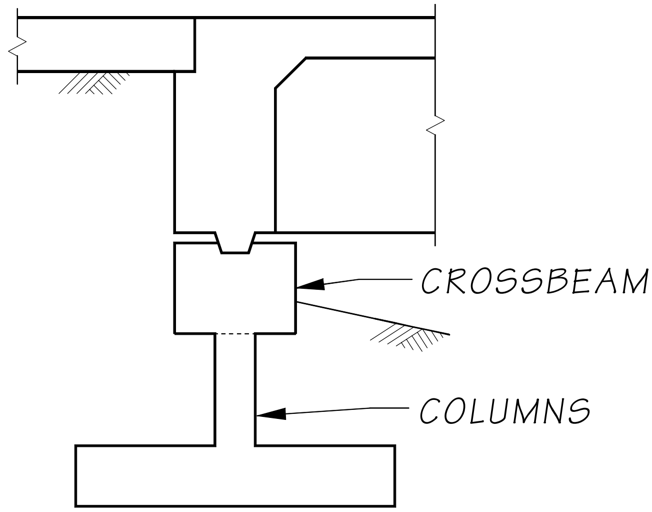
2. **Cantilever Abutments –** If the height of the wall from the bearing seat down to the bottom of the footing exceeds the clear distance between the girder bearings, the assumed 45° lines of influence from the girder reactions will overlap, and the dead load and live load from the superstructure can be assumed equally distributed over the abutment width. The design may then be carried out on a per-foot basis. The primary structural action takes place normal to the abutment, and the bending moment effect parallel to the abutment may be neglected in most cases. The wall is assumed to be a cantilever member fixed at the top of the footing and subjected to axial, shear, and bending loads see Figure 7.5.1-2.



Cantilever "L" Abutment Cantilever Abutment

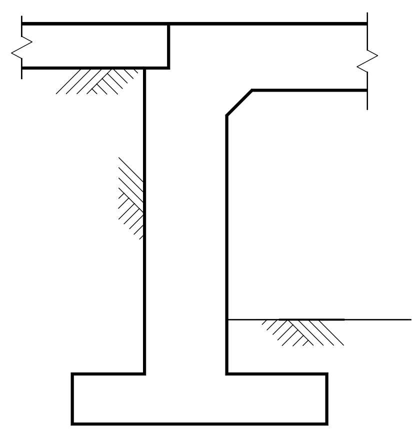
Figure 7.5.1-2

3. **Spill-Through Abutments –** The analysis of this type of abutment is similar to that of an intermediate pier, see Figure 7.5.1-3. The crossbeam shall be investigated for vertical loading as well as earth pressure and longitudinal effects transmitted from the superstructure. Columns shall be investigated for vertical loads combined with horizontal forces acting transversely and longitudinally. For earth pressure acting on rectangular columns, assume an effective column width equal to 1.5 times the actual column width. Short, stiff columns may require a hinge at the top or bottom to relieve excessive longitudinal moments.



Spill-Through Abutment
Figure 7.5.1-3

4. **Rigid Frame Abutments –** Abutments that are part of a rigid frame are generically shown in Figure 7.5.1-4. At-Rest earth pressures (EH) will apply to these structures. The abutment design should include the live load impact factor from the superstructure. However, impact shall not be included in the footing design. The rigid frame itself should be considered restrained against sidesway for live load only. AASHTO LRFD Chapter 12 addresses loading and analysis of rigid frames that are buried (box culverts).

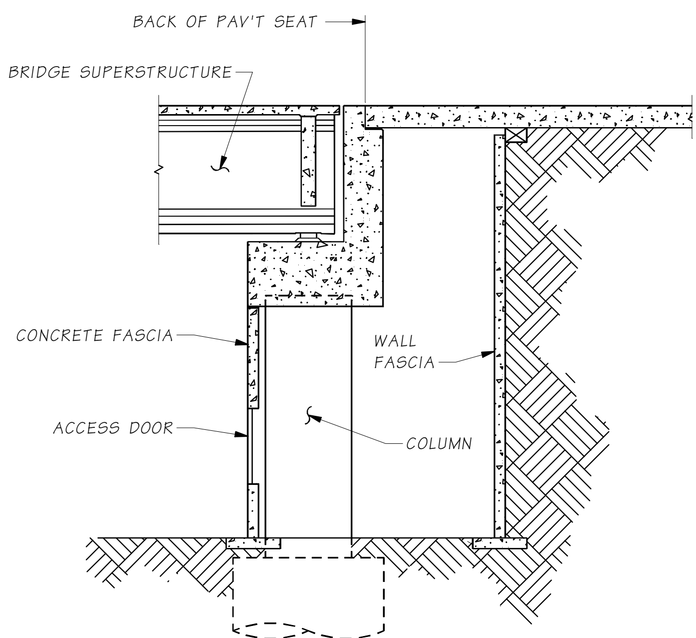


Rigid Frame Abutment
Figure 7.5.1-4

5. **Bent-Type Abutments** – An abutment that includes a bent cap supported on columns or extended piles or shafts is shown in Figure 7.5.1-5. For structural reasons it may be required to construct a complete wall behind a bridge abutment prior to bridge construction. Bent-type abutments may be used where the abutment requires protection from lateral and vertical loads and settlement. This configuration shall only be used with the approval of the WSDOT Bridge Design Engineer. It shall not be used where initial construction cost is the only determining incentive.

   A bridge approach slab shall span a maximum of 6′-0″ between the back of pavement seat and the face of the approach embankment wall. The approach slab shall be designed as a beam pinned at the back of pavement seat. The approach slab shall support traffic live loads and traffic barrier reactions. The approach embankment wall shall support the vertical live load surcharge. The approach slab shall not transfer loads to the approach embankment wall facing.

   An enclosing fascia wall is required to prohibit unwanted access with associated public health, maintenance staff safety, and law enforcement problems. The design shall include a concrete fascia enclosing the columns and void. The fascia shall have bridge inspection access. The access door shall be a minimum 3′-6″ square with the sill located 2′-6″ from finished grade. Contact the State Bridge and Structures Architect for configuration and concrete surface treatments.



Bent-Type Abutment
Figure 7.5.1-5

B. **Abutments on Structural Earth (SE) Walls and Geosynthetic Walls** – Bridge abutments may be supported on structural earth walls and geosynthetic walls. Abutments supported on these walls shall be designed in accordance with the requirements of this manual and the following documents (listed in order of importance):

- WSDOT *Geotechnical Design Manual* M 46-03 (see Section 15.5.3.5).
- AASHTO LRFD Bridge Design Specifications.
- Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Volume I and II, FHWA-NHI-10-024, FHWA-NHI-10-025.

Walls directly supporting bridge abutment spread footings shall be 30 feet or less in total height, measured from the top of the fascia leveling pad to the bottom of the bridge abutment footing. SE walls supporting spread footings shall be considered non-preapproved wall systems. Geosynthetic walls supporting spread footings shall be considered nonstandard. Deviations from the design requirements require approval from the State Bridge Design Engineer and the State Geotechnical Engineer.

Continuous superstructures shall be designed for differential settlement between piers. Walls supporting permanent bridges shall have precast or C.I.P concrete fascia panels. Walls supporting temporary bridges may use precast concrete block or welded wire facing.

7.5.2 Embankment at Abutments

A. **General Clearances** – The minimum clearances for the embankment at the front face of abutments shall be as indicated on *Standard Plans* A-50.10.00 through A-50.40.00. At the ends of the abutment, the fill may be contained with wing walls or in the case of concrete structures, placed against the exterior girders.

B. **Abutments on SE Walls and Geosynthetic Walls –** Clearances around bridge abutments shall be provided as shown in Figure 7.5.2-1, and shall supersede AASHTO LRFD Article 11.10.11. Concrete slope protection shall be provided. Fall protection shall be provided in accordance with *Design Manual* Section 730.

7.5.3 Abutment Loading

In general, bridge abutment loading shall be in accordance with AASHTO LRFD Chapter 3 and 11. The following simplifications and assumptions may be applied to the abutment design. See Section 7.7.4 for a force diagram of typical loads as they are applied to an abutment spread footing.

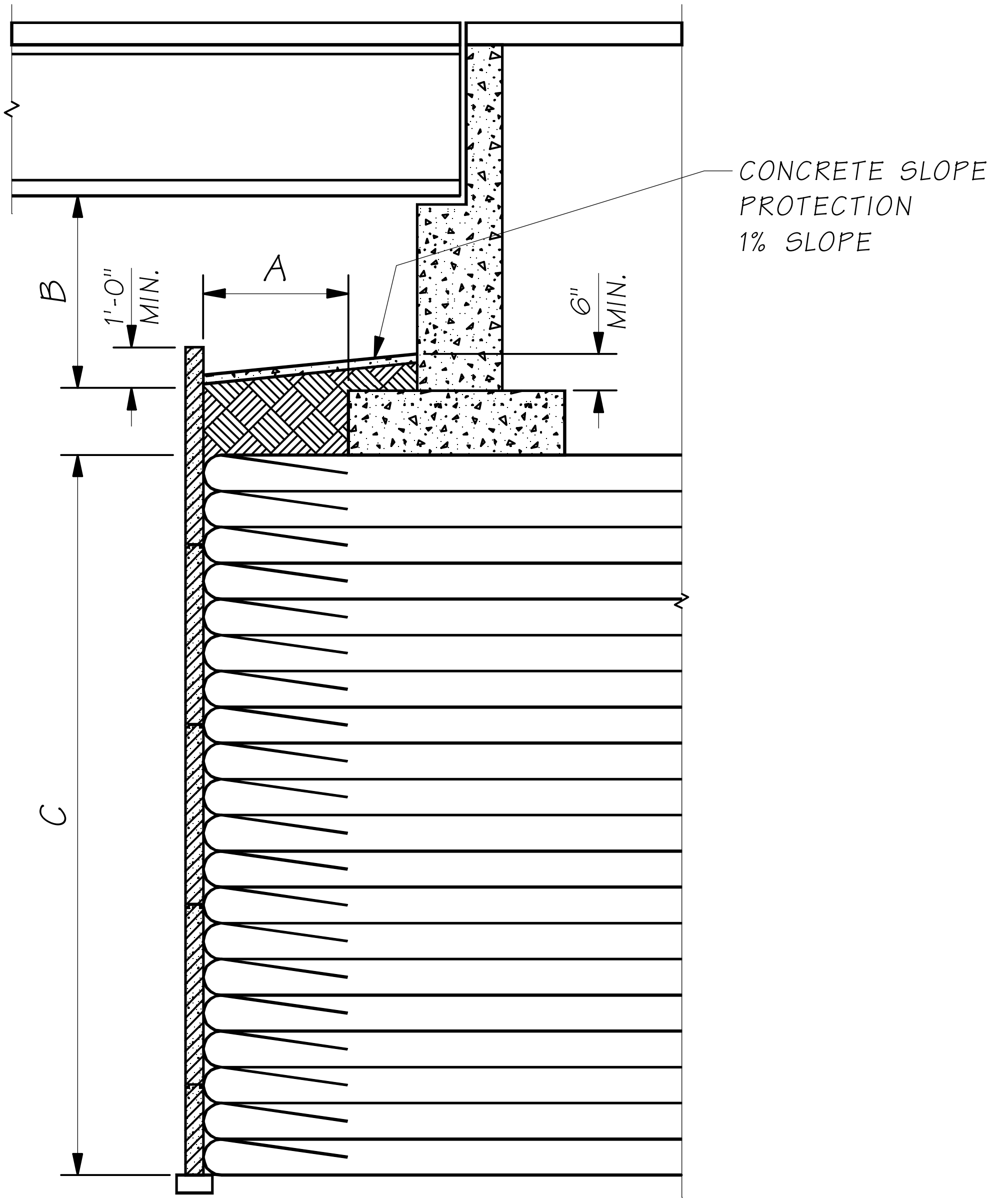
A. **Dead Load - DC –** Approach slab dead load reaction taken as 2 kips/foot of wall applied at the pavement seat.

B. **Live Load - LL –** Live load impact does not apply to the abutment. Bridge approach slab live load reaction (without *IM*) applied at the pavement seat may be assumed to be 4.5 kips per foot of wall for HL-93 loading, see Section 10.6 of this manual for bridge approach slab design assumptions. Abutment footing live loads may be reduced (by approximately one axle) if one design truck is placed at the bridge abutment with a bridge approach slab. Adding the pavement seat reaction to the bearing reaction duplicates the axle load from two different design truck configurations.

   If bridge approach slabs are not to be constructed in the project (e.g. bride approach slab details are not included in the bridge sheets of the Plans) a live load surcharge (*LS*) applies.

C. **Earth Pressure - EH, EV** – Active earth pressure (EH) and the unit weight of backfill on the heel and toe (EV) will be provided in a geotechnical report. The toe fill shall be included in the analysis for overturning if it adds to overturning.

   Passive earth pressure resistance (EH) in front of a footing may not be dependable due to potential for erosion, scour, or future excavation. Passive earth pressure may be considered for stability at the strength limit state only below the depth that is not likely to be disturbed over the structure's life. The Geotechnical Branch should be contacted to determine if passive resistance may be considered. The top two feet of passive earth pressure should be ignored.



A. 4'-0" MIN. FOR SE WALLS (PRECAST CONCRETE PANEL FACE OR CAST-IN-PLACE CONCRETE FACE) AND 2'-0" MIN. FOR SPECIAL DESIGNED GEOSYNTHETIC RETAINING WALLS WITH WRAPPED FACE.

B. 3'-0" MIN. FOR GIRDER BRIDGES AND 5'-0" MIN. FOR NON-GIRDER, SLAB, AND BOX GIRDER BRIDGES.

C. 30'-0" MAXIMUM

Abutment on SE Wall or Geosynthetic Wall
Figure 7.5.2-1

D. **Earthquake Load - EQ** – Seismic superstructure loads shall be transmitted to the substructure through bearings, girder stops or restrainers. As an alternative, the superstructure may be rigidly attached to the substructure. The Extreme Event I load factor for all EQ induced loads shall be 1.0.

For bearing pressure and wall stability checks, the seismic inertial force of the abutment, P_{IR}, shall be combined with the seismic lateral earth pressure force, P_{AE}, as described in AASHTO LRFD 11.6.5.1. The seismic inertial force acts horizontally at the mass centroid of the abutment in the same direction as the seismic lateral earth pressure. For structural design of the abutment, the seismic inertial force, P_{IR}, may be taken as 0.0.

For conventional footing supported abutments, the seismic horizontal acceleration coefficient, k_h, shall be taken as one half of the seismic horizontal acceleration coefficient assuming zero displacement, k_{h0}. Seismic active earth pressure, K_{AE}, shall be assumed to be a uniform pressure over the height, h, of the abutment. Thus, the resultant seismic lateral earth pressure force, P_{AE}, is located at 0.5h. The seismic active earth pressure shall be determined using the Mononobe-Okabe (M-O) method, as described in AASHTO LRFD Appendix A11. For more information on the M-O method and its applicability, see GDM Section 15.4.10.

For pile- or shaft-supported abutments or other abutments that are not free to translate 1.0 in. to 2.0 in. during a seismic event, use a seismic horizontal acceleration coefficient, k_h, of 1.5 times the site-adjusted peak ground acceleration. For more information on seismic lateral earth pressure on rigid abutments, see GDM Section 15.4.10.

The seismic vertical acceleration coefficient, k_v, shall be taken as 0.0 for abutment design.

E. **Bearing Forces - TU** – For strength design, the bearing shear forces shall be based on ½ of the annual temperature range. This force is applied in the direction that causes the worst case loading.

For extreme event load cases, calculate the maximum friction force (when the bearing slips) and apply in the direction that causes the worst case loading.

7.5.4 Temporary Construction Load Cases

A. **Superstructure Built after Backfill at Abutment** – If the superstructure is to be built after the backfill is placed at the abutments, the resulting temporary loading would be the maximum horizontal force with the minimum vertical force. During the abutment design, a load case shall be considered to check the stability and sliding of abutments after placing backfill but prior to superstructure placement. This load case is intended as a check for a temporary construction stage, and not meant to be a controlling load case that would govern the final design of the abutment and footing. This loading will generally determine the tensile reinforcement in the top of the footing heel.

If this load case check is found to be satisfactory, a note shall be added to the general notes in the contract plans and the contactor will not be required to make a submittal requesting approval for early backfill placement. This load case shall include a 2′-0″ deep soil surcharge for the backfill placement equipment (LS) as covered by the WSDOT *Standard Specifications* Section 2-03.3(14)I.

B. **Wingwall Overturning** – It is usually advantageous in sizing the footing to release the falsework from under the wing walls after some portion of the superstructure load is applied to the abutment. A note can cover this item, when applicable, in the sequence of construction on the plans.

7.5.5 Abutment Bearings and Girder Stops

All structures shall be provided with some means of restraint against lateral displacement at the abutments due to temperature, shrinkage, wind, earth pressure, and earthquake loads, etc. Such restraints may be in the form of concrete girder stops with vertical elastomeric pads, concrete hinges, or bearings restrained against movement.

All prestressed girder bridges in Western Washington (within and west of the Cascade mountain range) shall have girder stops between all girders at abutments and intermediate expansion piers. This policy is based on fact that the February 28, 2001 Nisqually earthquake caused significant damage to girder stops at bridges where girder stops were not provided between all girders. In cases where girder stops were cast prior to placement of girders and the 3″ grout pads were placed after setting the girders, the 3″ grout pads were severely damaged and displaced from their original position.

A. **Abutment Bearings** – Longitudinal forces from the superstructure are normally transferred to the abutments through the bearings. The calculated longitudinal movement shall be used to determine the shear force developed by the bearing pads. The shear modulus of Neoprene at 70°F (21°C) shall be used for determining the shear force. However, the force transmitted through a bearing pad shall be limited to that which causes the bearing pad to slip. Normally, the maximum percentage of the vertical load reaction transferred in shear is assumed to be 6 percent for PTFE sliding bearings and 20 percent for elastomeric bearing pads. For semi-integral abutments, the horizontal earth pressure acting on the end diaphragm is transferred through the bearings.

   When the force transmitted through the bearing pads is very large, the designer should consider increasing the bearing pad thickness, using PTFE sliding bearings and/or utilizing the flexibility of the abutment as a means of reducing the horizontal design force. When the flexibility of the abutment is considered, it is intended that a simple approximation of the abutment deformation be made.

   For semi-integral abutments with overhanging end diaphragms at the Extreme Event, the designer shall consider that longitudinal force may be transmitted through the end diaphragm. If the gap provided is less than the longitudinal displacement demand, assume the end diaphragm is in contact with abutment wall. In this case, the bearing force shall not be added to seismic earth pressure force.

B. **Bearing Seats** – The bearing seats shall be wide enough to accommodate the size of the bearings used with a minimum edge dimension of 3″ and satisfy the requirements of LRFD Section 4.7.4.4. On L abutments, the bearing seat shall be sloped away from the bearings to prevent ponding at the bearings. The superelevation and profile grade of the structure should be considered for drainage protection. Normally, a ¼″ drop across the width of the bearing seat is sufficient.

C. **Transverse Girder Stops** – Transverse girder stops are required for all abutments in order to transfer lateral loads from the superstructure to the abutment. Abutments shall normally be considered as part of the Earthquake Resisting System (ERS). Girder stops shall be full width between girder flanges except to accommodate bearing replacement requirements as specified in Chapter 9 of this manual. The girder stop shall be designed to resist loads at the Extreme Limit State for the earthquake loading, Strength loads (wind etc.) and any transverse earth pressure from skewed abutments, etc. Girder stops are designed using shear friction theory and the shear strength resistance factor shall be $\varphi_s = 0.9$. The possibility of torsion combined with horizontal shear when the load does not pass through the centroid of the girder stop shall also be investigated.

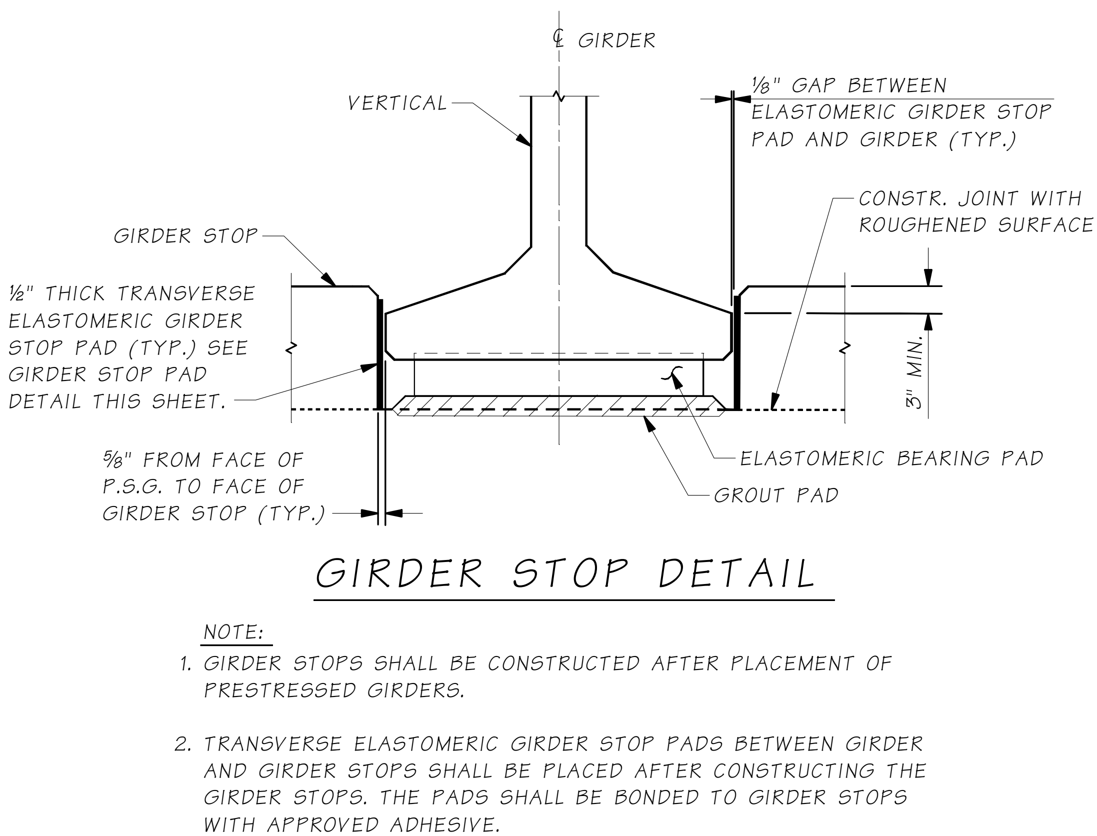
   In cases where the WSDOT Bridge Design Engineer permits use of ERS #3 described in Section 4.2.2 of this manual, which includes a fusing mechanism between the superstructure and substructure, the following requirements shall be followed:

   - The abutment shall not be included in the ERS system, the girder stops shall be designed to fuse, and the shear strength resistance factor shall be $\varphi_s = 1.0$.

- If a girder stop fusing mechanism is used on a pile supported abutment, the combined overstrength capacity of the girder stops per AASHTO Seismic 4.14 shall be less than the combined plastic shear capacity of the piles.

The detail shown in Figure 7.5.5-1 may be used for prestressed girder bridges. Prestressed girders shall be placed in their final position before girder stops are cast to eliminate alignment conflicts between the girders and girder stops. Elastomeric girder stop pads shall run the full length of the girder stop. All girder stops shall provide ⅛″ clearance between the prestressed girder flange and the elastomeric girder stop pad.

For skewed bridges with semi-integral or end type A diaphragms, the designer shall evaluate the effects of earth pressure forces on the elastomeric girder stop pads. These pads transfer the skew component of the earth pressure to the abutment without restricting the movement of the superstructure in the direction parallel to centerline. The performance of elastomeric girder stop pads shall be investigated at Service Limit State. In some cases bearing assemblies containing sliding surfaces may be necessary to accommodate large superstructure movements.



Girder Stop Details
Figure 7.5.5-1

7.5.6 Abutment Expansion Joints

The compressibility of abutment expansion joints shall be considered in the design of the abutment when temperature, shrinkage, and earthquake forces may increase the design load. For structures without abutment expansion joints, the earth pressure against the end diaphragm is transmitted through the superstructure.

7.5.7 Open Joint Details

Vertical expansion joints extending from the top of footings to the top of the abutment are usually required between abutments and adjacent retaining walls to handle anticipated movements. The expansion joint is normally filled with premolded joint filler which is not water tight. There may be circumstances when this joint must be water tight; ⅛ butyl rubber may be used to cover the joint. The open joint in the barrier shall contain a compression seal to create a watertight joint. Figure 7.5.7-1 shows typical details that may be used. Aesthetic considerations may require that vertical expansion joints between abutments and retaining walls be omitted. This is generally possible if the retaining wall is less than 60 feet long.

The footing beneath the joint may be monolithic or cast with a construction joint. In addition, dowel bars may be located across the footing joint parallel to the wall elements to guard against differential settlement or deflection.

On semi-integral abutments with overhanging end diaphragms, the open joints must be protected from the fill spilling through the joint. Normally butyl rubber is used to seal the openings. See the end diaphragm details in the Appendices of Chapter 5 for details.

7.5.8 Construction Joints

To simplify construction, vertical construction joints are often necessary, particularly between the abutment and adjacent wing walls. Construction joints should also be provided between the footing and the stem of the wall. Shear keys shall be provided at construction joints between the footing and the stem, at vertical construction joints or at any construction joint that requires shear transfer. The *Standard Specifications* cover the size and placement of shear keys. The location of such joints shall be detailed on the plans. Construction joints with roughened surface can be used at locations (except where needed for shear transfer) to simplify construction. These should be shown on the plans and labeled "Construction Joint With Roughened Surface." When construction joints are located in the middle of the abutment wall, a pour strip or an architectural reveal should be used for a clean appearance. Details should be shown in the plans.

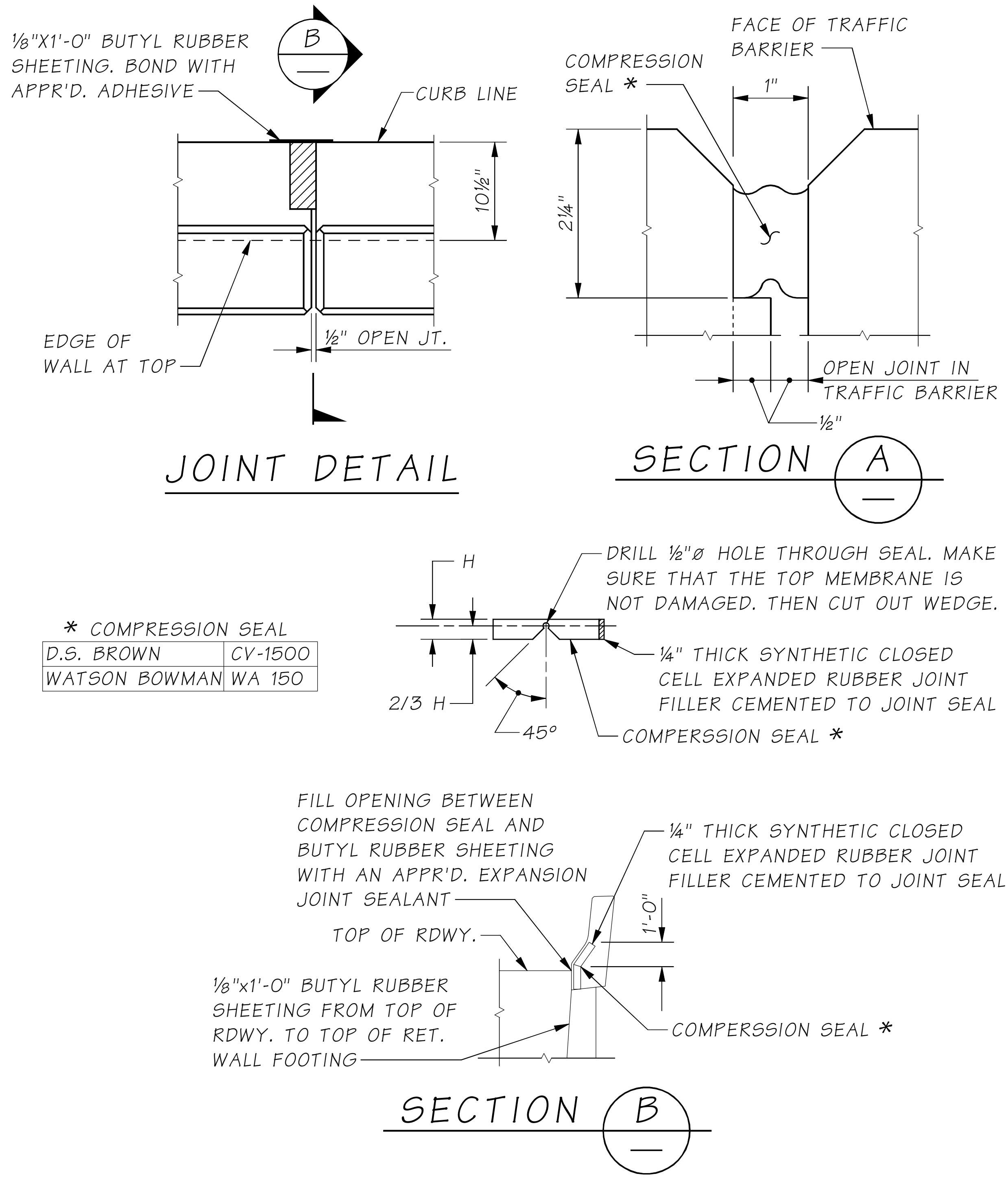
7.5.9 Abutment Wall Design

When the primary structural action is parallel to the superstructure or normal to the abutment face, the wall shall be treated as a column subjected to combined axial load and bending moment. Compressive reinforcement need not be included in the design of cantilever walls, but the possibility of bending moment in the direction of the span as well as towards the backfill shall be considered. A portion of the vertical bars may be cut off where they are no longer needed for stress.

A. **General** – In general, horizontal reinforcement should be placed outside of vertical reinforcement to facilitate easier placement of reinforcement.

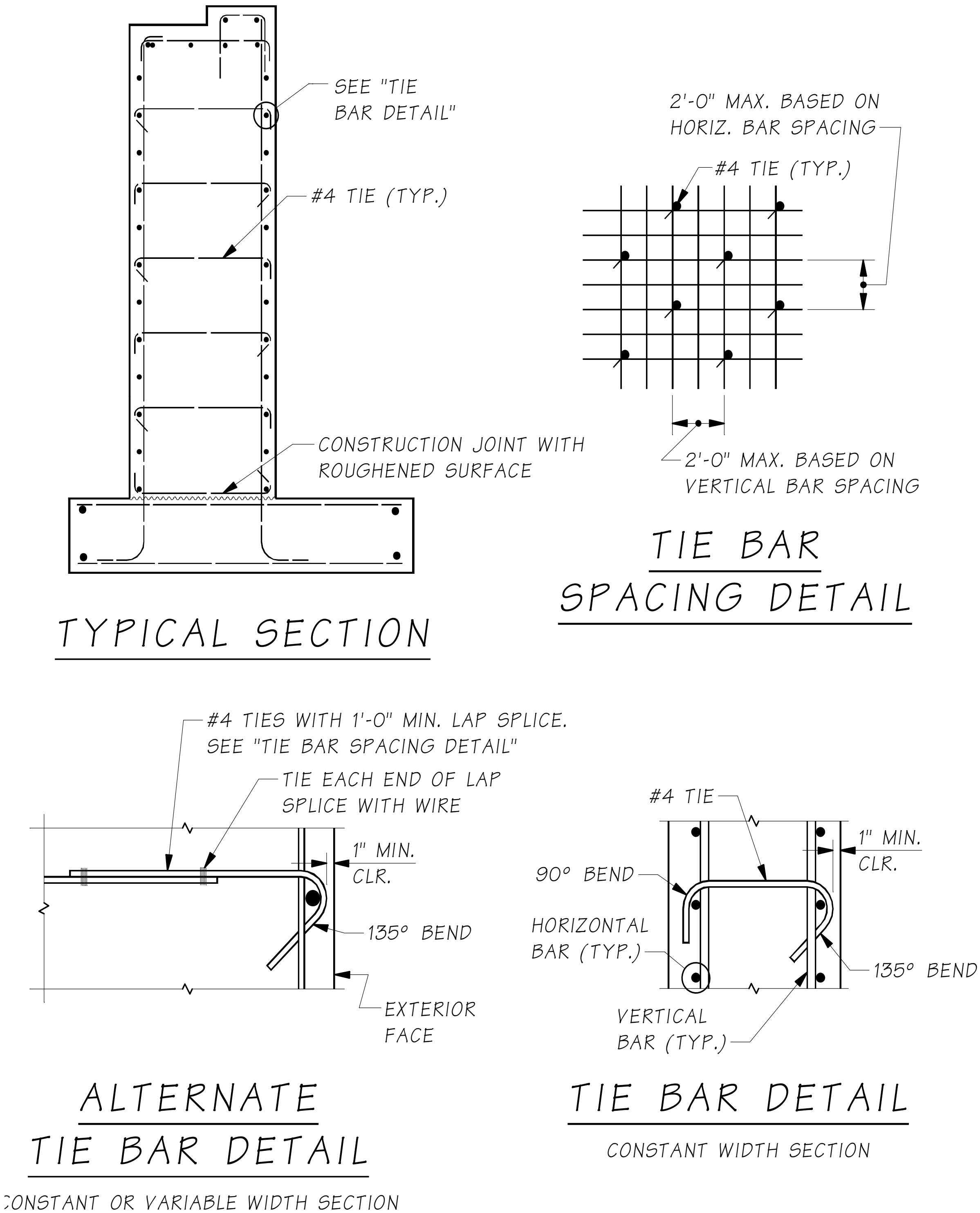
B. **Temperature and Shrinkage Reinforcement** – AASHTO LRFD 5.10.8 shall be followed for providing the minimum temperature and shrinkage steel near surfaces of concrete exposed to daily temperature changes and in structural mass concrete. On abutments that are longer than 60′, consideration should be given to have vertical construction joints to minimize shrinkage cracks.

C. **Cross Ties** – The minimum cross tie reinforcement in the abutment wall is as follows: #4 tie bars with 135° hooks, spaced at approximately 2′-0″ center to center vertically and at approximately 2′-0″ center to center horizontally shall be furnished throughout the abutment stem in all but stub abutments, see Figure 7.5.9-1.



Open Joint Details Between Abutment and Retaining Walls

Figure 7.5.7-1



Cross Tie Details
Figure 7.5.9-1

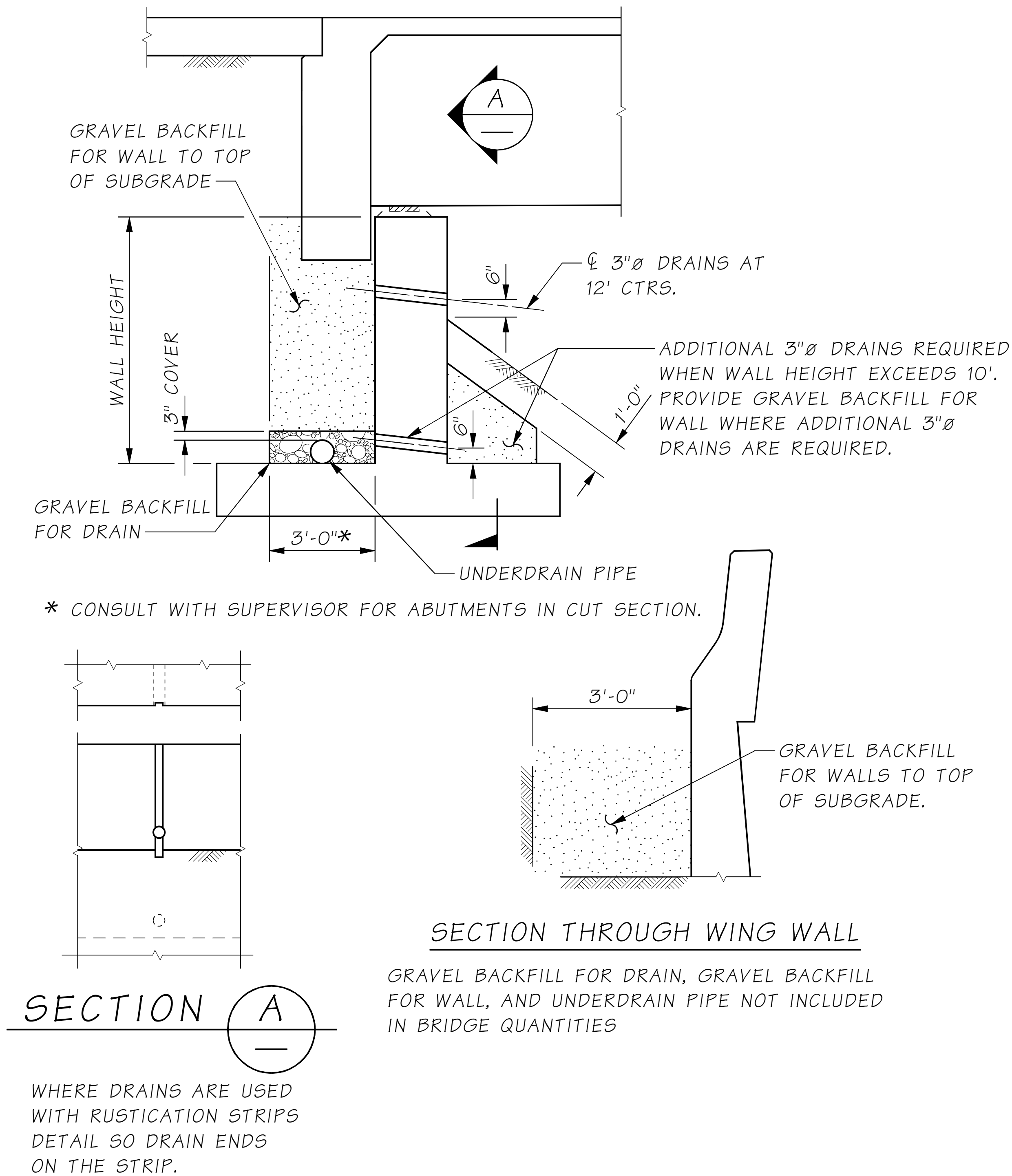
7.5.10 Drainage and Backfilling

3″ diameter weep holes shall be provided in all bridge abutment walls. These shall be located 6″ above the finish ground line at about 12′ on center. In cases where the vertical distance between the top of the footing and the finish groundline is greater than 10′, additional weep holes shall be provided 6″ above the top of the footing. No weep holes are necessary in cantilever wing walls where a wall footing is not used.

The details for gravel backfill for wall, underdrain pipe and backfill for drain shall be indicated on the plans. The gravel backfill for wall shall be provided behind all bridge abutments. The underdrain pipe and gravel backfill for drain shall be provided behind all bridge abutments except abutments on fills with a stem wall height of 5′ or less. When retaining walls with footings are attached to the abutment, a blockout may be required for the underdrain pipe outfall. Cooperation between Bridge and Structures Office and the Design PE Office as to the drainage requirements is needed to guarantee proper blockout locations.

Underdrain pipe and gravel backfill for drain are not necessary behind cantilever wing walls. A 3′ thickness of gravel backfill for wall behind the cantilever wing walls shall be shown in the plans.

The backfill for wall, underdrain pipe and gravel backfill for drain are not included in bridge quantities, the size of the underdrain pipe should not be shown on the bridge plans, as this is a Design PE Office design item and is subject to change during the design phase. Figure 7.5.10-1 illustrates backfill details.



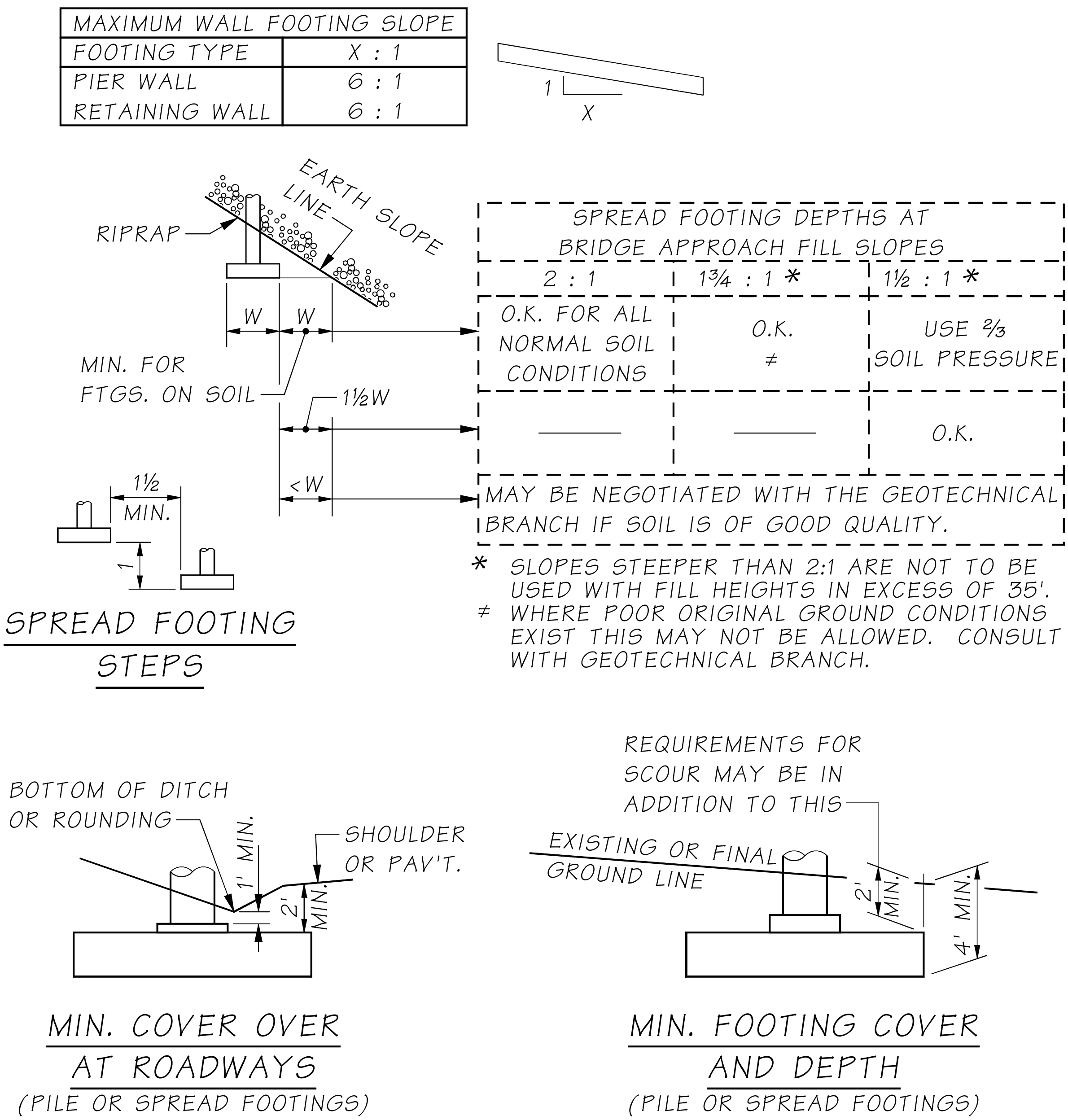
Drainage and Backfill Details
Figure 7.5.10-1

7.7 Footing Design

7.7.1 General Footing Criteria

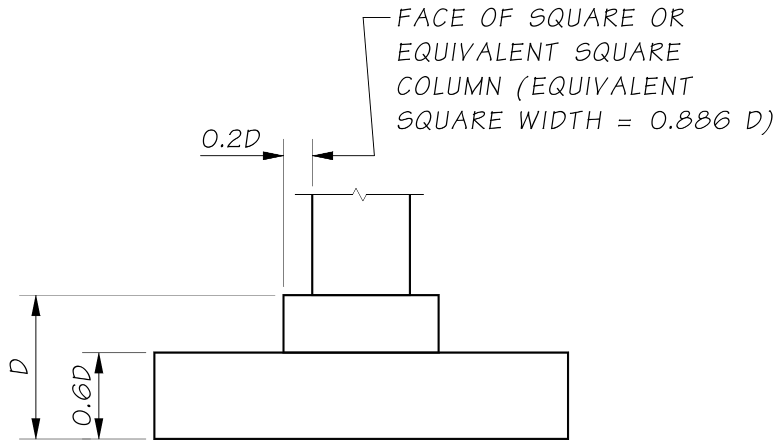
The provisions given in this section pertain to both spread footings and pile supported footings.

A. **Minimum Cover and Footing Depth** – The geotechnical report may specify a minimum footing depth in order to ensure adequate bearing pressure. Stream crossings may require additional cover depth as protection against scour. The HQ Hydraulic Section shall be consulted on this matter. Footings set too low result in large increases in cost. The end slope on the bridge approach fill is usually set at the preliminary plan stage but affects the depth of footings placed in the fill. Figure 7.7.1-1 illustrates footing criteria when setting footing elevations. Footings supported on SE walls or geosynthetic walls shall have a minimum of 6″ of cover for frost protection.



Guidelines for Footing Cover and Depth
Figure 7.7.1-1

B. **Pedestals –** A pedestal is sometimes used as an extension of the footing in order to provide additional depth for shear near the column. Its purpose is to provide adequate structural depth while saving concrete. For proportions of pedestals, see Figure 7.7.1-2. Since additional forming is required to construct pedestals, careful thought must be given to the tradeoff between the cost of the extra forming involved and the cost of additional footing concrete. Also, additional foundation depth may be needed for footing cover. Whenever a pedestal is used, the plans shall note that a construction joint will be permitted between the pedestal and the footing. This construction joint should be indicated as a construction joint with roughened surface.



Pedestal Dimensions
Figure 7.7.1-2

7.7.2 Loads and Load Factors

The following Table 7.7.2-1 is a general application of minimum and maximum load factors as they apply to a generic footing design. Footing design must select the maximum or minimum Load Factors for various modes of failure for the Strength and Extreme Event Limit States.

The dead load includes the load due to structural components and non-structural attachments (*DC*), and the dead load of wearing surfaces and utilities (*DW*). The live load (*LL*) does not include vehicular dynamic load allowance (*IM*).

Designers are to note, if column design uses magnified moments, then footing design must use magnified column moments.

Sliding and Overturning, e_o	Bearing Stress (e_c, s_v)
LL_{min} = 0	LL_{max}
DC_{min}, DW_{min} for resisting forces, DC_{max}, DW_{max} for causing forces,	DC_{max}, DW_{max} for causing forces DC_{min}, DW_{min} for resisting forces
EV_{min}	EV_{max}
EH_{max}	EH_{max}
LS	*LS*

Load Factors
Table 7.7.2-1

7.7.3 Geotechnical Report Summary

The Geotechnical Branch will evaluate overall bridge site stability. Slope stability normally applies to steep embankments at the abutment. If stability is in question, a maximum service limit state load will be specified in the report. Bridge design will determine the maximum total service load applied to the embankment. The total load must be less than the load specified in the geotechnical report.

Based on the foundations required in the Preliminary Plan and structural information available at this stage, the Report provides the following geotechnical engineering results. For all design limit states, the total factored footing load must be less than factored resistance.

A. **Plan Detailing –** The bridge plans shall include the nominal bearing resistance in the General Notes as shown in Figure 7.7.3-1. This information is included in the Plans for future reference by the Bridge and Structures Office.

Nominal Bearing Resistance of the Spread Footings Shall Be Taken As, In KSF:		
Pier No.	Service-I Limit State	Strength and Extreme Event-I Limit States
1	====	====
2	====	====

Figure 7.7.3-1

B. **Bearing Resistance - Service, Strength, and Extreme Event Limit States –** The nominal bearing resistance (q_n) may be increased or reduced based on previous experience for the given soils. The geotechnical report will contain the following information:

- Nominal bearing resistance (q_n) for anticipated effective footing widths, which is the same for the strength and extreme event limit states.
- Service bearing resistance (q_{ser}) and amount of assumed settlement.
- Resistance factors for strength and extreme event limit states (ϕ_b).
- Embedment depth requirements or footing elevations to obtain the recommended q_n.

Spread footings supported on SE walls or geosynthetic walls shall be designed with nominal bearing resistances not to exceed 6.0 ksf at service limit states and 9.0 ksf at strength and extreme event limit states. A vertical settlement monitoring program shall be conducted where nominal bearing resistance exceeds 4.0 ksf at service limit states or 7.0 ksf at strength or extreme event limit states. See GDM Section 15.5.3.5 for additional requirements.

C. **Sliding Resistance - Strength and Extreme Event Limit States –** The geotechnical report will contain the following information to determine earth loads and the factored sliding resistance ($R_R = \phi R_n$)

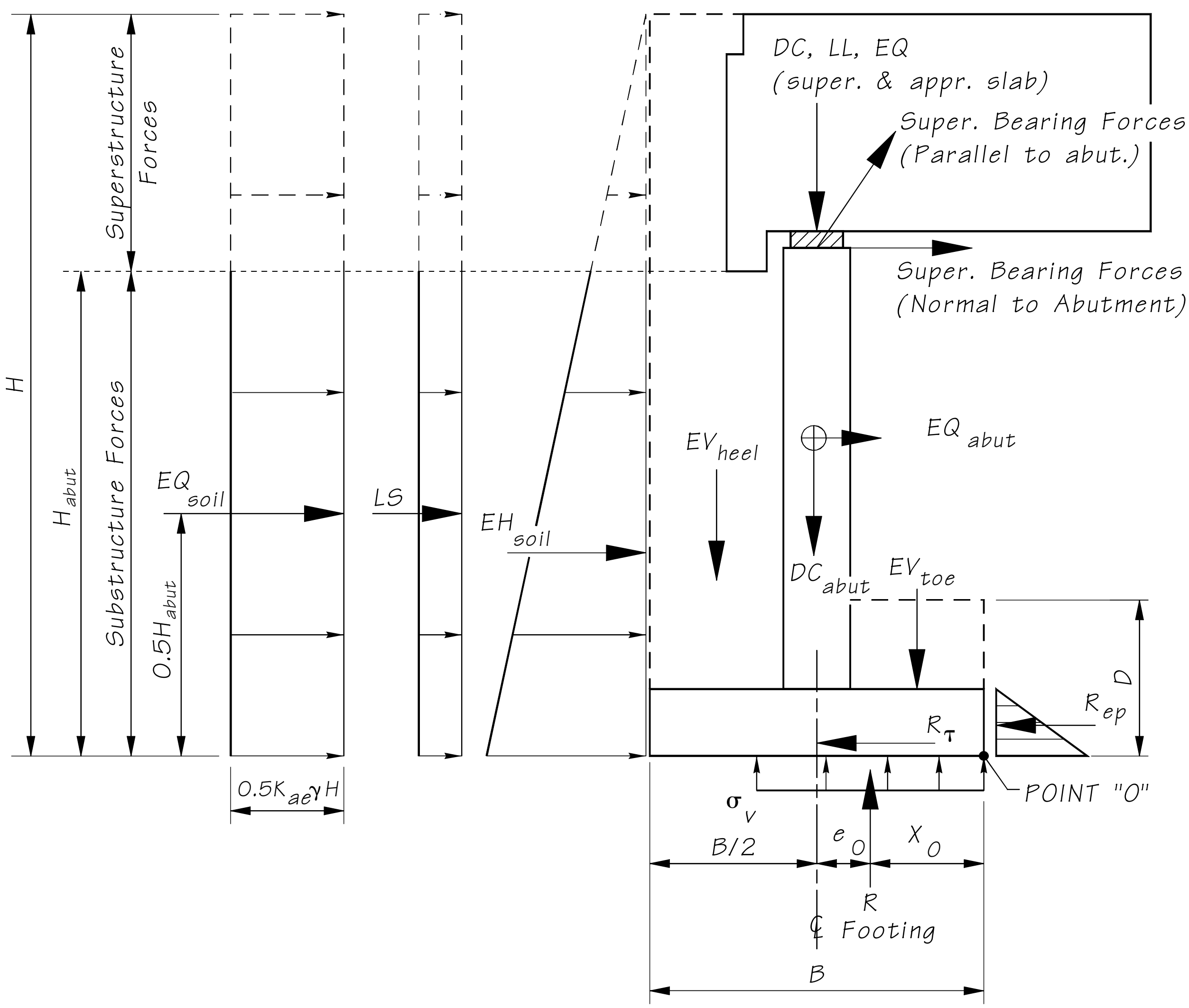
- Resistance factors for strength and extreme event limit states (ϕ_τ, ϕ_{ep})
- If passive earth pressure (R_{ep}) is reliably mobilized on a footing: ϕ_f or S_u and σ'_v, and the depth of soil in front of footing that may be considered to provide passive resistance.

D. **Foundation Springs - Extreme Event Limit States –** When a structural evaluation of soil response is required for a bridge analysis, the Geotechnical Branch will determine foundation soil/rock shear modulus and Poisson's ratio (G and μ). These values will typically be determined for shear strain levels of 2 to 0.2 percent, which are typical strain levels for large magnitude earthquakes.

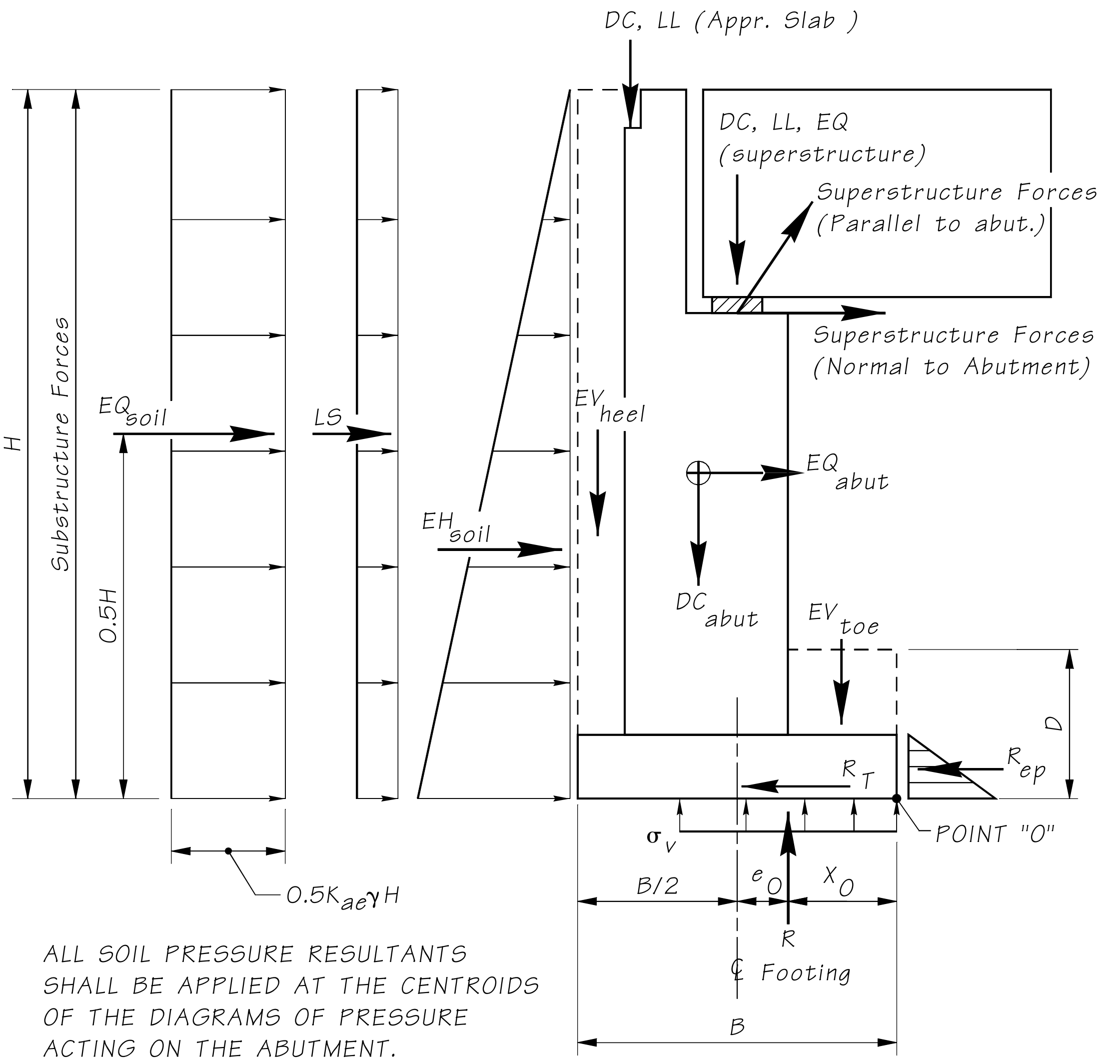
7.7.4 Spread Footing Design

The following section is oriented toward abutment spread footing design. Spread footing designs for intermediate piers or other applications use the same concepts with the appropriate structural analysis. Structural designers should complete all design checks before consulting with the geotechnical engineer about any design problem. There may be several problem criteria that should be addressed in the solution.

A. **Abutment Spread Footing Force Diagram –** Figures 7.7.4-1 and 7.7.4-2 diagram the forces that act on abutment footings. Each limit state design check will require calculation of a reaction (R) and the location (X_o) or eccentricity (e_o). The ultimate soil passive resistance (Q_{ep}) at the toe is determined by the geotechnical engineer and is project specific.



Cantilever (End Diaphragm) Abutment Force Diagram
Figure 7.7.4-1



L-Abutment Force Diagram

Figure 7.7.4-2

B. **Bearing Stress –** For geotechnical and structural footings design, the bearing stress calculation assumes a uniform bearing pressure distribution. For footing designs on rock, the bearing stress is based on a triangular or trapezoidal bearing pressure distribution. The procedure to calculate bearing stress is summarized in the following outline. See Abutment Spread Footing Force Diagrams for typical loads and eccentricity.

Step 1: Calculate the Resultant force (R_{str}), location (Xo_{str}) and eccentricity for Strength (e_{str}).

Xo_{str} = (factored moments about the footing base)/(factored vertical loads)

Step 2A: For Footings on Soil:

Calculate the maximum soil stress (σ_{str}) based on a uniform pressure distribution. Note that this calculation method applies in both directions for biaxially loaded footings. See AASHTO LRFD 10.6.3.1.5 for guidance on biaxial loading. The maximum footing pressure on soil with a uniform distribution is:

$\sigma_{str} = R/B' = R/2Xo = R/(B\text{-}2e)$, where B' is the effective footing width.

Step 2B: For Footings on Rock:

If the reaction is outside the middle ⅓ of the base, use a triangular distribution.

σ_{str} max = $2R/3\ Xo$, where “R” is the factored limit state reaction.

If the reaction is within the middle ⅓ of the base, use a trapezoidal distribution.

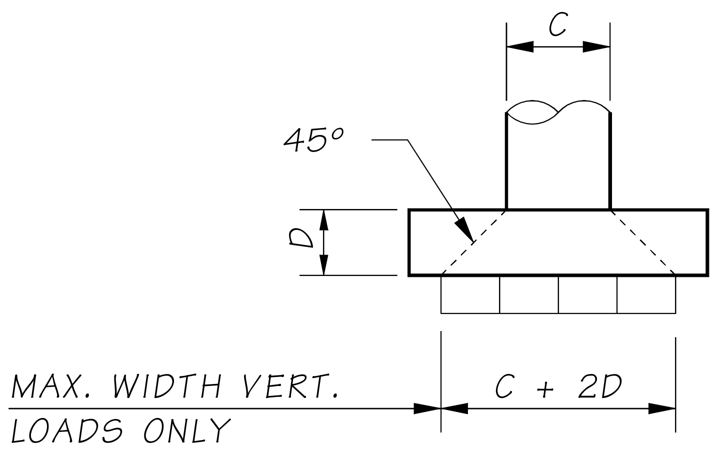
σ_{str} max = $R/B\ (1+ 6\ e/\mathrm{B})$

In addition, WSDOT limits the maximum stress (P/A) applied to rock due to vertical loads only. This is because the rock stiffness approaches infinity relative to the footing concrete. The maximum width of uniform stress is limited to C+2D as shown in Figure 7.7.4-3.

Step 3: Compare the factored bearing stress (σ_{str}) to the factored bearing resistance ($\phi b_c q_n$) of the soil or rock. The factored bearing stress must be less than or equal to the factored bearing resistance.

$\sigma_{str} \leq \phi b_c q_n$

Step 4: Repeat steps 1 thru 3 for the Extreme Event limit state. Calculate Xo_{ext}, e_{ext}, and σ_{ext} using Extreme Event factors and compare the factored stress to the factored bearing ($\phi_b q_n$).



Footings on Rock
Figure 7.7.4-3

C. **Failure By Sliding** – The factored sliding resistance (Q_R) is comprised of a frictional component (φ_τ Q_τ) and the Geotechnical Branch may allow a passive earth pressure component (ϕ_{ep} Q_{ep}). The designer shall calculate Q_R based on the soil properties specified in the geotechnical report. The frictional component acts along the base of the footing, and the passive component acts on the vertical face of a buried footing element. The factored sliding resistance shall be greater than or equal to the factored horizontal applied loads.

$$Q_R = \phi_T Q_T + \phi_{ep} Q_{ep}$$

The Strength Limit State ϕ_τ and ϕ_{ep} are provided in the geotechnical report or AASHTO LRFD 10.5.5.2.2-1. The Extreme Event Limit State Q_τ and ϕ_{ep} are generally equal to 1.0.

Where:

Q_τ	=	(R) tan δ
tan δ	=	Coefficient of friction between the footing base and the soil
tan δ	=	*tan* ϕ for cast-in-place concrete against soil
tan δ	=	(0.8)*tan* ϕ for precast concrete
R	=	Vertical force – Minimum Strength and Extreme Event factors are used to calculate R
ϕ	=	angle of internal friction for soil

D. **Overturning Stability** – Calculate the locations of the overturning reaction (R) for strength and extreme event limit states. Minimum load factors are applied to forces and moments resisting overturning. Maximum load factors are applied to forces and moments causing overturning. Note that for footings subjected to biaxial loading, the following eccentricity requirements apply in both directions.

See AASHTO LRFD 11.6.3.3 (Strength Limit State) and 11.6.5 (Extreme Event Limit State) for the appropriate requirements for the location of the overturning reaction (R).

E. **Footing Settlement** – The service limit state bearing resistance (q_{ser}) will be a settlement-limited value, typically 1″.

Bearing Stress = $\sigma_{ser} < \phi q_{ser}$ = Factored nominal bearing

Where, q_{ser} is the unfactored service limit state bearing resistance and ϕ is the service resistance factor. In general, the resistance factor (ϕ) shall be equal to 1.0.

For immediate settlement (not time dependent), both permanent dead load and live load should be considered for sizing footings for the service limit state. For long-term settlement (on clays), only the permanent dead loads should be considered.

If the structural analysis yields a bearing stress (σ_{ser}) greater than the bearing resistance, then the footing must be re-evaluated. The first step would be to increase the footing size to meet bearing resistance. If this leads to a solution, recheck layout criteria and inform the geotechnical engineer the footing size has increased. If the footing size cannot be increased, consult the geotechnical engineer for other solutions.

F. **Concrete Design –** Footing design shall be in accordance with AASHTO LRFD 5.13.3 for footings and the general concrete design of AASHTO LRFD Chapter 5. The following Figure 7.7.4-4 illustrates the modes of failure checked in the footing concrete design.

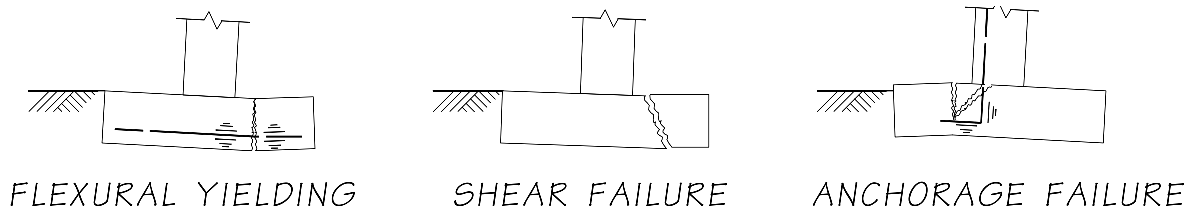


Figure 7.7.4-4

1. **Footing Thickness and Shear** – The minimum footing thickness shall be 1′-0″. The minimum plan dimension shall be 4′-0″. Footing thickness may be governed by the development length of the column dowels, or by concrete shear requirements (with or without reinforcement). If concrete shear governs the thickness, it is the engineer's judgment, based on economics, as to whether to use a thick footing unreinforced for shear or a thinner footing with shear reinforcement. Generally, shear reinforcement should be avoided but not at excessive cost in concrete, excavation, and shoring requirements. Where stirrups are required, place the first stirrup at $d/2$ from the face of the column or pedestal. For large footings, consider discontinuing the stirrups at the point where $vu = vc$.

2. **Footing Force Distribution** – The maximum shear stress in the footing concrete shall be determined based on a triangular or trapezoidal bearing pressure distribution, see AASHTO LRFD 5.13.3.6. This is the same pressure distribution as for footing on rock, see Section 7.7.4B.

3. **Vertical Reinforcement (Column or Wall)** – Vertical reinforcement shall be developed into the footing to adequately transfer loads to the footing. Vertical rebar shall be bent 90° and extend to the top of the bottom mat of footing reinforcement. This facilitates placement and minimizes footing thickness. Bars in tension shall be developed using 1.25 *Ld*. Bars in compression shall develop a length of 1.25 *Ld*, prior to the bend. Where bars are not fully stressed, lengths may be reduced in proportion, but shall not be less than ¾ *Ld*.

   The concrete strength used to compute development length of the bar in the footing shall be the strength of the concrete in the footing. The concrete strength to be used to compute the section strength at the interface between footing and a column concrete shall be that of the column concrete. This is allowed because of the confinement effect of the wider footing.

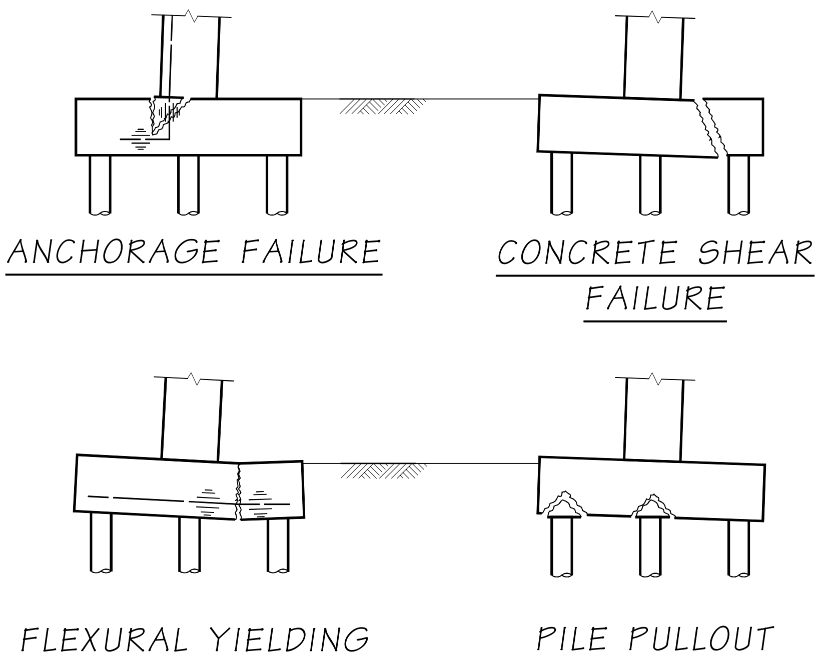
4. **Bottom Reinforcement** – Concrete design shall be in accordance with AASHTO LRFD Specifications. Reinforcement shall not be less than #6 bars at 12″ centers to account for uneven soil conditions and shrinkage stresses.

5. **Top Reinforcement** – Top reinforcement shall be used in any case where tension forces in the top of the footing are developed. Where columns and bearing walls are connected to the superstructure, sufficient reinforcement shall be provided in the tops of footings to carry the weight of the footing and overburden assuming zero pressure under the footing. This is the uplift earthquake condition described under "Superstructure Loads." This assumes that the strength of the connection to the superstructure will carry such load. Where the connection to the superstructure will not support the weight of the substructure and overburden, the strength of the connection may be used as the limiting value for determining top reinforcement. For these conditions, the AASHTO LRFD requirement for minimum percentage of reinforcement will be waived. Regardless of whether or not the columns and bearing walls are connected to the superstructure, a mat of reinforcement shall normally be provided at the tops of footings. On short stub abutment walls (4′ from girder seat to top of footing), these bars may be omitted. In this case, any tension at the top of the footing, due to the weight of the small overburden, must be taken by the concrete in tension.

Top reinforcement for column or bearing wall footings designed for two-way action shall not be less than #6 bars at 12″ centers, in each direction while top reinforcement for bearing wall footings designed for one-way action shall not be less than #5 bars at 12″ centers in each direction.

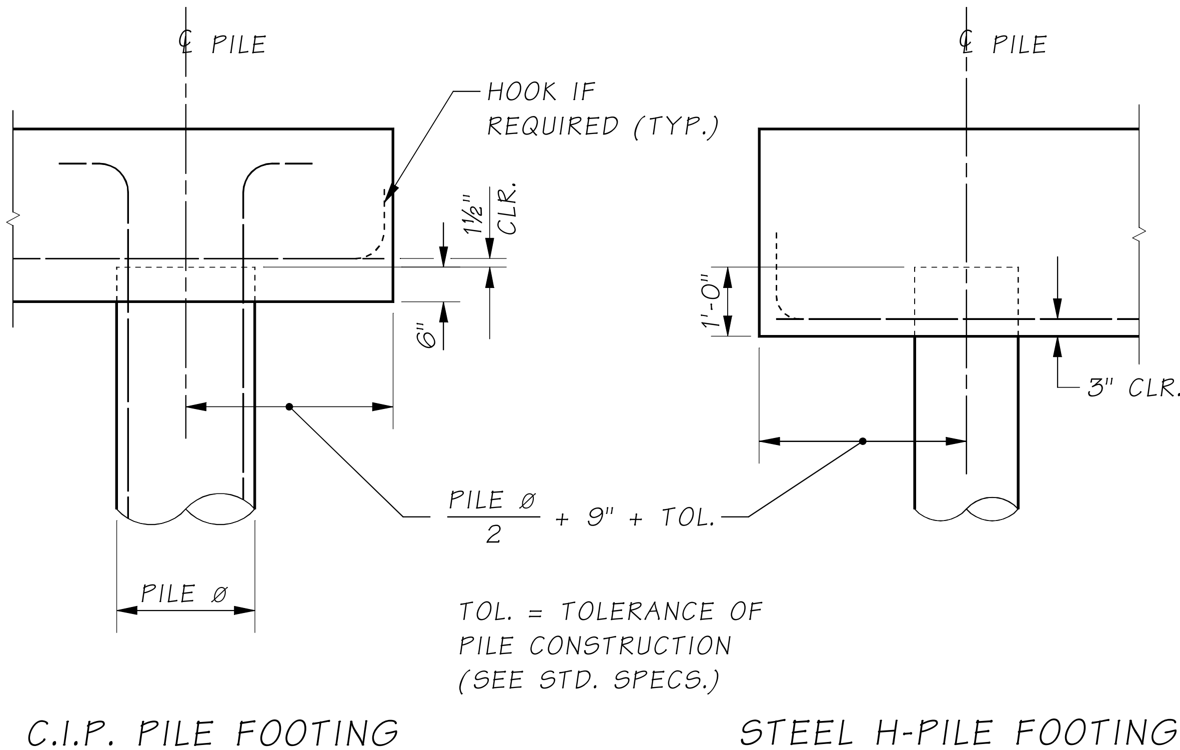
7.7.5 Pile-Supported Footing Design

The minimum footing thickness shall be 2′-0″. The minimum plan dimension shall be 4′-0″. Footing thickness may be governed by the development length of the column dowels, or by concrete shear requirements. The use of strut and tie modeling is recommended for the design of all pile caps and pile footings. Figure 7.7.5-1 identifies the modes of failure that should be investigated for general pile cap/ footing design.



Pile Footing Modes of Failure
Figure 7.7.5-1

A. **Pile Embedment, Clearance, and Rebar Mat Location –** All piles shall have an embedment in the concrete sufficient to resist moment, shear, and axial loads. Cast-in-place concrete piles with reinforcing extending into footings are embedded a minimum of 6″. The clearance for the bottom mat of footing reinforcement shall be 1½″ between the reinforcing and the top of the pile for CIP pile footings. See Figure 7.7.5-2 for the minimum pile clearance to the edge of footing.



Pile Embedment and Reinforcing Placement
Figure 7.7.5-2

B. **Concrete Design –** In determining the proportion of pile load to be used for calculation of shear stress on the footing, any pile with its center 6″ or more outside the critical section shall be taken as fully acting on that section. Any pile with its center 6″ or more inside the critical section shall be taken as not acting for that section. For locations in between, the pile load acting shall be proportioned between these two extremes. The critical section shall be taken as the effective shear depth (d_v) as defined in AASHTO LRFD 5.8.2.9. The distance from the column/wall face to the allowable construction centerline of pile (design location plus or minus the tolerance) shall be used to determine the design moment of the footing. The strut and tie design method should be used where appropriate.

7.8 Drilled Shafts

7.8.1 Axial Resistance

The factored axial resistance of the drilled shaft (R) is generally composed of two parts: the nominal end bearing (R_p) and the nominal skin friction (R_s). The general formula is as follows, where φ is the limit state resistance factor.

$$R = \varphi_p R_p + \varphi_s R_s \quad (7.8.1\text{-}1)$$

The total factored shaft loading must be less than the factored axial resistance. R_p and R_s are treated as independent quantities although research has shown that the end bearing and skin friction resistance have some interdependence. R_p and R_s shown as a function of depth will be stated in the geotechnical report for the bridge.

The designer shall consider all applicable factored load combination limit states and shaft resistances when determining shaft axial capacity and demand and shaft tip elevations. For some shaft designs, liquefiable soils, scour conditions and/or downdrag forces may need to be considered. Determining which limit states to include these conditions or forces can be complex. The Hydraulics Branch and the geotechnical engineer shall be consulted to ensure overly and/or under conservative load combinations and resistances are not being considered. Open and frequent communication is essential during design.

Although the AASHTO LRFD Specifications include water loads, *WA*, in Extreme Event I limit states, in most cases the loss of soil resistance due to scour conditions is not combined with Extreme Event I load combinations. The probability of a design earthquake occurring in the presence of the maximum scour event is low. However, in some instances it is appropriate to include some scour effects. When scour is included with Extreme Event I load combinations, the skin resistance of the soil, up to a maximum of 25 percent of the scour depth for the design flood (l00 year event), shall be deducted from the resistance of the shaft. The loss of skin resistance for the full scour depth for the design flood shall be considered when checking axial capacity of the shaft for all strength and service limit states. The loss of skin resistance for the full scour depth of the check flood (500 year event) shall be considered when checking the axial capacity of the shaft for Extreme Event II limit states. It should be noted that scour does not produce a load effect on the structure but changes the geometry of the bridge pier and available soil resistance so that effects of other loads are amplified. The engineer may also need to consider scour effects on piers that are currently outside of the ordinary high water zones due to potential migration of rivers or streams during flood events. The Hydraulics Branch will provide guidance for these rare cases.

Downdrag forces may also need to be considered in some designs. Downdrag forces are most often caused by the placement of fill adjacent to shafts, which causes consolidation and settlement of underlying soils. This situation is applicable to service and strength limit states. Downdrag forces can also be caused by liquefaction-induced settlement caused by a seismic event. Pore water pressure builds up in liquefiable soils during ground shaking. And as pore water pressure dissipates, the soil layer(s) may settle, causing downdrag forces on the shaft to develop. These liquefaction induced downdrag forces are only considered in the Extreme Event I limit state. However, downdrag induced by consolidation settlement is never combined with downdrag forces induced by liquefaction, but are only considered separately in their applicable limit states.

The downdrag is treated as a load applied to the shaft foundations. The settling soil, whether it is caused by consolidation under soil stresses (caused, for example, by the placement of fill), or caused by liquefaction, creates a downward acting shear force on the foundations. This shear force is essentially the skin friction acting on the shaft, but reversed in direction by the settlement. This means that the skin friction along the length of the shaft within the zone of soil that is contributing to downdrag is no longer available for resisting downward axial forces and must **not** be included with the soil resistance available to resist the total downward axial (i.e., compression) loads acting on the foundation.

In general, the geotechnical engineer will provide drilled shaft soil resistance plots as a function of depth that includes skin friction along the full length of the shaft. Therefore, when using those plots to estimate the shaft foundation depth required to resist the axial compressive foundation loads, this “skin friction lost” due to downdrag must be subtracted from the resistance indicated in the geotechnical shaft resistance plots, and the downdrag load per shaft must be added to the other axial compression loads acting on the shaft.

Similarly, if scour is an issue that must be considered in the design of the foundation, with regard to axial resistance (both in compression and in uplift), the skin friction lost due to removal of the soil within the scour depth must be subtracted from the shaft axial resistance plots provided by the geotechnical engineer. If there is any doubt as to whether or not this skin friction lost must be subtracted from the shaft resistance plots, it is important to contact the geotechnical engineer for clarification on this issue. Note that if both scour and downdrag forces must be considered, it is likely that the downdrag forces will be reduced by the scour. This needs to be considered when considering combination of these two conditions, and assistance from the geotechnical engineer should be obtained.

The WSDOT *Geotechnical Design Manual* M 46-03, Chapters 6, 8, and 23, should be consulted for additional explanation regarding these issues.

Following is a summary of potential load combination limit states that shall be checked if scour effects, liquefiable soils and/or downdrag forces are included in the design. The geotechnical report will provide the appropriate resistance factors to use with each limit state.

A. **Condition** – Embankment downdrag from fill or the presence of compressible material below the foundations; no liquefaction.

   Checks:

   1. Include embankment induced downdrag loads with all Strength and Service Limit States. Do not include with Extreme Limit States. Use maximum load factor unless checking an uplift case, where the minimum shall be used. Subtract the skin friction lost within the downdrag zone from the shaft axial resistance plots provided by the geotechnical engineer.

B. **Condition** – Liquefiable soils with post-earthquake downdrag forces. No embankment downdrag. ***Note:*** If embankment downdrag is present, it shall not be included with liquefaction-induced downdrag therefore it would not be included in Check 3 below.

   Checks:

   1. Extreme Event I Limit State – Use static soil resistances (no loss of resistance due to liquefaction) and no downdrag forces. Use a live load factor of 0.5.
   2. Extreme Event I Limit State – Use reduced soil resistance due to liquefaction and no downdrag forces. Use a live load factor of 0.5. The soils in the liquefied zone will not provide the static skin friction resistance but will in most cases have a reduced resistance that will be provided by the geotechnical engineer.
   3. Extreme Event I Limit State – Post liquefaction. Include downdrag forces, a live load factor of 0.5 and a reduced post-liquefaction soil resistance provided by the geotechnical engineer. Do not include seismic inertia forces from the structure since it is a post earthquake check. There will be no skin resistance in the post earthquake liquefied zone. Therefore, subtract the skin friction lost within the downdrag zone from the shaft axial resistance plots provided by the geotechnical engineer.

C. **Condition** – Scour from design flood (100 year events) and check floods (500 year events.) The shaft shall be designed so that shaft penetration below the scour of the applicable flood event provides enough axial resistance to satisfy demands. Since in general the geotechnical engineer will provide shaft resistance plots that include the skin friction within the scour zone, the skin friction lost will need to be subtracted from the axial resistance plots provided to determine the shaft resistance acting below the scour depth. A special case would include scour with Extreme Event I limit states without liquefiable soils and downdrag. It is overly conservative to include liquefied soil induced downdrag and scour with the Extreme Event I limit states. The Hydraulics Branch and the geotechnical engineer will need to be consulted for this special case.

Checks:

1. Service and Strength Limit States – Subtract the skin friction lost within the scour depth (i.e., 100 percent of the scour depth for the 100 year design flood) from the shaft axial resistance plots provided by the geotechnical engineer, to estimate the shaft depth required to resist all service and strength limit demands.
2. Extreme Event II Limit State – Subtract the skin friction lost within the scour depth (i.e., 100 percent of the scour depth for the 500 year check flood event) from the shaft axial resistance plots provided by the geotechnical engineer, to estimate the shaft depth required to resist all Extreme Event II limit demands. Use a live load factor of 0.5. Do not include ice load, *IC*, vessel collision force, *CV*, and vehicular collision force, *CT*.
3. Extreme Event II Limit State – Subtract the skin friction lost within the scour depth (in this case only 50 percent of the scour depth for the 500 year check flood event) from the shaft axial resistance plots provided by the geotechnical engineer, to estimate the shaft depth required to resist all Extreme Event II limit demands. Use a live load factor of 0.5. In this case, include ice load, *IC*, vessel collision force, *CV*, and vehicular collision force, *CT*.
4. Extreme Event I Limit State (special case - no liquefaction) – Subtract the skin friction lost within the scour depth (i.e., in this case 25 percent of the scour depth for the 100 year design flood) from the shaft axial resistance plots provided by the geotechnical engineer, to estimate the shaft depth required to resist the Extreme Event I limit state demands.

The bridge plans shall include the end bearing and skin friction nominal shaft resistance for the service, strength, and extreme event limit states in the General Notes, as shown in Figure 7.8-1-1. The nominal shaft resistances presented in Figure 7.8-1-1 are not factored by resistance factors.

The Nominal Shaft Resistance shall be taken as, in kips:

Service-I Limit State		
Pier No.	**Skin Friction Resistance**	**End Bearing Resistance**
1	====	====
2	====	====

Strength Limit State		
Pier No.	**Skin Friction Resistance**	**End Bearing Resistance**
1	====	====
2	====	====

Extreme Event-I Limit State		
Pier No.	**Skin Friction Resistance**	**End Bearing Resistance**
1	====	====
2	====	====

Figure 7.8.1-1

7.8.1.1 Axial Resistance Group Reduction Factors

The group reduction factors for axial resistance of shafts for the strength and extreme event limit states shall be taken as shown in Table 7.8.1.1-1 unless otherwise specified by the geotechnical engineer. These reduction factors presume that good shaft installation practices are used to minimize or eliminate the relaxation of the soil between shafts and caving. If this cannot be adequately controlled due to difficult soils conditions or for other constructability reasons, lower group reduction factors shall be used as recommended by the geotechnical engineer. Alternatively, steps could be required during and/or after shaft construction to restore the soil to its original condition. The geotechnical engineer will provide these recommendations, which could include but is not limited to, pressure grouting of the tip, grouting along side of the shaft or full length casing.

Soil Type	Shaft Group Configuration	Shaft Center-to-Center Spacing	Special Conditions	Group Reduction factor, η
Cohesionless (Sands, gravels, etc.)	Single row	2*D*		0.90
		2.5*D*		0.95
		3*D* or more		1.0
	Multiple row	2.5*D**		0.67
		3*D*		0.80
		4*D* or more		1.0
	Single and multiple rows	2*D* or more	Shaft group cap in intimate contact with ground consisting of medium-dense or denser soil	1.0
	Single and multiple rows	2*D* or more	Full depth casing is used and augering ahead of the casing is not allowed, or pressure grouting is used along the shaft sides to restore lateral stress losses caused by shaft installation, and the shaft tip is pressure grouted	1.0
Cohesive (Clays, clayey sands, and glacially overridden, well-graded soils such as glacial till)	Single or multiple rows	2*D* or more		1.0

*Minimum spacing for multiple row configurations.

Group Reduction Factors for Axial Resistance of Shafts
Table 7.8.1.1-1

These group reduction factors apply to both strength and extreme event limit states. For the service limit state the influence of the group on settlement as required in the AASHTO LRFD Specifications and the WSDOT *Geotechnical Design Manual* M 46-03 are still applicable.

7.8.2 Structural Design and Detailing

Section 6-19 of the *Standard Specifications* should be reviewed as part of the design of drilled shafts. The structural design of drilled shafts is similar to column design. The following guidelines shall be followed:

A. Drilled shafts shall be designed for the lesser of the plastic forces or 1.2 times the elastic seismic forces of the column above in single column/single shaft foundations. This applies to all Seismic Design Categories (SDCs) in Washington State.

B. Concrete Class 4000P shall be specified for the entire length of the shaft for wet or dry conditions of placement.

C. When shafts are constructed in water, the concrete specified for the casing shoring seal shall be Class 4000W.

D. The assumed concrete compressive strength shall be $0.85f'_c$ for structural design of shafts. Most shafts in the State are constructed with the wet method using slurries to stabilize caving soils. A reduction in concrete strength is used to account for the unknown shaft concrete quality that results.

E. The presence of permanent steel casing shall be taken into account in the shaft design (i.e. for stiffness, and etc.), but the structural capacity of permanent steel casing shall not be considered for structural design of drilled shafts.

F. Cover requirements vary depending on the drilled shaft diameter and shall be as specified below:

   - Diameter less than or equal to 3′-0″ = 3″
   - Diameter greater than 3′-0″ and less than 5′-0” = 4″
   - Diameter greater than or equal to 5′-0″ = 6″

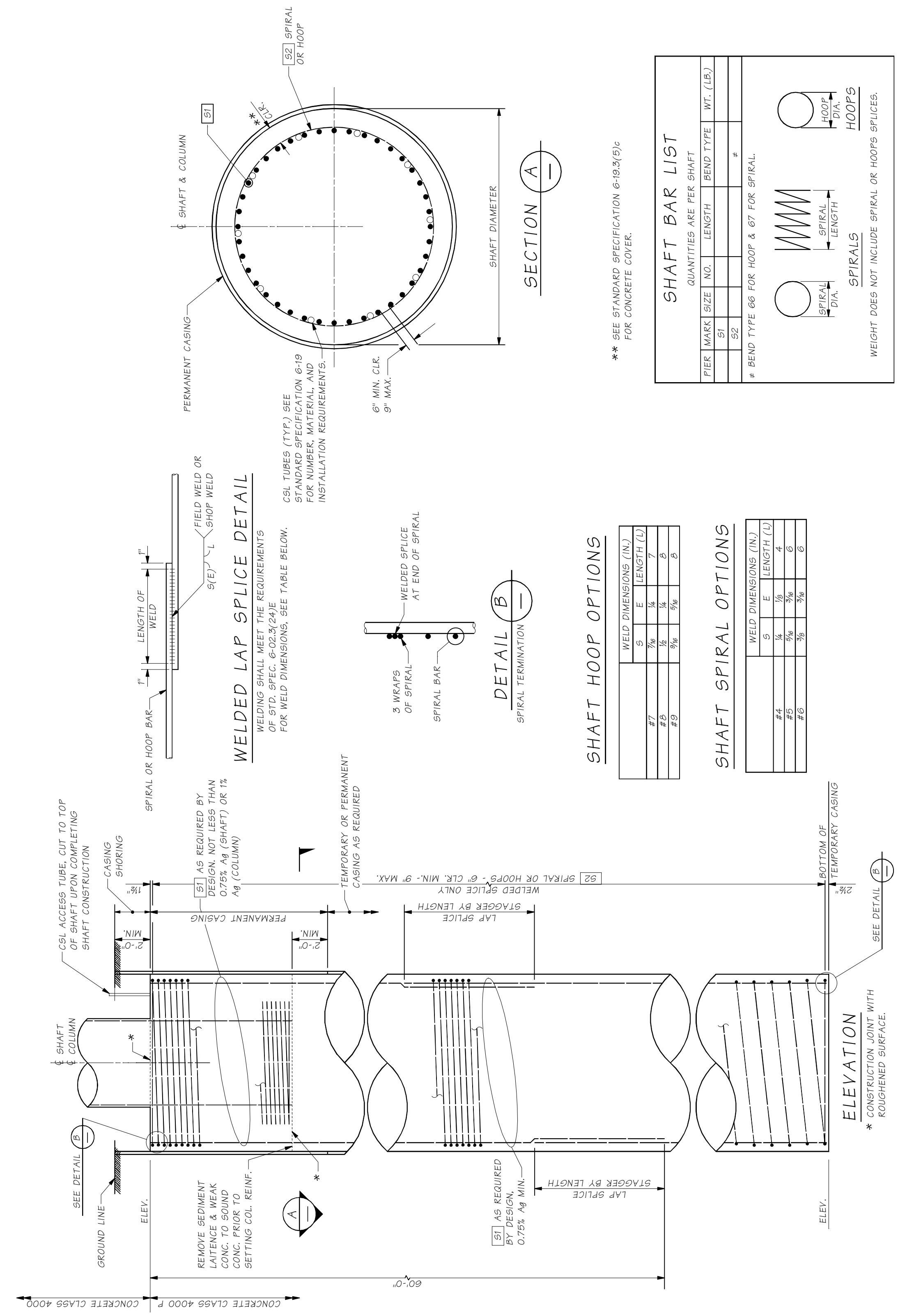
   Section 6-19 of the *Standard Specifications* lists exceptions to these cover requirements when permanent slip casings are used in column splice zones.

G. In general, drilled shaft reinforcing shall be detailed to minimize congestion, facilitate concrete placement by tremie, and maximize consolidation of concrete.

H. The clear spacing between spirals and hoops shall not be less than 6″ or more than 9″, with the following exception. The clear spacing between spirals or hoops may be reduced in the splice zone in single column/single shaft connections because shaft concrete may be vibrated in this area, negating the need for larger openings to facilitate good flow of concrete through the reinforcing cage.

I. The volumetric ratio and spacing requirements of the AASHTO Seismic Specifications for confinement need not be met. The top of shafts in typical WSDOT single column/single shaft connections remains elastic under seismic loads due to the larger shaft diameter (as compared to the column). Therefore this requirement does not need to be met.

J. Shaft transverse reinforcement may be constructed as hoops or spirals. Spiral reinforcement is preferred for shaft transverse reinforcement. However, if #6 spirals at 6″ (excluding the exception in 7.8.2.H) clear do not satisfy the shear design, circular hoops may be used. Circular hoops in shafts up to #9 bars may be lap spliced using the details as shown in Figure 7.8.2-1. ***Note:*** Welded lap splices for spirals are currently acceptable under the AWS D1.4 up to bar size #6. Recent testing has been performed by WSDOT for bar sizes #7 through #9. All tests achieved full tensile capacity (including 125 percent of yield strength.) Therefore, #7 through #9 welded lap spliced hoops are acceptable to use provided they are not located in possible plastic hinge regions. Circular hoops may also be fabricated using a manual direct butt weld, resistance butt weld, or mechanical coupler. Weld splicing of hoops for shafts shall be completed prior to assembly of the shaft steel reinforcing cage. Refer to Section 7.4.5 of this manual for additional discussion on circular hoops. Mechanical couplers may be considered provided cover and clearance requirements are accounted for in the shaft details. When welded hoops or mechanical couplers are used, the plans shall show a staggered pattern around the perimeter of the shaft so that no two adjacent welded splices or couplers are located at the same location.



Typical Drilled Shaft Details
Figure 7.8.2-1

K. In single column/single shaft configurations, the spacing of the shaft transverse reinforcement in the splice zone shall meet the requirements of the following equation, which comes from the TRAC Report titled, "NONCONTACT LAP SPLICES IN BRIDGE COLUMN-SHAFT CONNECTIONS":

$$S_{max} = \frac{2\pi A_{sh}\, f_{ytr} l_s}{k A_l\, f_{ul}} \tag{7.8.2-1}$$

Where:

S_{max} = Spacing of transverse shaft reinforcement
A_{sh} = Area of shaft spiral or transverse reinforcement
f_{ytr} = Yield strength of shaft transverse reinforcement
l_s = Standard splice length of the column reinforcement
A_l = Area of longitudinal column reinforcement
f_{ul} = Specified minimum tensile strength of column longitudinal reinforcement (ksi), 90 ksi for A615 and 80 ksi for A706
k = Factor representing the ratio of column tensile reinforcement to total column reinforcement at the nominal resistance. This ratio could be determined from the column moment-curvature analysis using computer programs Xtract or SAP 2000. To simplify this process, $k = 0.5$ could safely be used in most applications

L. Longitudinal reinforcement shall be provided for the full length of drilled shafts. The minimum longitudinal reinforcement in the splice zone of single column/single shaft connections shall be the larger of 0.75 percent A_g of the shaft or 1.0% A_g of the attached column. The minimum longitudinal reinforcement beyond the splice zone shall be 0.75% A_g of the shaft. The minimum longitudinal reinforcement in shafts without single column/single shaft connections shall be 0.75% A_g of the shaft.

M. The clear spacing between longitudinal reinforcement shall not be less than 6″ or more than 9″. If a shaft design is unable to meet this minimum requirement, a larger diameter shaft shall be considered.

N. Longitudinal reinforcing in drilled shafts should be straight with no hooks to facilitate concrete placement and removal of casing. If hooks are necessary to develop moment at the top of a drilled shaft (in a shaft cap situation) the hooks should be turned toward the center of the shaft while leaving enough opening to allow concrete placement with a tremie.

O. Locations of longitudinal splices shall be shown in the contract plans. Mechanical splices shall be staggered 2′-0″.

P. Use of two concentric circular rebar cages shall be avoided.

Q. Resistance factors for Strength Limit States shall be per the latest AASHTO LRFD Specifications. Resistance factors for Extreme Event Limit States shall be per the latest AASHTO Seismic Specifications. The resistance factor for shear shall conform to the AASHTO LRFD Specifications.

R. The axial load along the shaft varies due to the side friction. It is considered conservative, however, to design the shaft for the full axial load plus the maximum moment. The entire shaft normally is then reinforced for this axial load and moment.

S. Access tubes for Crosshole Sonic Log (CSL) testing shall be provided in all shafts. One tube shall be furnished and installed for each foot of shaft diameter, rounded to the nearest whole number, and shown in the plans. The number of access tubes for shaft diameters specified as "X feet 6 inches" shall be rounded up to the next higher whole number. The access tubes shall be placed around the shaft, inside the spiral or hoop reinforcement and three inches clear of the vertical reinforcement, at a uniform spacing measured along the circle passing through the centers of the access tubes. If the vertical reinforcement is not bundled and each bar is not more than one inch in diameter, the access tubes shall be placed two inches clear of the vertical reinforcement. If these minimums cannot be met due to close spacing of the vertical reinforcement, then access tubes shall be bundled with the vertical reinforcement.

T. Shafts shall be specified in English dimensions and shall be specified in sizes that do not preclude any drilling method. Shafts shall be specified in whole foot increments except as allowed here. The tolerances in *Standard Specifications* Section 6-19 accommodate metric casing sizes and/or oversized English casing sizes. Oversized English casings are often used so that tooling for drilling the shafts, which are the nominal English diameter, will fit inside the casing. There are a few exceptions, which will be discussed below. See Table 7.8.2-1 for casing sizes and tolerances.

Column A	Column B	Column C	Column D	Column E	Column F	Column G	Column H
Nominal (Outside) English Casing Diameter		***Maximum Increase in Casing Inside Diameter**	***Maximum Decrease in Casing Inside Diameter**	**Maximum English Casing Diameter**	**Nominal (Outside) Metric Casing Diameter**		
Feet	Inches	Inches	Inches	Inches	Meters	Feet	Inches
12.0	144	6	0	150	3.73	12.24	146.85
11.0	132	6	0	138	3.43	11.25	135.0
10.0	120	6	2	126	3.00	9.84#	118.11
9.5	114	6	0	120	3.00	9.84	118.11
9.0	108	6	0	114	2.80	9.19	110.23
8.0	96	6	0	102	2.50	8.20	98.42
7.0	84	6	0	90	2.20	7.22	86.61
6.5	78	6	0	84	2.00	6.56	78.74
6.0	72	6.75##	0	78	2.00	6.56	78.74
5.5	66	6	0	72			
5.0	60	12	1	72	1.5	4.92#	59.05
4.5	54	12	0	66	1.50	4.92	59.05
4.0 **	48	12	0	60	1.5	4.92	59.05
4.0**	48	12	1	60	1.2	3.94#	47.28
3.0	36	12	0	48	1.00	3.28	39.37
3.0	36	12	0	48	0.915	3.00	36.02
2.5	30	12	0	42			
2.0	24	12	0	36	0.70	2.30	27.56

*Check *Standard Specifications* Section 6-19.

**Construction tolerances would allow either 1.2 or 1.5 meter casing to be used.

Designer shall check that undersize shaft meets the design demands.

Exception to typical construction tolerance of 6″.

Table 7.8.2-1

As seen in Table 7.8.2-1, construction tolerances shown in Column “C” allow shaft diameters to be increased up to 12″ for shafts 5′-0″ diameter or less and increased up to 6″ for shafts greater than 5′-0″ in diameter. In most cases these construction tolerances allow either metric or English casings to be used for installation of the shafts.

There are a few exceptions to these typical tolerances. These exceptions are as follows:

1. 4.0′ Diameter Shafts – The tolerances in Columns "C" and "D" of Table 7.8.2-1 allow either an oversized 4.92′ diameter shaft or an undersized 3.94′ shaft to be constructed. The reinforcement cage shall be sized to provide a minimum of 3″ of cover to the undersized diameter.
2. 5.0′ Diameter Shafts – The tolerances in Columns "C" and "D" of Table 7.8.2-1 allow either an oversized 6.0′ diameter shaft or an undersized 4.92′ diameter shaft to be constructed. The reinforcement cage shall be sized to provide a minimum of 4″ of cover to the undersized diameter.
3. 6.0′ Diameter Shafts – Maximum oversize tolerance of 6 ¾″ is allowed.
4. 10.0′ Diameter Shafts – The tolerances in Columns "C" and "D" of Table 7.8.2-1 allow either an oversized 10.5′ diameter shaft or an undersized 9.84′ diameter shaft to be constructed. The reinforcement cage shall be sized to provide a minimum of 4″ of cover to the undersized diameter.

For all shaft diameters, the designer should bracket the design so that all possible shaft diameters, when considering the construction tolerances, will satisfy the design demands. The minimum shaft diameter (nominal or undersized) shall be used for design of the flexural and shear reinforcement.

The nominal English shaft diameter shall be specified on the plans. When requesting shaft capacity charts from the geotechnical engineer, the designer should request charts for the nominal English shaft diameter.

U. Shafts supporting a single column shall be sized to allow for construction tolerances, as illustrated in Figure 7.8.2-2.

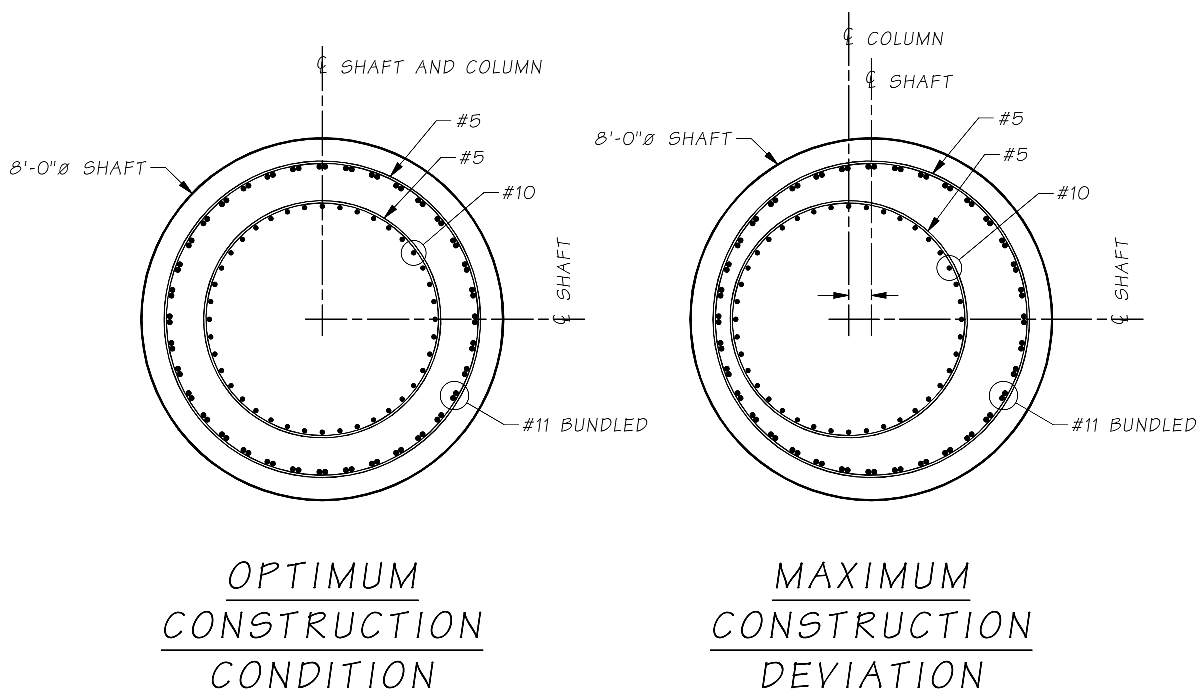


Figure 7.8.2-2

The shaft diameter shall be based on the maximum column diameter allowed by the following equation,

Maximum Column Diameter = Shaft Diameter – 2*(Shaft Concrete Cover) – 2*(Shaft Horizontal Construction Tolerance) – 2*(Shaft Cage Thickness)

The shaft horizontal construction tolerance and shaft concrete cover shall conform to *Standard Specifications* Section 6-19.

If the column diameter used in design is larger than the maximum allowed for a given shaft size, as defined by the equation above, a larger shaft diameter shall be used.

The shaft diameter specified here should not be confused with the desirable casing shoring diameter discussed below.

V. Casing shoring shall be provided for all shafts below grade or waterline. However, casing shoring requirements are different for shafts in shallow excavations and deep excavations. Shafts in deep excavations require a larger diameter casing shoring to allow access to the top of the shaft for column form placement and removal. The top of shafts in shallow excavations (approximately 4′ or less) can be accessed from the ground line above, by reaching in or by “glory-holing”, and therefore do not require larger diameter casing shoring. See Figure 7.8.2-3.

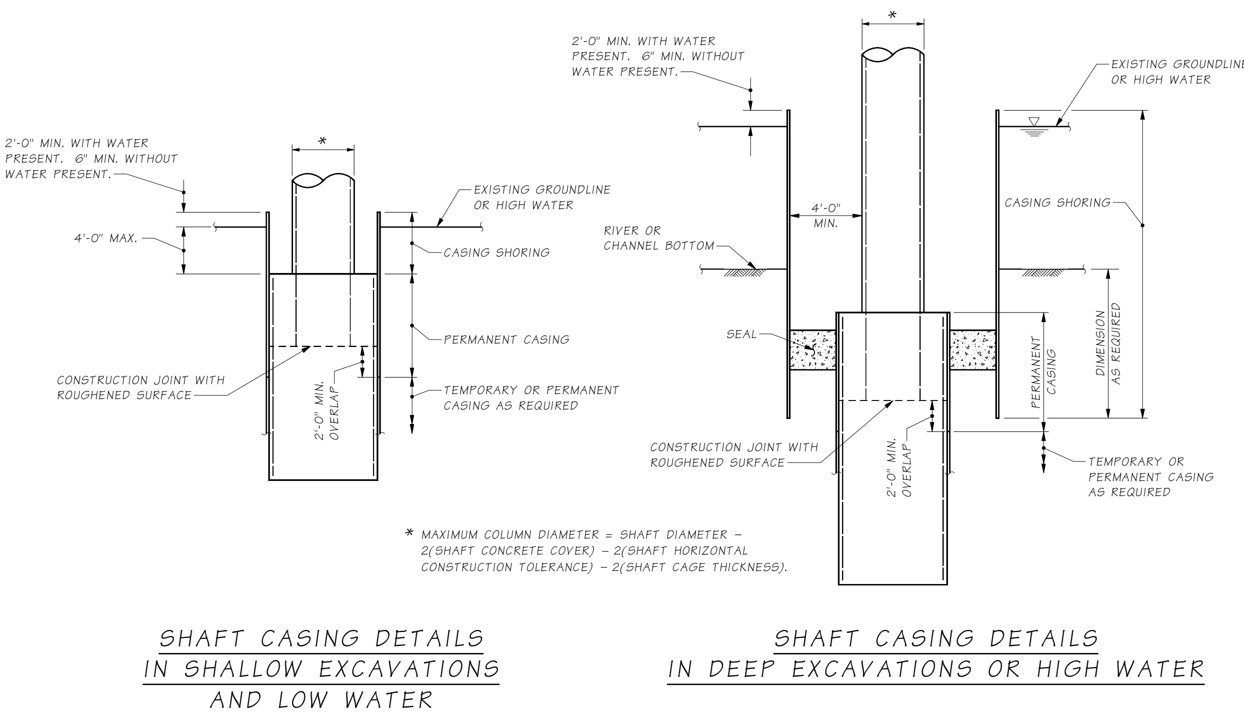
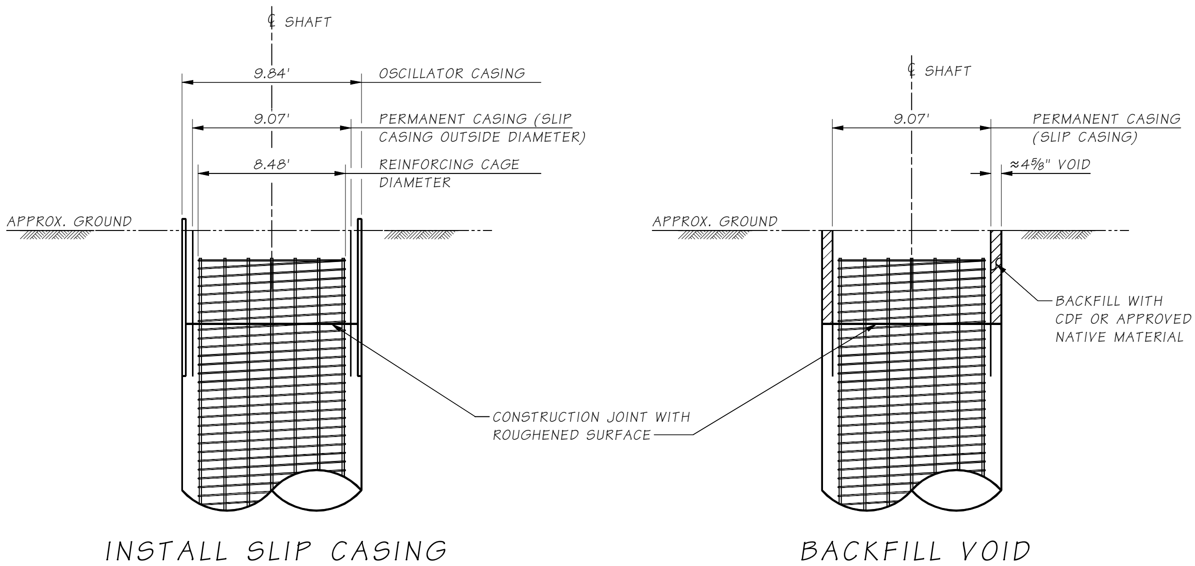


Figure 7.8.2-3

W. Changes in shaft diameters due to construction tolerances shall not result in a reinforcing steel cage diameter different from the diameter shown in the plans (plan shaft diameter minus concrete cover). For example, metric casing diameters used in lieu of English casing diameters shall only result in an increase in concrete cover, except as noted below for single column/single shaft connections requiring slip casings. There are also exceptions for 4′-0″, 5′-0″, and 10′-0″ diameter shafts, see Table 7.8.2-1.

X. Rotator and Oscillator drilling methods typically use a slip casing for permanent casing in single column/single shaft connections, as shown in Figure 7.8.2-4.



10'-0" Ø Shaft Constructed With The Oscillator Method
Figure 7.8.2-4

The use of the slip casing typically requires a modification to the reinforcing cage diameter. This should be considered during the structural design of the shaft. The slip casing also results in less concrete cover than the area of the shaft below the slip casing. See Table 7.8.2-2 for expected reinforcing cage diameters and clear cover. Shafts shall be designed such that the reduced concrete cover is acceptable in this area because the casing is permanent. A minimum of 3″ of concrete cover is achievable in this area for shafts 4′-0″diameter and larger and 1½″of cover for shafts less than 4′-0″. These concrete cover requirements shall be kept as a minimum requirement. The reduction in strength (compared to the area below the slip casing) associated with the reduced shaft diameter that results from the slip casing is bounded within the shaft analysis and design methods prescribed here and elsewhere. Therefore the reduction in strength in this area can be ignored.

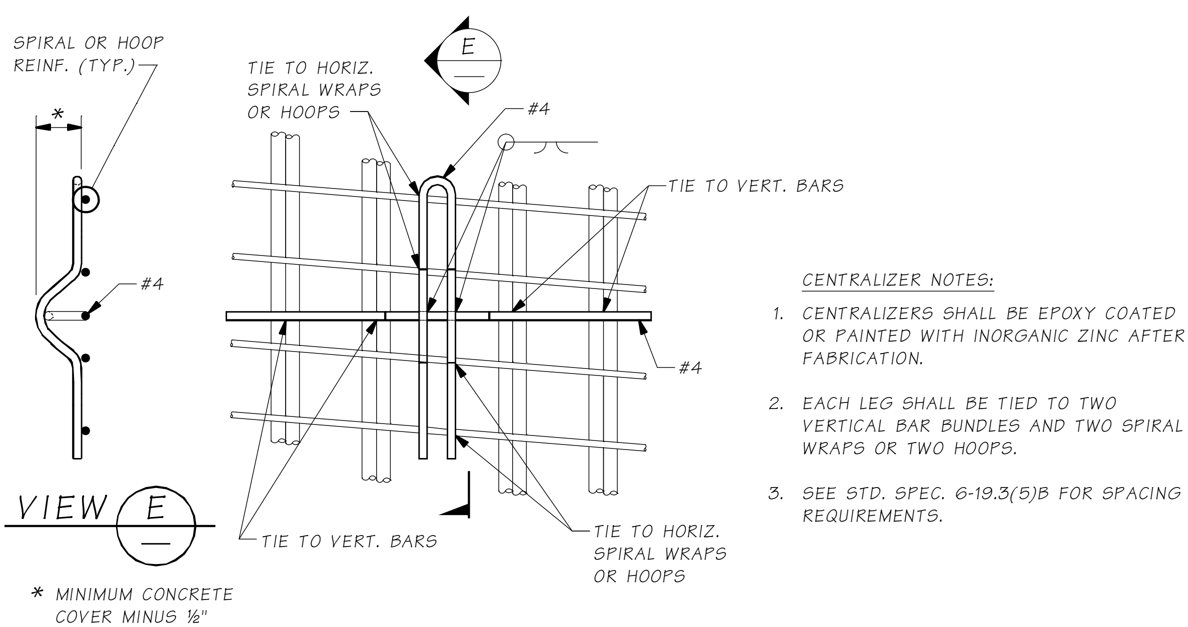
Y. Reinforcing bar centralizers shall be detailed in the plans as shown in Figure 7.8.2-5.

Nominal (Outside) Metric Casing Diameter		Maximum (Outside) Reinf. Cage Diameter to Accommodate Metric Casing[1]		Inside Diameter of Metric Casing[2]	Nominal (Outside) Metric Slip Casing Diameter[3]			Cage Clearance Below Slip Casing	Cage Clearance at Slip Casing[4]
Meters	Feet	Inches	Feet	Inches	Inches	Feet	Meters	Inches	Inches
3.73	12.24	130.52	10.88	140.52	137.52	11.46	3.49	8.16	3.0
3.43	11.25	118.71	9.89	128.71	125.71	10.48	3.19	8.16	3.0
3.00	9.84	101.81	8.48	111.84	108.81	9.07	2.76	8.15	3.0
2.80	9.19	95.51	7.96	105.51	102.51	8.54	2.60	7.36	3.0
2.50	8.20	83.70	6.98	93.70	90.70	7.56	2.30	7.36	3.0
2.20	7.22	71.89	5.99	81.89	78.89	6.57	2.00	7.36	3.0
2.00	6.56	64.02	5.34	74.02	71.02	5.92	1.80	7.36	3.0
1.50	4.92	45.12	3.76	55.12	52.12	4.34	1.32	6.97	3.0
1.2	3.94	34.08	2.84	44.09	41.09	3.42	1.044	6.57	3.0
1.00	3.28	30.22	2.52	36.22	34.22	2.85	0.87	4.57	1.5
0.92	3.00	26.87	2.24	32.87	30.87	2.57	0.78	4.57	1.5

Notes:

1. Provided by Malcolm Drilling. Assumes minimum of 5″ clearance to inside of oscillator casing on 4′ and larger and uses 3″ of clearance on smaller than 4′ (1.2 meters).
2. Provided by Malcolm Drilling.
3. Provided by Malcolm Drilling. Slip casing is 3″ smaller than inside diameter of temporary casing from 1.2 meters to 3 meters. 1 meter on down is 2″ smaller in diameter.
4. Slip casing is typically ⅜″ to ½″ thick (provided by Malcolm Drilling). Cage clearance assumes ½″ thick casing.

Table 7.8.2-2



Centralizer Detail
Figure 7.8.2-5

7.9 Piles and Piling

7.9.1 Pile Types

This section describes the piling used by the Bridge and Structures Office and their applications.
In general, piles should not be used where spread footings can be used. However, where heavy scour conditions may occur, pile foundations should be considered in lieu of spread footings. Also, where large amounts of excavation may be necessary to place a spread footing, pile support may be more economical.

A. **Cast-in-place Concrete Piles –** Cast-in-place (CIP) concrete piles utilize driven steel pipe casings, which are then filled with reinforcing steel and concrete. The bottom of the casing is typically capped with a suitable flat plate for driving. However, the Geotechnical Branch may specify special tips when difficult driving is expected.

   The Geotechnical Branch will determine the minimum wall thickness of the steel pipe casings based on driving conditions. However, the *Standard Specifications* require the contractor to provide a wall thickness that will prevent damage during driving.

B. **Precast, Prestressed Concrete Piles –** Precast, prestressed concrete piles are octagonal, or square in cross-section and are prestressed to allow longer handling lengths and resist driving stresses. Standard Plans are available for these types of piles.

C. **Steel H Piles –** Steel piles have been used where there are hard layers that must be penetrated in order to reach an adequate point bearing stratum. Steel stress is generally limited to 9.0 ksi (working stress) on the tip. H piling can act efficiently as friction piling due to its large surface area. Do not use steel H piling where the soil consists of only moderately dense material. In such conditions, it may be difficult to develop the friction capacity of the H piles and excessive pile length may result.

D. **Timber Piles –** Timber piles may be untreated or treated. Untreated piles are used only for temporary applications or where the entire pile will be permanently below the water line. Where composite piles are used, the splice must be located below the permanent water table. If doubt exists as to the location of the permanent water table, treated timber piles shall be used.

   Where dense material exists, consideration should be given to allowing jetting (with loss of uplift capacity), use of shoes, or use of other pile types.

E. **Steel Sheet Piles –** Steel sheet piles are typically used for cofferdams and shoring and cribbing, but are usually not made a part of permanent construction.

   CIP concrete piles consisting of steel casing filled with reinforcing steel and concrete are the preferred type of piling for WSDOT's permanent bridges. Other pile types such as precast, prestressed concrete piles, steel H piles, timber piles, auger cast piles, and steel pipe piles shall not be used for WSDOT permanent bridge structures. These types of piles may be used for temporary bridges and other non-bridge applications subject to approval by the State Geotechnical Engineer and the State Bridge Design Engineer.

   Micropiles shall not be used for new bridge foundations. This type of pile may be used for foundation strengthening of existing bridges, temporary bridges and other non-bridge applications subject to approval by the State Geotechnical Engineer and the State Bridge Design Engineer.

   Battered piles shall not be used for bridge foundations to resist lateral loads.

   The above limitations apply to all WSDOT bridges including mega projects and design-build contracts.

   The above policy on pile types is the outcome of lengthy discussions and meetings between the bridge design, construction and geotechnical engineers. These limitations are to ensure improved durability, design and construction for WSDOT pile foundations.

In seismic applications there is a need for bi-directional demands. Steel H piles have proven to have little bending capacity for the purposes of resisting seismic load while circular CIP piles provide consistent capacities in all directions. Also, CIP pile casing is generally available in a full range of casing diameters. CIP piles are easily inspected after driving to ensure the quality of the finished pile prior to placing reinforcing steel and concrete. All bending strength is supplied by elements other than the casing in accordance with WSDOT *Bridge Design Manual* policy.

Precast, prestressed concrete piles, and timber piles are difficult to splice and for establishing moment connections into the pile cap.

Micropiles have little bending capacity for the purposes of resisting lateral loads in seismic applications.

7.9.2 Single Pile Axial Resistance

The geotechnical report will provide the nominal axial resistance (R_n) and resistance factor (ϕ) for pile design. The factored pile load ($P_{U\,pile}$) must be less than the factored resistance, ϕR_n, specified in the geotechnical report.

Pile axial loading ($P_{U\,pile}$) due to loads applied to a pile cap are determined as follows:

$$\left(P_{U\ pile}\right) = \left(P_{U\ pile\ group}\right)/N + M_{U\ group}\ C/I_{group} + \gamma DD \tag{7.9.2-1}$$

Where:

$M_{U\,group}$	=	Factored moment applied to the pile group. This includes eccentric *LL*, *DC*, centrifugal force (*CE*), etc. Generally, the dynamic load allowance (*IM*) does not apply.
C	=	Distance from the centroid of the pile group to the center of the pile under consideration.
I_{group}	=	Moment of inertia of the pile group
N	=	Number of piles in the pile group
$P_{U\,pile\,group}$	=	Factored axial load to the pile group
DD	=	Downdrag force specified in the geotechnical report
γ	=	Load factor specified in the geotechnical report

Pile selfweight is typically neglected. As shown above, downdrag forces are treated as load to the pile when designing for axial capacity. However, it should not be included in the structural analysis of the bridge.

See Section 7.8.1 “Axial Resistance” of drilled shafts for discussion on load combinations when considering liquefaction, scour and on downdrag effects. These guidelines are also applicable to piles.

7.9.3 Block Failure

For the strength and extreme event limit states, if the soil is characterized as cohesive, the pile group capacity shall also be checked for the potential for a “block” failure, as described in AASHTO LRFD 10.7.3.9. This check requires interaction between the designer and the geotechnical engineer. The check is performed by the geotechnical engineer based on loads provided by the designer. If a block failure appears likely, the pile group size shall be increased so that a block failure is prevented.

7.9.4 Pile Uplift

Piles may be designed for uplift if specified in the geotechnical report. In general, pile construction methods that require preboring, jetting, or spudding will reduce uplift capacity.

7.9.5 Pile Spacing

Pile spacing determination is typically determined collaboratively with the geotechnical engineer. The WSDOT *Geotechnical Design Manual* M 46-03 specifies a minimum center-to-center spacing of 30″ or 2.5 pile diameters. However, center-to-center spacings of less than 2.5 pile diameters may be considered on a case-by-case basis.

7.9.6 Structural Design and Detailing of CIP Concrete Piles

The structural design and detailing of CIP Concrete piles is similar to column design with the following guidelines:

A. Class 4000P Concrete shall be specified for CIP concrete piles. The top 10′ of concrete in the pile is to be vibrated. Use 1.0 f'_c for the structural design.

B. For structural design, the reinforcement alone shall be designed to resist the total moment throughout the length of pile without considering strength of the steel casing. The minimum reinforcement shall be 0.75 percent A_g for SDC B, C, and D and shall be provided for the full length of the pile unless approved by the WSDOT Bridge Design Engineer. Minimum clearance between longitudinal bars shall meet the requirements in Chapter 5, Appendix 5.1-A2.

C. If the pile to footing/cap connection is not a plastic hinge zone longitudinal reinforcement need only extend above the pile into the footing/cap a distance equal to 1.0 l_d (tension). If the pile to footing/cap connection is a plastic hinge zone longitudinal reinforcement shall extend above the pile into the footing/cap a distance equal to 1.25 l_d.

D. Since the diameter of the concrete portion of the pile is dependent on the steel casing thickness, the as-built diameter will not be known during design (since the casing thickness is determined by the contractor). As such, a casing thickness must be assumed for design. The structural engineer should work closely with the geotechnical engineer to determine a suitable casing thickness to assume based on expected driving conditions. A pile drivability analysis may be required for this. Otherwise, the following can typically be assumed:
   - ¼″ for piles less than 14″ in diameter
   - ⅜″ for piles 14″ to 18″ in diameter
   - ½″ for larger piles

E. Steel casing for 24″ diameter and smaller CIP piling should be designated by nominal diameter rather than inside diameter. *Standard Specification* Section 9-10.5 requires steel casings to meet ASTM A252 Grade 2, which is purchased by nominal diameter (outside diameter) and wall thickness. A pile thickness should not be stated in the plans. As stated previously, the *Standard Specifications* require the contractor to determine the pile casing thickness required for driving.

F. Transverse spiral reinforcement shall be designed to resist the maximum shear in the pile. Avoid a spiral pitch of less than 3″. The minimum spiral shall be a #4 bar at 9″ pitch. If the pile to footing/cap connection is not a plastic hinge zone the volumetric requirements of AASHTO LRFD 5.13.4.6 need not be met.

G. Resistance factors for Strength Limit States shall be per the latest AASHTO LRFD Specifications. Resistance factors for Extreme Event Limit States shall be per the latest AASHTO Seismic Specifications.

H. Piles are typically assumed to be continuously supported. Normally, the soil surrounding a foundation element provides sufficient bracing against a buckling failure. Piles that are driven through very weak soils should be designed for reduced lateral support, using information from the Geotechnical Branch as appropriate. AASHTO LRFD 10.7.3.13.4 may be used to estimate the column length for buckling. Piles driven through firm material normally can be considered fully supported for column action (buckling not critical) below the ground.

I. The axial load along the pile varies due to side friction. It is considered conservative, however, to design the pile for the full axial load plus the maximum moment. The entire pile is then typically reinforced for this axial load and moment.

J. In all cases of uplift, the connection between the pile and the footing must be carefully designed and detailed. The bond between the pile and the seal may be considered as contributing to the uplift resistance. This bond value shall be limited to 10 psi. The pile must be adequate to carry tension throughout its length. For example, a timber pile with a splice sleeve could not be used.

7.9.7 Pile Splices

Pile splices shall be avoided where possible. If splices may be required in timber piling, a splice shall be detailed on the plans. Splices between treated and untreated timber shall always be located below the permanent water line. Concrete pile splices shall have the same strength as unspliced piles.

7.9.8 Pile Lateral Design

The strength limit state for lateral resistance is only structural, though the determination of pile fixity is the result of soil-structure interaction. A failure of the soil does not occur; the soil will continue to displace at constant or slightly increasing resistance. Failure occurs when the pile reaches the structural limit state and this limit state is reached, in the general case, when the nominal combined bending, shear, and axial resistance is reached.

Piles resist horizontal forces by a combination of internal strength and the passive pressure resistance of the surrounding soil. The capacity of the pile to carry horizontal loads should be investigated using a soil/structural analysis. For more information on modeling individual piles or pile groups, see Section 7.2, Foundation Modeling and Section 7.2.6 Lateral Analysis of Piles and Shafts.

7.9.9 Battered Piles

As stated previously, battered piles shall not be used to resist lateral loads for new bridge foundations. Where battered piles are used, the maximum batter shall be 4½:12. Piles with batters in excess of this become very difficult to drive and the bearing values become difficult to predict. Ensure that battered piling do not intersect piling from adjacent footings within the maximum length of the piles.

7.9.10 Pile Tip Elevations and Quantities

Pile length quantities provided to PS&E are based on the estimated tip elevation given in the geotechnical report or the depth required for design whichever is greater. If the estimated tip elevation given in the geotechnical report is greater than the design tip elevation, overdriving the pile will be required. The geotechnical engineer shall be contacted to evaluate driving conditions. Bridge Special Provision BSP050311D5.FB6 is required in the Special Provisions to alert the contractor of the additional effort needed to drive these piles.

Minimum pile tip elevations provided in the geotechnical report may need to be adjusted to lower elevations depending on the results of the lateral, axial, and uplift analysis. This would become the minimum pile tip elevation requirement for the contract specifications. If adjustment in the minimum tip elevations is necessary, or if the pile diameter needed is different than what was assumed for the geotechnical report, the Geotechnical Branch MUST be informed so that pile drivability can be re-evaluated.

Note that lateral loading and uplift requirements may influence (possibly increase) the number of piles required in the group if the capacity available at a reasonable minimum tip elevation is not adequate. This will depend on the soil conditions and the loading requirements. For example, if the upper soil is very soft or will liquefy, making the minimum tip elevation deeper is unlikely to improve the lateral response of the piles enough to be adequate. Adding more piles to the group or using a larger pile diameter to increase the pile stiffness may be the only solution.

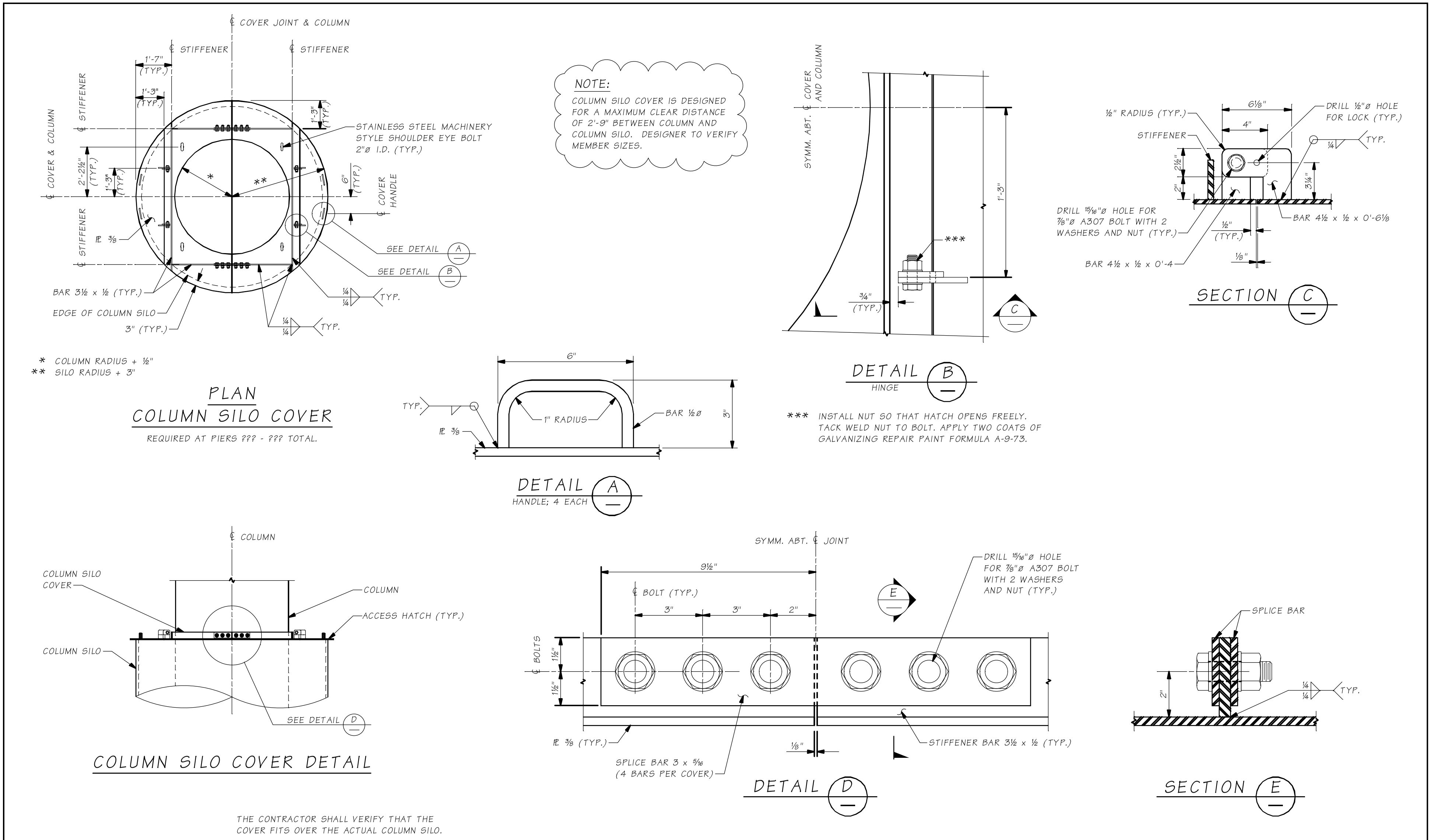
7.9.11 Plan Pile Resistance

The Bridge Plan General Notes shall list the Ultimate Bearing Capacity (Nominal Driving Resistance, R_{ndr}) in tons. This information is used by the contractor to determine the pile casing thickness and size the hammer to drive the piles. The resistance for several piers may be presented in a table as shown in Figure 7.9.11-1. If overdriving the piles is required to reach the minimum tip elevation, the estimated amount of overdriving (tons) shall be specified in the Special Provisions with BSP050311D5.FB6.

THE PILES SHALL BE DRIVEN TO AN ULTIMATE BEARING CAPACITY AS FOLLOWS:	
PIER NO.	**ULTIMATE BEARING CAPACITY (TONS)**
1	====
4	====

Figure 7.9.11-1

The total factored pile axial loading must be less than ϕR_n for the pile design. Designers should note that the driving resistance might be greater than the design loading for liquefied soil conditions. This is not an overdriving condition. This is due to the resistance liquefied soils being ignored for design, but included in the driving criteria to place the piles.



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Supervisor					
Designed By					
Checked By					
Detailed By					
Bridge Projects Engr.					
Prelim. Plan By					
Architect/Specialist	DATE	REVISION	BY	APP'D	

REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
10	WASH.			
JOB NUMBER				

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

COLUMN SILO COVER

BRIDGE SHEET NO.

SHEET OF SHEETS

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Chapter 8 Walls and Buried Structures Contents

8.2 Miscellaneous Underground Structures

8.2.1 General

Miscellaneous underground structures consist of box culverts, precast reinforced concrete three-sided structures, detention vaults, and metal pipe arches.

Where miscellaneous underground structures pass under or support roadways and other structures, they shall be designed for seismic effects as follows:

- Seismic effects need not be considered for structures with span lengths of 20 feet or less.
- Seismic effects shall be considered for structures with span lengths more than 20 feet. The potential effects of unstable ground conditions (e.g., liquefaction, liquefaction induced settlement, landslides, ground motion attenuation with depth, and fault displacements) on the function of the underground structures shall be considered. The *AASHTO LRFD Bridge Design Specifications* Section 12.6.1 exemption from seismic loading shall not apply.

As with any structure, a geotechnical soils report with loading or pressure diagrams, settlement criteria, and ground water levels will be needed from the Materials Laboratory Geotechnical Office in order to complete the design. The requirement of BDM Section 3.5 for inclusion of live load in Extreme Event-I load combination is applicable.

In addition to the *AASHTO LRFD Bridge Design Specifications*, the FHWA Publication No. FHWA-NHI-09-010 dated November 2008, *Technical Manual for Design and Construction of Road Tunnels Civil Elements*, may also be used as a design specification reference for the seismic design requirement.

8.2.2 Design

A. **Box Culverts** – Box culverts are four-sided rigid frame structures and are either made from cast-in-place (CIP) reinforced concrete or precast concrete. In the past, standardized box culvert plan details were shown in the WSDOT *Standard Plans*, under the responsibility of the Hydraulics Branch. These former Standard Plans have been deleted and are no longer available. Now box culvert design is standardized under applicable AASHTO material specifications, and design plans are not required in the PS&E. Box culverts shall be in accordance with ASTM C1433.

B. **Precast Reinforced Concrete Three-Sided Structures** – Precast reinforced concrete three-sided structures are patented or trademarked rigid frame structures made from precast concrete. Some fabricators of these systems are: Utility Vault Company, Central Pre-Mix Prestress Company, and Bridge Tek, LLC. These systems require a CIP concrete or precast footing that must provide sufficient resistance to the horizontal reaction or thrust at the base of the vertical legs.

The precast concrete fabricators are responsible for the structural design and the preparation of shop plans. Precast reinforced concrete three sided structures, constructed in accordance with the current WSDOT General Special Provisions (GSP's) for these structures, shall be designed under AASHTO LRFD Bridge Specifications. The fabricators of systems which have received WSDOT pre-approval are specified in the GSP's. The bridge designer reviewing the project will be responsible for reviewing the fabricator's design calculations and details with consultation from the Construction Support Unit. Under the current GSP, precast reinforced concrete three sided structures are limited to spans of 26 feet or less. However, in special cases it may be necessary to allow longer spans, with the specific approval of the Bridge and Structures Office. Several manufacturers advertise spans over 40 feet.

C. **Detention Vaults** – Detention vaults are used for stormwater storage and are to be watertight. These structures can be open at the top like a swimming pool, or completely enclosed and buried below ground. Detention vaults shall be designed by the AASHTO LRFD Bridge Design Specification and the following: Seismic design effects shall satisfy the requirements of ACI 350.3-06 "Seismic Design of Liquid-Containing Concrete Structures." Requirements for Joints and jointing shall satisfy the requirements of ACI 350-06. Two references for tank design are the PCA publications *Rectangular Concrete Tanks*, Revised 5th Edition (1998) and *Design of Liquid-Containing Structures for Earthquake Forces* (2002).

The geotechnical field investigations and recommendations shall comply with the requirements given in 8.16 of the WSDOT *Geotechnical Design Manual* M 46-03. In addition to earth pressures, water tables, seismic design, and uplift, special consideration should be given to ensure differential settlement either does not occur or is included in the calculations for forces, crack control and water stops.

Buoyant forces from high ground water conditions should be investigated for permanent as well as construction load cases so the vault does not float. Controlling loading conditions may include: backfilling an empty vault, filling the vault with stormwater before it is backfilled, or seasonal maintenance that requires draining the vault when there is a high water table. In all Limit States, the buoyancy force (*WA*) load factor shall be taken as γ_{WA} = 1.25 in AASHTO LRFD Table 3.4.1-1. In the Strength Limit State, the load factors that resist buoyancy (γ_{DC}, γ_{DW}, γ_{ES}, Etc.) shall be their minimum values, per AASHTO LRFD Table 3.4.1-2 and the entire vault shall be considered empty. During the vault construction, the water table shall be taken as the seal vent elevation or the top of the vault, if open at the top. In this case the load factors that resist buoyancy shall be their minimum values, except where specified as a construction load, per AASHTO LRFD Section 3.4.2. In certain situations tie-downs may be required to resist buoyancy forces. The resisting force (R_n) and resistance factors (ø) for tie-downs shall be provided by the Geotechnical Engineers. The buoyancy check shall be as follows:

For Buoyancy without tie-downs:

$$(R_{RES} / R_{UPLIFT}) \geq 1.0$$

For Buoyancy with tie-downs:

$$(R_{\mathrm{RES}} / [R_{UPLIFT} + øR_n]) \geq 1.0$$

Where:

$$R_{RES} = \left| \gamma_{DC}\, DC + \gamma_{DW}\, DW + \gamma_{ES}\, ES + \gamma_{\mathrm{i}}\, Q_{\mathrm{i}} \right|$$

$$R_{UPLIFT} = \left| \gamma_{\mathrm{WA}}\, WA \right|$$

ACI 350-06 has stricter criteria for cover and spacing of joints than the AASHTO LRFD Specifications. Cover is not to be less than 2 inches (ACI 7.7.1), no metal or other material is to be within 1½ inches from the formed surface, and the maximum bar spacing shall not exceed 12 inches (ACI 7.6.5). Crack control criteria is per AASHTO LRFD 5.7.3.4 with γ_e = 0.5 (in order to maintain a crack width of 0.0085 inches, per the commentary of 5.7.3.4).

Joints in the vault's top slab, bottom slab and walls shall allow dissipation of temperature and shrinkage stresses, thereby reducing cracking. The amount of temperature and shrinkage reinforcement is a function of reinforcing steel grade "and length between joints (ACI Table 7.12.2-1). All joints shall have a shear key and a continuous and integral PVC waterstop with a 4-inch minimum width. The purpose of the waterstop is to prevent water infiltration and exfiltration. Joints having welded shear connectors with grouted keyways shall use details from WSDOT Precast Prestressed Slab Details or approved equivalent, with weld ties spaced at 4′-0″ on center. Modifications to the above joints shall be justified with calculations. Calculations shall be provided for all grouted shear connections. The width of precast panels shall be increased to minimize the number of joints between precast units.

For cast-in-place walls in contact with liquid that are over 10′ in height, the minimum wall thickness is 12″. This minimum thickness is generally good practice for all external walls, regardless of height, to allow for 2 inches of cover as well as space for concrete placement and vibration.

After the forms are placed, the void left from the form ties shall be coned shaped, at least 1 inch in diameter and 1½ inches deep, to allow proper patching.

Detention vaults that need to be located within the prism supporting the roadway are required to meet the following maintenance criteria. A by-pass piping system is required. Each cell in the vault shall hold no more than 6,000 gallons of water to facilitate maintenance and cleanout operations. Baffles shall be water tight. Access hatches shall be spaced no more than 50 feet apart. There shall be an access near both the inlet and the outfall. These two accesses shall allow for visual inspection of the inlet and outfall elements, in such a manner that a person standing on the ladder, out of any standing water, will be in reach of any grab handles, grates or screens. All other access hatches shall be over sump areas. All access hatches shall be a minimum 30 inch in diameter, have ladders that extend to the vault floor, and shall be designed to resist HS-20 wheel loads with applicable impact factors as described below.

Detention vaults that need to be located in the roadway shall be oriented so that the access hatches are located outside the traveled lanes. Lane closures are usually required next to each access hatch for maintenance and inspection, even when the hatches are in 12′-0″ wide shoulders.

A 16 kip wheel load having the dynamic load allowance for deck joints, in AASHTO LRFD Table 3.6.2.1-1, shall be applied at the top of access hatches and risers. The load path of this impact force shall be shown in the calculations.

Minimum vault dimensions shall be 4′-0″ wide and 7′-0″ tall; inside dimensions.

Original signed plans of all closed top detention vaults with access shall be forwarded to the Bridge Plans Engineer in the Bridge Project Unit (see Section 12.4.10.B of this manual). This ensures that the Bridge Preservation Office will have the necessary inventory information for inspection requirements. A set of plans must be submitted to both the WSDOT Hydraulics Office and the Regional WSDOT Maintenance Office for plans approval.

D. **Metal Pipe Arches –** Soil ph should be investigated prior to selecting this type of structure. Metal Pipe arches are not generally recommended under high volume highways or under large fills.

Pipe arch systems are similar to precast reinforced concrete three sided structures in that these are generally proprietary systems provided by several manufacturers, and that their design includes interaction with the surrounding soil. Pipe arch systems shall be designed in accordance with the *AASHTO Standard Specifications for Highway Bridges*, and applicable ACI design and ASTM material specifications.

E. **Tunnels –** Tunnels are unique structures in that the surrounding ground material is the structural material that carries most of the ground load. Therefore, geology has even more importance in tunnel construction than with above ground bridge structures. In short, geotechnical site investigation is the most important process in planning, design and construction of a tunnel. These structures are designed in accordance with the *AASHTO LRFD Bridge Design Specifications*.

Tunnels are not a conventional structure, and estimation of costs is more variable as size and length increase. Ventilation, safety access, fire suppression facilities, warning signs, lighting, emergency egress, drainage, operation and maintenance are extremely critical issues associated with the design of tunnels and will require the expertise of geologists, tunnel experts and mechanical engineers.

For motor vehicle fire protection, a standard has been produced by the National Fire Protection Association. This document, *NFPA 502 – Standard for Road Tunnels, Bridges, and Other Limited Access Highways*, uses tunnel length to dictate minimum fire protection requirements:

300 feet or less: no fire protection requirements
300 to 800 feet: minor fire protection requirements
800 feet or more: major fire protection requirements

Some recent WSDOT tunnel projects are:

I-90 Mt. Baker Ridge Tunnel Bore Contract: 3105 Bridge No: 90/24N

This 1500 foot long tunnel is part of the major improvement of Interstate 90. Work was started in 1983 and completed in 1988. The net interior diameter of the bored portion, which is sized for vehicular traffic on two levels with a bike/pedestrian corridor on the third level, is 63.5 feet. The project is the world's largest diameter tunnel in soft ground, which is predominantly stiff clay. Construction by a stacked-drift method resulted in minimal distortion of the liner and insignificant disturbance at the ground surface above.

Jct I-5 SR 526 E-N Tunnel Ramp Contract: 4372 Bridge No: 526/22E-N

This 465 foot long tunnel, an example of the cut and cover method, was constructed in 1995. The interior dimensions were sized for a 25 foot wide one lane ramp roadway with a vertical height of 18 feet. The tunnel was constructed in three stages. 3 and 4 foot diameter shafts for the walls were placed first, a 2 foot thick cast-in-place top slab was placed second and then the tunnel was excavated, lined and finished.

I-5 Sleater-Kinney Bike/Ped. Tunnel Contract: 6031 Bridge No: 5/335P

This 122 foot long bike and pedestrian tunnel was constructed in 2002 to link an existing path along I-5 under busy Sleater-Kinney Road. The project consisted of precast prestressed slab units and soldier pile walls. Construction was staged to minimize traffic disruptions.

8.2.3 References

1. *AASHTO LRFD Bridge Design Specifications*, 5th Edition, American Association of State Highway and Transportation Officials, Washington, D.C.
2. AASHTO *Standard Specifications for Highway Bridges*, 17th Ed., 2002
3. WSDOT *Standard Specifications for Highway Bridges and Municipal Construction*, Olympia, Washington 98501.
4. ACI 350/350R-06 Code Requirements for Environmental Engineering Concrete Structures, ACI, 2006.
5. Munshi, Javeed A. *Rectangular Concrete Tanks*, Rev. 5th Ed., PCA, 1998.
6. Miller, C. A. and Constantino, C. J. "Seismic Induced Earth Pressure in Buried Vaults", PVP-Vol.271, *Natural Hazard Phenomena and Mitigation*, ASME, 1994, pp. 3-11.
7. Munshi, J. A. *Design of Liquid-Containing Concrete Structures for Earthquake Forces*, PCA, 2002.
8. NFPA 502, *Standard for Road Tunnels, Bridges, and Other Limited Access Highways*.

Chapter 10 Signs, Barriers, Approach Slabs, and Utilities — Contents

10.1 Sign and Luminaire Supports

10.1.1 Loads

A. **General –** The reference used in developing the following office criteria is the AASHTO "*Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*," Fourth Edition Dated 2001 including interims, and shall be the basis for analysis and design.

B. **Dead Loads**

Sign (including panel and windbeams, does not include vert. bracing)	3.25 lbs/ft^2
Luminaire (effective projected area of head = 3.3 sq ft)	60 lbs/each
Fluorescent Lighting	3.0 lbs/in ft
Standard Signal Head	60 lbs/each
Mercury Vapor Lighting	6.0 lbs/in ft
Sign Brackets	Calc.
Structural Members	Calc.
5 foot wide maintenance walkway (including sign mounting brackets and handrail)	160 lbs/in ft
Signal Head w/3 lenses (effective projected area with backing plate = 9.2 sq ft)	60 lbs each

C. **Wind Loads –** A major change in the AASHTO 2001 Specification wind pressure equation is the use of a 3 second gust wind speed in place of a fastest-mile wind speed used in the previous specification. The 3 second wind gust map in AASHTO is based on the wind map in ANSI/ASCE 7-95.

Basic wind speed of 90 mph shall be used in computing design wind pressure using Equation 3-1 of AASHTO Section 3.8.1.

Do not use the Alternate Method of Wind Pressures given in Appendix C of the AASHTO 2001 Specifications.

D. **Design Life and Recurrence Interval** – (Table 3-3, AASHTO 2001)

50 years for luminaire supports, overhead sign structures, and traffic signal structures.
10 years for roadside sign structures.

E. **Ice Loads** – 3 psf applied around all the surfaces of structural supports, horizontal members, and luminaires, but applied to only one face of sign panels (AASHTO Section 3.7).

Walk-through VMS shall not be installed in areas where appreciable snow loads may accumulate on top of the sign, unless positive steps are taken to prevent snow build-up.

F. **Fatigue Design** – Fatigue design shall conform to AASHTO Section 11. Fatigue Categories are listed in Table 11-1. Cantilever structures, poles, and bridge mounted sign brackets shall conform to the following fatigue categories.

Fatigue Category I for overhead cantilever sign structures (maximum span of 30 feet and no VMS installation), high level (high mast) lighting poles 100 feet or taller in height, bridge-mounted sign brackets, and all signal bridges.

Fatigue Category II for high level (high mast) lighting poles between 51 feet and 99 feet in height.

Fatigue Category III for lighting poles 50 feet or less in height with rectangular, square or non-tapered round cross sections, and overhead cantilever traffic signals at intersections (maximum cantilever length 65 feet). If vehicle speeds are posted at 45 mph or greater, then overhead cantilever traffic signal structures shall be designed for Fatigue Category I.

Sign bridges, cantilever sign structures, signal bridges, and overhead cantilever traffic signals mounted on bridges shall be either attached to substructure elements (e.g., crossbeam extensions) or to the bridge superstructure at pier locations. Mounting these features to bridges as described above will help to avoid resonance concerns between the bridge structure and the signing or signal structure.

The “XYZ” limitation shown in Table 10.1.4-2 shall be met for Monotube Cantilevers. The “XYZ” limitation consists of the product of the sign area (XY) and the arm from the centerline of the posts to the centerline of the sign (Z). See Appendix 10.1-A2-1 for details.

G. **Live Load** – A live load consisting of a single load of 500 lb distributed over 2.0 feet transversely to the member shall be used for designing members for walkways and platforms. The load shall be applied at the most critical location where a worker or equipment could be placed, see AASHTO 2001, Section 3.6.

F. **Group Load Combinations** – Sign, luminaire, and signal support structures are designed using the maximum of the following four load groups (AASHTO Section 3.4 and Table 3-1):

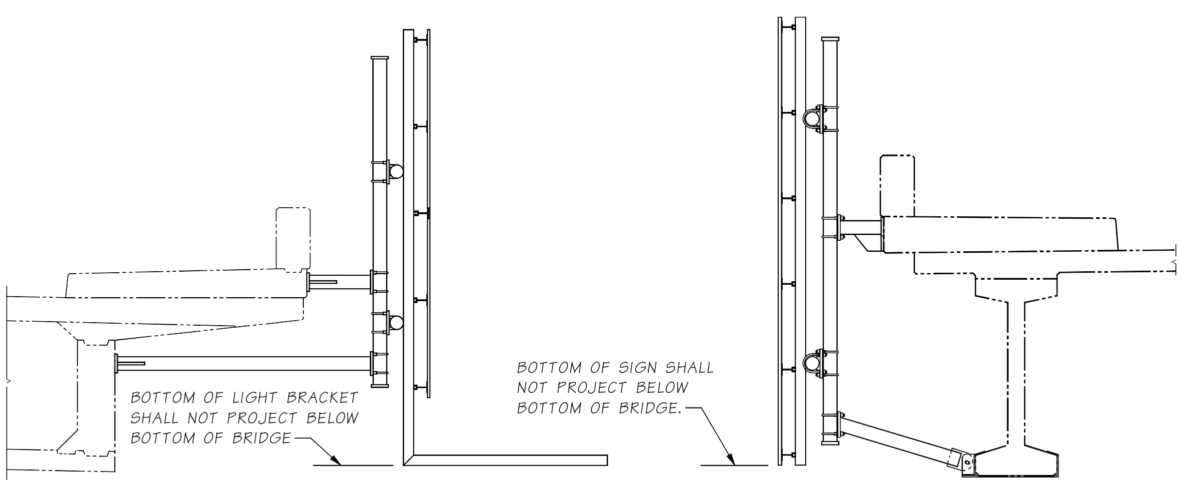
Group Load	Load Combination	Percent of *Allowable Stress
I	DL	100
II	DL+W**	133
III	DL+Ice+½(W**)	133
IV	Fatigue	See AASHTO Section 11 for Fatigue loads and stress range

* No load reduction factors shall be applied in conjunction with these increased allowable stresses.
** W – Wind Load

10.1.2 Bridge Mounted Signs

A. **Vertical Clearance** – All new signs mounted on bridge structures shall be positioned such that the bottom of the sign or lighting bracket does not extend below the bottom of the bridge as shown in Figure 10.1.2-1. The position of the sign does not need to allow for the future placement of lights below the sign. If lights are to be added in the future they will be mounted above the sign. To ensure that the bottom of the sign or lighting bracket is above the bottom of the bridge, the designer should maintain at least a nominal 2 inch dimension between the bottom of the sign or lighting and the bottom of the bridge. Maximum sign height shall be decided by the Region. If the structure is too high above the roadway, then the sign should not be placed on the structure.

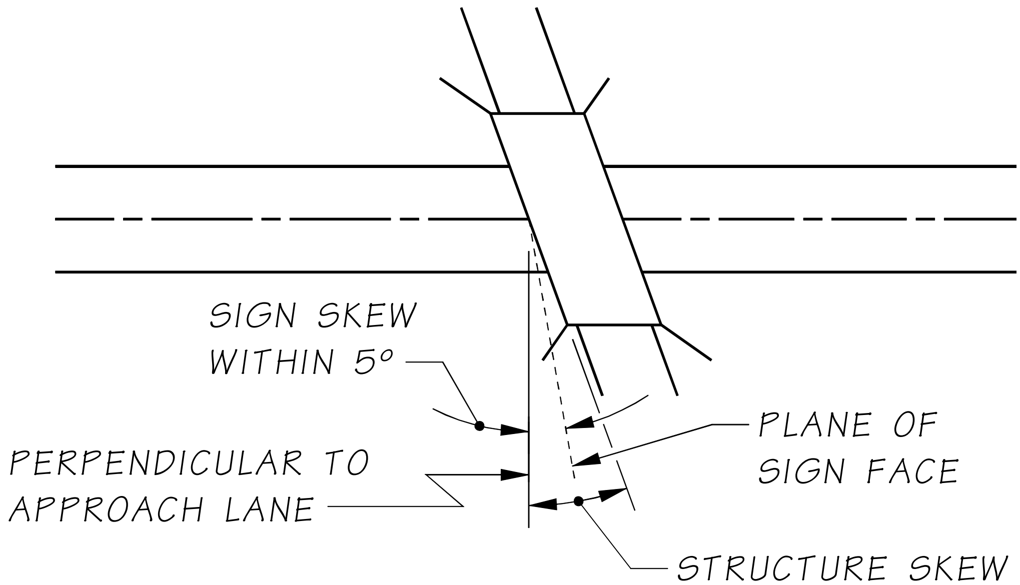
Bridge mounted sign brackets shall be designed to account for the weight of added lights, and for the wind affects on the lights to ensure bracket adequacy if lighting is attached in the future.



Sign Vertical Clearance
Figure 10.1.2-1

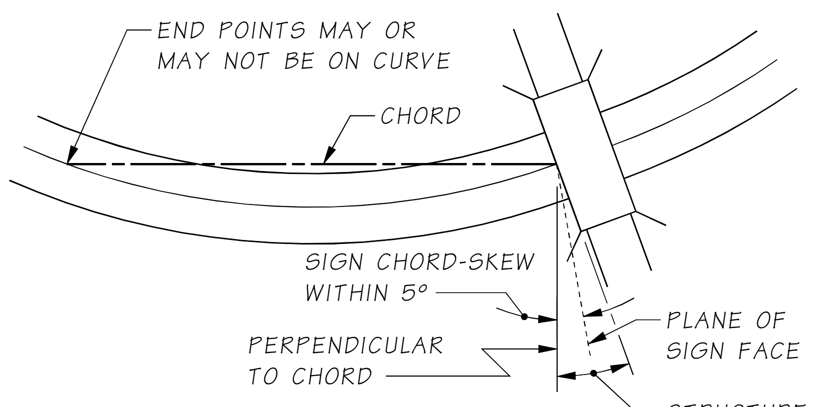
B. **Geometrics**

1. Signs should be installed at approximate right angles to approaching motorists. For structures above a tangent section of roadway, signs shall be designed to provide a sign skew within 5° from perpendicular to the lower roadway (see Figure 10.1.2-2).



Sign Skew on Tangent Roadway
Figure 10.1.2-2

2. For structures located on or just beyond a horizontal curve of the lower roadway, signs shall be designed to provide a sign chord skew within 5° from perpendicular to the chord-point determined by the approach speed (see Figure 10.1.2-3).
3. The top of the sign shall be level.



SPEED LIMIT	35 MPH OR LESS	MORE THAN 35 MPH
CHORD LENGTH	300'	500'

Sign Skew on Curved Roadway
Figure 10.1.2-3

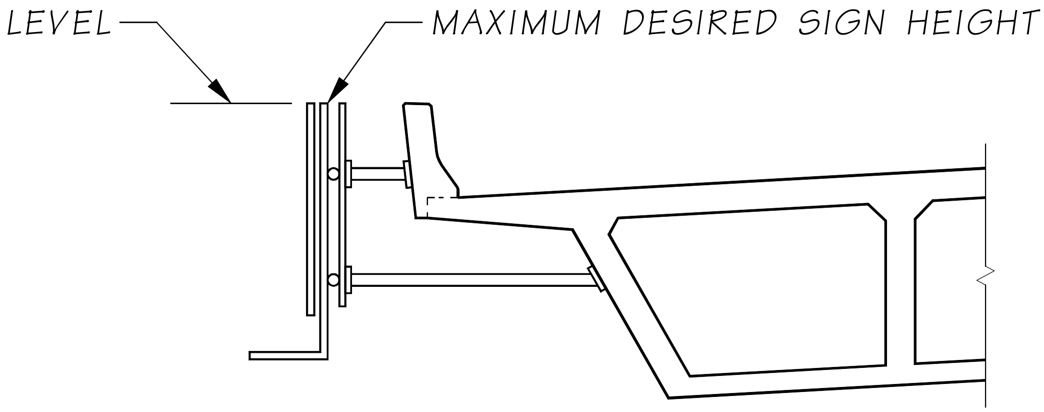


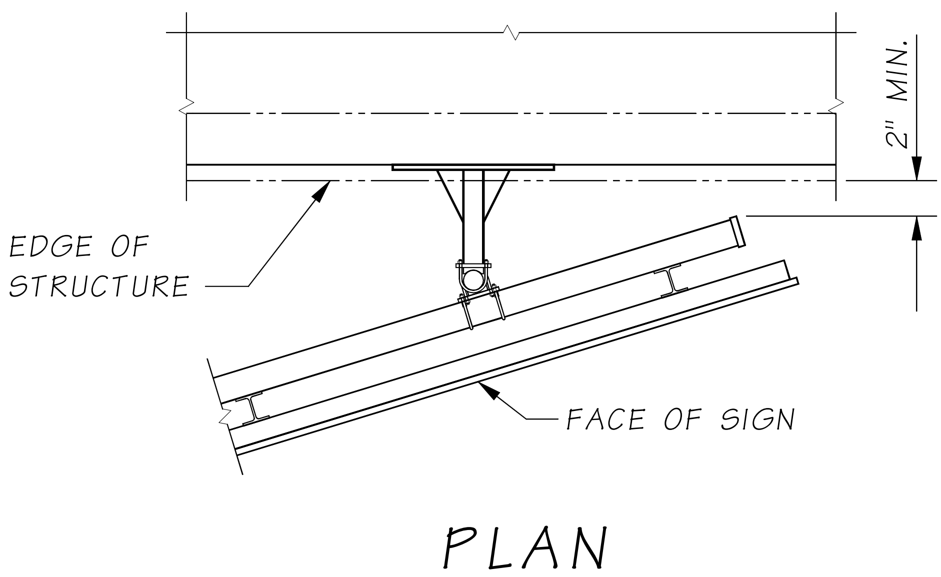
Figure 10.1.2-4

C. **Aesthetics**

1. When possible, the support structure should be hidden from view of traffic.
2. The sign support shall be detailed in such a manner that will permit the sign and lighting bracket to be installed level.
3. When the sign support will be exposed to view, special consideration is required in determining member sizes and connections to provide as pleasing an appearance as possible.

D. **Sign Placement**

1. When possible, the designer should avoid locating signs under bridge overhangs. This causes partial shading or partial exposure to the elements and problems in lifting the material into position and making the required connections. Signs shall never be placed directly under the drip-line of the structure. These conditions may result in uneven fading, discoloring, and difficulty in reading. When necessary to place a sign under a bridge due to structural or height requirements, the installation should be reviewed by the Region Traffic Design Office.
2. A minimum of 2 inches of clearance shall be provided between back side of the sign support and edge of the structure. See Figure 10.1.2-5.



Sign Horizontal Location
Figure 10.1.2-5

E. **Installation**

1. Resin bonded anchors or cast-in-place ASTM A 307 anchor rods should be used to install the sign brackets on the structure. Size and minimum installation depth shall be given in the plans. The resin bonded anchors should be installed normal to the concrete surface. Resin bonded anchors shall not be placed through the webs or flanges of presstressed or post-tensioned girders unless approved by the WSDOT Bridge Design Engineer.
2. Bridge mounted sign structures shall not be placed on bridges with steel superstructures unless approved by the WSDOT Bridge Design Engineer.

10.1.3 Monotube Sign Structures Mounted on Bridges

A. **Design Loads** – Design loads for the supports of the Sign Bridges shall be calculated based on assuming a 12-foot-deep sign over the entire roadway width, under the sign bridge. This will account for any signs that may be added in the future. For Cantilever design loads, guidelines specified in Section 10.1.1 shall be followed. The design loads shall follow the same criteria as described in Section 10.1.1. Loads from the sign bridge shall be included in the design of the supporting bridge.

In cases where a sign structure is mounted on a bridge, the sign structure, from the anchor bolt group and above, shall be designed to AASHTO “Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals,” Fourth Edition, dated 2001 including interims. The concrete around the anchor bolt group and the connecting elements to the bridge structure shall be designed to the specifications in this manual and *AASHTO LRFD Bridge Design Specifications*. Loads from the sign structure design code shall be taken as unfactored loads for use in LFRD bridge design.

B. **Vertical Clearance** – Vertical clearance for Monotube Sign Structures shall be 20′-0″ minimum from the bottom of the lowest sign to the highest point in the traveled lanes. See Appendix 10.1-A1-1, 10.1-A2-1, and 10.1-A3-1 for sample locations of Minimum Vertical Clearances.

C. **Geometrics –** Sign structures shall be placed at approximate right angles to approaching motorists. Dimensions and details of sign structures are shown in the *Standard Plans* G-60.10, G-60.20, G-60.30, G-70.10, G-70.20, G-70.30 and Appendix 10.1-A1-1, 2, and 3 and 10.1-A2-1, 2, and 3. When maintenance walkways are included, refer to *Standard Plans* G-95.10, G-95.20, G-95.30.

10.1.4 Monotube Sign Structures

A. **Sign Bridge Standard Design** – Table 10.1.4-1 provides the standard structural design information to be used for a Sign Bridge Layout, Appendix 10.1-A1-1; along with the Structural Detail sheets, which are Appendix 10.1-A1-2 and Appendix 10.1-A1-3; and General Notes, Appendix 10.1-A0-1.

B. **Cantilever Standard Design** – Table 10.1.4-2 provides the standard structural design information to be used for a Cantilever Layout, Appendix 10.1-A2-1; along with the Structural Detail sheets, which are Appendix 10.1-A2-2 and Appendix 10.1-A2-3; and General Notes, Appendix 10.1-A0-1.

STANDARD MONOTUBE SIGN BRIDGES

SPAN LENGTH	POSTS Ⓥ				BEAM A Ⓥ				BEAM B Ⓥ				BEAM C Ⓥ				CAMBER
"S"	"H"	"A"	"B"	"T1"	"L1"	"B"	"C"	"T2"	"L2"	"B"	"C"	"T2"	"L3"	"B"	"C"	"T2"	
LESS THAN 60'-0"	30'-0" OR LESS	1'-6"	2'-0"	1/2"	6'-0"	2'-0"	2'-0"	3/8"	0'-0"	2'-0"	2'-0"	3/8"	13'-0" TO 48'-0"	2'-0"	2'-0"	3/8"	2 3/4"
60'-0" TO 75'-0"	30'-0" OR LESS	1'-6"	2'-3"	1/2"	6'-0"	2'-3"	2'-0"	3/8"	9'-0" TO 14'-0"	2'-3"	2'-0"	3/8"	30'-0" TO 35'-0"	2'-3"	2'-0"	3/8"	3 3/4"
+75'-0" TO 90'-0"	30'-0" OR LESS	1'-6"	2'-3"	5/8"	6'-0"	2'-3"	2'-0"	3/8"	14'-0" TO 19'-0"	2'-3"	2'-0"	3/8"	35'-0" TO 40'-0"	2'-3"	2'-0"	3/8"	5"
+90'-0" TO 105'-0"	30'-0" OR LESS	1'-9"	2'-6"	5/8"	6'-0"	2'-6"	2'-3"	1/2"	19'-0" TO 26'-6"	2'-6"	2'-3"	1/2"	40'-0"	2'-6"	2'-3"	1/2"	6"
+105'-0" TO 120'-0"	30'-0" OR LESS	1'-9"	2'-6"	5/8"	6'-0"	2'-6"	2'-3"	1/2"	26'-6" TO 34'-0"	2'-6"	2'-3"	1/2"	40'-0"	2'-6"	2'-3"	1/2"	7 1/2"
+120'-0" TO 135'-0"	30'-0" OR LESS	2'-0"	2'-6"	5/8"	6'-0"	2'-6"	2'-6"	1/2"	34'-0" TO 41'-6"	2'-6"	2'-6"	1/2"	40'-0"	2'-6"	2'-6"	1/2"	8 1/2"
+135'-0" TO 150'-0"	30'-0" OR LESS	2'-0"	2'-6"	5/8"	6'-0"	2'-6"	2'-6"	1/2"	41'-6" TO 49'-0"	2'-6"	2'-6"	1/2"	40'-0"	2'-6"	2'-6"	1/2"	10 1/2"

SPAN LENGTH	POST BASE Ⓥ					BOLTED SPLICE #1 L1 TO L2 AND L1 TO L3						BOLTED SPLICE #2 L2 TO L3						MAXIMUM SIGN AREA
"S"	"D1"	"S5"	"S6"	"T3"	"T6"	"S1"	"S2"	"S3"	"S4"	"T4"	"T5"	"S1"	"S2"	"S3"	"S4"	"T4"	"T5"	
LESS THAN 60'-0"	1 1/2"	4	4	2 1/4"	3/4"	5	-	5	-	2"	5/8"	-	-	-	-	-	-	500 SQ. FT.
60'-0" TO 75'-0"	1 3/4"	4	4	2 1/4"	3/4"	6	-	5	-	2"	5/8"	6	-	5	-	2 1/4"	3/4"	600 SQ. FT.
+75'-0" TO 90'-0"	1 3/4"	4	4	2 1/2"	3/4"	6	-	5	-	2"	5/8"	6	-	5	-	2 1/4"	3/4"	750 SQ. FT.
+90'-0" TO 105'-0"	1 3/4"	4	5	2 1/2"	1"	7	-	6	-	2"	5/8"	7	5	6	4	2 1/2"	1"	750 SQ. FT.
+105'-0" TO 120'-0"	1 3/4"	4	5	2 1/2"	1"	7	-	6	-	2"	5/8"	7	5	6	4	2 1/2"	1"	850 SQ. FT.
+120'-0" TO 135'-0"	2"	4	5	2 1/2"	1"	7	-	7	-	2"	5/8"	7	5	7	5	2 1/2"	1"	800 SQ. FT.
+135'-0" TO 150'-0"	2"	4	5	2 1/2"	1"	7	-	7	-	2"	5/8"	7	5	7	5	2 1/2"	1"	800 SQ. FT.

Ⓥ NOTE: DENOTES MAIN LOAD CARRYING TENSILE MEMBERS OR TENSION COMPONENTS OF FLEXURAL MEMBERS.

Table 10.1.4-1

STANDARD MONOTUBE CANTILEVERS

SPAN LENGTH	POSTS Ⓥ				BEAM A Ⓥ				BEAM B Ⓥ				CAMBER
"S"	"H"	"A"	"B"	"T1"	"L1"	"B"	"C"	"T2"	"L2"	"B"	"C"	"T2"	
LESS THAN 20'-0"	30'-0" OR LESS	1'-6"	2'-0"	⅜"	6'-0"	2'-0"	2'-0"	⅜"	14'-0"	2'-0"	2'-0"	⅜"	2"
20'-0" TO 30'-0"	30'-0" OR LESS	1'-6"	2'-0"	½"	6'-0"	2'-0"	2'-0"	⅜"	14'-0" TO 24'-0"	2'-0"	2'-0"	⅜"	3½"

SPAN LENGTH	POST BASE Ⓥ					BOLTED SPLICE						MAXIMUMS		
"S"	"D1"	"S5"	"S6"	"T3"	"T6"	"S1"	"S2"	"S3"	"S4"	"T4"	"T5"	SIGN AREA	"XYZ"	"Z"
LESS THAN 20'-0"	1½"	4	4	2"	¾"	5	-	5	-	2"	⅝"	168 SQ. FT.	2604 C.F.	15'-6"
20'-0" TO 30'-0"	2"	4	4	2"	¾"	5	3	5	3	2½"	⅝"	252 SQ. FT.	4410 C.F.	17'-6"

Ⓥ NOTE: DENOTES MAIN LOAD CARRYING TENSILE MEMBERS OR TENSION COMPONENTS OF FLEXURAL MEMBERS.

Table 10.1.4-2

C. **Balanced Cantilever Standard Design** – Appendix 10.1-A3-1; along with the Structural Detail sheets, Appendix 10.1-A3-2 and Appendix 10.1-A3-3, and General Notes, Appendix 10.1-A0-1, provides the standard structural design information to be used for a Balanced Cantilever Layout, Balanced Cantilevers are typically for VMS sign applications and shall have the sign dead load balanced with a maximum difference one third to two thirds distribution.

D. **Monotube Sheet Guidelines** – The following guidelines apply when using the Monotube Sign Structure Appendix 10.1-A0-1; 10.1-A1-1, 2, and 3; 10.1-A2-1, 2, and 3; 10.1-A3-1, 2, and 3; 10.1-A4-1, 2, and 3; and 10.1-A5-1.

1. Each sign structure shall be detailed and must specify:
    a. Sign structure base Elevation, Station, and Number.
    b. Type of Foundation 1, 2, or 3 shall be used for the Monotube Sign Structures, unless a special design is required. The average Lateral Bearing Pressure for each foundation shall be noted on the Foundation sheet(s).
    c. If applicable, label the Elevation View "Looking Back on Stationing."
2. Designers shall verify the cross-referenced page numbers and details are correct.

E. **Monotube Quantities** – Quantities for structural steel are given in Table 10.1.4-3.

Sign Structure Material Quantities										
ASTM A572 GR. 50 or ASTM 588	Cantilever			Sign Bridge						
	20' ≤	20' to 30'	Balanced	60' ≤	60' to 75'	75' to 90'	90' to 105'	105' to 120'	120' to 135'	135' to 150'
Post (plf)	99	132	132	132	144	176	204	204	215	215
Base PL (ea)	431	490	490	490	578	585	654	654	688	688
Beam, near Post (plf)	116	116	116	116	124	124	139	139	171	195
Span Beam (plf)	116	116	116	116	124	124	162	185	195	195
Corner Stiff. (ea set)	209	204	115	204	238	236	312	312	371	369
Splice Pl #1 (1pr)	482	482	482	482	692	692	892	883	780	780
Splice Pl #2 (1pr)	--	--	--	--	615	615	718	802	715	780
Brackets (ea)	60	60	60	60	65	65	69	69	70	70
6" Hand Hole (ea)	18	18	18	18	18	18	18	18	18	18
6" x 11" Hand Hole (ea)	30	30	30	30	30	30	18	30	30	30
Anchor Bolt PL (ea)	175	175	175	175	185	185	311	311	326	326
Seal Plates (1 bridge)	217	216	216	--	--	--	--	--	--	--

Sign Structure Steel Quantities
Table 10.1.4-3

10.1.5 Foundations

A. **Monotube Sign Bridge and Cantilever Sign Structure Foundation Types** – The Geotechnical Branch shall be consulted as to which foundation type is to be used. Standard foundation designs for standard plan truss-type sign structures are provided in WSDOT Standard Plans G-60.20 and G-60.30 and G-70.20 and G-70.30; and in Section 10.1.5 of this manual. The following paragraphs describe the four types of foundations detailed in this section.

1. The Foundation Type 1, a drilled shaft, is the preferred foundation type. The standard drilled shafts are designed for a lateral bearing pressure of 2,500 psf. See Appendix 10.1-A4-1 and 10.1-A4-2 for Foundation Type 1 standard design information. The Geotech report for this foundation should include the soil friction angle and if temporary casing is required for shaft construction, in additional to the allowable lateral bearing pressures. When the Geotechnical engineer specifies temporary casing, it shall be clearly shown on shaft plans, for each required shaft.
2. The Foundation Type 2 is an alternate to Type 1 when drilled shafts are not suitable to the site. Foundation Type 2 is designed for a lateral bearing pressure of 2,500 psf. See Appendix 10.1-A4-3 for Foundation Type 2 standard design information.
3. The Foundation Type 3 replaces the foundation Type 2 for poor soil conditions where the lateral bearing pressure is between 2,500 psf and 1,500 psf. See Appendix 10.1-A4-3 for Type 3 Foundation standard design information.
4. Barrier Foundations are foundations that include a barrier in the top portion of Foundation Types 1, 2, and 3. Foundation details shall be modified to include Barrier Foundation details. Appendix 10.1-A5-1 details a single slope barrier.

B. **Luminaire, Signal Standard, and Camera Pole Foundation Types** – Luminaire foundation options are shown on Standard Plan J-28.30. Signal Standard and Camera Pole foundation options are provided on Standard Plans J-26.10 and J-29.10 respectively.

C. **Foundation Design** – Shaft type foundations constructed in soil for sign bridges, cantilever sign structures, luminaires, signal standards and strain poles are designed per the current edition of the AASHTO Standard Specifications For Highway Signs, Luminaires, and Traffic Signals; Section 13.10; Embedment of Lightly Loaded Small Poles And Posts. This design method assumes the presence of uniform soil properties with depth, including a single value for Allowable Lateral Bearing Pressure. For foundation locations with multiple soil layers within the anticipated foundation depth (and multiple values of allowable lateral bearing pressure), consideration should be given to using a single “weighted average” value of allowable lateral bearing pressure for design. For foundation locations where a soft soil (with low allowable lateral bearing pressures) is overlaid by a stronger soil (with higher allowable lateral bearing pressures), the foundation can be conservatively designed for the lower allowable lateral bearing pressure value. This design method accounts for the lateral loads applied to the foundation due to the soil pressure (increasing with depth) and the lateral loads applied from the structure above. An additional increase in lateral resistance should not be added for increasing soil lateral pressures with depth.

No provisions for foundation torsional capacity are provided in Section 10.13 of the AASHTO Standard Specifications For Highway Signs, Luminaires, and Traffic Signals. The following approach can be used to calculate torsional capacity of sign structure, luminaire, and signal standard foundations:

Torsional Capacity, T_u,

$$Tu = F^* \tan\varphi D \qquad 10.1.5(1)$$

Where:

F = Total force normal to shaft surface (kip)
D = Diameter of shaft (feet)
φ = Soil friction angle (degree), use smallest for variable soils

1. Monotube Sign Bridge and Cantilever Sign Structures Foundation Type 1 Design – The standard embedment depth "Z", shown in the table on Appendix 10.1-A4-1, shall be used as a minimum embedment depth and shall be increased if the shaft is placed on a sloped surface, or if the allowable lateral bearing pressures are reduced from the standard 2500 psf. The standard depth assumed that the top 4 feet of the C.I.P. cap is not included in the lateral resistance (i.e., shaft depth "D" in the code mentioned above), but is included in the overturning length of the sign structure. Bridge Special Provisions 210201A1.GB8, 210501.GB8, and 210309F2.FB8 shall be included with all Foundation Type 1 shafts.
2. Monotube Sign Bridge and Cantilever Structures Foundation Type 2 and 3 – These foundation designs are standards and shall not be adjusted or redesigned. They are used in conditions where a Foundation Type 1 (shaft) would be impractical due to difficult drilling or construction and when the Geotechnical Engineer specifies their use. The concept is that the foundation excavation would maintain a vertical face in the shape of the Foundation Type 2 or 3. Contractors often request to over-excavate and backfill the hole, after formwork has been used to construct this foundation type. This is only allowed with the Geotechnical engineer's approval, if the forming material is completely removed, and if the backfill material is either CDF or concrete class 3000 or better.
3. Monotube Sign Bridge and Cantilever Structures Special Design Foundations – The Geotechnical Engineer will identify conditions where the foundation types (1, 2, or 3) will not work. In this case, the design forces are calculated, using the AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires, and traffic Signals, and applied at the bottom of the structure base plate. These forces are then considered service loads and the special design foundation is designed with the appropriate Service, Strength, and Extreme Load Combination Limit States and current design practices of the *AASHTO LRFD Bridge Design Specifications* and this manual. Some examples of these foundations are spread footings, columns and shafts that extend above ground adjacent to retaining walls, or connections to traffic barriers on bridges. The anchor rod array shall be used from Tables 10.1.4-1 and 10.1.4-2 of this manual and shall be long enough to develop the rods into the confined concrete core of the foundation. The rod length and the reinforcement for concrete confinement, shown in the top four feet of the Foundation Type 1, shall be used as a minimum.
4. Signal Foundation Design – Bridge Special Provisions 20021.GB8, 20051.GB8, and 20034041.FB8 shall be included with these foundation designs when specified by the Geotechnical engineer.

D. **Foundation Quantities**

1. Barrier quantities are approximate and can be used for all Foundation Types:

Class 4000 Concrete	7.15 CY (over shaft foundation)
Grade 60 rebar	372 lbs

2. Miscellaneous steel quantities (anchor rods, anchor plate, and template) for all Monotube Sign Structure foundation types are listed below (per foundation). Quantities vary with span lengths as shown.

60 feet and under	= 1,002 pounds
61 feet to 90 feet	= 1,401 pounds
91 feet to 120 feet	= 1,503 pounds
121 feet to 150 feet	Barrier mounted sign bridge not recommended for these spans.

3. Monotube Sign Bridge and Cantilever Sign Structure Type 1-3 Foundation quantities for concrete, rebar and excavation are given in Table 10.1.5-1. For Sign Bridges, the quantities shown below are for one foundation and there are two foundations per Sign Bridge. If the depth “Z” shown in the table on Appendix 10.1-A4-1 is increased, these values should be recalculated.

Sign Structure Foundation Material Quantities						
	Cantilever Signs		Sign Bridges			
Concrete Cl. 4000 (cu. yard)	20′ and Under	20′ – 30′	60′ and Under	60′ – 90′	90′ – 120′	120′ – 150′
Type 1	6.3	7.5	7.7	9.4	10.6	11.4
Type 2	8.0	10.5	10.0	12.2	14.1	15.0
Type 3	11.1	14.1	13.0	16.1	18.6	20.0
Rebar Gr. 60 Pounds						
Type 1	685	1,027	1,168	2,251	3,256	4,255
Type 2	772	1,233	1,190	1,724	2,385	2,838
Type 3	917	1,509	1,421	2,136	2,946	3,572
Excavation (cu. yard)						
Type 1	9.8	10.9	10.9	12.8	14.1	14.9
Type 2	20.7	25.7	24.6	29.0	32.9	34.6
Type 3	29.0	34.6	32.9	39.0	44.0	47.8

Table 10.1.5-1

10.1.6 Truss Sign Bridges: Foundation Sheet Design Guidelines

If a Truss sign structure is used, refer to WSDOT *Standard Plans* for foundation details. There are four items that should be addressed when using the WSDOT *Standard Plans*, which are outlined below. For details for F-shape barrier details not shown in *Standard Plans* contact Bridge Office to access archived Bridge Office details.

1. Determine conduit needs. If none exist, delete all references to conduit. If conduit is required, verify with the Region as to size and quantity.
2. Show sign bridge base elevation, number, dimension and station.
3. Transition section shall be per *Standard Plans*.
4. The quantities shall be based on the *Standard Plans* details as needed.

10.2 Bridge Traffic Barriers

10.2.1 General Guidelines

The design criteria for traffic barriers on structures shall be in accordance with Chapter 13 of the *LRFD Bridge Design Specifications* adopted by AASHTO. The following guidelines supplement the requirements in AASHTO.

The WSDOT Bridge and Structures standard for new traffic barriers on structures is a 34″ high Single Slope concrete barrier. It shall be used on all interstates, major highway routes, and over National Highway System (NHS) routes unless special conditions apply.

It shall be the Bridge and Structures Office policy to design traffic barriers for new structures using the Test Level 4 (TL-4) design criteria regardless of the height of the barrier safety shape (e.g., 2′-8″, 2′-10″, or 3′-6″). Loads shall be applied at the top of the barrier safety shape. General notes for traffic barriers on structures shall include the test level of the barrier.

Use of an F Shape concrete bridge traffic barrier shall be limited to locations where there is F Shape concrete barrier on the approach grade to a bridge or for continuity within a corridor.

A Test Level 5 (TL-5) traffic barrier shall be used on new structures under the following conditions:

- “T” intersections on a structure.
- Barriers on structures with a radius of curvature less than 500 ft, greater than 10% Average Daily Truck Traffic (ADTT), and where approach speeds are 50 mph or greater (e.g., freeway off-ramps). TL-4 is adequate for the barrier on the inside of the curve.

See Chapter 13 of the *AASHTO LRFD Bridge Design Specifications* for additional Test Level selection criteria.

A list of crash tested barriers can be found through the FHWA website at: http://safety.fhwa.dot.gov/roadway_dept/policy_guide/road_hardware/barriers/bridgerailings/index.cfm

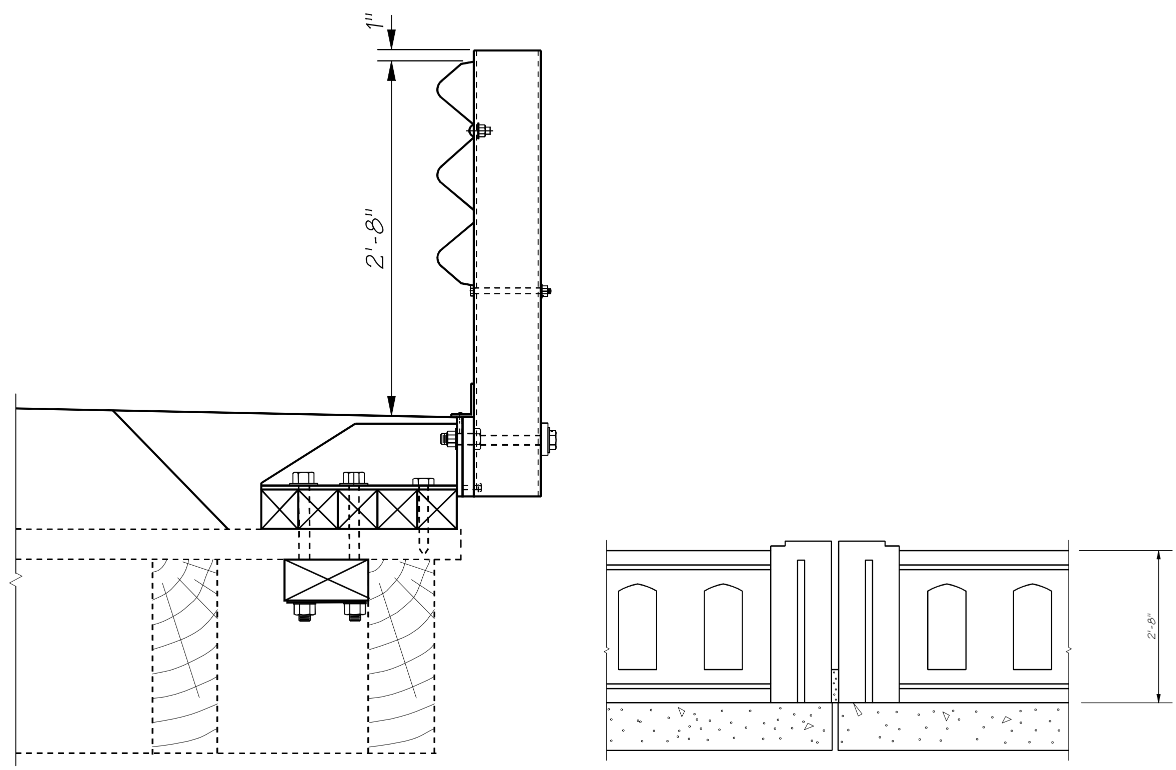
10.2.2 Bridge Railing Test Levels

It must be recognized that bridge traffic barrier performance needs differ greatly from site to site. Barrier designs and costs should match facility needs. This concept is embodied in the *AASHTO LRFD Bridge Design Specifications*. Six different bridge railing test levels, TL-1 thru TL-6, and associated crash test/performance requirements are given in Chapter 13 of these design specifications along with guidance for determining the appropriate test level for a given bridge.

10.2.3 Available WSDOT Designs

A. **Service Level 1 (SL-1) Weak Post Guardrail (TL-2)** – This bridge traffic barrier is a crash tested weak post rail system. It was developed by Southwest Research Institute and reported in NCHRP Report 239 for low-volume rural roadways with little accident history. This design has been utilized on a number of short concrete spans and timber bridges. A failure mechanism is built into this rail system such that upon a 10 kip applied impact load, the post will break away from the mounting bracket. The thrie beam guardrail will contain the vehicle by virtue of its ribbon strength. To ensure minimal or no damage to the bridge deck and stringers, the breakaway connection may be modified for a lower impact load (2 kip minimum) with approval of the Bridge Design Engineer. The 2 kip minimum equivalent impact load is based on evaluation of the wood rail post strength tested in NCHRP Report 239. The appropriate guardrail approach transition shall be a Case 14 placement as shown on WSDOT Standard Plan C-2h. For complete details see Appendix 10.4-A1.

B. **Texas T-411 Aesthetic Concrete Baluster (TL-2)** – Texas developed this standard for a section of highway that was considered to be a historic landmark. The existing deficient concrete baluster rail was replaced with a much stronger concrete baluster that satisfactorily passed the crash test performance criteria set forth by the NCHRP Report 230. For details, visit TXDOT’s Bridge and Structures website at www.txdot.gov/contact_us/bridge.htm.

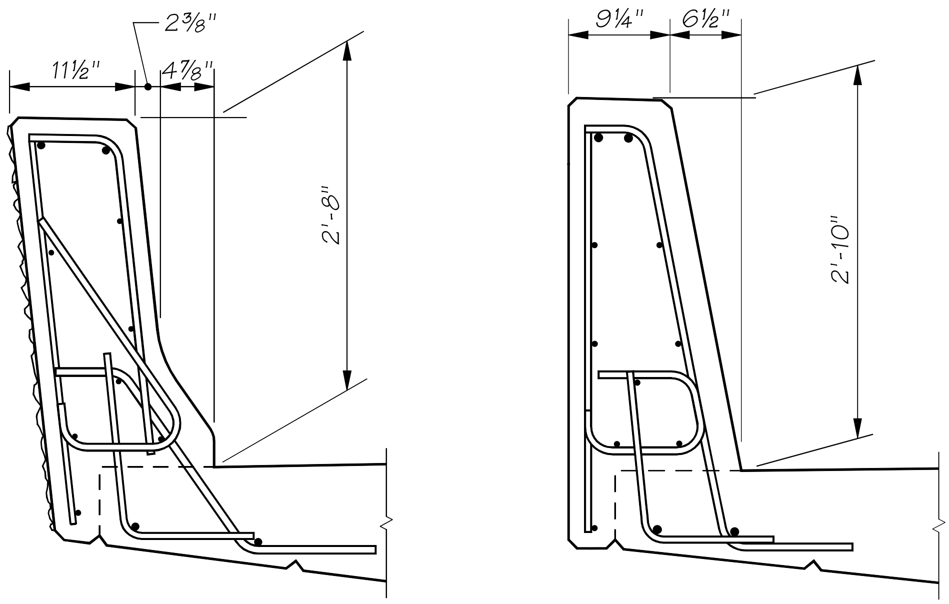


SL-1 Weak Post **Texas T-411**

Figure 10.2.3-1

C. **Traffic Barrier – 32″ Shape F (TL-4)** – This configuration was crash tested in the late 1960s, along with the New Jersey Shape, under NCHRP 230 and again at this test level under NCHRP 350. The steeper vertical shape tested better than the New Jersey face and had less of an inclination to roll vehicles over upon impact. The 3″ toe of the traffic barrier is the maximum depth that an ACP or HMA overlay can be placed. For complete details see Appendix 10.2-A1 and A2.

D. **Traffic Barrier – 34″ Single Slope (TL-4)** – This concrete traffic barrier system was designed by the state of California in the 1990s to speed up construction by using the “slip forming” method of construction. It was tested under NCHRP 350. WSDOT has increased the height from 32″ to 34″ to match the approach traffic barrier height and to allow the placement of one HMA overlay. Due to inherent problems with the “slip forming” method of traffic barrier construction WSDOT has increased the concrete cover on the traffic side from 1½″ to 2½″. For complete details, see Appendix 10.2-A3.

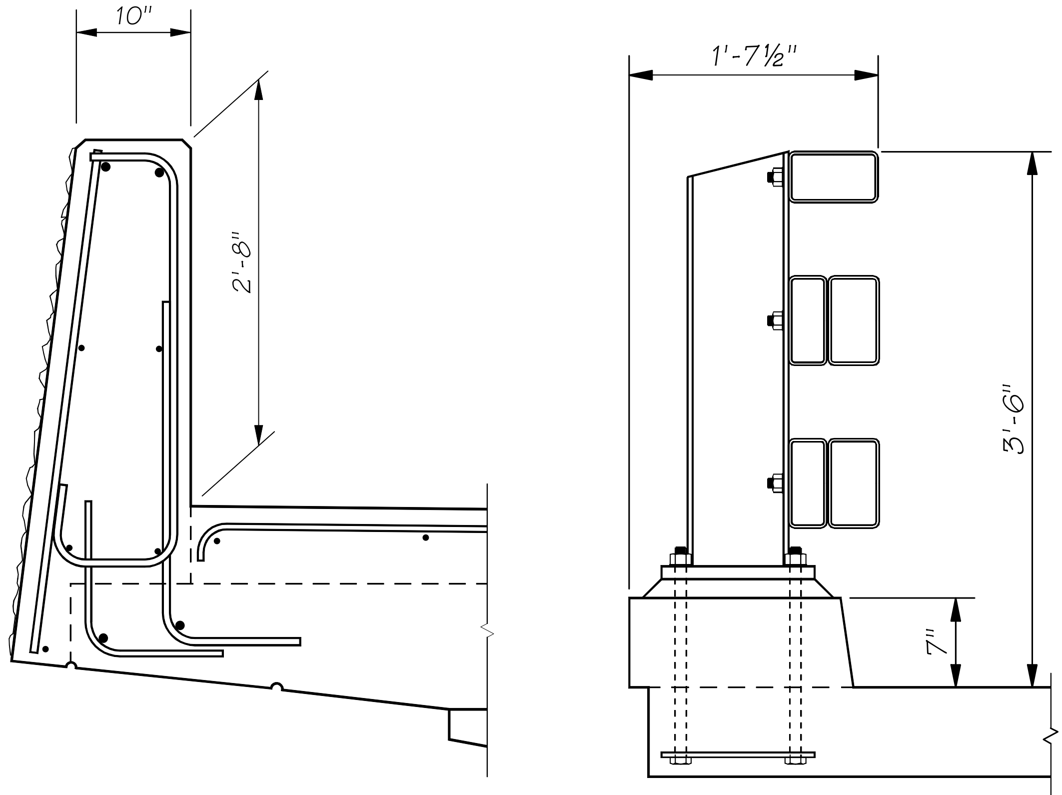


32″ F-Shape **34″ Single Slope**

Figure 10.2.3-2

E. **Pedestrian Barrier (TL-4)** – This crash tested rail system offers a simple to build concrete alternative to the New Jersey and F-Shape configurations. This system was crash tested under both NCHRP 230 and 350. Since the traffic face geometry is better for pedestrians and bicyclists, WSDOT uses this system primarily in conjunction with a sidewalk. For complete details, see Appendix 10.2-A4.

F. **Oregon 3-Tube Curb Mounted Traffic Barrier (TL-4)** – This is another crash tested traffic barrier that offers a lightweight, see-through option. This system was crash tested under both NCHRP 230 and 350. A rigid thrie beam guardrail transition is required at the bridge ends. For details, see the Oregon Bridge and Structure website at http://egov.oregon.gov/odot/hwy/engservices/bridge_drawings.shtml#bridge_200___bridge_rails.

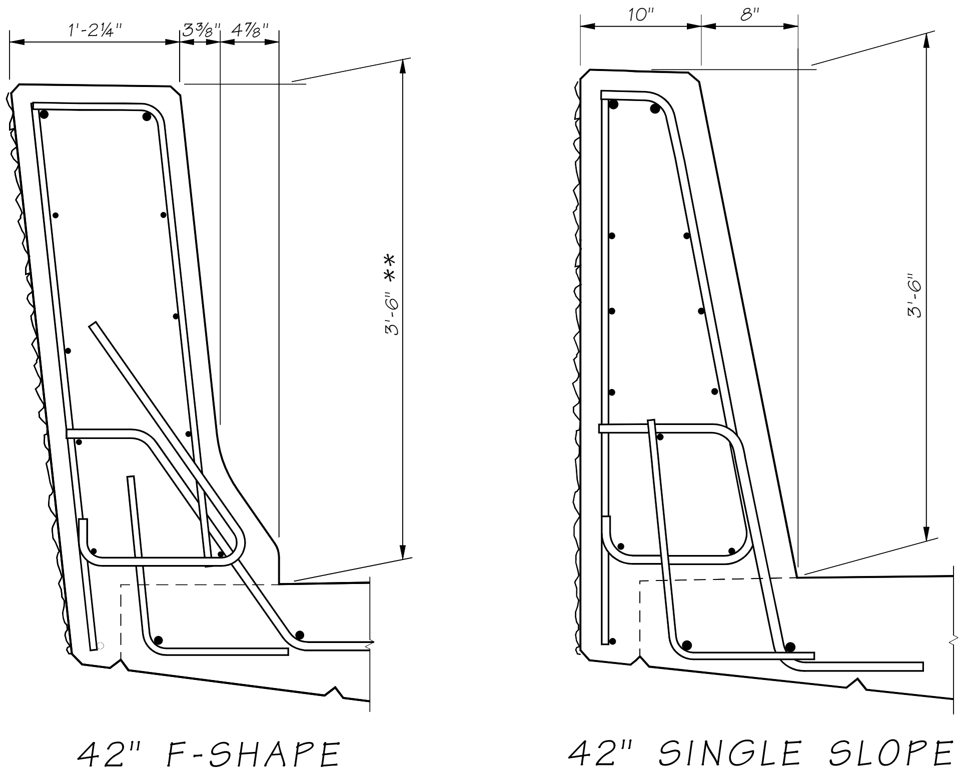


32″ Vertical **Oregon 3-Tube**

Figure 10.2.3-3

G. **Traffic Barrier – 42″ Shape F (TL-4 and TL-5)** – This barrier is very similar to the 32″ F-shape concrete barrier in that the slope of the front surface is the same except for height. For complete details, see Appendix 10.2-A5.

H. **Traffic Barrier – 42″ Single Slope (TL-4 and TL-5)** – This option offers a simple to build alternative to the Shape F configuration. For complete details see Appendix 10.2-A6.



42″ F-Shape **42″ Single Slope**

Figure 10.2.3-4

10.2.4 Design Criteria

A. **Design Values** – AASHTO LRFD Appendix A13 shall be used to design bridge traffic barriers and their supporting elements (i.e. the deck).

Concrete traffic barriers shall be designed using yield line analysis as described in AASHTO LRFD A13.3.1. WSDOT Standard F Shape and Single Slope barriers meet these requirements.

Deck overhangs supporting traffic barriers shall be designed per AASHTO LRFD A13.4. For concrete traffic barriers in Design Case 1, AASHTO requires M_S, the deck overhang flexural resistance, to be greater than M_c of the concrete traffic barrier base. This requirement is consistent with yield line analysis (see AASHTO LRFD CA13.3.1), but results in overconservative deck overhang designs.

In order to prevent this unnecessary overdesign of the deck overhang, the nominal traffic barrier resistance to transverse load R_W (*AASHTO LRFD Bridge Design Specifications* A13.3.1) transferred from the traffic barrier to deck overhang shall not exceed 120 percent of the design force F_t (AASHTO LFRD Table A13.2-1) required for a traffic barrier.

The deck overhang shall be designed in accordance with the requirements of AASHTO LRFD A13.4.2 to provide a flexural resistance M_s, acting coincident with the tensile force T. At the inside face of the barrier M_s may be taken as:

for an interior barrier segment - $M_s = \dfrac{R_w \cdot H}{L_C + 2 \cdot H}$

and for an end barrier segment - $M_s = \dfrac{R_w \cdot H}{L_C + H}$

However, M_s need not be taken greater than M_c at the base. T shall be taken as:

for an interior barrier segment - $T = \dfrac{R_w}{L_C + 2 \cdot H}$

and for an end barrier segment - $T = \dfrac{R_w}{L_C + H}$

The end segment requirement may be waived if continuity between adjacent barriers is provided.

When an HMA overlay is required for initial construction, increase the weight for Shape F traffic barrier. See Section 10.2.4.C for details.

B. **Geometry** – The traffic face geometry is part of the crash test and shall not be modified. Contact the WSDOT Bridge and Structure Office Traffic Barrier Specialist for further guidance.

Thickening of the traffic barrier is permissible for architectural reasons. Concrete clear cover must meet minimum concrete cover requirements but can be increased to accommodate rustication grooves or patterns.

C. **Standard Detail Sheet Modifications** – When designing and detailing a bridge traffic barrier on a superelevated bridge deck the following guidelines shall be used:

- For bridge decks with a superelevation of 8% or less, the traffic barriers (and the median barrier, if any) shall be oriented perpendicular to the bridge deck.
- For bridge decks with a superelevation of more than 8%, the traffic barrier on the low side of the bridge (and median barrier, if any) shall be oriented perpendicular to an 8% superelevated bridge deck. For this situation, the traffic barrier on the high side of the bridge shall be oriented perpendicular to the bridge deck.

The standard detail sheets are generic and may need to be modified for each project. The permissible modifications are:

- Removal of the electrical conduit, junction box, and deflection fitting details.
- Removal of design notes.
- If the traffic barrier does not continue on to a wall, remove W1 and W2 rebar references.
- Removal of the non-applicable guardrail end connection details and verbiage.
- If guardrail is attached to the traffic barrier use either the thrie beam design “F” detail or the W-beam design “F” detail.

If the traffic barrier continues off the bridge, approach slab, or wall, remove the following:

- Guardrail details from all sheets.
- Conduit end flare detail.
- Modified end section detail and R1A or R2A rebar details from all sheets.
- End section bevel.
- Increase the 3″ toe dimension of the Shape F traffic barriers up to 6″ to accommodate HMA overlays.

Barrier Impact Design Forces on Traffic Barrier & Deck Overhang

Parameters		Type F 32 in. (TL-4) Interior	Type F 32 in. (TL-4) End*	Single Slope 34 in. (TL-4) Interior	Single Slope 34 in. (TL-4) End*	Type F 42 in. (TL-4) Interior	Type F 42 in. (TL-4) End*	Single Slope 42 in. (TL-4) Interior	Single Slope 42 in. (TL-4) End*	Type F 42 in. (TL-5) Interior	Type F 42 in. (TL-5) End*	Single Slope 42 in. (TL-5) Interior	Single Slope 42 in. (TL-5) End*
Traffic Barrier Design	Average M_c (ft-kips/ft)	20.55	20.55	19.33	19.33	25.93	25.93	22.42	22.42	29.09	29.09	25.14	25.14
	M_c at Base (ft-kips/ft)	27.15	27.15	26.03	26.03	32.87	32.87	30.66	30.66	36.89	36.89	34.41	34.41
	M_w (ft-kips)	42.47	46.04	46.01	43.16	72.54	71.72	60.66	57.26	98.23	96.93	83.85	79.12
	L_c (ft)	8.62	4.76	9.30	4.81	10.77	5.32	10.63	5.21	14.51	9.26	14.46	9.20
	R_w (kips)	132.82	73.32	126.92	65.69	159.62	78.83	136.17	66.81	241.26	153.91	207.70	132.12
	F_t (kips)	54.00	54.00	54.00	54.00	54.00	54.00	54.00	54.00	124.00	124.00	124.00	124.00
Deck Overhang Design	$1.2{*}F_t$ (kips)	64.80	64.80	64.80	64.80	64.80	64.80	64.80	64.80	148.80	148.80	148.80	148.80
	Design R_w (kips)	64.80	64.80	64.80	64.80	64.80	64.80	64.80	64.80	148.80	148.80	148.80	132.12
	$R_w{*}H/(L_c+aH)$ (ft-kips/ft)**	12.39	23.28	12.27	24.01	12.76	25.72	12.86	26.03	24.21	40.82	24.27	36.42
	Design M_s (ft-kips/ft)	12.39	23.28	12.27	24.01	12.76	25.72	12.86	26.03	24.21	36.89	24.27	34.41
	Design T (kips/ft)	4.65	8.73	4.33	8.47	3.65	7.35	3.68	7.44	6.92	11.66	6.93	10.41
Deck to Barrier Reinforcement	A_s required (in^2/ft)	0.29	0.57	0.30	0.59	0.23	0.47	0.26	0.54	0.44	0.68	0.51	0.73
	A_s provided (in^2/ft)	0.41	0.62	0.41	0.62	0.41	0.62	0.41	0.62	0.67	0.76	0.67	0.76
	S_1 Bars	#5 @ 9 in	#5 @ 6 in	#5 @ 9 in	#5 @ 6 in	#5 @ 9 in	#5 @ 6 in	#5 @ 9 in	#5 @ 6 in	#6 @ 8 in	#6 @ 7 in	#6 @ 8 in	#6 @ 7 in

*Traffic barrier cross sectional dimensions and reinforcement used for calculation of end segment parameters are the same as interior segments. Parameters for modified end segments shall be calculated per AASHTO-LRFD article A13.3, A13.4, and the WSDOT BDM.

**a = 1 for an end segment and 2 for an interior segment

Loads are based on vehicle impact only. For deck overhang design, the designer must also check other limit states per LRFD A13.4.1.

f_y = 60 ksi

f'_c = 4 ksi

Table 10.2.4-1

D. **Miscellaneous Design Information**

- Show the back of pavement seat in the “Plan – Traffic Barrier” detail.
- At roadway expansion joints, show traffic barrier joints normal to centerline except as shown on sheets in Chapter 9 Appendix 9.1-A1-1 and A2-1.
- When an overlay is required the 2′-8″ minimum dimension shown in the “Typical Section – Traffic Barrier” shall be referenced to the top of the overlay.
- When bridge lighting is part of the contract include the lighting bracket anchorage detail sheet.
- Approximate quantities for the traffic barrier sheets are:

Barrier Type	Concrete Weight (lb/ft)	Steel Weight (lb/ft)
32″ F-shape (3″ toe)	455	18.6
32″ F-shape (6″ toe)	510	19.1
34″ Single Slope	490	16.1
42″ F-shape (3″ toe)	710	25.8
42″ F-shape (6″ toe)	765	28.4
42″ Single Slope	670	22.9
32″ Pedestrian	640*	14.7
Using concrete class 4000 with a unit weight of 155 lb/ft^3 *with 6″ sidewalk, will vary with sidewalk thickness		

- Steel Reinforcement Bars:

  S_1 & S_2 or S_3 & S_4 and W_1 & W_2 bars (if used) shall be included in the Bar List. S_1, S_3, and W_1 bars shall be epoxy coated.

10.3 At Grade Traffic Barriers

10.3.1 Median Barriers

The top of the median traffic barrier shall have a minimum width of 6″. If a luminaire or sign is to be mounted on top of the median traffic barrier, then the width shall be increased to accommodate the mounting plate and 6″ of clear distance on each side of the luminaire or sign pole. The transition flare rate shall follow the WSDOT *Design Manual* M 22-01.

A. **Differential Grade Median Barriers** – Barriers at grade are sometimes required in median areas with different roadway elevations on each side. The standard Single Slope barrier can be used for a grade difference up to 10″ for a 2′-10″ safety shape and up to 6″ for a 3′-6″ safety shape. See WSDOT Standard Plans C-13 and C-14a for details.

If the difference in grade elevations is 4′-0″ or less, then the barrier shall be designed using AASHTO LFD barrier loading with the following requirements:

1. The differential grade traffic barrier shall be designed to 10 kips, as a minimum loading.
2. For soil loads without traffic impact, the barrier shall be designed as a combination of a standard retaining wall (barrier weight resists overturning and sliding) and a cantilever retaining wall (passive soil resistance resists overturning and sliding) using the factors of safety for retaining walls found in this manual. Earthquake Group VII loads need not be considered.
3. Traffic impact loads shall be added to the side of the barrier retaining soil.
4. For soil loads with traffic impact, the barrier shall be designed as a combination of a standard retaining wall (barrier weight resists overturning and sliding) and a cantilever retaining wall (passive soil resistance resists overturning and sliding). The design shall be based on stability requirements for overturning with a factor of safety $M_R/M_O \geq 1.3$ and sliding $F_R/F_S \geq 1.3$.
5. To meet the stability requirements of item 4, several feet of barrier length are required. The length of the barrier required for stability shall be no more than 10 times the overall height. The barrier shall be designed for lateral bending over this length assuming it acts as a beam on an elastic foundation for this length with a 10K point load. The barrier shall also be designed for torsion from the moment induced by the 10K load about the barrier c.g.
6. A special impact analysis shall be performed at the barrier ends if the barrier terminates without being connected to a rigid object or dowelled to another barrier. Differential barrier deflection from traffic impact may cause a vehicle to “snag” on the undeflected barrier. The barrier depth may need to be increased at the end to prevent this deflection.
7. The differential grade traffic barrier shall have dummy joints at 8 to 12 foot centers based on project requirements.
8. Full depth expansion joints with shear dowels at the top will be required at intervals based on analysis but not to exceed a 120′-0″ maximum spacing.

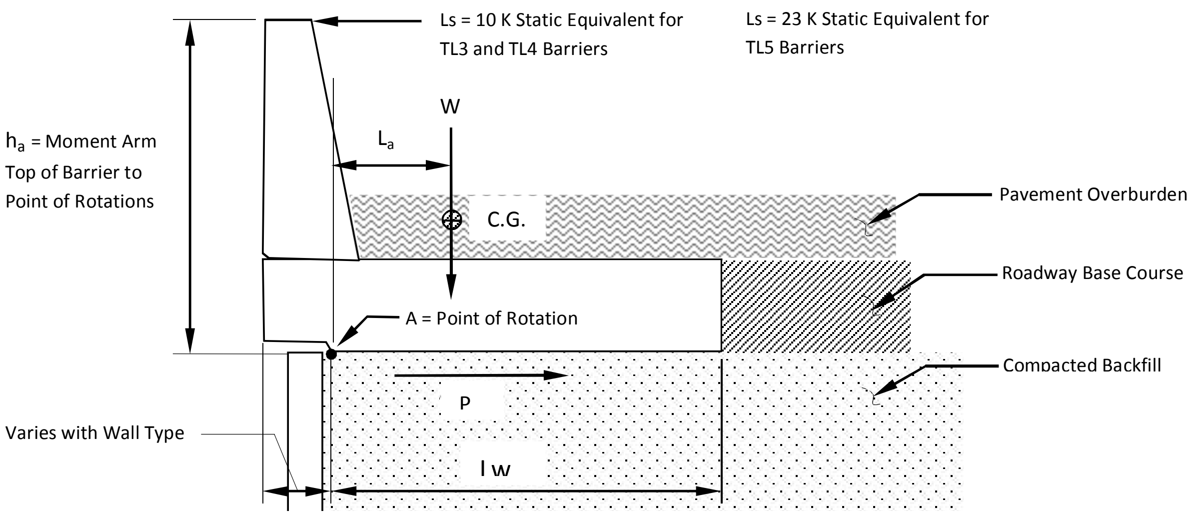
Median traffic barriers with a grade difference greater that 4′-0″ shall be designed as standard plan retaining walls with a traffic barrier at the top and a barrier shape at the cut face.

10.3.2 Shoulder Barriers

At grade CIP shoulder barriers are sometimes used adjacent to bridge sidewalk barriers in lieu of standard precast Type 2 barriers. This barrier cross section has an equivalent mass and resisting moment for stability as the embedded double-face New Jersey Traffic Barrier which has been satisfactorily crash tested. A wire rope and pin connection shall be made at the bridge barrier end section per Standard Plan C-8. If a connection is made to an existing traffic barrier or parapet on the bridge, 15-inch long holes shall be drilled for the wire rope connection and shall be filled with an epoxy bonding agent.

10.3.3 Traffic Barrier Moment Slab

A. **General** – The guidelines provided herein are based on NCHRP Report 663 with the exception that a re-sistance factor of 0.5 shall be used to determine rotational resistance. This guideline is applicable for TL-3, TL-4, and TL-5 barrier systems as defined in Section 13 of *AASHTO LRFD Bridge Design Specifications*.



Global Stability of Barrier–Moment Slab System
Figure 10.3.3-1

B. **Guidelines for Moment Slab Design**

1. **Structural Capacity** – The structural capacity of the barrier and concrete moment slab shall be designed using impulse Traffic Loads (TL-3, TL-4, TL-5) in accordance with Sections 5 and 13 of *AASHTO LRFD Bridge Design Specifications*. Any section along the moment slab should not fail in shear, bending, or torsion when the barrier is subjected to the design impact load. The moment slab reinforcement shall be designed to resist forces developed at the base of the barrier. The torsion capacity of the moment slab must be equal to or greater than the traffic barrier moment generated by the specified TL impulse load (TL-3, TL-4, TL-5) applied to the top of the barrier.

2. **Global Stability** – Sliding and overturning stability of the moment slab shall be based on an Equivalent Static Load (ESL) applied to the top of the traffic barrier. For TL-3 and TL-4 barrier systems, the ESL shall be 10 kips. For TL-5 barrier systems, the ESL shall be 23 kips.

   The Equivalent Static Load (ESL) is assumed to distribute over the length of the moment slab through rigid body behavior. Any coupling between adjacent moment slabs or friction that may exist between free edges of the moment slab and the surrounding soil should be neglected.

3. **Minimum and Maximum Dimensions** – Moment slabs shall have a minimum width of 4.0 feet measured from the point of rotation to the heel of the slab and a minimum average depth of 0.83 feet. Moment slabs meeting these minimum requirements are assumed to provide rigid body behavior up to a length of 60 feet for end barrier and interior barrier impacts.

   Rigid body behavior may be increased from 60 feet to a maximum of 120 feet if the torsional rigidity constant of the moment slab is proportionately increased and the reinforcing steel is designed to resist combined shear, moment, and torsion from TL impulse loads.

   For example: Rigid Body Length = (J'/J60)x(60 ft.) < 120 feet

   The torsional rigidity constant for moment slabs shall be based on a solid rectangle using the following formula:

$$J = a \cdot b^3 \left[\frac{16}{3} - 3.36\left(\frac{b}{a}\right)\left(1 - \frac{b^4}{12a^4}\right)\right]$$

   where:

   2a = total width of moment slab
   2b = average depth of moment slab

   For example:

   Minimum Moment Slab Width = 48 inches: a = 24 inches
   Minimum Moment Slab Average Depth = 10 inches: b = 5 inches
   J = J60 = 13,900 in^4

4. **Sliding of the Barrier** – The factored static resistance to sliding (ϕP) of the barrier-moment slab system along its base shall satisfy the following condition (Figure 2).

$$\phi P \geq \gamma Ls \qquad (1)$$

   where:

   Ls = equivalent static load (10 kips for TL-3 and TL-4) (23 kips for TL-5)
   ϕ = resistance factor (0.8) Supersedes AASHTO 10.5.5.3.3—Other Extreme Limit States
   γ = load factor (1.0) for TL-3 and TL-4 [crash tested extreme event]
   load factor (1.2) for TL-5 [untested extreme event]
   P = static resistance (kips)
   P shall be calculated as:

$$P = W \tan \phi_r \qquad (2)$$

   where:

   W = weight of the monolithic section of barrier and moment slab between joints or assumed length of rigid body behavior whichever is less, plus any material laying on top of the moment slab
   ϕ_r = friction angle of the soil on the moment slab interface (°)

   If the soil-moment slab interface is rough (e.g., cast in place), ϕ_r is equal to the friction angle of the soil ϕ_s. If the soil-moment slab interface is smooth (e.g., precast), tan ϕ_r shall be reduced accordingly (0.8 tan ϕ_s).

5. **Overturning of the Barrier** – The factored static moment resistance (ϕM) of the barrier-moment slab system to over-turning shall satisfy the following condition (Figure 1).

   The factored static moment resistance (ϕM) of the barrier-moment slab system to overturning shall satisfy the following condition (Figure 1).

   $$\phi M \geq \gamma L_s h_a \qquad (3)$$

   where:

   A = point of rotation, where the toe of the moment slab makes contact with compacted backfill adjacent to the fascia wall
   L_w = width of moment slab
   L_s = equivalent static load (10 kips for TL-3 and TL-4) (23 kips for TL-5)
   φ = resistance factor (0.5) Supersedes AASHTO 10.5.5.3.3—Other Extreme Limit States and NCHRP Report 663
   γ = load factor (1.0) for TL-3 and TL-4 [crash tested extreme event]
   load factor (1.2) for TL-5 [untested extreme event]
   h_a = moment arm taken as the vertical distance from the point of impact due to the dynamic force (top of the barrier) to the point of rotation A
   M = static moment resistance (kips-ft)
   M shall be calculated as:

   $$M = W (L_a) \qquad (4)$$

   W = weight of the monolithic section of barrier and moment slab between joints or assumed length of rigid body behavior whichever is less, plus any material laying on top of the moment slab
   L_a = horizontal distance from the center of gravity of the weight W to point of rotation A

   The moment contribution due to any coupling between adjacent moment slabs, shear strength of the overburden soil, or friction which may exist between the backside of the moment slab and the surrounding soil should be neglected.

C. **Guidelines for the Soil Reinforcement** – Design of the soil reinforcement shall be in accordance with the WSDOT *Geotechnical Design Manual* M 46-03, Chapter 15.

D. **Design of the Wall Panel** – The wall panels shall be designed to resist the dynamic pressure distributions as defined in the WSDOT *Geotechnical Design Manual*, Chapter 15.

   The wall panel shall have sufficient structural capacity to resist the maximum design rupture load for the wall reinforcement designed in accordance with the WSDOT *Geotechnical Design Manual*, Chapter 15.

   The static load is not included because it is not located at the panel connection.

10.3.4 Precast Traffic Barrier

A. **Concrete Barrier Type 2** – “Concrete Barrier Type 2” (see WSDOT Standard Plan C-8) may be used on bridges for median applications or for temporary traffic control based on the following guidelines:

   1. For temporary applications, no anchorage is required if there is 2 feet or greater slide distance between the back of the traffic barrier and an object and 3 feet or greater to the edge of the bridge deck or a severe drop off (see WSDOT *Design Manual* M 22-01).

   2. For permanent applications in the median, no anchorage will be required if there is 2 feet or greater slide distance between the traffic barrier and the traffic lane.

3. For temporary applications, the traffic barrier shall not be placed closer than 9 inches or 6 inches to the edge of a bridge deck or substantial drop-off and shall be anchored (see WSDOT Standard Plans K-80.35 and K-80.37).
4. The traffic barrier shall not be used to retain soil that is sloped or greater than the barrier height or soil that supports a traffic surcharge.

B. **Concrete Barrier Type 4 and Alternative Temporary Concrete Barrier** – "Concrete Barrier Type 4 (see the WSDOT Standard Plan C-8a), is not a free standing traffic barrier. This barrier shall be placed against a rigid vertical surface that is at least as tall as the traffic barrier. In addition, Alternative Temporary Concrete Barrier Type 4 – Narrow Base (WSDOT Standard Plan K-80.30) shall be anchored to the bridge deck as shown in WSDOT Standard Plan K-80.37. The "Concrete Barrier Type 4 and Alternative Temporary Concrete Barrier" are not designed for soil retention.

10.4 Bridge Traffic Barrier Rehabilitation

10.4.1 Policy

The bridge traffic barrier retrofit policy is: “to systematically improve or replace existing deficient rails within the limits of roadway resurfacing projects.” This is accomplished by:

- Utilizing an approved crash tested rail system that is appropriate for the site or
- Designing a traffic barrier system to the strength requirements set forth by Section 2 of AASHTO Standard Specifications for Highway Bridges, 17th edition.”

10.4.2 Guidelines

A strength and geometric review is required for all bridge rail rehabilitation projects. If the strength of the existing bridge rail is unable to resist an impact of 10 kips or has not been crash tested, then modifications or replacement will be required to improve its redirectional characteristics and strength. Bridges that have deficient bridge traffic barriers were designed to older codes. The AASHTO LFD load of 10 kips shall be used in the retrofit of existing traffic barrier systems constructed prior to the year 2000. The use of the AASHTO LRFD criteria to design traffic barrier rehabs will result in a bridge deck that has insufficient reinforcement to resist moment from a traffic barrier impact load and will increase the retrofit cost due to expensive deck modifications.

10.4.3 Design Criteria

Standard thrie beam guardrail post spacing is 6′-3″ except for the SL-1 Weak Post, which is at 8′-4″. Post spacing can be increased up to 10′-0″ if the thrie beam guardrail is nested (doubled up).

Gaps in the guardrail are not allowed because they produce snagging hazards. The exceptions to this are:

- Movable bridges at the expansion joints of the movable sections.
- At traffic gates and drop down net barriers.
- At stairways.

Design F guardrail end sections will be used at the approach and trailing end of these gaps.

For Bridge Traffic Barrier Rehabilitation the following information will be needed from the Region Design office:

- Bridge Site Data Rehabilitation Sheet – DOT Form 235-002A.
- Photos, preferably digital Jpegs.
- Layout with existing dimensions.
- Standard Plan thrie beam guardrail transitions (selected by Region Design office) to be used at each corner of the bridge (contact bridges and structures office for thrie beam height).
- Location of any existing utilities.
- Measurements of existing ACP to top of curb at the four corners, midpoints and the locations of minimum and maximum difference (five locations each side as a minimum).
- Diagram of the location of Type 3 anchors, if present, including a plan view with vertical and horizontal dimensions of the location of the Type 3 anchor connection relative to the intersecting point of the back of pavement seat with the curb line.
- The proposed overlay type, quantities of removal and placement.
- For timber bridges, the field measurement of the distance from the edge of bridge deck to the first and second stringer is required for mounting plate design.

Placement of the retrofit system will be determined from the WSDOT *Design Manual* M 22-01. Exceptions to this are bridges with sidewalk strength problems, pedestrian access issues, or vehicle snagging problems.

10.4.4 WSDOT Bridge Inventory of Bridge Rails

The WSDOT Bridge Preservation office maintains an inventory of all bridges in the state on the State of Washington Inventory of Bridges.

Concrete balusters are deficient for current lateral load capacity requirements. They have approximately 3 kips of capacity whereas 10 kips is required.

The combination high-base concrete parapet and metal rail may or may not be considered adequate depending upon the rail type. The metal rail Type R, S, and SB attached to the top of the high-base parapet are considered capable of resisting the required 5 kips of lateral load. Types 3, 1B, and 3A are considered inadequate. See the WSDOT *Design Manual* M 22-01 for replacement criteria.

10.4.5 Available Retrofit Designs

A. **Washington Thrie Beam Retrofit of Concrete Balusters** – This system consists of thrie beam guardrail stiffening of existing concrete baluster rails with timber blockouts. The Southwest Research Institute conducted full-scale crash tests of this retrofit in 1987. Results of the tests were satisfactory and complied with criteria for a Test Level 2 (TL2) category in the Guide Specifications. For complete details see Appendix 10.4-A1-1.

B. **New York Thrie Beam Guardrail** – This crash tested rail system can be utilized at the top of a raised concrete sidewalk to separate pedestrian traffic from the vehicular traffic or can be mounted directly to the top of the concrete deck. For complete details see Thrie Beam Retrofit Concrete Curb in Appendix 10.4-A1-3.

C. **Concrete Parapet Retrofit** – This is similar to the New York system. For complete details see Appendix 10.4-A1-2.

D. **SL-1 Weak Post** – This design has been utilized on some short concrete spans and timber bridges. A failure mechanism is built into this rail system so that upon impact with a 10 kip load the post will break away from the mounting bracket. The thrie beam guardrail will contain the vehicle by virtue of its ribbon strength. To ensure minimal damage to the bridge deck and stringers, the breakaway connection may be modified for a lower impact load (2 kip minimum) with approval of the Bridge Design Engineer. For complete details, see Appendix 10.4-A1-4.

10.4.6 Available Replacement Designs

A. **Traffic Barrier – Shape F Retrofit** – This is WSDOT's preferred replacement of deficient traffic barriers and parapets on high volume highways with a large truck percentage. All interstate highway bridges shall use this type of barrier unless special conditions apply. For complete details see Appendix 10.4-A2.

10.5 Bridge Railing

10.5.1 Design

WSDOT pedestrian and bike/pedestrian railings are designed in accordance with Chapter 13 in the AASHTO LRFD *Bridge Design Specifications*. The AASHTO LRFD *Bridge Design Specifications* calls for a minimum of 42″ for bicycle railings whereas WSDOT requires a minimum height of 54″ on structures. The railings in Section 10.5.2 are not designed for vehicular impact loads assuming location is low speed, location is outside of Design Clear Zone as defined in Chapter 1600 in WSDOT *Design Manual* M 21-01, or location has minimal safety consequence from collapse of railing. Railings for other locations shall be designed for vehicular impact loads in accordance with Chapter 13 and/or 15 in the AASHTO LRFD *Bridge Design Specifications*. Emergency and maintenance access shall be considered.

10.5.2 Railing Types

A. **Bridge Railing Type Pedestrian** – This pedestrian railing is designed to sit on top of the 32″ and 34″ traffic barriers and to meet pedestrian height requirements of 42″. For complete details see Appendix 10.5-A1.

B. **Bridge Railing Type BP and S-BP** – These railings are designed to meet WSDOT's minimum bicycle height requirements of 54″, and sit on top of the 32″ and 34″ traffic barriers.

   There are two versions—the BP and S-BP. The BP is the standard railing and is made out of aluminum. The S-BP is the steel version designed for use in rural areas because of aluminum theft. For complete details see Appendix 10.5-A2 and A3.

C. **Pedestrian Railing** – This railing is designed to sit on top of a six-inch curb on the exterior of a bridge sidewalk. It meets the bicycle height requirements of 54″. For complete details see Appendix 10.5-A4.

D. **Bridge Railing Type Chain Link Snow Fence and Bridge Railing Type Snow Fence** – This railing is designed to prevent large chunks of plowed snow from falling off the bridge on to traffic below. For complete details see Appendix 10.5-A5-1 through 10.5-A5-3.

E. **Bridge Railing Type Chain Link Fence** – This railing is designed to minimize the amount of objects falling off the bridge on to traffic below. For complete details see Appendix 10.5-A5-4.

10.6 Bridge Approach Slabs

Bridge approaches typically experience two types of settlement, global and local. Global settlement is consolidation of the deeper natural foundation soils. Local settlement is mainly compression of fill materials directly beneath the approach pavement due to construction. The combination of global and local settlements adjacent to the bridge end piers form the characteristic “bump” in the pavement at the bridge. The approach slab significantly reduces local settlement and will provide a transition to the long term roadway differential settlements. Generally, abutments with a deep foundation will have greater differential roadway settlements than spread footing foundations.

When Are Approach Slabs Required – Bridge approach slabs are required for all new and widened bridges, except when concurrence is reached between the Geotechnical Branch, the Region Design Project Engineer Office, and the Bridge and Structures Office, that approach slabs are not appropriate for a particular site. In accordance with WSDOT *Design Manual* M 21-01, the State Geotechnical Engineer will include a recommendation in the geotechnical report for a bridge on whether or not bridge approach slabs should be used at the bridge site. Factors considered while evaluating the need for bridge approach slabs include the amount of expected settlement and the type of bridge structure.

Standard Plan A-40.50 – The Standard Plan A-40.50 is available for the Local Agencies (or others) to use or reference in a contract. Bridge and Structures Office designs will provide detailed information in a customized approach slab Plan View and show the approach slab length on the Bridge Layout Sheet.

Bridge Runoff – Bridge runoff at the abutments shall be carried off and collected at least 10 feet beyond the bridge approach slab. Drainage structures such as grate inlets and catch basins shall be located in accordance with Standard Plan B-95.40 and the recommendations of the Hydraulics Branch.

Approach Pay Item –All costs in connection with constructing bridge approach slabs are included in the unit contract price per square yard for “Bridge Approach Slab.” The pay item includes steel reinforcing bars, approach slab anchors, concrete, and compression seals.

10.6.1 Notes to Region for Preliminary Plan

All bridge preliminary plans shall show approach slabs at the ends of the bridges. In the Notes to Region in the first submittal of the Preliminary Plan to the Region, the designer shall ask the following questions:

1. Bridge approach slabs are shown for this bridge, and will be included in the Bridge PS&E. Do you concur?
2. The approach ends of the bridge approach slabs are shown normal to the survey line (a) with or (b) without steps (the designer shall propose one alternative). Do you concur?
3. Please indicate the pavement type for the approach roadway.

Depending on the type and number of other roadway features present at the bridge site (such as approach curbs and barriers, drainage structures, sidewalks, utilities and conduit pipes) or special construction requirements such as staged construction, other questions in the Notes to Region pertaining to the bridge approach slabs may be appropriate.

Special staging conditions exist when the abutment skew is greater than 30° and for wide roadway widths. This includes bridge widenings with (or without) existing bridge approach slabs. The preliminary plan should include details showing how these conditions are being addressed for the bridge approach slabs, and the designer shall include appropriate questions in the Notes to Region asking for concurrence with the proposed design.

10.6.2 Approach Slab Design Criteria

The standard bridge approach slab design is based on the following criteria:

1. The bridge approach slab is designed as a slab in accordance with AASHTO LRFD. (Strength Limit State, IM = 1.33, no skew).
2. The support at the roadway end is assumed to be a uniform soil reaction with a bearing length that is approximately ⅓ the length of the approach slab, or 25′/3 = 8′.
3. The Effective Span Length (S_{eff}), regardless of approach length, is assumed to be:

   25′ approach – 8′ = 17′
4. Longitudinal reinforcing bars do not require modification for skewed approaches up to 30° or for slab lengths greater than 25′.
5. The approach slab is designed with a 2″ concrete cover to the bottom reinforcing.

10.6.3 Bridge Approach Slab Detailing

The bridge approach slab and length along center line of project shall be shown in the Plan View of the Bridge Layout sheet. The Bridge Plans will also include approach slab information as shown on Plan Sheets 10-A1-1, 10-A1-2, and 10-A1-3. The Approach Slab Plan sheets should be modified as appropriate to match the bridge site conditions. Approach slab Plan Views shall be customized for the specific project and all irrelevant details shall be removed.

Plan View dimensions need to define the plan area of the approach slab. The minimum dimension from the bridge is 25′. If there are skewed ends, then dimensions need to be provided for each side of the slab, or a skew angle and one side, in addition to the width. For slabs on a curve, the length along the project line and the width need to be shown.

Similar to Bridge Traffic Barrier detailing, approach slab steel detailing need only show size, spacing, and edge clearance. The number and total spaces can be determined by the contractor. If applicable, the traffic barrier AS1 and AS2 along with the extra top transverse bar in the slab need to be shown in the Plan View. AS1 bars shall be epoxy coated. Also remember that the spacing of the AS1 bars decreases near joints. When the skew is greater than 20°, then AP8 bars need to be rotated at the acute corners of the bridge approach slab.

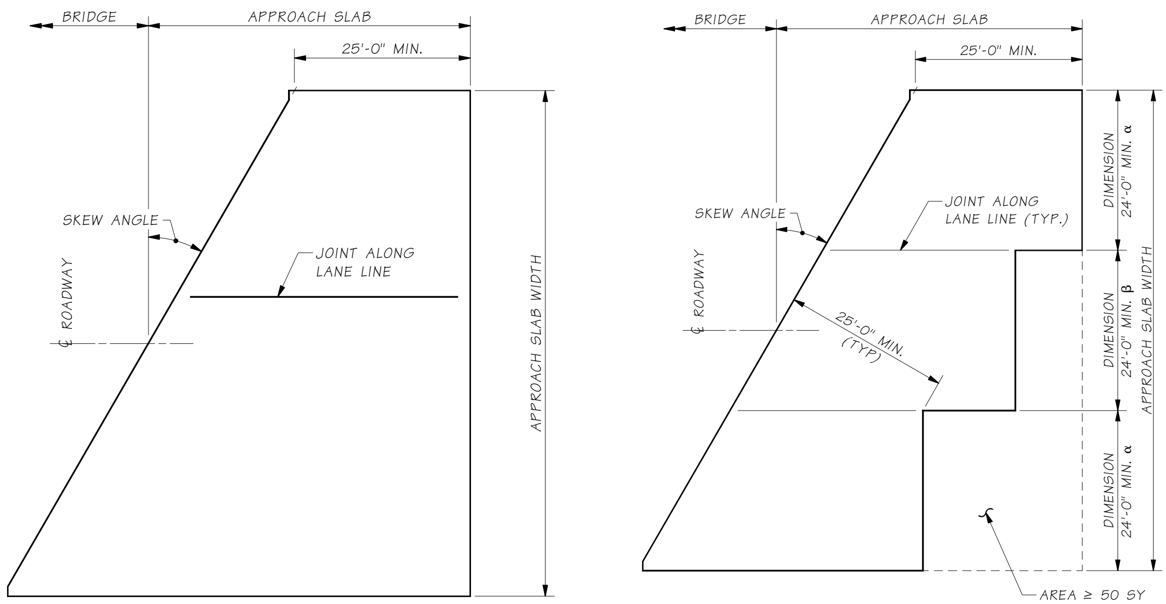
Bending diagrams shall be shown for all custom reinforcement. All Approach Slab sheets will have the AP2 and AP7 bars. If there is a traffic barrier, then AP8, AS1, and AS2 bars shall be shown.

Additional layout and details may be required to address special roadway features and construction requirements such as: roadway curbs and barriers, sidewalks, utilities and conduits and staging. This means, if sidewalks and interior barriers (such as traffic-pedestrian barriers) are present, special details will be required in the Bridge Plans to show how the sidewalks and interior barriers are connected to and constructed upon the bridge approach slab. If the bridge construction is staged, then the approach slabs will also require staged construction.

10.6.4 Skewed Approach Slabs

For all skewed abutments, the roadway end of the bridge approach slab shall be normal to the roadway centerline. The Bridge Design Engineer should be consulted when approach slab skew is greater than 30°. Higher skewed bridges require modifications to the bottom mat reinforcement, and may require expansion joint modifications.

The roadway end of the approach may be stepped to reduce the size or to accommodate staging construction widths. A general rule of thumb is that if the approach slab area can be reduced by 50 SY or more, then the slab should be stepped. At no point should the roadway end of the approach slab be closer than 25′ to the bridge. These criteria apply to both new and existing bridge approach slabs. If stepped, the design should provide the absolute minimum number of steps and the longitudinal construction joint shall be located on a lane line. See Figure 10.6.4-1 for clarification.



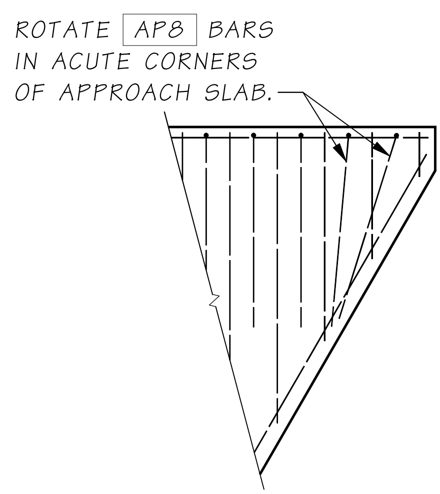
Skewed Approach ~ Typical **Skewed Approach ~ Stepped**

α - DIMENSION MAY BE ONE LANE WIDTH PLUS THE SHOULDER WIDTH IF THE SHOULDER ≥ 8'-0".

β - DIMENSION MAY BE TWO LANE WIDTHS.

Skewed Approach
Figure 10.6.4-1

In addition, for bridges with traffic barriers and skews greater than 20°, the AP8 bars shall be rotated in the acute corners of the bridge approach slabs. Typical placement is shown in the flared corner steel detail, Figure 10.6.4-2.



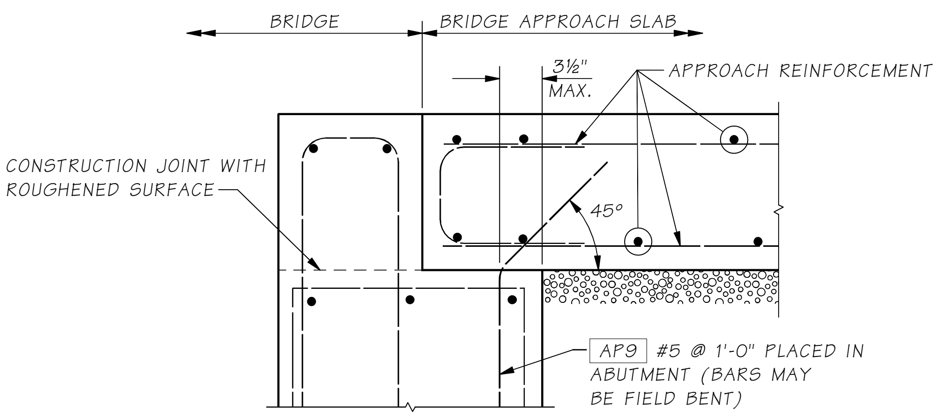
Flared Corner Steel
Figure 10.6.4-2

10.6.5 Approach Anchors and Expansion Joints

For semi-integral abutments or stub abutments, the Bridge Designer must check the joint design to make sure the movement of the standard joint is not exceeded. In general, the approach slab is assumed to be stationary and the joint gap is designed to vary with the bridge movement. Approach Slab Sheets 10-A1-3 and Standard Plan A-40.50 detail a typical 2½″ compression seal. For approach slabs with barrier, the compression seal should extend into the barrier.

Approach slab anchors installed at bridge abutments should be as shown in the Bridge Plans. For bridges with semi-integral type abutments, this can be accomplished by showing the approach slab anchors in the End Diaphragm or Pavement Seat details.

L Type Abutments – L type abutments do not require expansion joints or approach anchors because the abutment and approach slab are both considered stationary. A pinned connection is preferred. The L type abutment anchor detail, as shown sign in Figure 10.6.5-1, must be added to the abutment plan sheets. The pinned anchor for bridges with L type abutments shall be a #5 bar at one foot spacing, bent as shown, with 1′-0″ embedment into both the pier and the bridge approach slab. This bar shall be included in the bar list for the bridge substructure.



L Type Abutment Anchor Detail
Figure 10.6.5-1

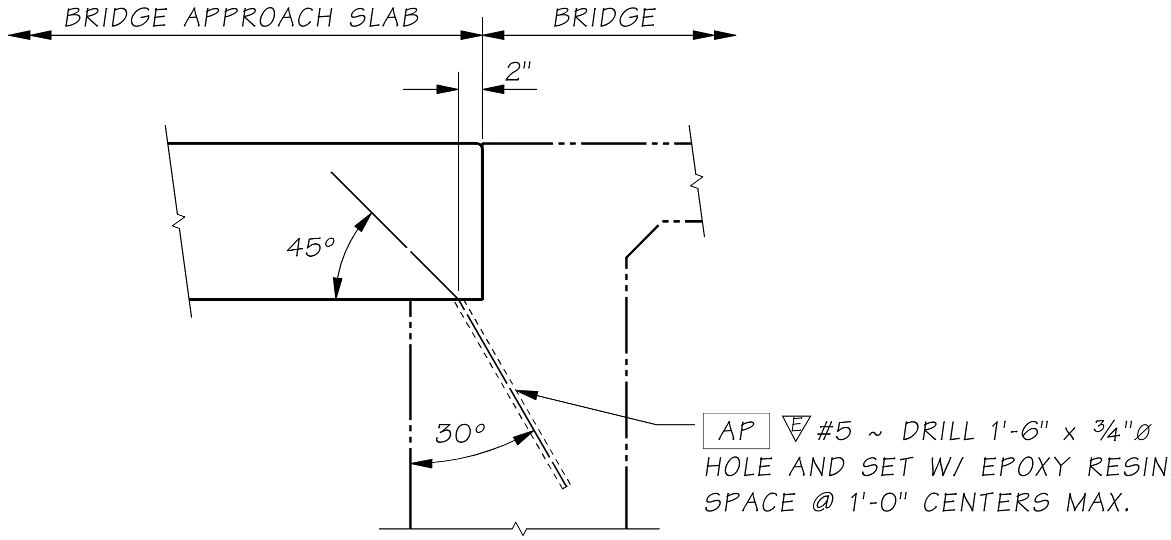
10.6.6 Approach Slab Addition or Retrofit to Existing Bridges

When approach slabs are to be added or replaced on existing bridges, modification may be required to the pavement seats. Either the new approach slab will be pinned to the existing pavement seat, or attached with approach anchors with a widened pavement seat. Pinning is a beneficial option when applicable as it reduces the construction cost and time.

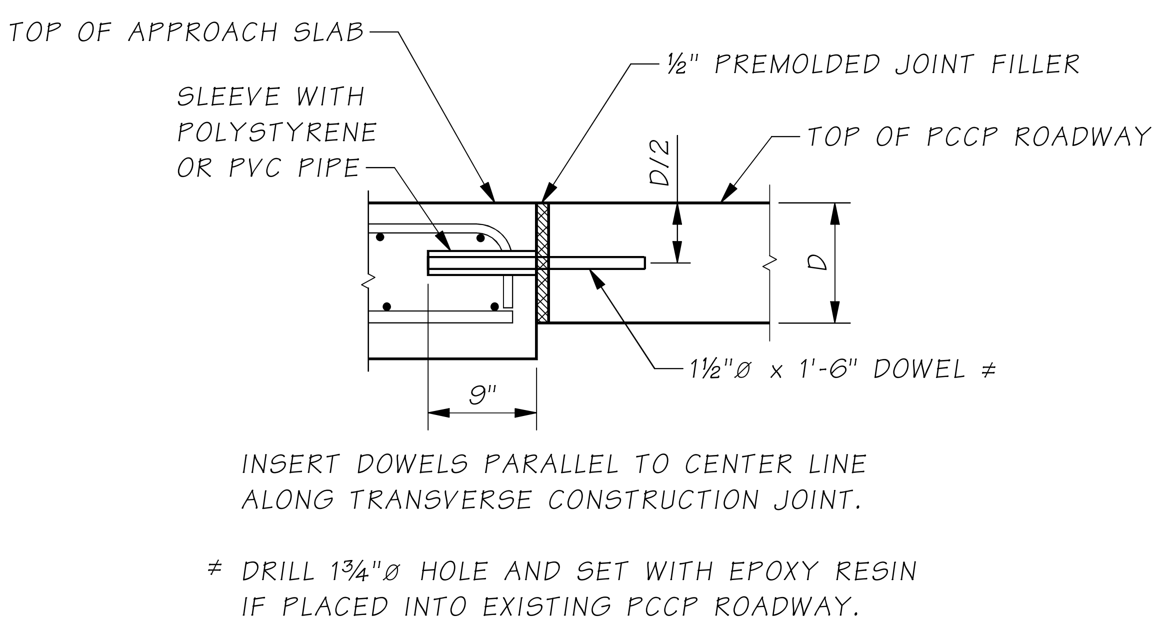
The pinning option is only allowed on semi-integral abutments as an approach slab addition or retrofit to an existing bridge. Figure 10.6.6-1 shows the pinning detail. As this detail eliminates the expansion joint between the approach slab and the bridge, the maximum bridge superstructure length is limited to 150′. The Bridge Design Engineer may modify this requirement on a case by case basis. Additionally, if the roadway end of the approach slab is adjacent to PCCP roadway, then the detail shown in Figure 10.6.6-2 applies. PCCP does not allow for as much movement as HMA and a joint is required to reduce the possibility of buckling.

When pinning is not applicable, then the approach slab must be attached to the bridge with approach anchors. If the existing pavement seat is less than 10 inches, the seat shall be replaced with an acceptable, wider pavement seat. The Bridge Design Engineer may modify this requirement on a site-specific basis. Generic pavement seat repair details are shown in Appendix 10.6-A2-1 for a concrete repair and Appendix 10.6-A2-2 for a steel T-section repair. These sheets can be customized for the project and added to the Bridge Plans.

When an approach slab is added to an existing bridge, the final grade of the approach slab concrete shall match the existing grade of the concrete bridge deck or concrete slab, including bridges with asphalt pavement. The existing depth of asphalt on the bridge must be shown in the Plans and an equal depth of asphalt placed on a new approach slab. If the existing depth of asphalt is increased or decreased, the final grade must also be shown on the Plans.



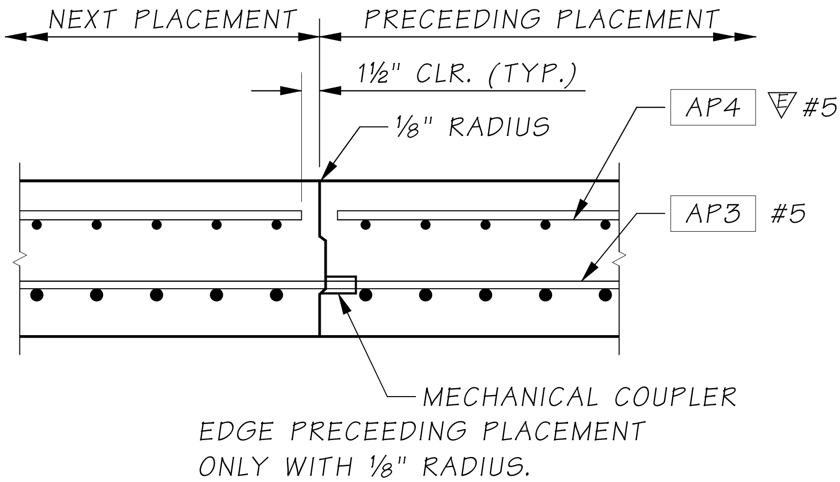
Pinned Approach Slab Detail
Figure 10.6.6-1



PCCP Roadway Dowel Bar Detail
Figure 10.6.6-2

10.6.7 Approach Slab Staging

Staging plans will most likely be required when adding or retrofitting approach slabs on existing bridges. The staging plans will be a part of the bridge plans and should be on their own sheet. Coordination with the Region is required to ensure agreement between the bridge staging sheet and the Region traffic control sheet. The longitudinal construction joints required for staging shall be located on lane lines. As there may not be enough room to allow for a lap splice in the bottom transverse bars, a mechanical splice option should be added. If a lap splice is not feasible, then only the mechanical splice option should be given. See Figure 10.6.6-3.



Alternate Longitudinal Joint Detail
Figure 10.6.6-3

GENERAL NOTES

1. ALL MATERIAL AND WORKMANSHIP SHALL BE IN ACCORDANCE WITH THE REQUIREMENTS OF THE WASHINGTON STATE DEPARTMENT OF TRANSPORTATION STANDARD SPECIFICATIONS FOR ROAD, BRIDGE, AND MUNICIPAL CONSTRUCTION DATED 20__ AND AMENDMENTS.

2. THE SIGN STRUCTURES DESIGN AND ANALYSIS HAS BEEN DONE IN ACCORDANCE WITH AASHTO STANDARD SPECIFICATIONS FOR STRUCTURAL SUPPORTS FOR HIGHWAY SIGNS, LUMINAIRES AND TRAFFIC SIGNALS - FOURTH EDITION - DATED 20__ AND INTERIMS, USING BASIC WIND SPEED OF 90 MPH AND 50 YEARS OF DESIGN LIFE. FATIGUE DESIGN OF THE STRUCTURE CONFORMS TO FATIGUE CATEGORY 1 OF THE SPECIFIED AASHTO STANDARD SPECIFICATIONS.

3. ALL BUTT JOINT WELDS SHALL BE FULL PENETRATION GROOVE WELDS WITH BACK-UP PLATES OF ¼" MIN. THICKNESS.

4. THE BACK-UP PLATES FOR ALL FULL PENETRATION WELDS SHALL BE WELDED CONTINUOUSLY TO THE JOINED PIECES. THIS CAN BE DONE BY EITHER A CONTINUOUS FILLET WELD ON THE BACK SIDE OF THE PIECE, OR BY A CONTINUOUS WELD IN THE ROOT OF THE FULL PENETRATION WELD, UNLESS OTHERWISE NOTED.

5. ALL BOLTS, RODS, AND RELATED HARDWARE SHALL BE GALVANIZED AFTER FABRICATION PER AASHTO M 232.

α 6. ALL STEEL SURFACES SHALL BE GALVANIZED AFTER FABRICATION IN ACCORDANCE WITH AASHTO M 111. ALL EXTERIOR STEEL SURFACES SHALL BE PAINTED IN ACCORDANCE WITH THE SPECIAL PROVISIONS. THE MAINTENANCE PLATFORM AND ASSOCIATED HAND RAILINGS SHALL NOT BE PAINTED. FOR MAINTENANCE PLATFORM ATTACHMENT BRACKET DETAILS FOR MONOTUBES SEE STANDARD PLAN G-95.20. PAINT ENTIRE ATTACHMENT BRACKET TO MATCH EXISTING STRUCTURE EXCEPT FOR MOUNTING BEAM. MAINTENANCE WALKWAY DETAILS SHALL BE DETERMINED FROM THE CONTRACT PLANS OR THE STANDARD PLANS.

π 6. ALL STEEL SURFACES SHALL BE GALVANIZED AFTER FABRICATION IN ACCORDANCE WITH AASHTO M 111. ALL EXTERIOR STEEL SURFACES SHALL BE PAINTED IN ACCORDANCE WITH THE SPECIAL PROVISIONS.

7. SIGN PANELS AS SHOWN IN THE CONTRACT PLANS SHALL BE INSTALLED WITH THE SIGN STRUCTURE OR IMMEDIATELY AFTER THE SIGN STRUCTURE IS ERECTED.

λ 8. FABRICATE BEAM TO PROVIDE SMOOTH PARABOLIC CAMBER CURVE. SEE CAMBER DIAGRAM. DO NOT SHIM AT BOLTED SPLICES.

β 8. FABRICATE BEAM TO PROVIDE STRAIGHT CAMBER, SEE CAMBER DIAGRAM. DO NOT SHIM AT BOLTED SPLICES.

9. FABRICATE POST STRAIGHT.

10. MATERIALS SPECIFICATIONS:

ALL STRUCTURAL STEEL EXCEPT AS OTHERWISE NOTED	ASTM A 572 GR. 50 OR ASTM A 588
ANCHOR RODS	ASTM F 1554 GR. 105
HANDHOLE COVER SCREWS	ASTM F 593 GR. 1
SPLICE BOLTS	AASHTO M 164
SIGN BRACKET RODS	ASTM A 307
MOUNTING BEAM BOLTS	AASHTO M 164
COVER PLATES	ASTM A 36

11. BOTTOM OF BASE PLATE ELEVATIONS AND POST HEIGHTS SHOWN ARE APPROXIMATE. THE CONTRACTOR SHALL FIELD MEASURE ANCHOR ROD LOCATIONS, ELEVATIONS, CLEARANCES AND ALL STEEL STRUCTURE DIMENSIONS, AND SUBMIT TO ENGINEER FOR APPROVAL PRIOR TO FABRICATION.

12. POSTS, BASE PLATES, BEAMS AND SPLICE PLATES ARE MAIN LOAD CARRYING TENSILE MEMBERS OR TENSION COMPONENTS OF FLEXURAL MEMBERS AND SHALL MEET THE LONGITUDINAL CHARPY V-NOTCH TEST AS DESCRIBED INSECTION 6-03.2 FOR AASHTO M 270 MATERIAL. NON-DESTRUCTIVE TEST ACCEPTANCE CRITERIA TO CONFORM TO TENSILE MEMBERS WITH CYCLIC LOAD.

13. SEE OTHER PLANS FOR CONDUIT PENETRATIONS AND HAND HOLES. REFER TO ELECTRICAL PLANS FOR INTERNAL ROUTING OF CONDUCTORS. CONDUIT CONDUCTORS SHALL NOT BE ATTACHED TO THE OUTSIDE OF THE SIGN STRUCTURE. ISOLATION SWITCH SHALL BE LOCATED NEAR THE SHOULDER OF ROADWAY ON THE OPPOSITE SIDE OF THE BEAM AS THE SIGNS. SEE NEMA 3R TERMINAL CABINET DETAIL ON BRIDGE SHEET ___. (10.1-A1-2, 10.1-A2-2 OR 10.1-A3-2

14. THE MAXIMUM SIGN AREA ON THE STRUCTURE SHALL BE AS NOTED.

α 15. FOR SIGN AND LIGHT ATTACHMENT BRACKET DETAILS FOR MONOTUBES SEE STANDARD PLAN G-90.20. PAINT ENTIRE ATTACHMENT BRACKET TO MATCH EXISTING STRUCTURE EXCEPT FOR MOUNTING BEAM. SIGN, BEAM LENGTHS, AND SIZE SHALL BE DETERMINED FROM THE STANDARD PLANS. SPACING SHALL BE DETERMINED FROM THE CONTRACT PLANS. VARIABLE MESSAGE SIGNS SHALL HAVE MOUNTING BEAMS @ 3'-0" MAXIMUM.

π 15. FOR SIGN AND LIGHT ATTACHMENT BRACKET DETAILS FOR MONOTUBES SEE STANDARD PLAN G-90.20. PAINT ENTIRE ATTACHMENT BRACKET TO MATCH EXISTING STRUCTURE EXCEPT FOR MOUNTING BEAM. SIGN, BEAM LENGTHS, AND SIZE SHALL BE DETERMINED FROM THE STANDARD PLANS. SPACING SHALL BE DETERMINED FROM THE CONTRACT PLANS.

β 16. THE TOTAL BEAM LENGTH "S" SHALL NOT EXCEED 30'-0".

17. ALL WELDING SHALL BE DONE TO MINIMIZE DISTORTION. PERMISSIBLE MONOTUBE DIMENSION VARIATIONS FOR OUTSIDE DIMENSIONS, WALL THICKNESS, LENGTH, STRAIGHTNESS, (PARABOLICALLY CAMBERED SIGN BRIDGE BEAMS EXCLUDED) SQUARENESS OF SIDES AND TWIST SHALL BE IN ACCORDANCE WITH SECTION 11 OF ASTM A500.

note to designer

π cantilever only
α balance "t" & sign bridges only
β balance "t" & cantilevers only
λ sign bridge only

(modify these notes to fit specific project structure type.)

note to designer

suggested sheet order:

layouts
- cantilever
- balanced "t"
- sign bridge

general notes & other misc. info.

structural details
- cantilever
- balanced "t"
- sign bridge

foundation
- cantilever
  - type 1
  - type 2 or 3
- balanced "t"
  - type 1
  - type 2 or 3
- sign bridge
  - type 1
  - type 2 or 3

barrier shape modification

LEGEND

B / 15 — IDENTIFIES SECTION, VIEW OR DETAIL — TAKEN OR SHOWN ON BRIDGE SHEET 15

B / — — TAKEN OR SHOWN ON THE SAME SHEET

Bridge Design Engr.	M:\STANDARDS\Sign Bridges\GENERAL NOTES.MAN	REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS	
Supervisor		10	WASH.				
Designed By		JOB NUMBER					
Checked By							
Detailed By							
Bridge Projects Engr.							
Prelim. Plan By							
Architect/Specialist	DATE	REVISION	BY	APP'D			

BRIDGE AND STRUCTURES OFFICE

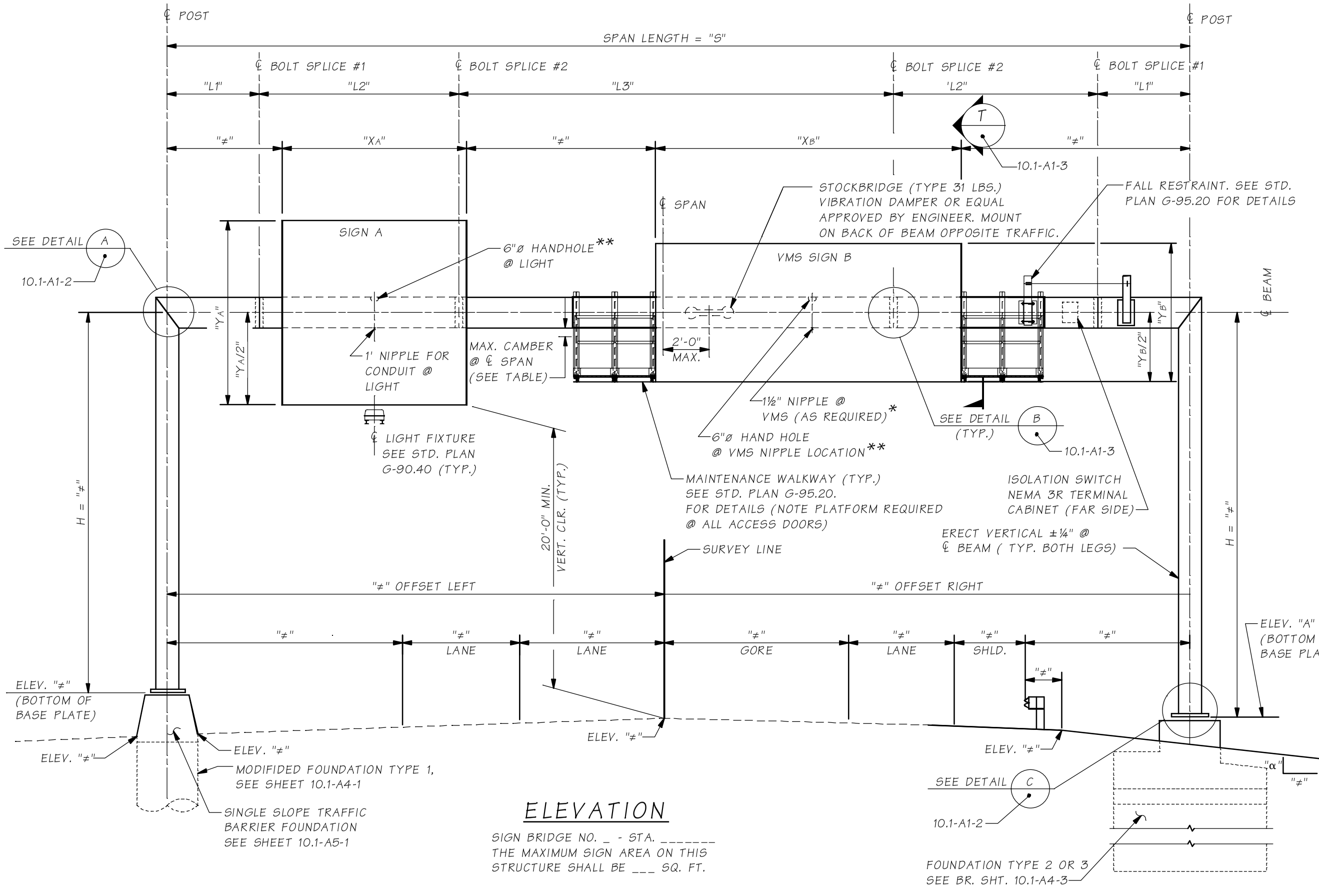
Washington State Department of Transportation

STANDARD MONOTUBE SIGN STRUCTURES

GENERAL NOTES

BRIDGE SHEET NO.
SHEET OF SHEETS

Wed Jul 18 10:12:34 2012



ELEVATION

SIGN BRIDGE NO. _ - STA. _______
THE MAXIMUM SIGN AREA ON THIS STRUCTURE SHALL BE ___ SQ. FT.

notes to the designer

delete all lighting and electrical details, callouts & notes if no electrical items required on the sign structure, except conduit in foundation.

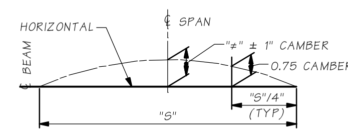
existing roadway lines dashed and new roadway is solid.

"≠" is shown where dimensions are required.

remove notes that are not applicable.

* CONTRACTOR TO VERIFY NIPPLE LOCATION TO MATCH VMS SIGN CONDUIT LOCATIONS, PRIOR TO SIGN STRUCTURE FABRICATION. NO FIELD WELDING OR DRILLING SHALL BE PERMITTED.

** HAND HOLE IS ONLY REQUIRED IF NIPPLE LOCATION IS GREATER THAN 1'-6" FROM ANOTHER HAND HOLE LOCATION.



BEAM CAMBER DIAGRAM

Bridge Design Engr.	M:\STANDARDS\Sign Bridges\MT_SIGN_BRIDGE_LAYOUT.MAN			
Supervisor		REGION NO.	STATE	FED. AID PROJ. NO.
Designed By		10	WASH.	
Checked By		JOB NUMBER		
Detailed By				
Bridge Projects Engr.				
Prelim. Plan By				
Architect/Specialist	DATE	REVISION	BY	APP'D

SHEET NO. TOTAL SHEETS

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

STANDARD MONOTUBE SIGN STRUCTURES

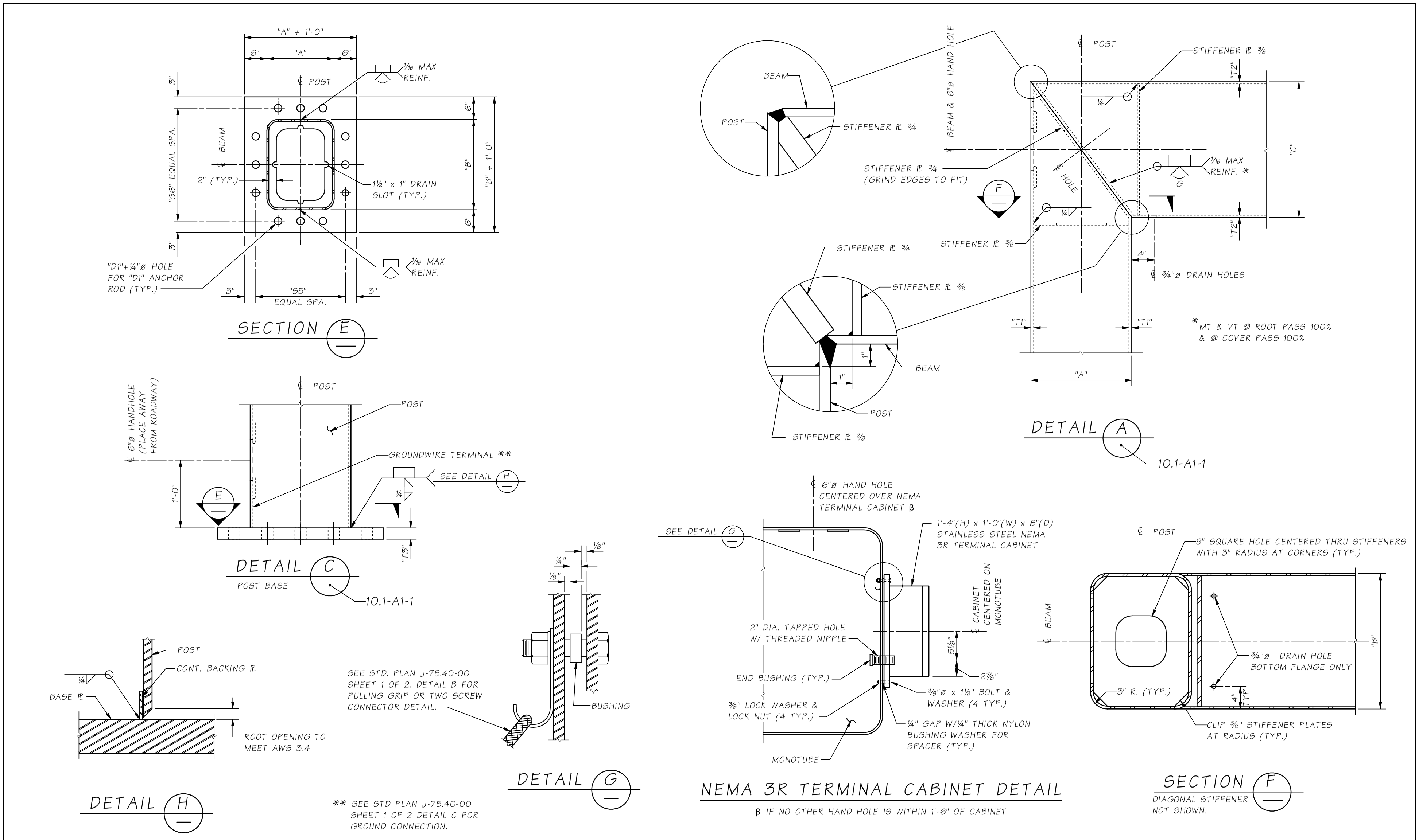
MONOTUBE SIGN BRIDGE LAYOUT

BRIDGE SHEET NO.
SHEET OF SHEETS

SR JOB NO. SHEET

10.1-A1-1

Wed Jul 18 10:12:45 2012



Bridge Design Engr.	M:\STANDARDS\Sign Bridges\MT_SIGN_BRIDGE_STRUCT_DETAILS_1.MAN			
Supervisor				
Designed By				
Checked By				
Detailed By				
Bridge Projects Engr.				
Prelim. Plan By				
Architect/Specialist	DATE	REVISION	BY	APP'D

REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
10	WASH.			
JOB NUMBER				

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

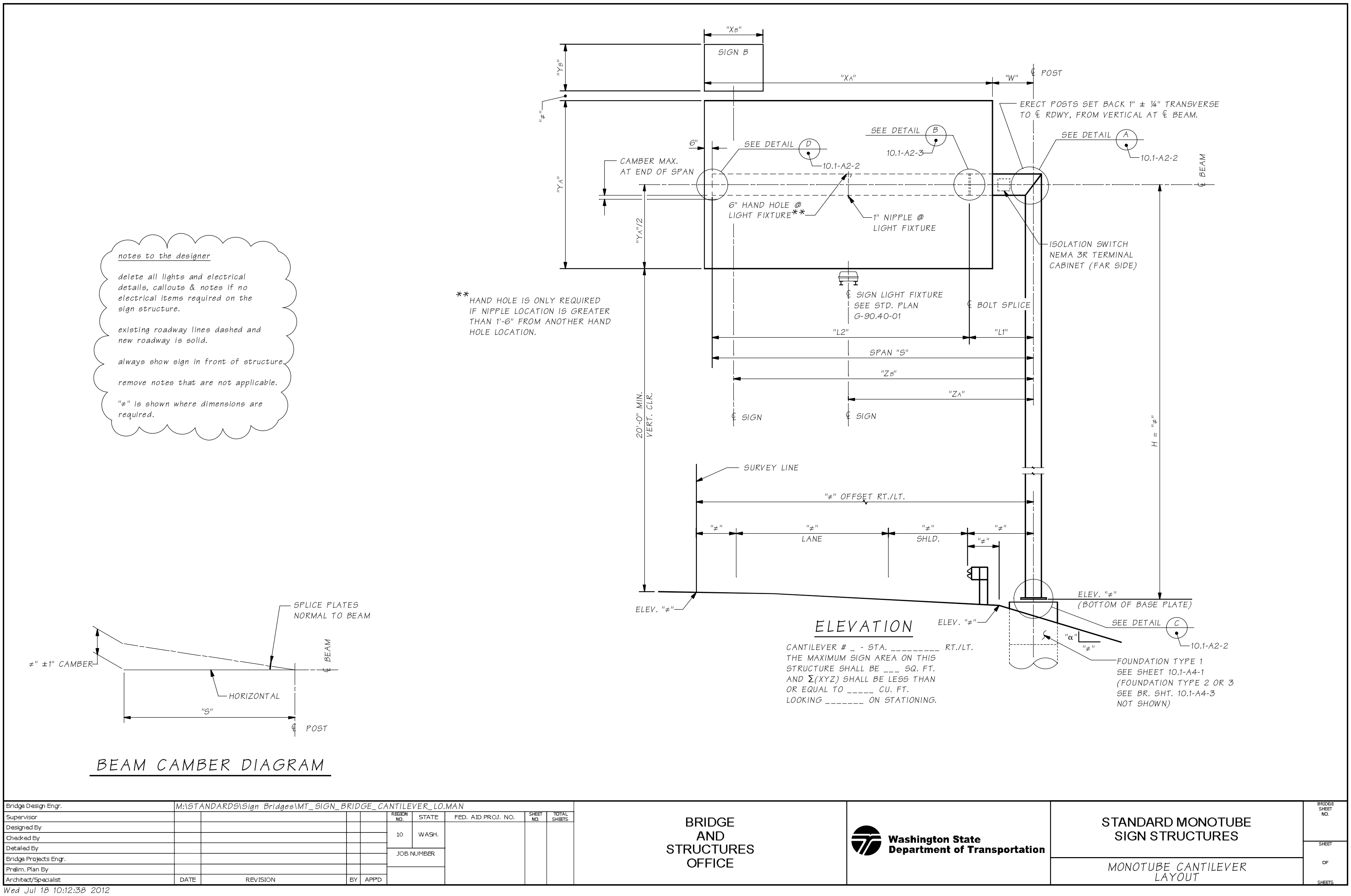
STANDARD SIGN BRIDGES

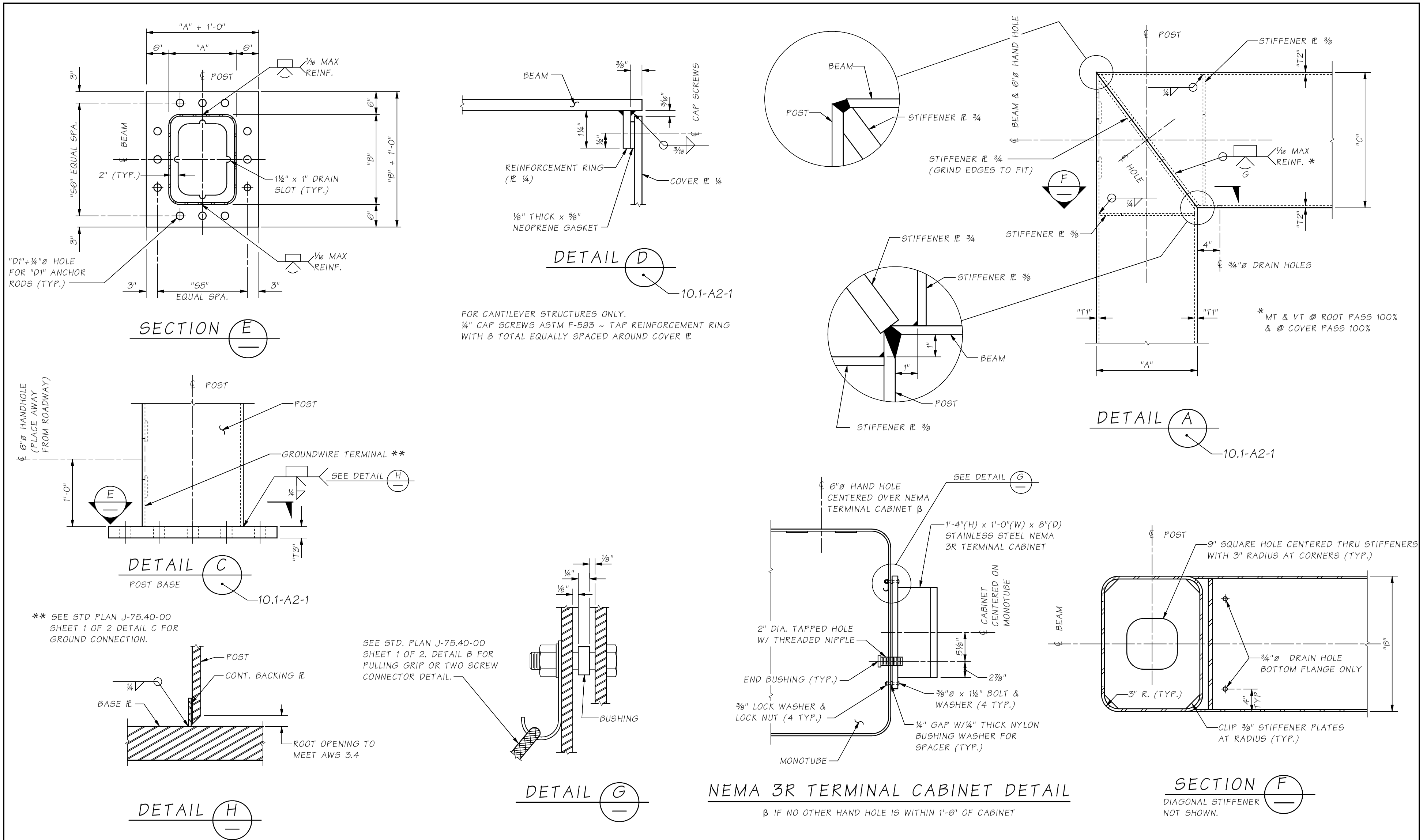
MONOTUBE SIGN BRIDGE STRUCTURAL DETAILS 1

BRIDGE SHEET NO.

SHEET OF SHEETS

Fri Sep 03 09:42:26 2010


"XB"
SIGN B
"YB"
"≠"
"XA"
"W"
℄ POST
ERECT POSTS SET BACK 1" ± ¼" TRANSVERSE TO ℄ RDWY, FROM VERTICAL AT ℄ BEAM.
SEE DETAIL A 10.1-A2-2
SEE DETAIL B 10.1-A2-3
SEE DETAIL D 10.1-A2-2
6"
CAMBER MAX. AT END OF SPAN
"YA"
"YA"/2
℄ BEAM
6" HAND HOLE @ LIGHT FIXTURE**
1" NIPPLE @ LIGHT FIXTURE
ISOLATION SWITCH NEMA 3R TERMINAL CABINET (FAR SIDE)
**HAND HOLE IS ONLY REQUIRED IF NIPPLE LOCATION IS GREATER THAN 1'-6" FROM ANOTHER HAND HOLE LOCATION.
℄ SIGN LIGHT FIXTURE SEE STD. PLAN G-90.40-01
℄ BOLT SPLICE
"L2"
"L1"
SPAN "S"
"ZB"
"ZA"
℄ SIGN
℄ SIGN
H = "≠"
20'-0" MIN. VERT. CLR.
SURVEY LINE
"≠" OFFSET RT./LT.
"≠"
LANE
SHLD.
ELEV. "≠"
ELEV. "≠" (BOTTOM OF BASE PLATE)
SEE DETAIL C 10.1-A2-2
"α"
FOUNDATION TYPE 1 SEE SHEET 10.1-A4-1 (FOUNDATION TYPE 2 OR 3 SEE BR. SHT. 10.1-A4-3 NOT SHOWN)
ELEVATION
CANTILEVER # _ - STA. _________ RT./LT.
THE MAXIMUM SIGN AREA ON THIS STRUCTURE SHALL BE ___ SQ. FT.
AND Σ(XYZ) SHALL BE LESS THAN OR EQUAL TO _____ CU. FT.
LOOKING _______ ON STATIONING.
notes to the designer
delete all lights and electrical details, callouts & notes if no electrical items required on the sign structure.
existing roadway lines dashed and new roadway is solid.
always show sign in front of structure.
remove notes that are not applicable.
"≠" is shown where dimensions are required.
SPLICE PLATES NORMAL TO BEAM
"≠" ±1" CAMBER
℄ BEAM
HORIZONTAL
"S"
℄ POST
BEAM CAMBER DIAGRAM
Bridge Design Engr.
Supervisor
Designed By
Checked By
Detailed By
Bridge Projects Engr.
Prelim. Plan By
Architect/Specialist
DATE
REVISION
BY
APP'D
M:\STANDARDS\Sign Bridges\MT_SIGN_BRIDGE_CANTILEVER_LO.MAN
REGION NO.
STATE
FED. AID PROJ. NO.
SHEET NO.
TOTAL SHEETS
10
WASH.
JOB NUMBER
BRIDGE AND STRUCTURES OFFICE
Washington State Department of Transportation
STANDARD MONOTUBE SIGN STRUCTURES
MONOTUBE CANTILEVER LAYOUT
BRIDGE SHEET NO.
SHEET
OF
SHEETS
SR
JOB NO.
SHEET
Wed Jul 18 10:12:38 2012



Bridge Design Engr.	M:\STANDARDS\Sign Bridges\MT_CANT_STRUCT_DETAILS_1.MAN							
Supervisor				REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
Designed By				10	WASH.			
Checked By								
Detailed By				JOB NUMBER				
Bridge Projects Engr.								
Prelim. Plan By								
Architect/Specialist	DATE	REVISION	BY	APP'D				

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

STANDARD SIGN BRIDGES

MONOTUBE CANTILEVER STRUCTURAL DETAILS 1

BRIDGE SHEET NO.
SHEET OF SHEETS

Fri Sep 03 09:41:56 2010

notes to the designer

camber and tolerance values shown are for "S1" = 20' and "S2" = 10'

use interpolation to determine camber and tolerance values, based on actual span length.

walk in vms with single access door is shown. for vms with two doors, maint. walkways are required at each door. fall restraint is only required at main walkway used for primary access.

verify that attachment brackets do not interfer with handholes, drain holes, nema 3 boxes, nipples etc.

delete all lights and electrical details, callouts & notes if no electrical items required on the sign structure.

existing roadway lines dashed and new roadway is solid.

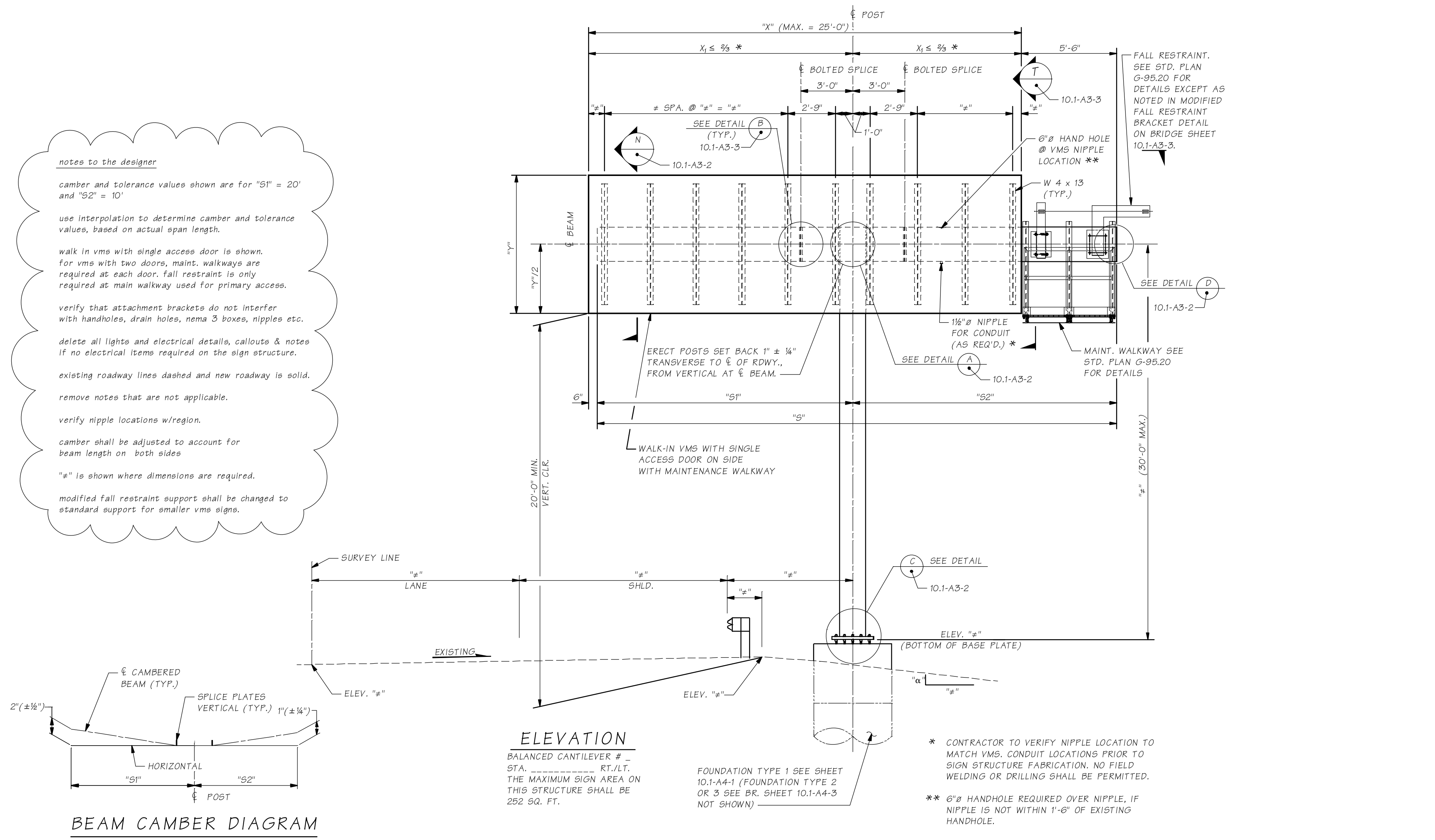
remove notes that are not applicable.

verify nipple locations w/region.

camber shall be adjusted to account for beam length on both sides

"≠" is shown where dimensions are required.

modified fall restraint support shall be changed to standard support for smaller vms signs.



BEAM CAMBER DIAGRAM

ELEVATION

BALANCED CANTILEVER # _
STA. __________ RT./LT.
THE MAXIMUM SIGN AREA ON THIS STRUCTURE SHALL BE 252 SQ. FT.

FOUNDATION TYPE 1 SEE SHEET 10.1-A4-1 (FOUNDATION TYPE 2 OR 3 SEE BR. SHEET 10.1-A4-3 NOT SHOWN)

* CONTRACTOR TO VERIFY NIPPLE LOCATION TO MATCH VMS. CONDUIT LOCATIONS PRIOR TO SIGN STRUCTURE FABRICATION. NO FIELD WELDING OR DRILLING SHALL BE PERMITTED.

** 6"ø HANDHOLE REQUIRED OVER NIPPLE, IF NIPPLE IS NOT WITHIN 1'-6" OF EXISTING HANDHOLE.

Bridge Design Engr.	M:\STANDARDS\Sign Bridges\Mt_CANTILEVER_T_LAYOUT.MAN			
Supervisor				
Designed By				
Checked By				
Detailed By				
Bridge Projects Engr.				
Prelim. Plan By				
Architect/Specialist	DATE	REVISION	BY	APP'D

REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
10	WASH.			
JOB NUMBER				

Wed Jul 18 10:12:41 2012

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

STANDARD MONOTUBE SIGN STRUCTURES

MONOTUBE BALANCED CANTILEVER LAYOUT

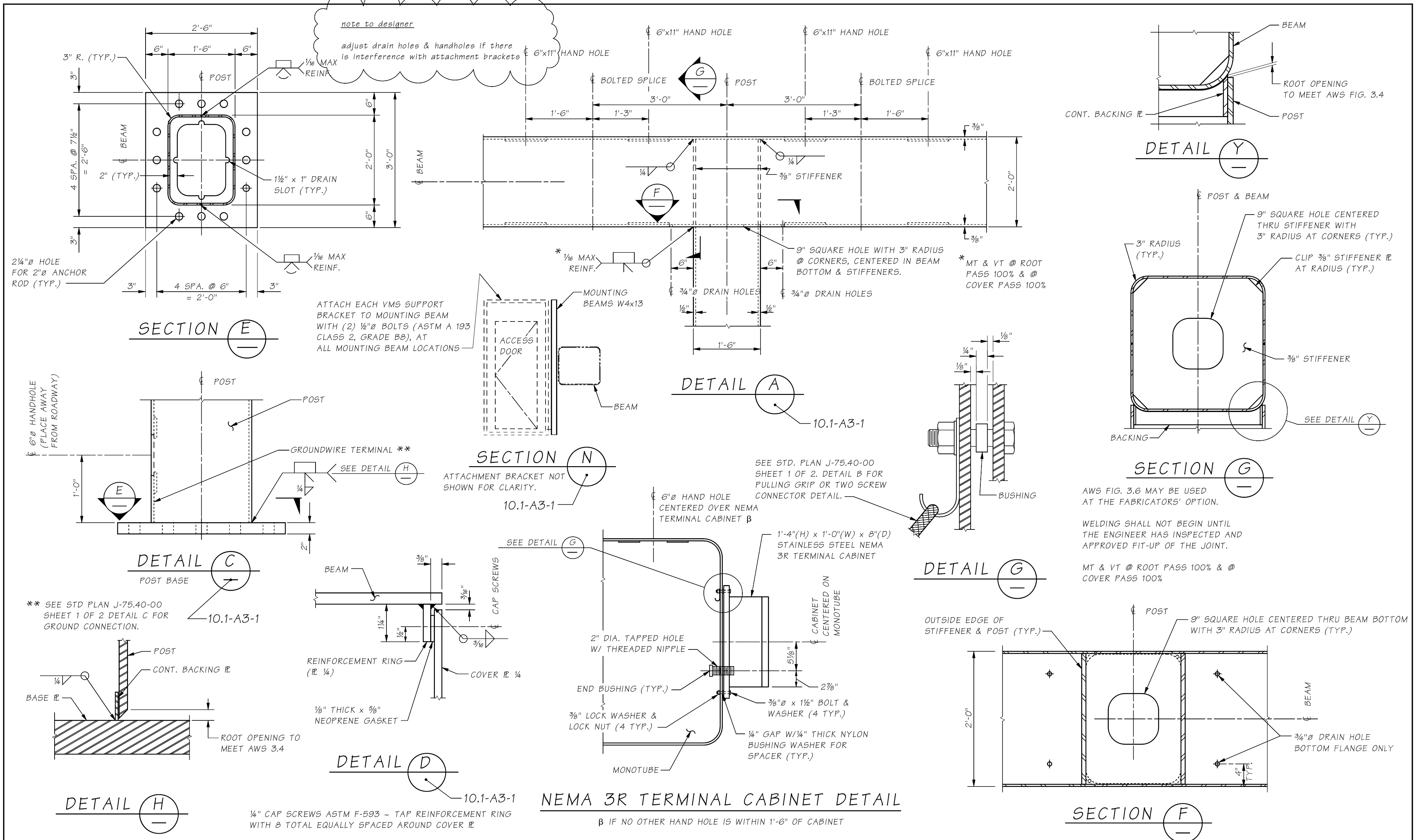
BRIDGE SHEET NO.

SHEET OF SHEETS

SR

FILE NO. 10.1-A3-1

SHEET



Bridge Design Engr.	M:\STANDARDS\Sign Bridges\MT_CANTILEVER_T_DETAILS_1.MAN				
Supervisor		REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO. / TOTAL SHEETS
Designed By		10	WASH.		
Checked By		JOB NUMBER			
Detailed By					
Bridge Projects Engr.					
Prelim. Plan By					
Architect/Specialist	DATE / REVISION / BY / APP'D				

BRIDGE AND STRUCTURES OFFICE

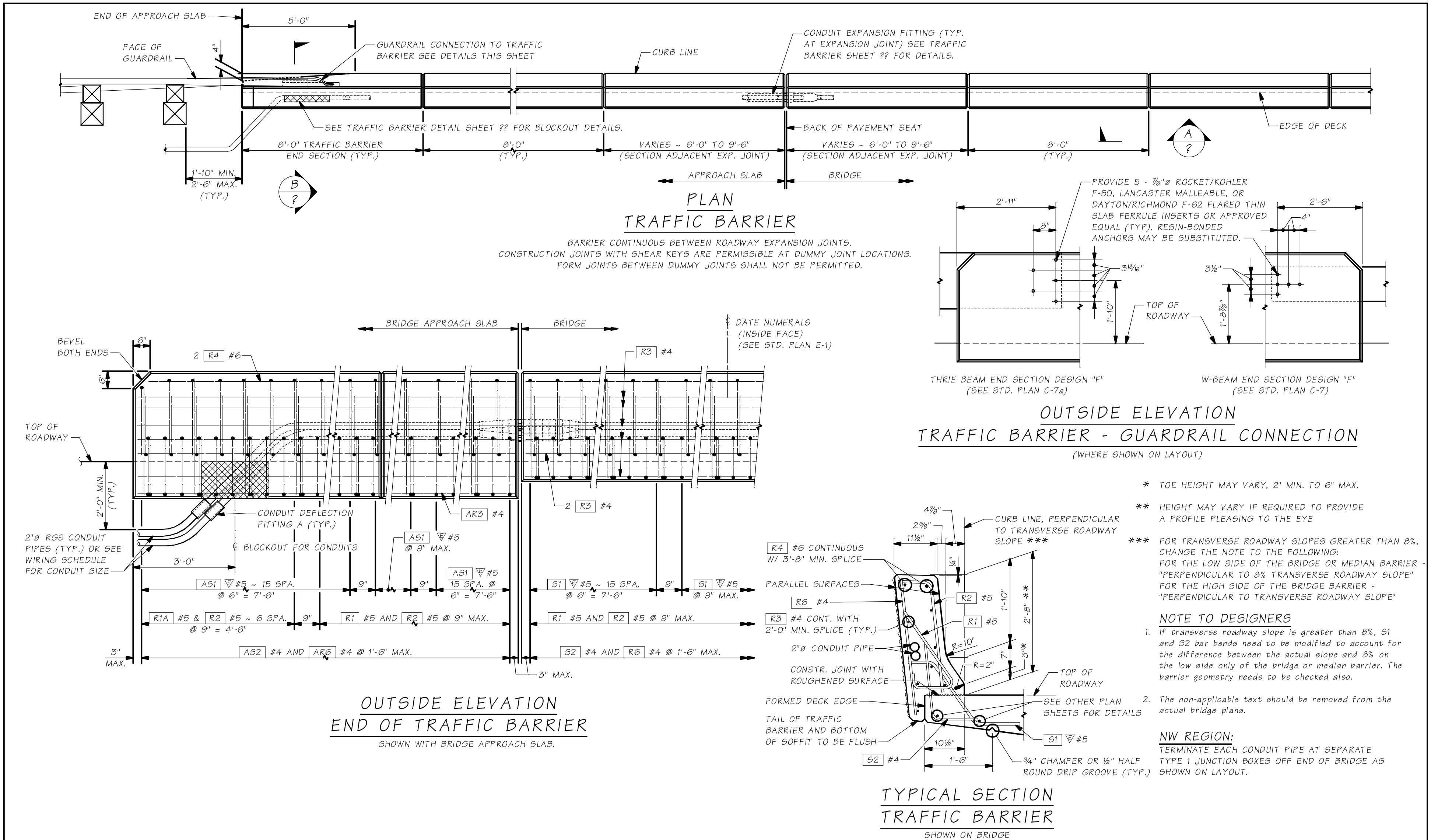
Washington State Department of Transportation

STANDARD SIGN BRIDGES

MONOTUBE BALANCED CANTILEVER STRUCTURAL DETAILS 1

BRIDGE SHEET NO. / SHEET OF SHEETS

Fri Sep 03 09:42:01 2010



* TOE HEIGHT MAY VARY, 2" MIN. TO 6" MAX.

** HEIGHT MAY VARY IF REQUIRED TO PROVIDE A PROFILE PLEASING TO THE EYE

*** FOR TRANSVERSE ROADWAY SLOPES GREATER THAN 8%, CHANGE THE NOTE TO THE FOLLOWING:
FOR THE LOW SIDE OF THE BRIDGE OR MEDIAN BARRIER - "PERPENDICULAR TO 8% TRANSVERSE ROADWAY SLOPE"
FOR THE HIGH SIDE OF THE BRIDGE BARRIER - "PERPENDICULAR TO TRANSVERSE ROADWAY SLOPE"

NOTE TO DESIGNERS

1. If transverse roadway slope is greater than 8%, S1 and S2 bar bends need to be modified to account for the difference between the actual slope and 8% on the low side only of the bridge or median barrier. The barrier geometry needs to be checked also.
2. The non-applicable text should be removed from the actual bridge plans.

NW REGION:

TERMINATE EACH CONDUIT PIPE AT SEPARATE TYPE 1 JUNCTION BOXES OFF END OF BRIDGE AS SHOWN ON LAYOUT.

Bridge Design Engr.	M:\STANDARDS\Traffic Barriers\Shape F\SHAPE F BARRIER SHT 1.MAN		
Supervisor	REGION NO. 10	STATE WASH.	FED. AID PROJ. NO.
Designed By	JOB NUMBER	SHEET NO.	TOTAL SHEETS
Checked By			
Detailed By			
Bridge Projects Engr.			
Prelim. Plan By			
Architect/Specialist	DATE	REVISION	BY APP'D

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

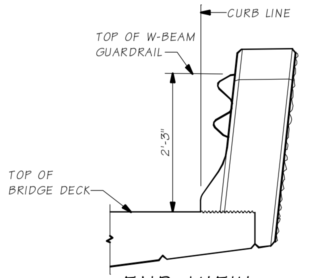
STANDARD TRAFFIC BARRIERS

TRAFFIC BARRIER - SHAPE F DETAILS 1 OF 3

BRIDGE SHEET NO. — SHEET OF SHEETS

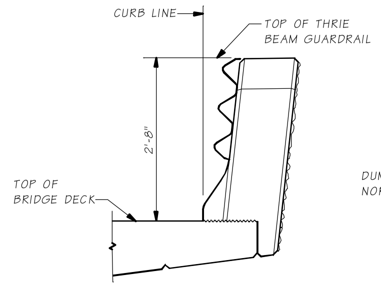
SR — JOB NO. — SHEET 1

Thu Jun 07 14:19:03 2012



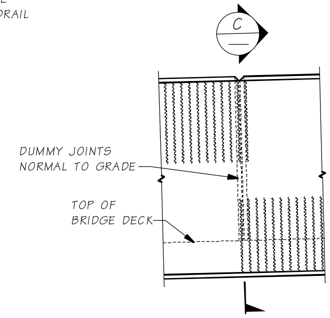
END VIEW

W-BEAM SHOWN WITH "D" CONNECTION OR "F" CONNECTION (SEE STD. PLAN C-5).

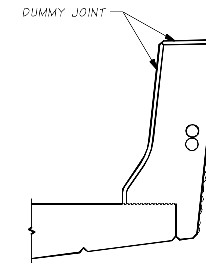


END VIEW

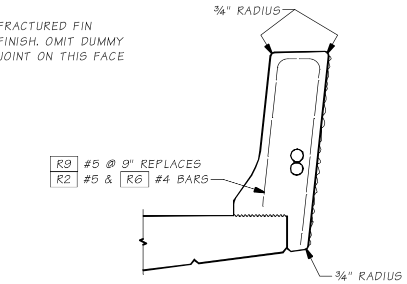
THRIE BEAM SHOWN WITH "D" CONNECTION OR "F" CONNECTION (SEE STD. PLAN C-5).



VIEW A/?



SECTION C/—



SLIPFORM ALTERNATE

SEE "TYPICAL SECTION - TRAFFIC BARRIER" FOR ADDITIONAL DETAILS

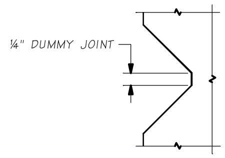
THE CONTRACTOR IS ADVISED THAT THE SLIPFORM CONSTRUCTION METHOD IS A PATENTED PROPRIETARY PROCESS FOR BARRIERS WITH A FRACTURED FIN FINISH.

JUNCTION BOX LOCATIONS		
STATION	OFFSET	"TS" OR "LT"

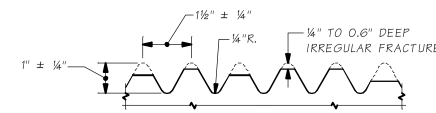
TS = TRAFFIC SYSTEM
LT = LIGHTING SYSTEM

JUNCTION BOX LOCATIONS SHOWN ARE APPROXIMATE. CENTER JUNCTION BOX INSTALLATION BETWEEN BARRIER DUMMY JOINTS.

INSTALL ALL CONDUIT RUNS TO DRAIN TO A BRIDGE END OR PROVIDE DRAIN AT ALL LOW POINTS IN CONDUIT RUN ON BRIDGE.



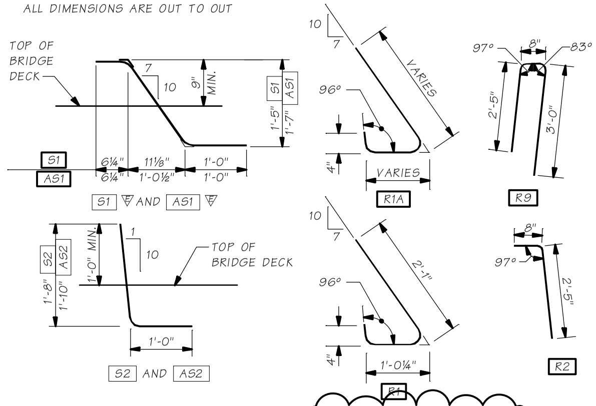
DUMMY JOINT DETAIL



FRACTURED FIN FINISH

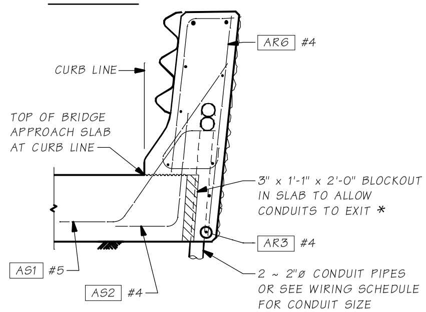
BENDING DIAGRAM

ALL DIMENSIONS ARE OUT TO OUT



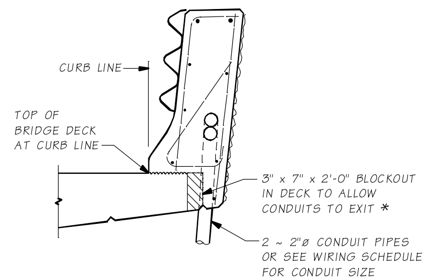
FOR W1 & W2 BARS SEE WINGWALL OR RETAINING WALL PLANS.

NOTE TO DESIGNER:
S1 AND S2 LENGTH BASED ON STANDARD DECK THICKNESS.



SECTION B/? APPROACH SLAB

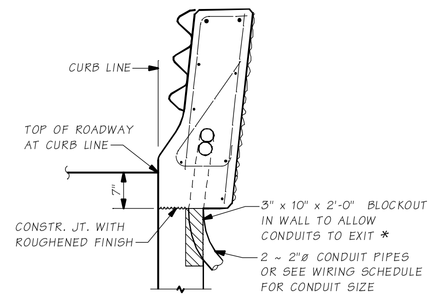
FOR DETAILS NOT SHOWN SEE "OUTSIDE ELEVATION" AND "TYPICAL SECTION - TRAFFIC BARRIER"



SECTION B/? BRIDGE

FOR DETAILS NOT SHOWN SEE "OUTSIDE ELEVATION" AND "TYPICAL SECTION - TRAFFIC BARRIER"

* BLOCKOUT WIDTH MAY BE INCREASED TO 6" TO ALLOW CONDUITS OF A LARGER DIAMETER THAN 2" TO EXIT BARRIER OR WALL WITHOUT REBAR STEEL CONFLICT



SECTION B/? WALL

DETAIL FOR RETAINING WALL OR WINGWALL. FOR REINFORCING NOT SHOWN SEE STD. PLAN D-15 OR WINGWALL PLAN.

Bridge Design Engr.	M:\STANDARDS\Traffic Barriers\Shape F\SHAPE F BARRIER SHT 2.man			
Supervisor				
Designed By				
Checked By				
Detailed By				
Bridge Projects Engr.				
Prelim. Plan By				
Architect/Specialist	DATE	REVISION	BY	APP'D

REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
10	WASH.			
JOB NUMBER				

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

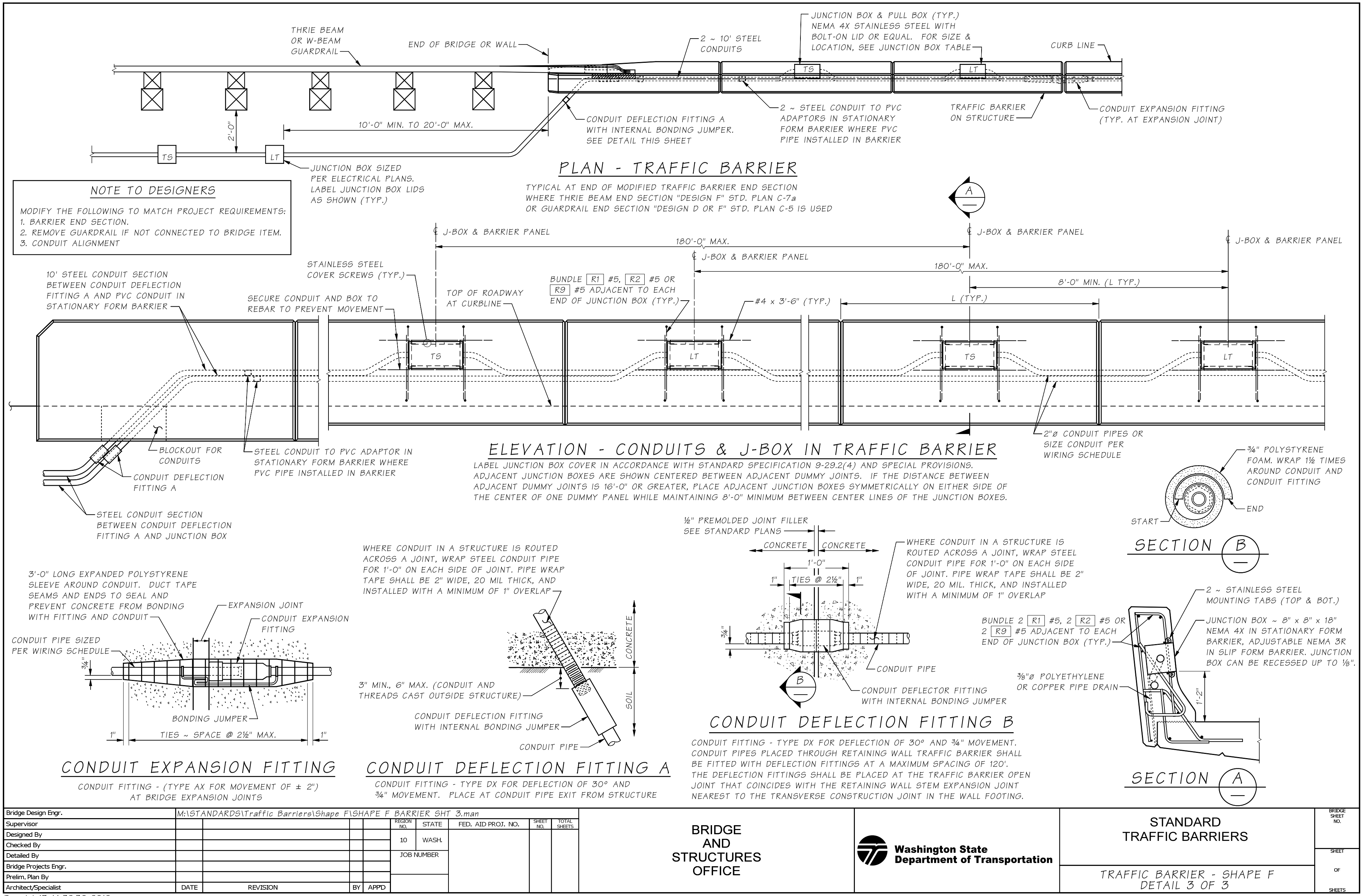
STANDARD TRAFFIC BARRIERS

TRAFFIC BARRIER - SHAPE F DETAILS 2 OF 3

BRIDGE SHEET NO.
SHEET
OF
SHEETS

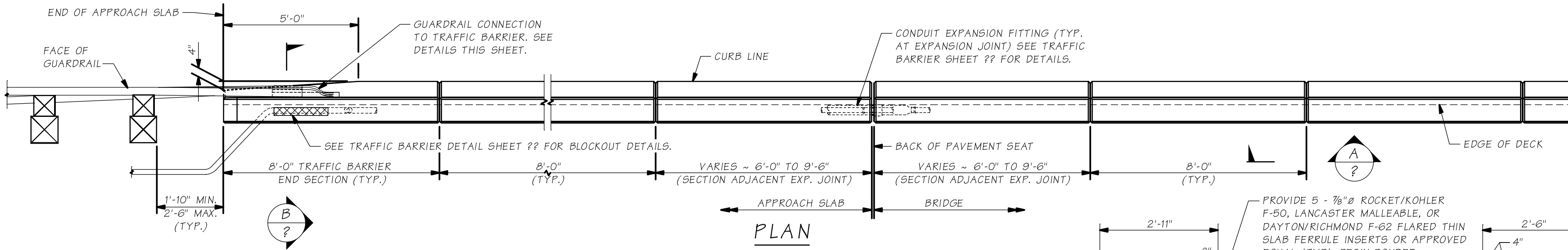
Thu Jun 07 14:19:04 2012

SR
JOB NO.
SHEET



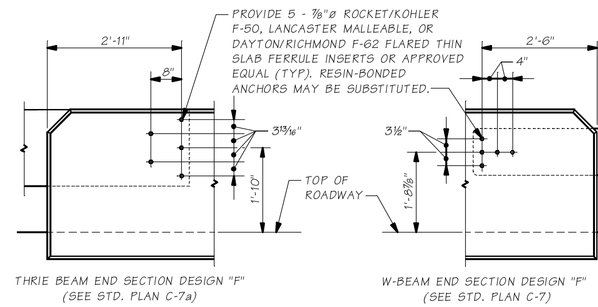
SR
JOB NO.
SHEET

Last revised on : 7/17/2012



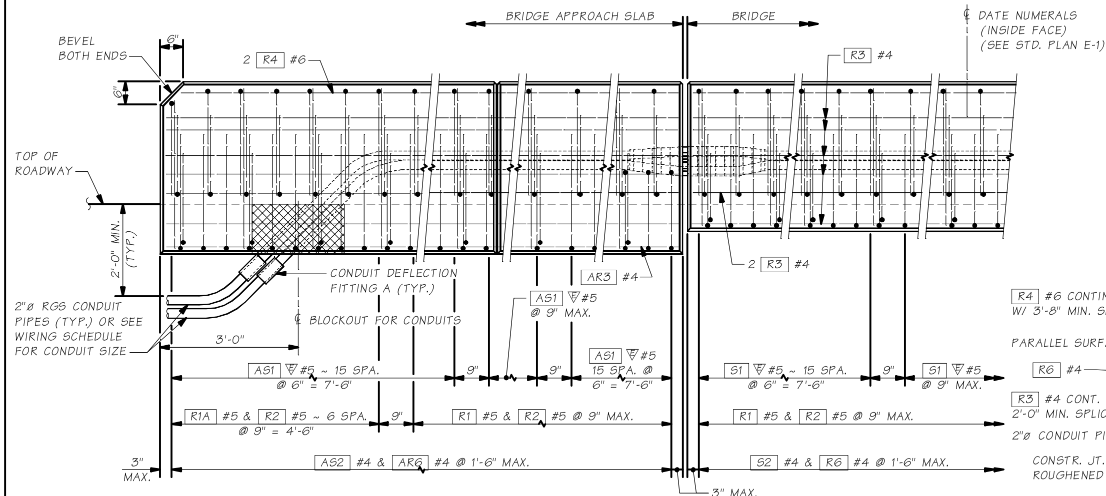
PLAN
TRAFFIC BARRIER

BARRIER CONTINUOUS BETWEEN ROADWAY EXPANSION JOINTS.
CONSTRUCTION JOINTS WITH SHEAR KEYS ARE PERMISSIBLE AT DUMMY JOINT LOCATIONS.
FORM JOINTS BETWEEN DUMMY JOINTS SHALL NOT BE PERMITTED.



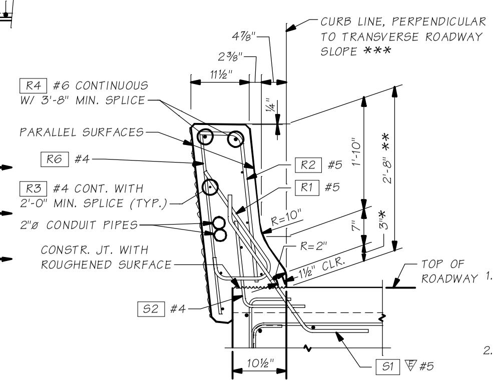
OUTSIDE ELEVATION
TRAFFIC BARRIER - GUARDRAIL CONNECTION

(WHERE SHOWN ON LAYOUT)



OUTSIDE ELEVATION
END OF TRAFFIC BARRIER

SHOWN WITH APPROACH SLAB



TYPICAL SECTION
TRAFFIC BARRIER

SHOWN ON BRIDGE

* TOE HEIGHT MAY VARY, 2" MIN. TO 6" MAX.

** HEIGHT MAY VARY IF REQUIRED TO PROVIDE A PROFILE PLEASING TO THE EYE

*** FOR TRANSVERSE ROADWAY SLOPES GREATER THAN 8%, CHANGE THE NOTE TO THE FOLLOWING:
FOR THE LOW SIDE OF THE BRIDGE OR MEDIAN BARRIER - "PERPENDICULAR TO 8% TRANSVERSE ROADWAY SLOPE"
FOR THE HIGH SIDE OF THE BRIDGE BARRIER - "PERPENDICULAR TO TRANSVERSE ROADWAY SLOPE"

NOTES TO DESIGNERS:

1. If transverse roadway slope is greater than 8%, S1 & S2 bar bends need to be modified to account for the difference between the actual slope & 8% on the low side only of the bridge or median barrier. The barrier geometry needs to be checked also.
2. The non-applicable text should be removed from the actual bridge plans.

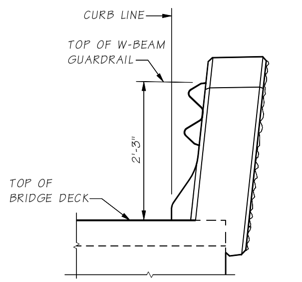
NW REGION:
TERMINATE EACH CONDUIT PIPE AT SEPARATE TYPE 1 JUNCTION BOXES OFF END OF BRIDGE AS SHOWN ON LAYOUT.

Bridge Design Engr.	M:\STANDARDS\Traffic Barriers\flat slab TB\FS SHAPE F BARRIER SHT 1.MAN	BRIDGE AND STRUCTURES OFFICE	Washington State Department of Transportation	STANDARD TRAFFIC BARRIER	BRIDGE SHEET NO.
Supervisor	REGION NO. 10 / STATE WASH. / FED. AID PROJ. NO. / SHEET NO. / TOTAL SHEETS				
Designed By					
Checked By	JOB NUMBER				SHEET
Detailed By				TRAFFIC BARRIER - SHAPE F	OF
Bridge Projects Engr.				FLAT SLAB - DETAILS 1 OF 3	SHEETS
Prelim. Plan By					
Architect/Specialist	DATE / REVISION / BY / APP'D				

Thu Jun 07 14:19:04 2012

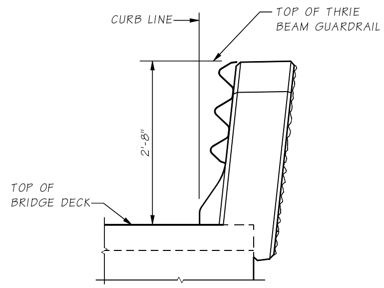
SR FILE NO. SHEET

Last revised on : 06/08/2011



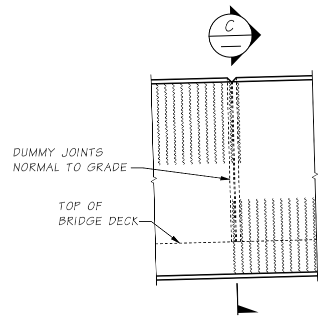
END VIEW

W-BEAM SHOWN WITH "D" CONNECTION OR "F" CONNECTION (SEE STD. PLAN C-5).

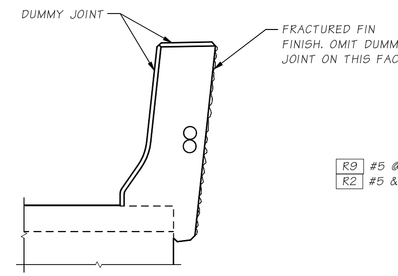


END VIEW

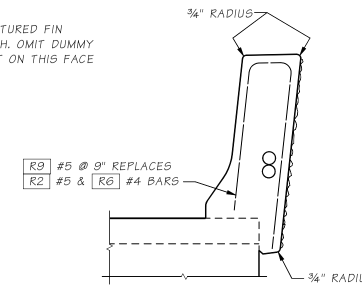
THRIE BEAM SHOWN WITH "D" CONNECTION OR "F" CONNECTION (SEE STD. PLAN C-5).



VIEW (A/?)



SECTION (C/—)



SLIPFORM ALTERNATE

SEE "TYPICAL SECTION - TRAFFIC BARRIER" FOR ADDITIONAL DETAILS

THE CONTRACTOR IS ADVISED THAT THE SLIPFORM CONSTRUCTION METHOD IS A PATENTED PROPRIETARY PROCESS FOR BARRIERS WITH A FRACTURED FIN FINISH.

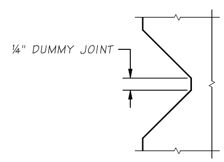
JUNCTION BOX LOCATIONS

STATION	OFFSET	"TS" OR "LT"

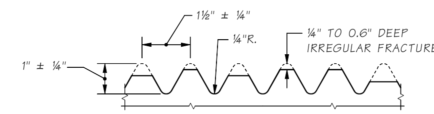
TS = TRAFFIC SYSTEM
LT = LIGHTING SYSTEM

JUNCTION BOX LOCATIONS SHOWN ARE APPROXIMATE. CENTER JUNCTION BOX INSTALLATION BETWEEN BARRIER DUMMY JOINTS.

INSTALL ALL CONDUIT RUNS TO DRAIN TO A BRIDGE END OR PROVIDE DRAIN AT ALL LOW POINTS IN CONDUIT RUN ON BRIDGE.



DUMMY JOINT DETAIL



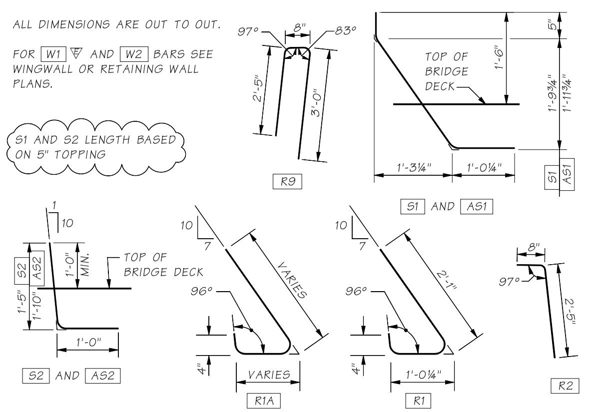
FRACTURED FIN FINISH

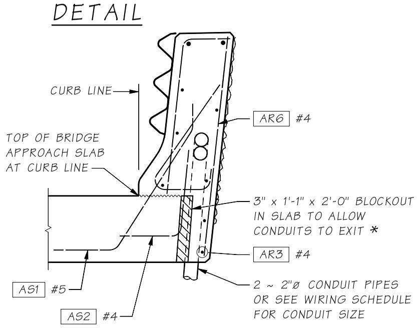
BENDING DIAGRAM

ALL DIMENSIONS ARE OUT TO OUT.

FOR W1 ▽ AND W2 BARS SEE WINGWALL OR RETAINING WALL PLANS.

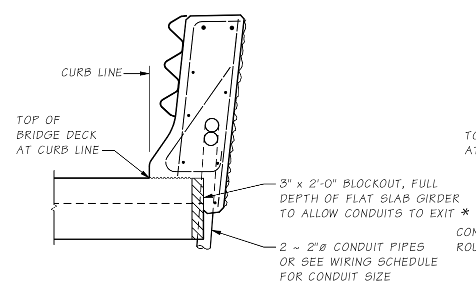
S1 AND S2 LENGTH BASED ON 5" TOPPING





SECTION (B/?) APPROACH SLAB

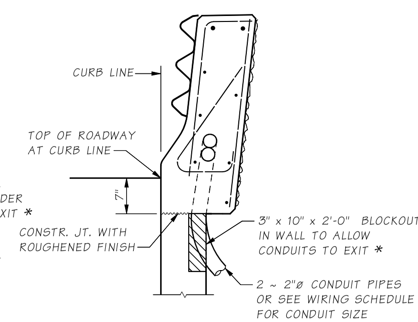
FOR DETAILS NOT SHOWN SEE "OUTSIDE ELEVATION" AND "TYPICAL SECTION - TRAFFIC BARRIER"



SECTION (B/?) BRIDGE

FOR DETAILS NOT SHOWN SEE "OUTSIDE ELEVATION" AND "TYPICAL SECTION - TRAFFIC BARRIER"

* BLOCKOUT WIDTH MAY BE INCREASED TO 6" TO ALLOW CONDUITS OF A LARGER DIAMETER THAN 2" TO EXIT BARRIER OR WALL WITHOUT REBAR STEEL CONFLICT



SECTION (B/?) WALL

DETAIL FOR RETAINING WALL OR WINGWALL. FOR REINFORCING NOT SHOWN SEE STD. PLAN D-15 OR WINGWALL PLANS.

Bridge Design Engr.	M:\STANDARDS\Traffic Barriers\flat slab TB\FS SHAPE F BARRIER SHT 2.MAN				
Supervisor					
Designed By					
Checked By					
Detailed By					
Bridge Projects Engr.					
Prelim. Plan By					
Architect/Specialist	DATE	REVISION	BY	APP'D	

REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
10	WASH.			
JOB NUMBER				

Tue Jul 17 15:13:41 2012

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

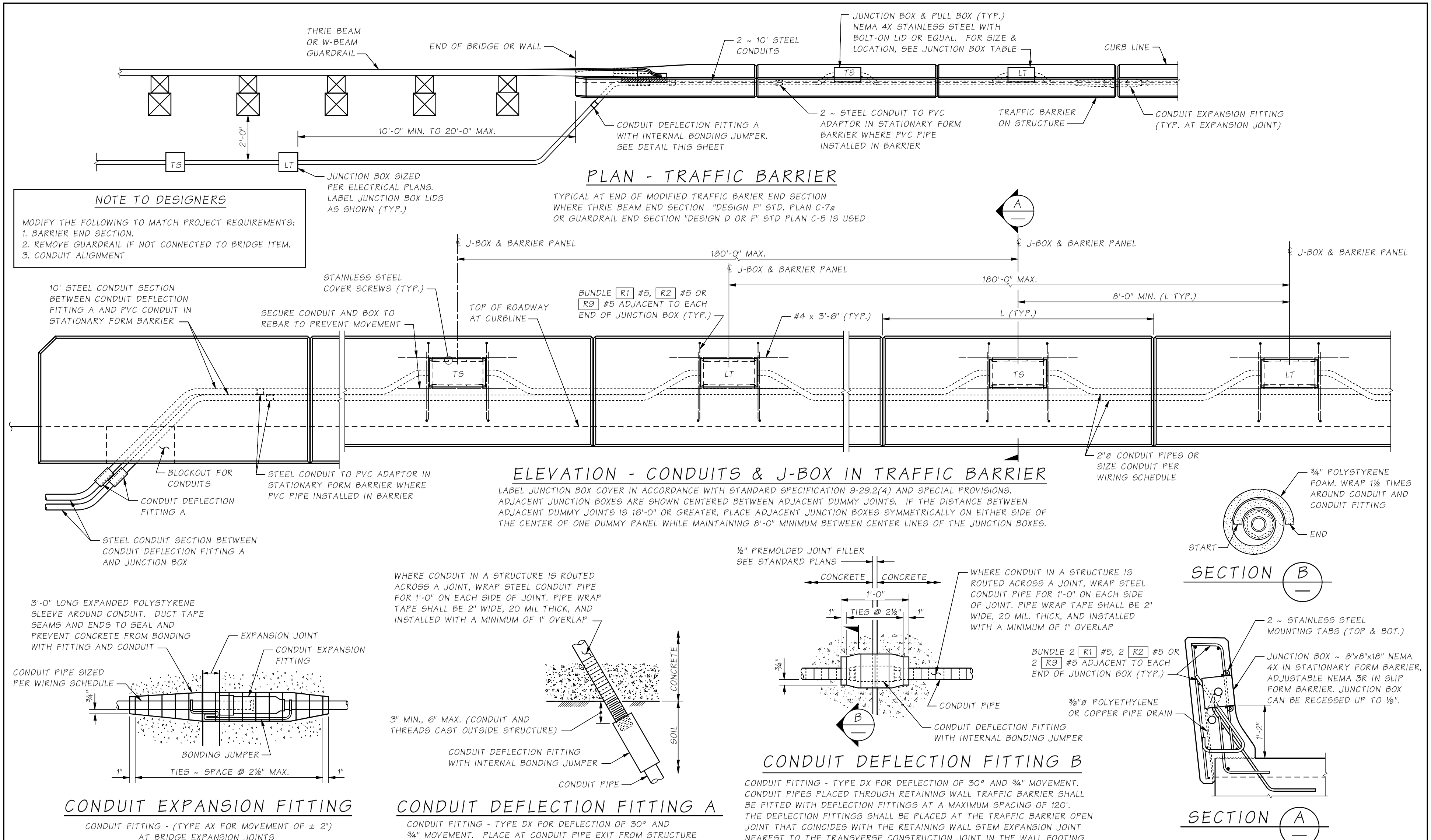
STANDARD TRAFFIC BARRIERS

TRAFFIC BARRIER - SHAPE F
FLAT SLAB - DETAILS 2 OF 3

BRIDGE SHEET NO.
SHEET OF SHEETS

SR ___ FILE NO. ___ SHEET 0

Last revised on : 6/5/2012



Bridge Design Engr.	M:\STANDARDS\Traffic Barriers\flat slab TB\FS SHAPE F BARRIER SHT 3.man								
Supervisor				REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS	
Designed By				10	WASH.				
Checked By				JOB NUMBER					
Detailed By									
Bridge Projects Engr.									
Prelim. Plan By									
Architect/Specialist	DATE	REVISION	BY	APP'D					

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

STANDARD TRAFFIC BARRIERS

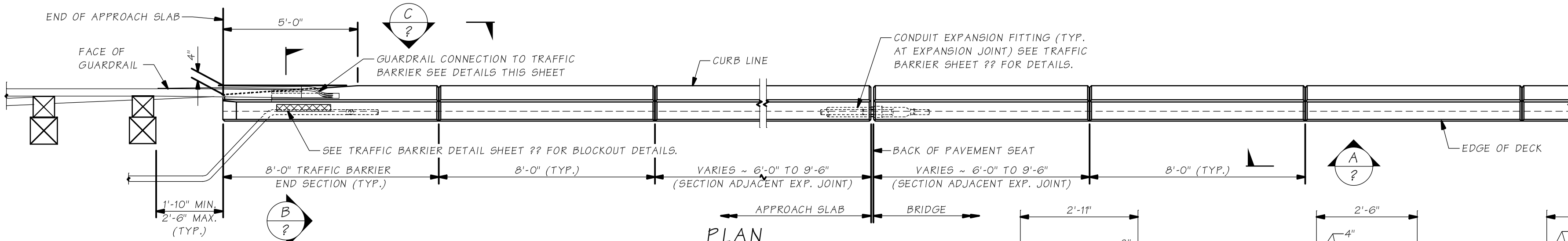
TRAFFIC BARRIER - FLAT SLAB
FLAT SLAB - DETAILS 3 OF 3

BRIDGE SHEET NO.
SHEET OF SHEETS

Tue Jul 17 14:32:30 2012

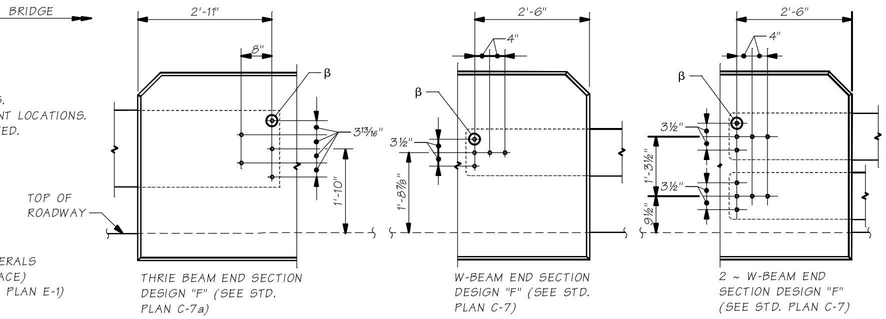
SR
FILE NO.
SHEET 0

Last revised on : 7/17/2012



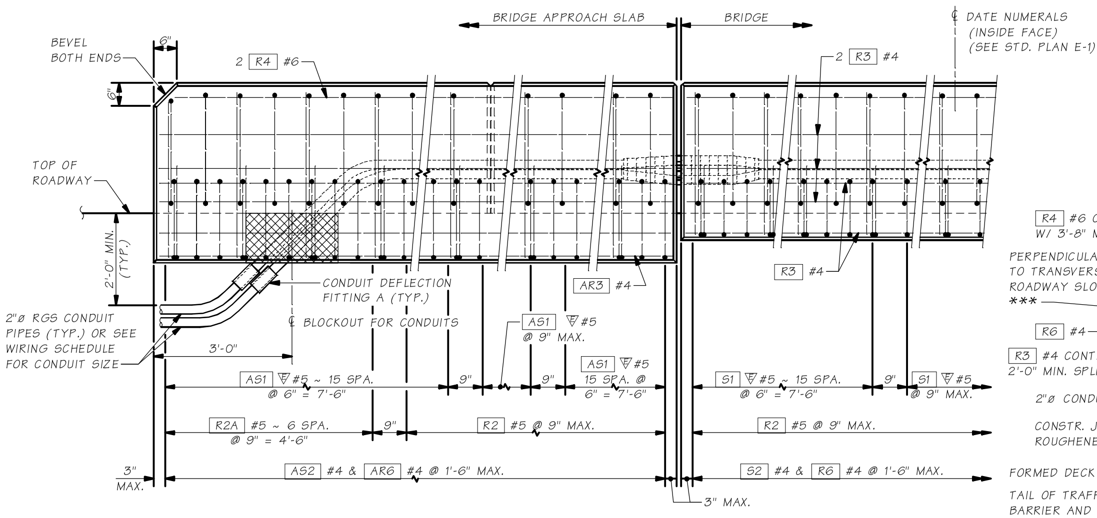
PLAN
TRAFFIC BARRIER

BARRIER CONTINUOUS BETWEEN ROADWAY EXPANSION JOINTS.
CONSTRUCTION JOINTS WITH SHEAR KEYS ARE PERMISSIBLE AT DUMMY JOINT LOCATIONS.
FORM JOINTS BETWEEN DUMMY JOINTS SHALL NOT BE PERMITTED.



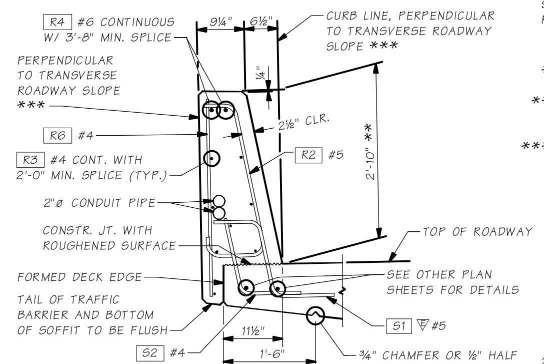
OUTSIDE ELEVATION
TRAFFIC BARRIER - GUARDRAIL CONNECTION

(WHERE SHOWN ON LAYOUT)



OUTSIDE ELEVATION
END OF TRAFFIC BARRIER

SHOWN WITH BRIDGE APPROACH SLAB



TYPICAL SECTION
TRAFFIC BARRIER

SHOWN ON BRIDGE

β PROVIDE ⅞"ø ROCKET/KOHLER F-50, LANCASTER MALLEABLE, OR DAYTON/RICHMOND F-62 FLARED THIN SLAB FERRULE INSERTS OR APPROVED EQUAL (TYP.). RESIN-BONDED ANCHORS MAY BE SUBSTITUTED.

* TOE HEIGHT MAY VARY, 2" MIN. TO 6" MAX.

** HEIGHT MAY VARY IF REQUIRED TO PROVIDE A PROFILE PLEASING TO THE EYE

*** FOR TRANSVERSE ROADWAY SLOPES GREATER THAN 8%, CHANGE THE NOTE TO THE FOLLOWING:
FOR THE LOW SIDE OF THE BRIDGE OR MEDIAN BARRIER - "PERPENDICULAR TO 8% TRANSVERSE ROADWAY SLOPE"
FOR THE HIGH SIDE OF THE BRIDGE BARRIER - "PERPENDICULAR TO TRANSVERSE ROADWAY SLOPE"

NOTE TO DESIGNERS

1. If transverse roadway slope is greater than 8%, S1 and S2 bar bends need to be modified to account for the difference between the actual slope and 8% on the low side only of the bridge or median barrier. The barrier geometry needs to be checked also.
2. The non-applicable text should be removed from the actual bridge plans.

NW REGION:

TERMINATE EACH CONDUIT PIPE AT SEPARATE TYPE 1 JUNCTION BOXES OFF END OF BRIDGE AS SHOWN ON LAYOUT.

Bridge Design Engr.	M:\STANDARDS\Traffic Barriers\Single Slope\SSTB SHEET 1.MAN									
Supervisor					REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS	
Designed By					10	WASH.				
Checked By					JOB NUMBER					
Detailed By										
Bridge Projects Engr.										
Prelim. Plan By										
Architect/Specialist	DATE	REVISION	BY	APP'D						

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

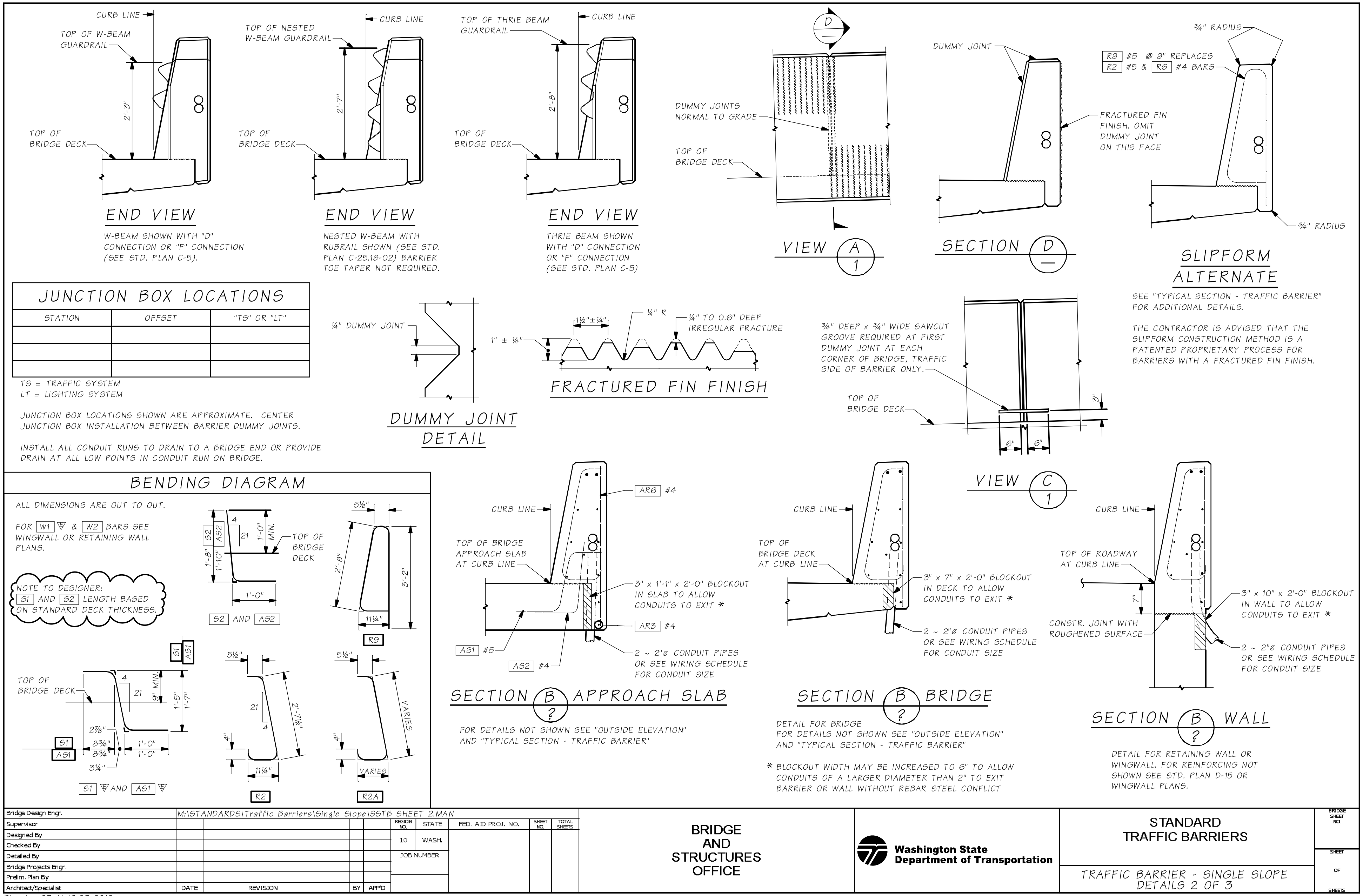
STANDARD TRAFFIC BARRIERS

TRAFFIC BARRIER - SINGLE SLOPE DETAILS 1 OF 3

BRIDGE SHEET NO.
SHEET OF SHEETS

Thu Jun 07 14:19:04 2012

JOB NO. SR SHEET 1



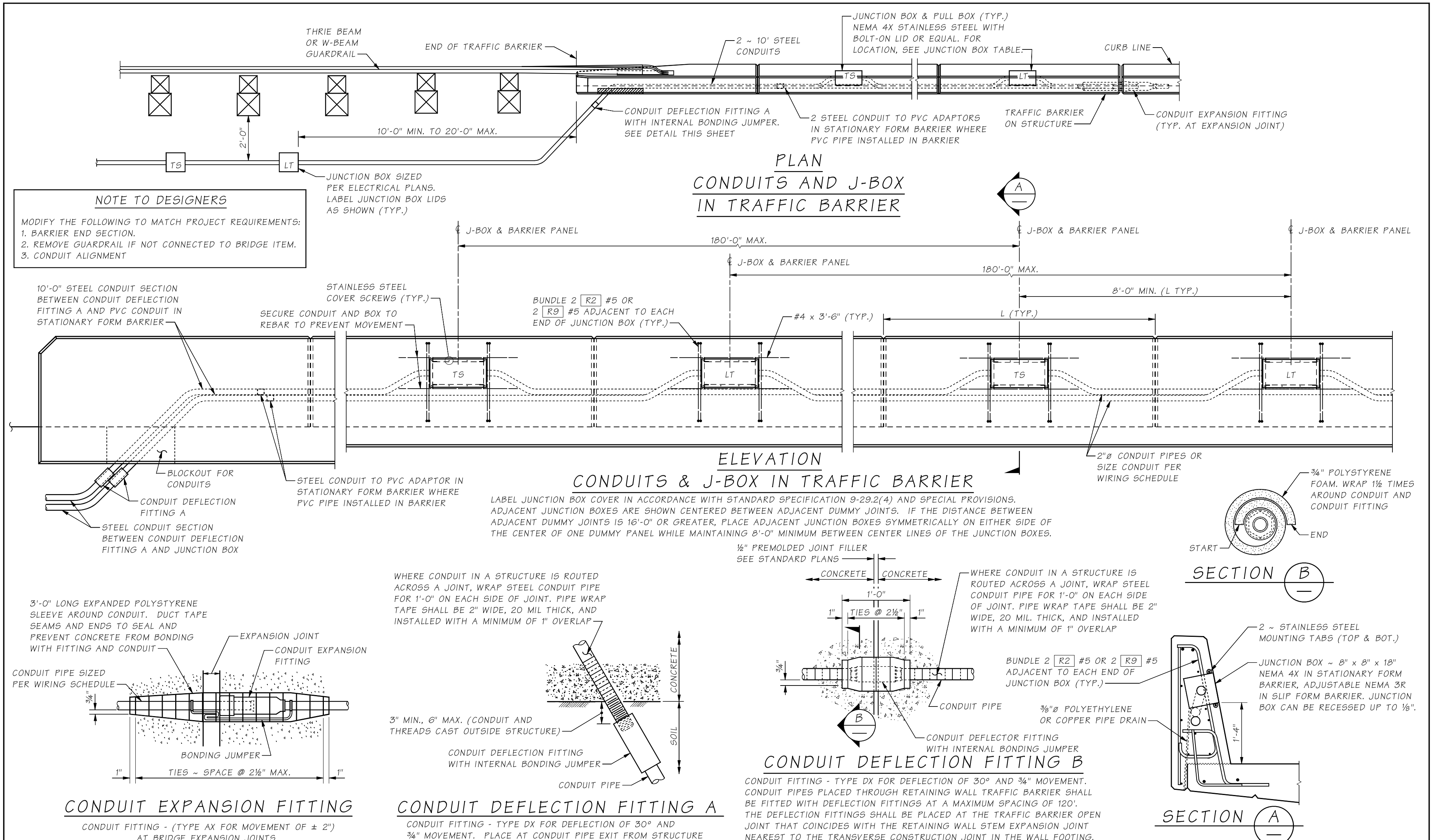
JUNCTION BOX LOCATIONS

STATION	OFFSET	"TS" OR "LT"

TS = TRAFFIC SYSTEM
LT = LIGHTING SYSTEM

JUNCTION BOX LOCATIONS SHOWN ARE APPROXIMATE. CENTER JUNCTION BOX INSTALLATION BETWEEN BARRIER DUMMY JOINTS.

INSTALL ALL CONDUIT RUNS TO DRAIN TO A BRIDGE END OR PROVIDE DRAIN AT ALL LOW POINTS IN CONDUIT RUN ON BRIDGE.



Bridge Design Engr.	M:\STANDARDS\Traffic Barriers\Single Slope\SSTB SHEET 3.MAN							
Supervisor				REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
Designed By				10	WASH.			
Checked By				JOB NUMBER				
Detailed By								
Bridge Projects Engr.								
Prelim. Plan By								
Architect/Specialist	DATE	REVISION	BY / APP'D					

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

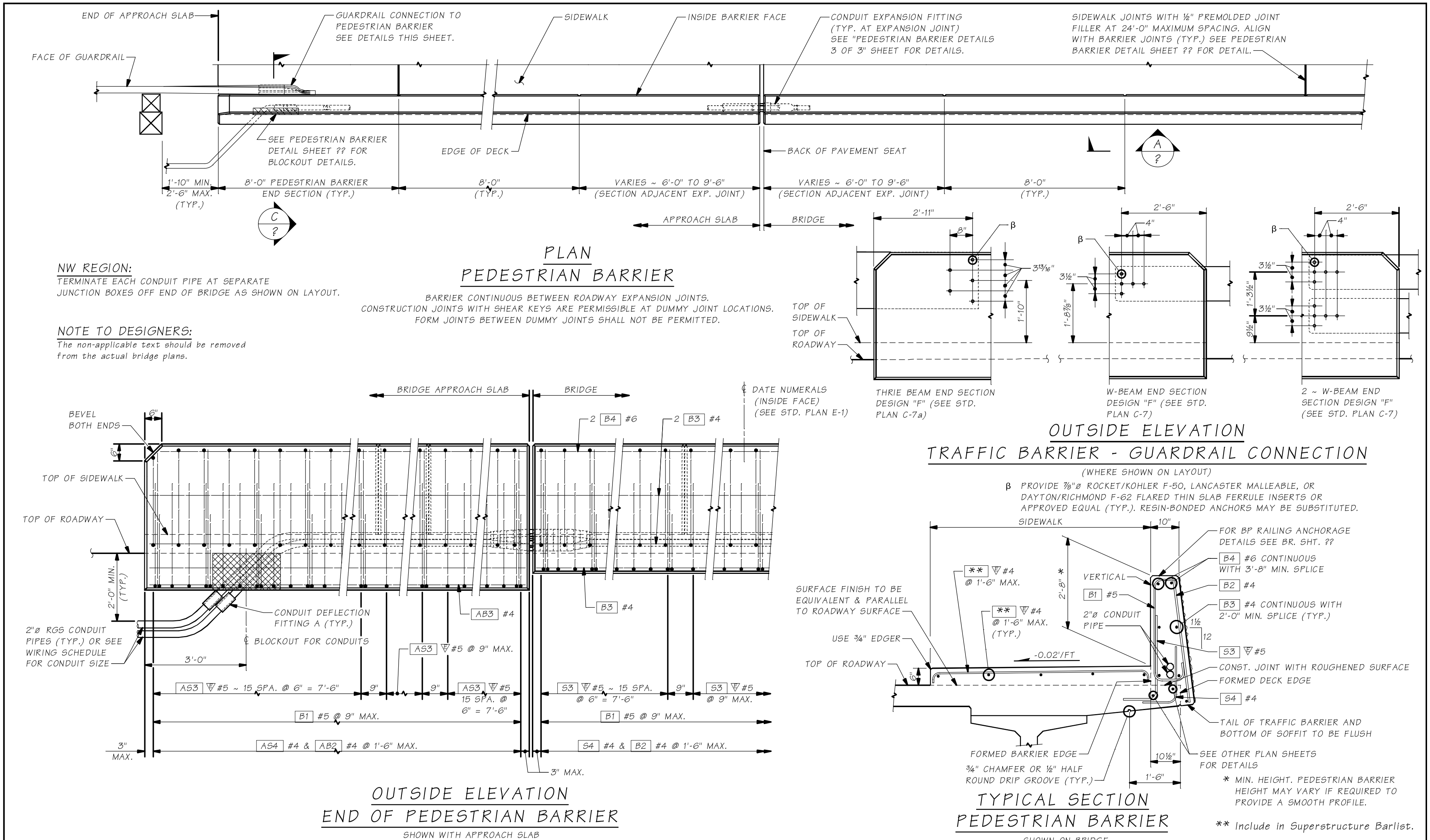
STANDARD TRAFFIC BARRIERS

TRAFFIC BARRIER - SINGLE SLOPE DETAILS 3 OF 3

BRIDGE SHEET NO. — SHEET — OF — SHEETS

Tue Jul 17 14:32:32 2012

SR — JOB NO. — SHEET —



Bridge Design Engr.	M:\STANDARDS\Traffic Barriers\Pedestrian Barrier\PED BARRIER 1.MAN			REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
Supervisor				10	WASH.			
Designed By				JOB NUMBER				
Checked By								
Detailed By								
Bridge Projects Engr.								
Prelim. Plan By								
Architect/Specialist	DATE	REVISION	BY	APP'D				

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

STANDARD TRAFFIC BARRIERS

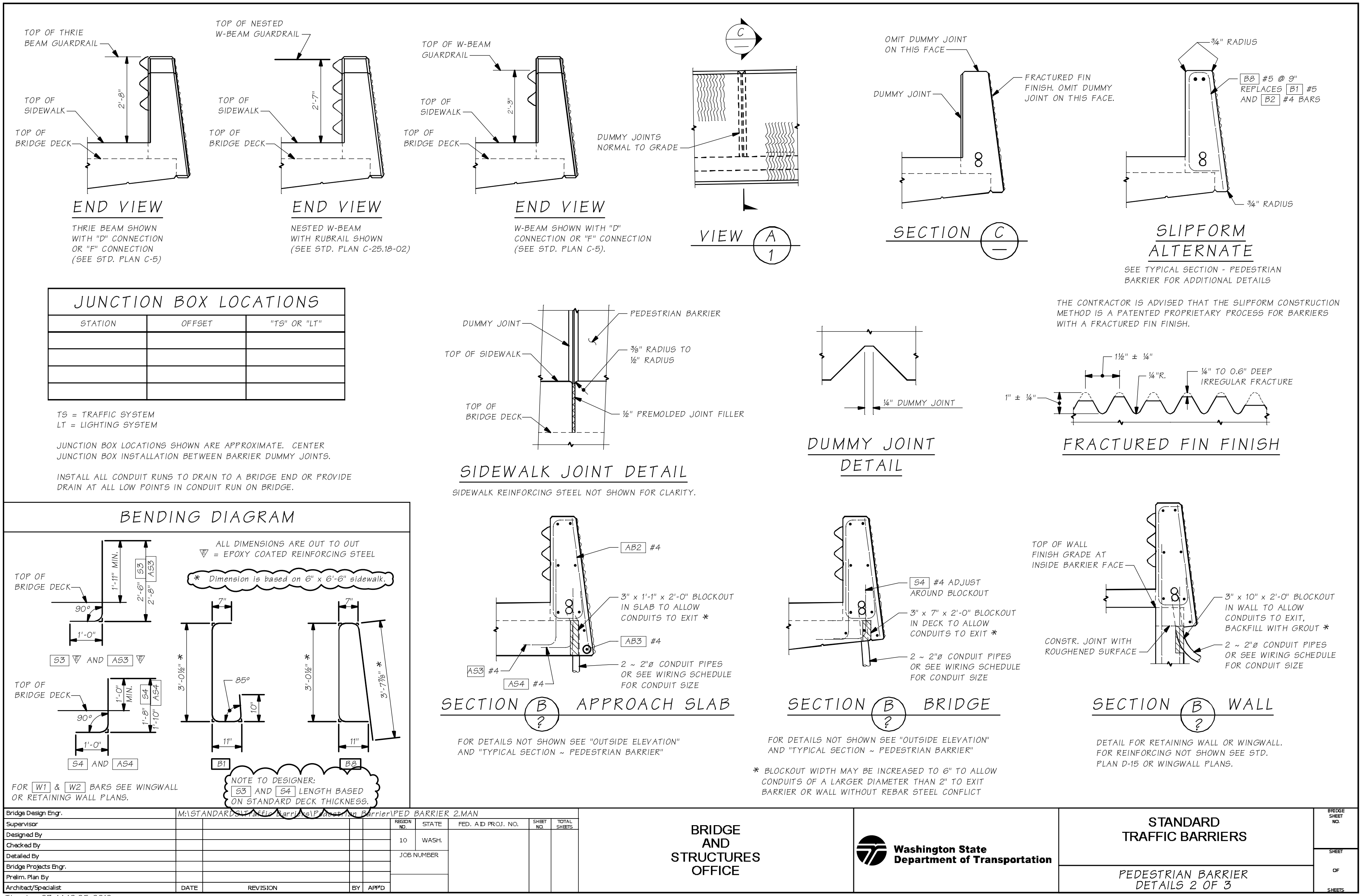
PEDESTRIAN BARRIER DETAILS 1 OF 3

BRIDGE SHEET NO.
SHEET OF SHEETS

Thu Jun 07 14:19:05 2012

SR JOB NO. SHEET

Last revised on : 06/23/2010

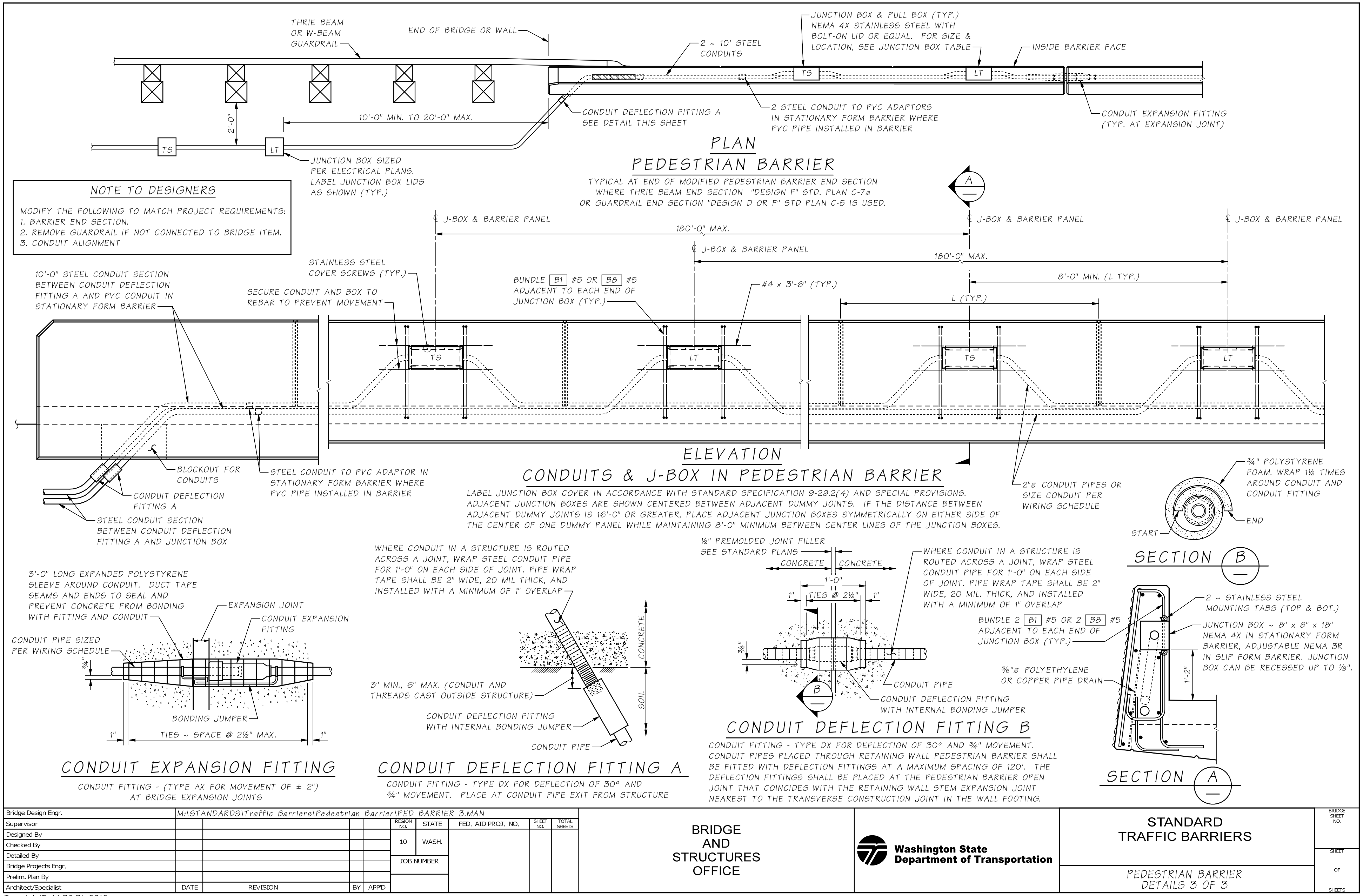


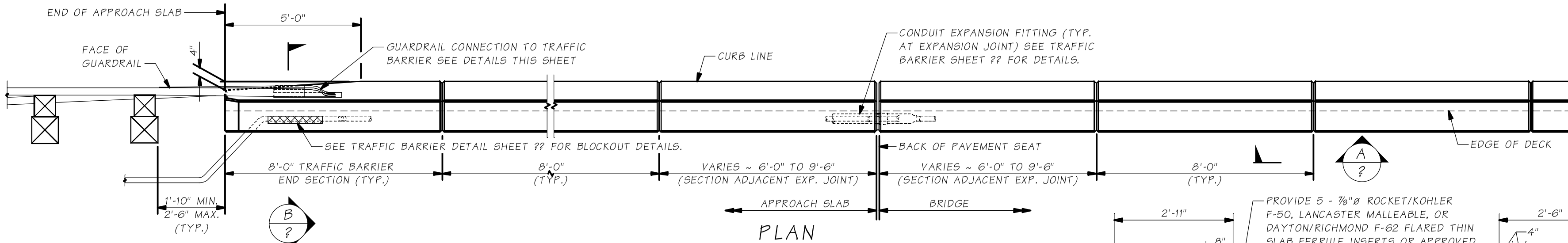
JUNCTION BOX LOCATIONS

STATION	OFFSET	"TS" OR "LT"

Last revised on : 6/5/2012

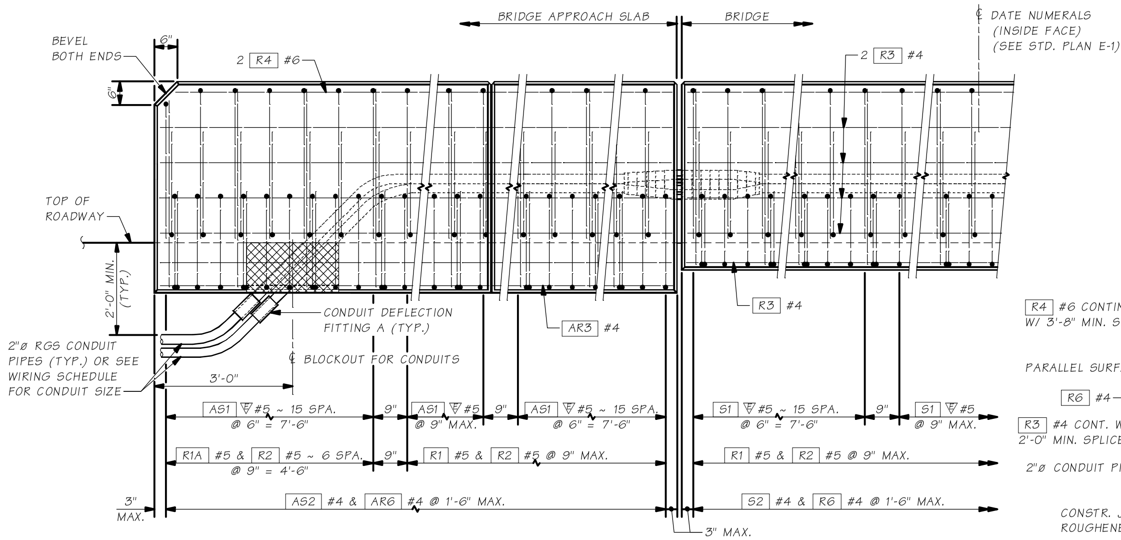
JOB NO. SR SHEET


THRIE BEAM OR W-BEAM GUARDRAIL
END OF BRIDGE OR WALL
2 ~ 10' STEEL CONDUITS
JUNCTION BOX & PULL BOX (TYP.) NEMA 4X STAINLESS STEEL WITH BOLT-ON LID OR EQUAL. FOR SIZE & LOCATION, SEE JUNCTION BOX TABLE
INSIDE BARRIER FACE
TS
LT
2'-0"
10'-0" MIN. TO 20'-0" MAX.
CONDUIT DEFLECTION FITTING A SEE DETAIL THIS SHEET
2 STEEL CONDUIT TO PVC ADAPTORS IN STATIONARY FORM BARRIER WHERE PVC PIPE INSTALLED IN BARRIER
CONDUIT EXPANSION FITTING (TYP. AT EXPANSION JOINT)
JUNCTION BOX SIZED PER ELECTRICAL PLANS. LABEL JUNCTION BOX LIDS AS SHOWN (TYP.)
PLAN
PEDESTRIAN BARRIER
TYPICAL AT END OF MODIFIED PEDESTRIAN BARRIER END SECTION WHERE THRIE BEAM END SECTION "DESIGN F" STD. PLAN C-7a OR GUARDRAIL END SECTION "DESIGN D OR F" STD PLAN C-5 IS USED.
NOTE TO DESIGNERS
MODIFY THE FOLLOWING TO MATCH PROJECT REQUIREMENTS:
1. BARRIER END SECTION.
2. REMOVE GUARDRAIL IF NOT CONNECTED TO BRIDGE ITEM.
3. CONDUIT ALIGNMENT
℄ J-BOX & BARRIER PANEL
180'-0" MAX.
8'-0" MIN. (L TYP.)
L (TYP.)
STAINLESS STEEL COVER SCREWS (TYP.)
SECURE CONDUIT AND BOX TO REBAR TO PREVENT MOVEMENT
BUNDLE B1 #5 OR B8 #5 ADJACENT TO EACH END OF JUNCTION BOX (TYP.)
#4 x 3'-6" (TYP.)
10'-0" STEEL CONDUIT SECTION BETWEEN CONDUIT DEFLECTION FITTING A AND PVC CONDUIT IN STATIONARY FORM BARRIER
BLOCKOUT FOR CONDUITS
CONDUIT DEFLECTION FITTING A
STEEL CONDUIT SECTION BETWEEN CONDUIT DEFLECTION FITTING A AND JUNCTION BOX
STEEL CONDUIT TO PVC ADAPTOR IN STATIONARY FORM BARRIER WHERE PVC PIPE INSTALLED IN BARRIER
ELEVATION
CONDUITS & J-BOX IN PEDESTRIAN BARRIER
LABEL JUNCTION BOX COVER IN ACCORDANCE WITH STANDARD SPECIFICATION 9-29.2(4) AND SPECIAL PROVISIONS. ADJACENT JUNCTION BOXES ARE SHOWN CENTERED BETWEEN ADJACENT DUMMY JOINTS. IF THE DISTANCE BETWEEN ADJACENT DUMMY JOINTS IS 16'-0" OR GREATER, PLACE ADJACENT JUNCTION BOXES SYMMETRICALLY ON EITHER SIDE OF THE CENTER OF ONE DUMMY PANEL WHILE MAINTAINING 8'-0" MINIMUM BETWEEN CENTER LINES OF THE JUNCTION BOXES.
2"ø CONDUIT PIPES OR SIZE CONDUIT PER WIRING SCHEDULE
¾" POLYSTYRENE FOAM. WRAP 1½ TIMES AROUND CONDUIT AND CONDUIT FITTING
START
END
SECTION B
3'-0" LONG EXPANDED POLYSTYRENE SLEEVE AROUND CONDUIT. DUCT TAPE SEAMS AND ENDS TO SEAL AND PREVENT CONCRETE FROM BONDING WITH FITTING AND CONDUIT
EXPANSION JOINT
CONDUIT EXPANSION FITTING
CONDUIT PIPE SIZED PER WIRING SCHEDULE
¾"
BONDING JUMPER
1"
TIES ~ SPACE @ 2½" MAX.
CONDUIT EXPANSION FITTING
CONDUIT FITTING - (TYPE AX FOR MOVEMENT OF ± 2") AT BRIDGE EXPANSION JOINTS
WHERE CONDUIT IN A STRUCTURE IS ROUTED ACROSS A JOINT, WRAP STEEL CONDUIT PIPE FOR 1'-0" ON EACH SIDE OF JOINT. PIPE WRAP TAPE SHALL BE 2" WIDE, 20 MIL THICK, AND INSTALLED WITH A MINIMUM OF 1" OVERLAP
CONCRETE
SOIL
3" MIN., 6" MAX. (CONDUIT AND THREADS CAST OUTSIDE STRUCTURE)
CONDUIT DEFLECTION FITTING WITH INTERNAL BONDING JUMPER
CONDUIT PIPE
CONDUIT DEFLECTION FITTING A
CONDUIT FITTING - TYPE DX FOR DEFLECTION OF 30° AND ¾" MOVEMENT. PLACE AT CONDUIT PIPE EXIT FROM STRUCTURE
½" PREMOLDED JOINT FILLER SEE STANDARD PLANS
CONCRETE
CONCRETE
1'-0"
1"
TIES @ 2½"
1"
¾"
WHERE CONDUIT IN A STRUCTURE IS ROUTED ACROSS A JOINT, WRAP STEEL CONDUIT PIPE FOR 1'-0" ON EACH SIDE OF JOINT. PIPE WRAP TAPE SHALL BE 2" WIDE, 20 MIL. THICK, AND INSTALLED WITH A MINIMUM OF 1" OVERLAP
CONDUIT PIPE
CONDUIT DEFLECTION FITTING WITH INTERNAL BONDING JUMPER
CONDUIT DEFLECTION FITTING B
CONDUIT FITTING - TYPE DX FOR DEFLECTION OF 30° AND ¾" MOVEMENT. CONDUIT PIPES PLACED THROUGH RETAINING WALL PEDESTRIAN BARRIER SHALL BE FITTED WITH DEFLECTION FITTINGS AT A MAXIMUM SPACING OF 120'. THE DEFLECTION FITTINGS SHALL BE PLACED AT THE PEDESTRIAN BARRIER OPEN JOINT THAT COINCIDES WITH THE RETAINING WALL STEM EXPANSION JOINT NEAREST TO THE TRANSVERSE CONSTRUCTION JOINT IN THE WALL FOOTING.
2 ~ STAINLESS STEEL MOUNTING TABS (TOP & BOT.)
BUNDLE 2 B1 #5 OR 2 B8 #5 ADJACENT TO EACH END OF JUNCTION BOX (TYP.)
JUNCTION BOX ~ 8" x 8" x 18" NEMA 4X IN STATIONARY FORM BARRIER, ADJUSTABLE NEMA 3R IN SLIP FORM BARRIER. JUNCTION BOX CAN BE RECESSED UP TO ⅛".
⅜"ø POLYETHYLENE OR COPPER PIPE DRAIN
1'-2"
SECTION A
Bridge Design Engr.
Supervisor
Designed By
Checked By
Detailed By
Bridge Projects Engr.
Prelim. Plan By
Architect/Specialist
DATE
REVISION
BY
APP'D
M:\STANDARDS\Traffic Barriers\Pedestrian Barrier\PED BARRIER 3.MAN
REGION NO.
STATE
FED. AID PROJ. NO.
SHEET NO.
TOTAL SHEETS
10
WASH.
JOB NUMBER
BRIDGE AND STRUCTURES OFFICE
Washington State Department of Transportation
STANDARD TRAFFIC BARRIERS
PEDESTRIAN BARRIER DETAILS 3 OF 3
BRIDGE SHEET NO.
SHEET
OF
SHEETS
Tue Jul 17 14:32:31 2012
SR
JOB NO.
SHEET



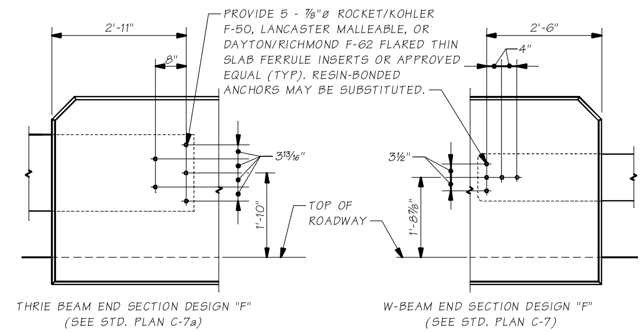
PLAN
TRAFFIC BARRIER

BARRIER CONTINUOUS BETWEEN ROADWAY EXPANSION JOINTS.
CONSTRUCTION JOINTS WITH SHEAR KEYS ARE PERMISSIBLE AT DUMMY JOINT LOCATIONS.
FORM JOINTS BETWEEN DUMMY JOINTS SHALL NOT BE PERMITTED.



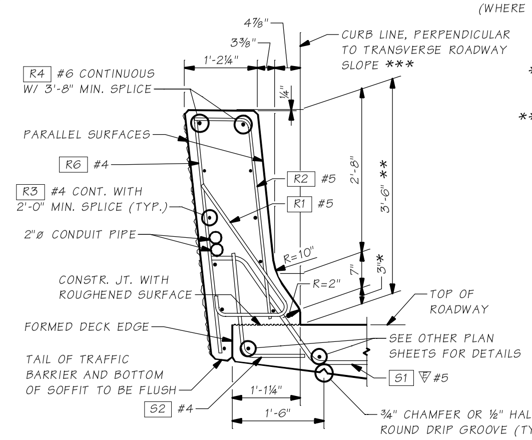
OUTSIDE ELEVATION
END OF TRAFFIC BARRIER

SHOWN WITH BRIDGE APPROACH SLAB.



OUTSIDE ELEVATION
TRAFFIC BARRIER - GUARDRAIL CONNECTION

(WHERE SHOWN ON LAYOUT)



TYPICAL SECTION
TRAFFIC BARRIER

SHOWN ON BRIDGE

* TOE HEIGHT MAY VARY, 2" MIN. TO 6" MAX.

** HEIGHT MAY VARY IF REQUIRED TO PROVIDE A PROFILE PLEASING TO THE EYE.

*** FOR TRANSVERSE ROADWAY SLOPES GREATER THAN 8%, CHANGE THE NOTE TO THE FOLLOWING:
FOR THE LOW SIDE OF THE BRIDGE OR MEDIAN BARRIER - "PERPENDICULAR TO 8% TRANSVERSE ROADWAY SLOPE"
FOR THE HIGH SIDE OF THE BRIDGE BARRIER - "PERPENDICULAR TO TRANSVERSE ROADWAY SLOPE".

NOTE TO DESIGNERS

1. If transverse roadway slope is greater than 8%, S1 and S2 bar bends need to be modified to account for the difference between the actual slope and 8% on the low side only of the bridge or median barrier. The barrier geometry needs to be checked also.
2. The non-applicable text should be removed from the actual bridge plans.

NW REGION:

TERMINATE EACH CONDUIT PIPE AT SEPARATE TYPE 1 JUNCTION BOXES OFF END OF BRIDGE AS SHOWN ON LAYOUT.

Bridge Design Engr.	M:\STANDARDS\Traffic Barriers\Shape F\42IN SHAPE F BARRIER SHT 1 - TL 4.MAN				
Supervisor	REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
Designed By	10	WASH.			
Checked By	JOB NUMBER				
Detailed By					
Bridge Projects Engr.					
Prelim. Plan By					
Architect/Specialist	DATE	REVISION	BY	APP'D	

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

STANDARD TRAFFIC BARRIERS

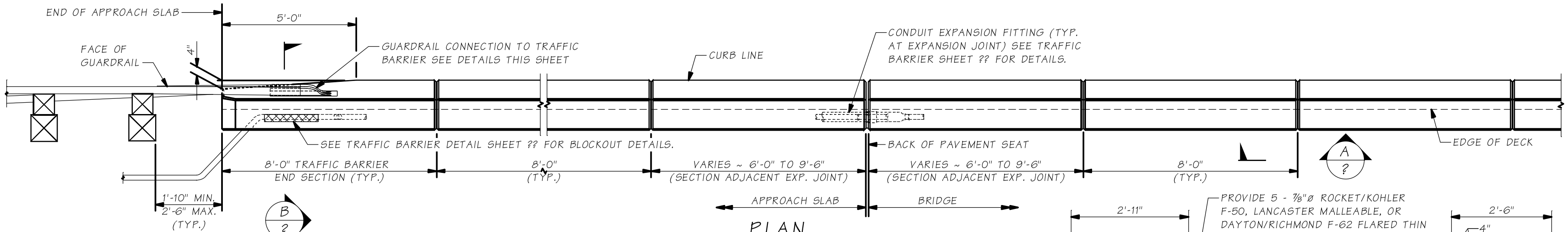
TRAFFIC BARRIER - SHAPE F 42" DETAILS 1 OF 3

BRIDGE SHEET NO. SHEET OF SHEETS

Thu Jun 07 14:19:05 2012

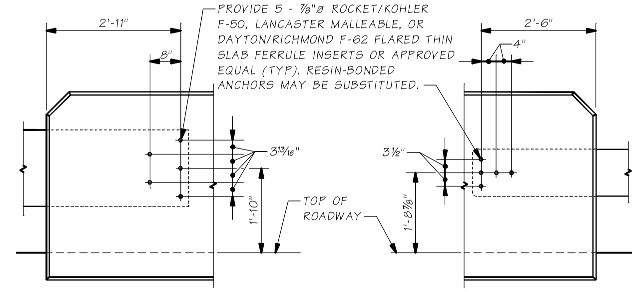
SR JOB NO. SHEET 1

Last revised on : 06/23/2010



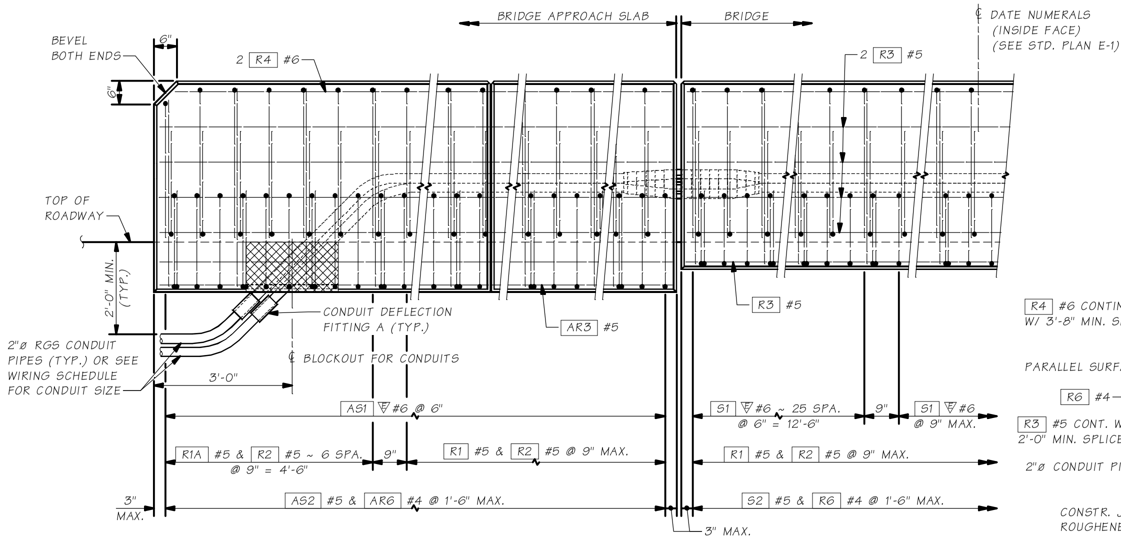
PLAN
TRAFFIC BARRIER

BARRIER CONTINUOUS BETWEEN ROADWAY EXPANSION JOINTS.
CONSTRUCTION JOINTS WITH SHEAR KEYS ARE PERMISSIBLE AT DUMMY JOINT LOCATIONS.
FORM JOINTS BETWEEN DUMMY JOINTS SHALL NOT BE PERMITTED.

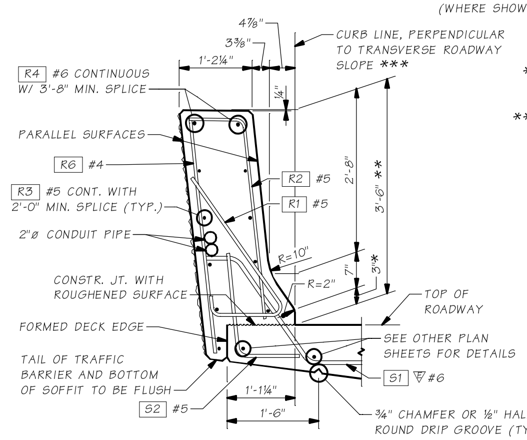


THRIE BEAM END SECTION DESIGN "F" (SEE STD. PLAN C-7a)
W-BEAM END SECTION DESIGN "F" (SEE STD. PLAN C-7)

OUTSIDE ELEVATION
TRAFFIC BARRIER - GUARDRAIL CONNECTION
(WHERE SHOWN ON LAYOUT)



OUTSIDE ELEVATION
END OF TRAFFIC BARRIER
SHOWN WITH BRIDGE APPROACH SLAB



TYPICAL SECTION
TRAFFIC BARRIER
SHOWN ON BRIDGE

* TOE HEIGHT MAY VARY, 2" MIN. TO 6" MAX.

** HEIGHT MAY VARY IF REQUIRED TO PROVIDE A PROFILE PLEASING TO THE EYE.

*** FOR TRANSVERSE ROADWAY SLOPES GREATER THAN 8%, CHANGE THE NOTE TO THE FOLLOWING:
FOR THE LOW SIDE OF THE BRIDGE OR MEDIAN BARRIER - "PERPENDICULAR TO 8% TRANSVERSE ROADWAY SLOPE"
FOR THE HIGH SIDE OF THE BRIDGE BARRIER - "PERPENDICULAR TO TRANSVERSE ROADWAY SLOPE".

NOTE TO DESIGNERS

1. If transverse roadway slope is greater than 8%, S1 and S2 bar bends need to be modified to account for the difference between the actual slope and 8% on the low side only of the bridge or median barrier. The barrier geometry needs to be checked also.
2. The non-applicable text should be removed from the actual bridge plans.

NW REGION:
TERMINATE EACH CONDUIT PIPE AT SEPARATE TYPE 1 JUNCTION BOXES OFF END OF BRIDGE AS SHOWN ON LAYOUT.

Bridge Design Engr.	M:\STANDARDS\Traffic Barriers\Shape F\42IN SHAPE F BARRIER SHT 1 - TL 5.MAN			
Supervisor		REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
Designed By		10	WASH.			
Checked By		JOB NUMBER				
Detailed By						
Bridge Projects Engr.						
Prelim. Plan By						
Architect/Specialist	DATE	REVISION	BY	APP'D		

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

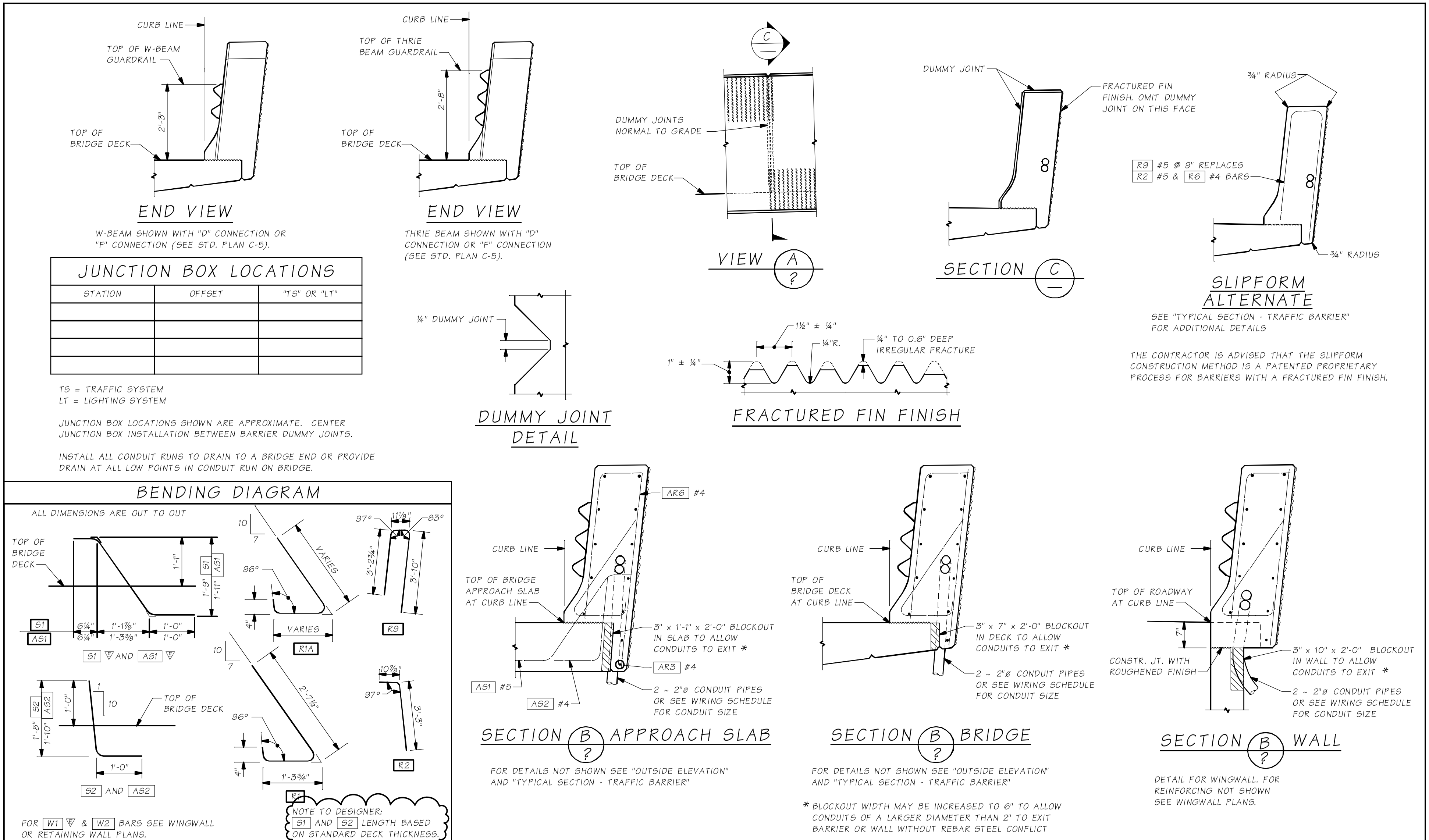
STANDARD TRAFFIC BARRIERS

TRAFFIC BARRIER - SHAPE F 42"
DETAILS 1 OF 3

BRIDGE SHEET NO.
SHEET
OF
SHEETS

Thu Jun 07 14:19:06 2012

SR
JOB NO.
SHEET 1



JUNCTION BOX LOCATIONS

STATION	OFFSET	"TS" OR "LT"

TS = TRAFFIC SYSTEM
LT = LIGHTING SYSTEM

JUNCTION BOX LOCATIONS SHOWN ARE APPROXIMATE. CENTER JUNCTION BOX INSTALLATION BETWEEN BARRIER DUMMY JOINTS.

INSTALL ALL CONDUIT RUNS TO DRAIN TO A BRIDGE END OR PROVIDE DRAIN AT ALL LOW POINTS IN CONDUIT RUN ON BRIDGE.

Bridge Design Engr. / Supervisor / Designed By / Checked By / Detailed By / Bridge Projects Engr. / Prelim. Plan By / Architect/Specialist — DATE — REVISION — BY — APP'D

M:\STANDARDS\Traffic Barriers\Shape F\42IN SHAPE F BARRIER SHT 2 - TL 4.MAN

REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
10	WASH.			

JOB NUMBER

BRIDGE AND STRUCTURES OFFICE

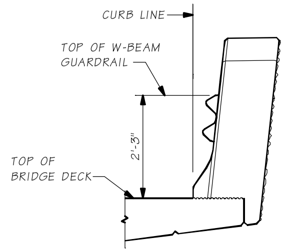
Washington State Department of Transportation

STANDARD TRAFFIC BARRIER

TRAFFIC BARRIER - SHAPE F 42"
DETAIL 2 OF 3

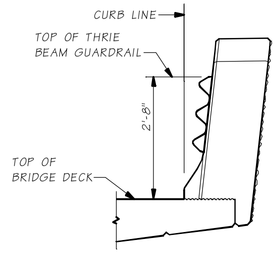
BRIDGE SHEET NO. / SHEET OF SHEETS

Thu Jun 07 14:19:06 2012



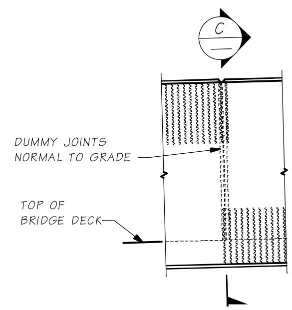
END VIEW

W-BEAM SHOWN WITH "D" CONNECTION OR "F" CONNECTION (SEE STD. PLAN C-5).

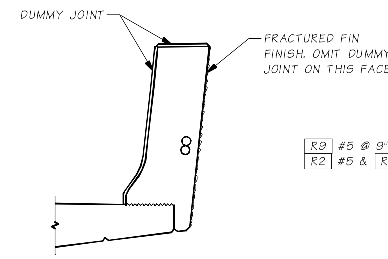


END VIEW

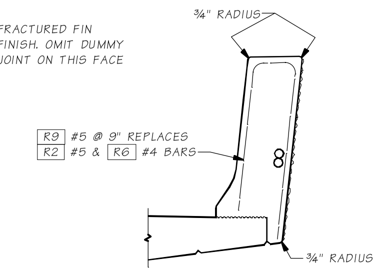
THRIE BEAM SHOWN WITH "D" CONNECTION OR "F" CONNECTION (SEE STD. PLAN C-5).



VIEW (A/?)



SECTION (C/—)



SLIPFORM ALTERNATE

SEE "TYPICAL SECTION - TRAFFIC BARRIER" FOR ADDITIONAL DETAILS

THE CONTRACTOR IS ADVISED THAT THE SLIPFORM CONSTRUCTION METHOD IS A PATENTED PROPRIETARY PROCESS FOR BARRIERS WITH A FRACTURED FIN FINISH.

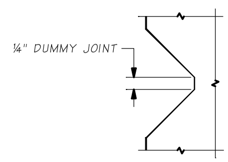
JUNCTION BOX LOCATIONS

STATION	OFFSET	"TS" OR "LT"

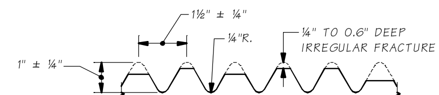
TS = TRAFFIC SYSTEM
LT = LIGHTING SYSTEM

JUNCTION BOX LOCATIONS SHOWN ARE APPROXIMATE. CENTER JUNCTION BOX INSTALLATION BETWEEN BARRIER DUMMY JOINTS.

INSTALL ALL CONDUIT RUNS TO DRAIN TO A BRIDGE END OR PROVIDE DRAIN AT ALL LOW POINTS IN CONDUIT RUN ON BRIDGE.

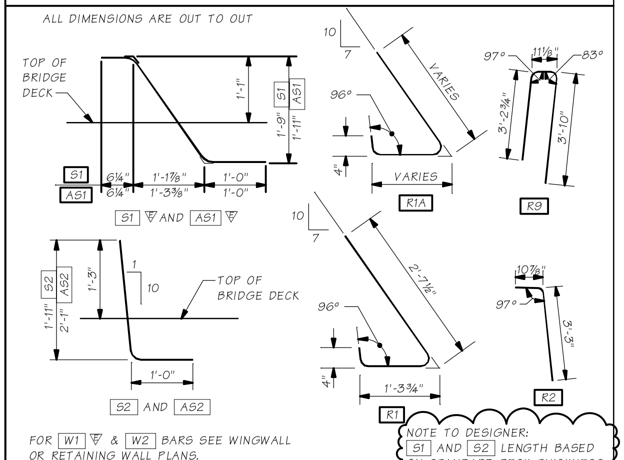


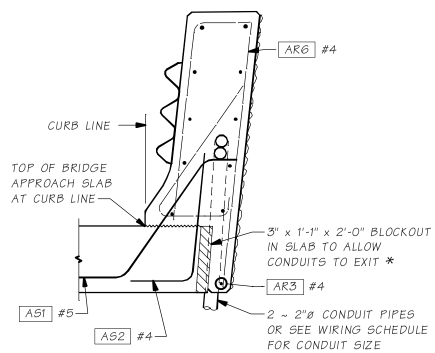
DUMMY JOINT DETAIL



FRACTURED FIN FINISH

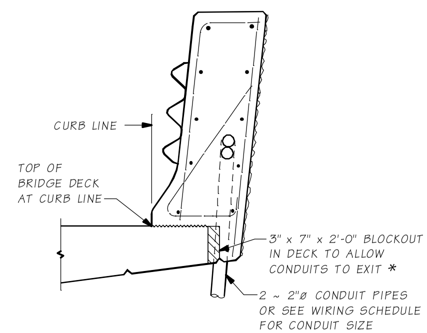
BENDING DIAGRAM





SECTION (B/?) APPROACH SLAB

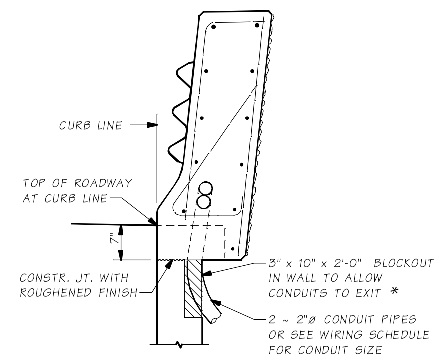
FOR DETAILS NOT SHOWN SEE "OUTSIDE ELEVATION" AND "TYPICAL SECTION - TRAFFIC BARRIER"



SECTION (B/?) BRIDGE

FOR DETAILS NOT SHOWN SEE "OUTSIDE ELEVATION" AND "TYPICAL SECTION - TRAFFIC BARRIER"

* BLOCKOUT WIDTH MAY BE INCREASED TO 6" TO ALLOW CONDUITS OF A LARGER DIAMETER THAN 2" TO EXIT BARRIER OR WALL WITHOUT REBAR STEEL CONFLICT



SECTION (B/?) WALL

DETAIL FOR WINGWALL. FOR REINFORCING NOT SHOWN SEE WINGWALL PLANS.

Bridge Design Engr.				
Supervisor				
Designed By				
Checked By				
Detailed By				
Bridge Projects Engr.				
Prelim. Plan By				
Architect/Specialist	DATE	REVISION	BY	APP'D

M:\STANDARDS\Traffic Barriers\Shape F\PIN SHAPE F BARRIER SHT 2 - TL 5.MAN

REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
10	WASH.			

JOB NUMBER

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

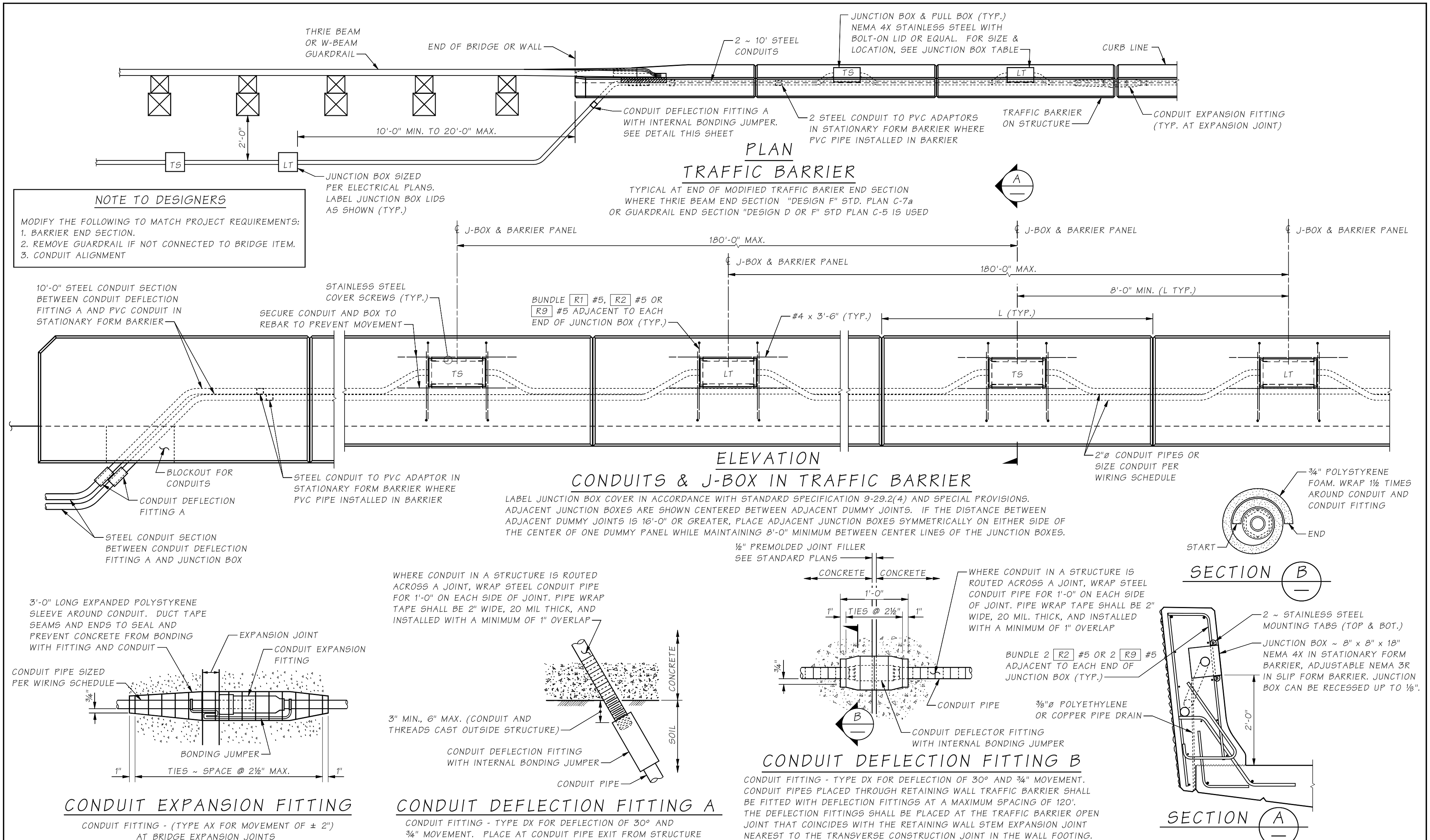
STANDARD TRAFFIC BARRIER

TRAFFIC BARRIER - SHAPE F 42"
DETAIL 2 OF 3

BRIDGE SHEET NO.
SHEET OF SHEETS

Thu Jun 07 14:19:06 2012

SR
JOB NO.
SHEET



Bridge Design Engr.	M:\STANDARDS\Traffic Barriers\Shape F\42IN SHAPE F BARRIER SHT 3.MAN				
Supervisor		REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO. / TOTAL SHEETS
Designed By		10	WASH.		
Checked By		JOB NUMBER			
Detailed By					
Bridge Projects Engr.					
Prelim. Plan By					
Architect/Specialist	DATE / REVISION / BY / APP'D				

BRIDGE AND STRUCTURES OFFICE

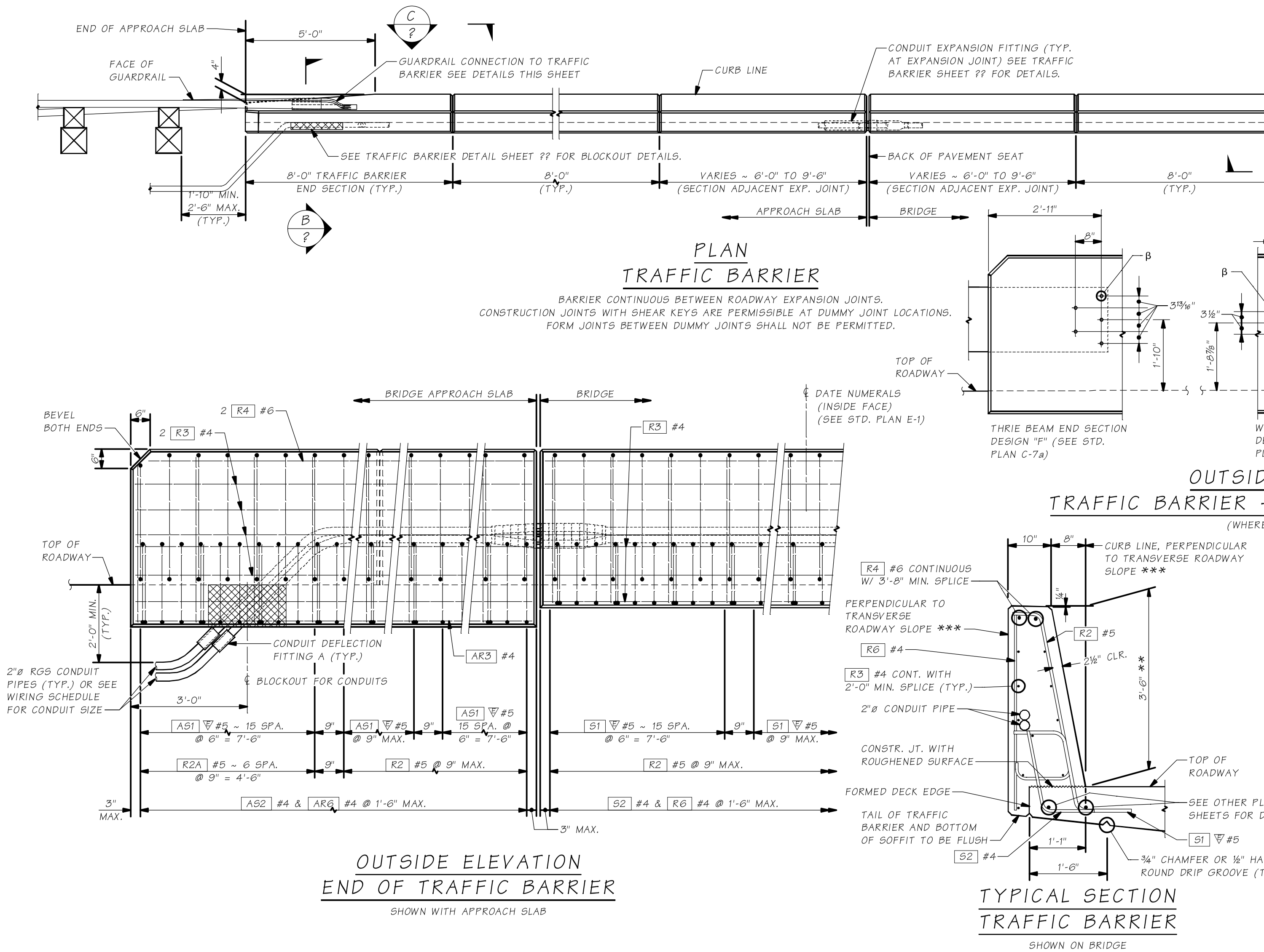
Washington State Department of Transportation

STANDARD TRAFFIC BARRIER

TRAFFIC BARRIER - SHAPE F 42"
DETAIL 3 OF 3

BRIDGE SHEET NO.
SHEET OF SHEETS

Tue Jul 17 14:32:32 2012



β PROVIDE ⅞"ø ROCKET/KOHLER F-50, LANCASTER MALLEABLE, OR DAYTON/RICHMOND F-62 FLARED THIN SLAB FERRULE INSERTS OR APPROVED EQUAL (TYP.). RESIN-BONDED ANCHORS MAY BE SUBSTITUTED.

* TOE HEIGHT MAY VARY, 2" MIN. TO 6" MAX.

** HEIGHT MAY VARY IF REQUIRED TO PROVIDE A PROFILE PLEASING TO THE EYE

*** FOR TRANSVERSE ROADWAY SLOPES GREATER THAN 8%, CHANGE THE NOTE TO THE FOLLOWING:
FOR THE LOW SIDE OF THE BRIDGE OR MEDIAN BARRIER - "PERPENDICULAR TO 8% TRANSVERSE ROADWAY SLOPE"
FOR THE HIGH SIDE OF THE BRIDGE BARRIER - "PERPENDICULAR TO TRANSVERSE ROADWAY SLOPE"

NOTE TO DESIGNERS

1. If transverse roadway slope is greater than 8%, S1 and S2 bar bends need to be modified to account for the difference between the actual slope and 8% on the low side only of the bridge or median barrier. The barrier geometry needs to be checked also.
2. The non-applicable text should be removed from the actual bridge plans.

NW REGION:

TERMINATE EACH CONDUIT PIPE AT SEPARATE TYPE 1 JUNCTION BOXES OFF END OF BRIDGE AS SHOWN ON LAYOUT.

Bridge Design Engr.	M:\STANDARDS\Traffic Barriers\Single Slope\SS42TB1-TL4.MAN						
Supervisor		REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS	
Designed By		10	WASH.				
Checked By		JOB NUMBER					
Detailed By							
Bridge Projects Engr.							
Prelim. Plan By							
Architect/Specialist	DATE	REVISION	BY	APP'D			

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

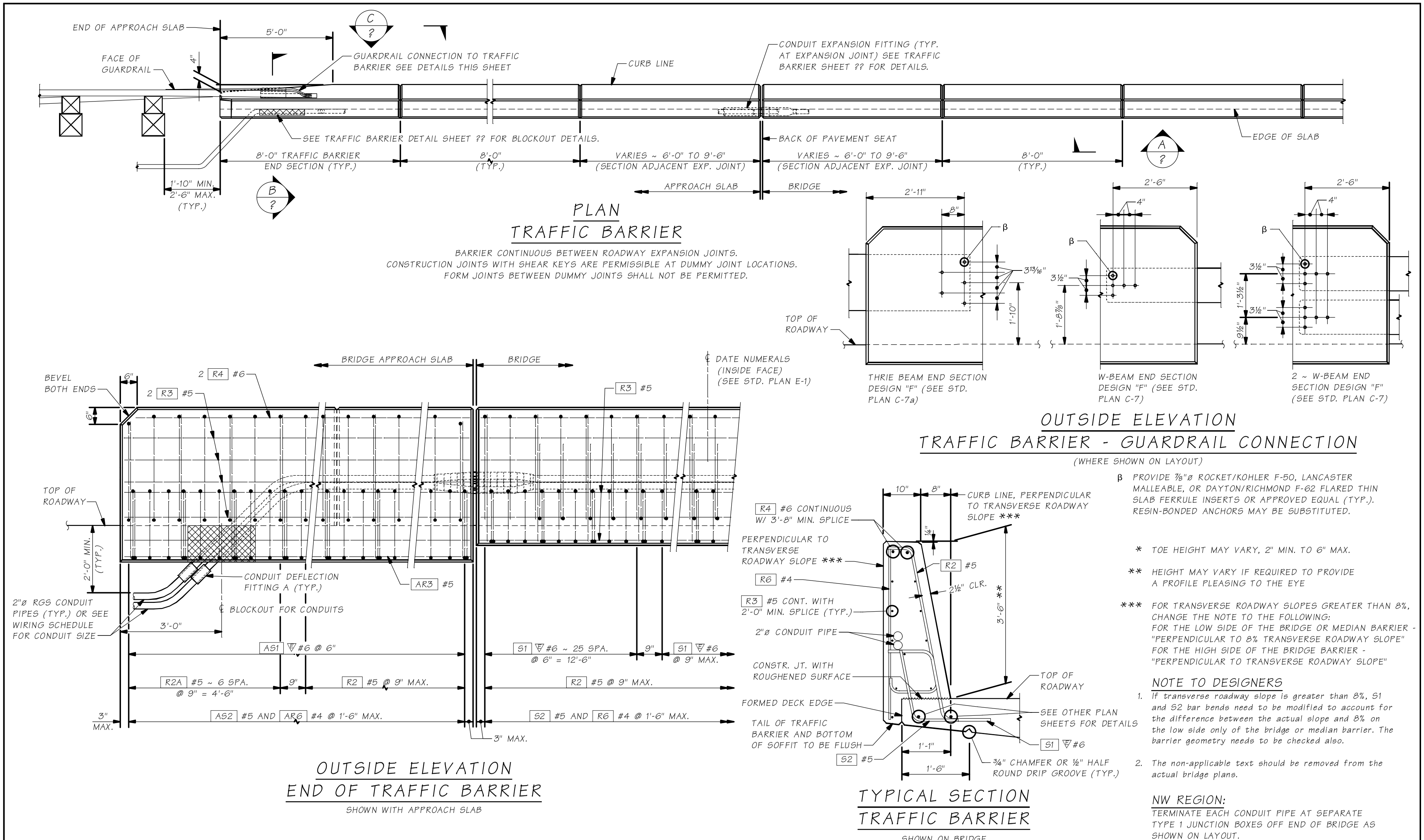
STANDARD TRAFFIC BARRIERS

TRAFFIC BARRIER - SINGLE SLOPE 42" DETAILS 1 OF 3

BRIDGE SHEET NO. | SHEET OF SHEETS

Thu Jun 07 14:19:06 2012

JOB NO. SR SHEET



Bridge Design Engr.	M:\STANDARDS\Traffic Barriers\Single Slope\SS42TB1-TL5.MAN	
Supervisor	REGION NO. 10	STATE WASH.
Designed By	FED. AID PROJ. NO.	
Checked By	JOB NUMBER	
Detailed By		
Bridge Projects Engr.		
Prelim. Plan By		
Architect/Specialist	DATE / REVISION / BY / APP'D	

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

STANDARD TRAFFIC BARRIERS

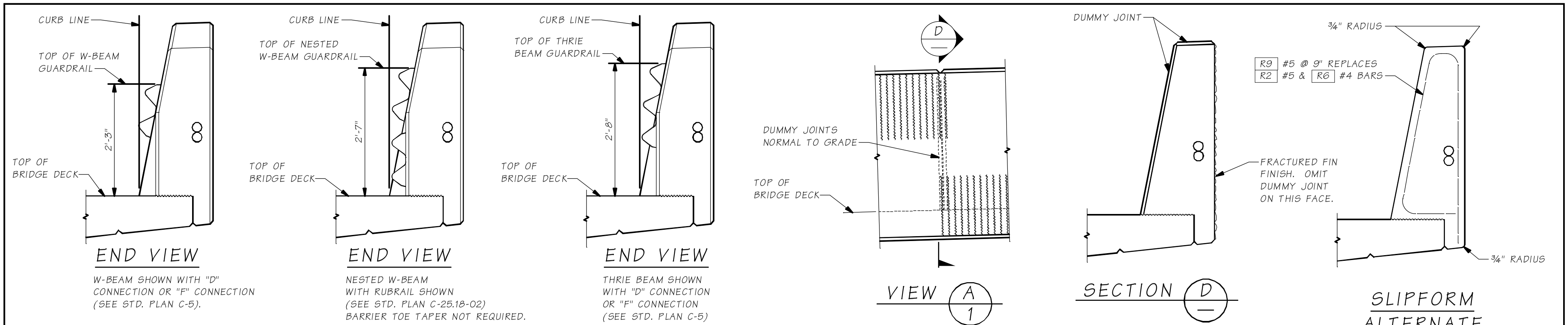
TRAFFIC BARRIER - SINGLE SLOPE 42" DETAILS 1 OF 3

BRIDGE SHEET NO. / SHEET OF SHEETS

Thu Jun 07 14:19:07 2012

Last revised on : 8/23/2011

SR ____ JOB NO. ____ SHEET ____



SLIPFORM ALTERNATE

SEE "TYPICAL SECTION - TRAFFIC BARRIER" FOR ADDITIONAL DETAILS.

THE CONTRACTOR IS ADVISED THAT THE SLIPFORM CONSTRUCTION METHOD IS A PATENTED PROPRIETARY PROCESS FOR BARRIERS WITH A FRACTURED FIN FINISH.

JUNCTION BOX LOCATIONS

STATION	OFFSET	"TS" OR "LT"

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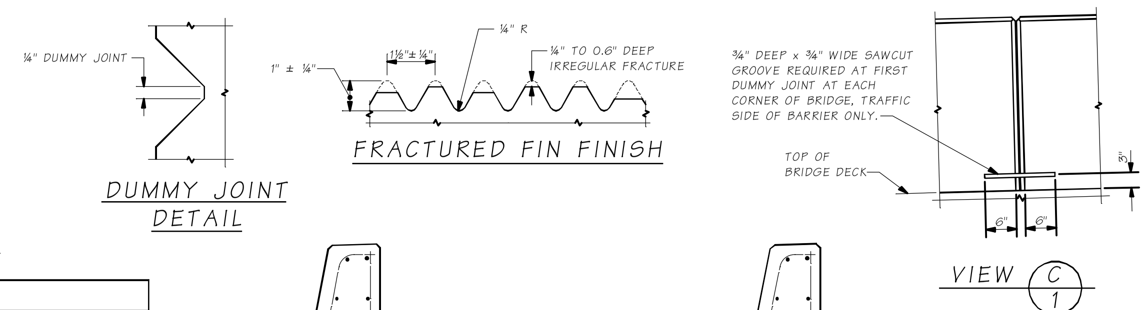
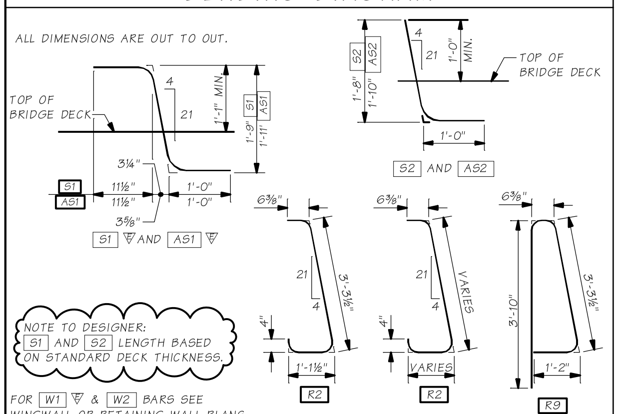
JUNCTION BOX LOCATIONS SHOWN ARE APPROXIMATE. CENTER JUNCTION BOX INSTALLATION BETWEEN BARRIER DUMMY JOINTS.

INSTALL ALL CONDUIT RUNS TO DRAIN TO A BRIDGE END OR PROVIDE DRAIN AT ALL LOW POINTS IN CONDUIT RUN ON BRIDGE.



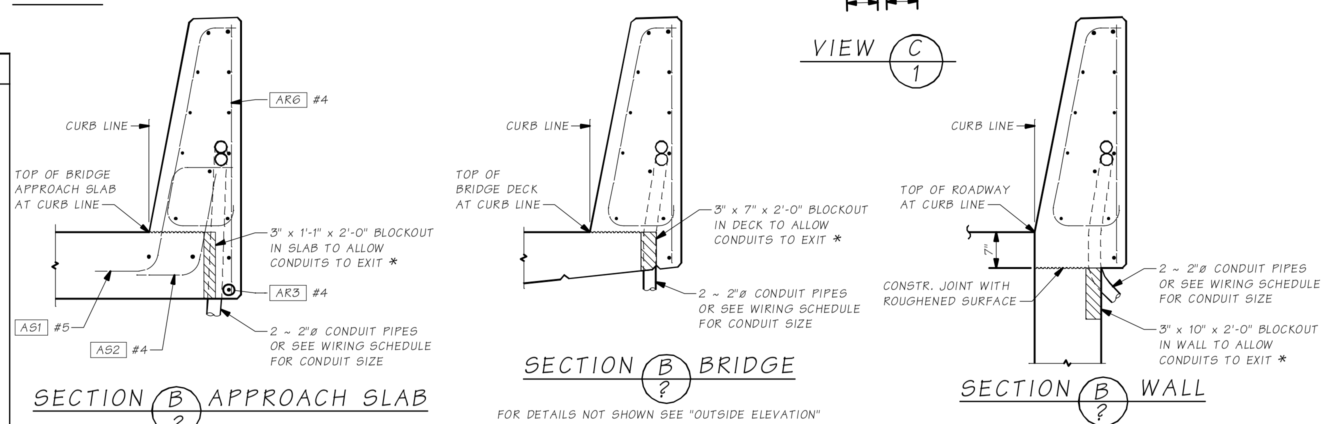
BENDING DIAGRAM

ALL DIMENSIONS ARE OUT TO OUT.



NOTE TO DESIGNER:
S1 AND S2 LENGTH BASED ON STANDARD DECK THICKNESS.

FOR W1 & W2 BARS SEE WINGWALL OR RETAINING WALL PLANS.



FOR DETAILS NOT SHOWN SEE "OUTSIDE ELEVATION" AND "TYPICAL SECTION - TRAFFIC BARRIER"

FOR DETAILS NOT SHOWN SEE "OUTSIDE ELEVATION" AND "TYPICAL SECTION - TRAFFIC BARRIER"

* BLOCKOUT WIDTH MAY BE INCREASED TO 6" TO ALLOW CONDUITS OF A LARGER DIAMETER THAN 2" TO EXIT BARRIER OR WALL WITHOUT REBAR STEEL CONFLICT

DETAIL FOR WINGWALL. FOR REINFORCING NOT SHOWN SEE WINGWALL PLANS.

Bridge Design Engr.	M:\STANDARDS\Traffic Barriers\Single Slope\SS42TB2-TL4.MAN			
Supervisor				
Designed By				
Checked By				
Detailed By				
Bridge Projects Engr.				
Prelim. Plan By				
Architect/Specialist	DATE	REVISION	BY	APP'D

REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
10	WASH.			

JOB NUMBER

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

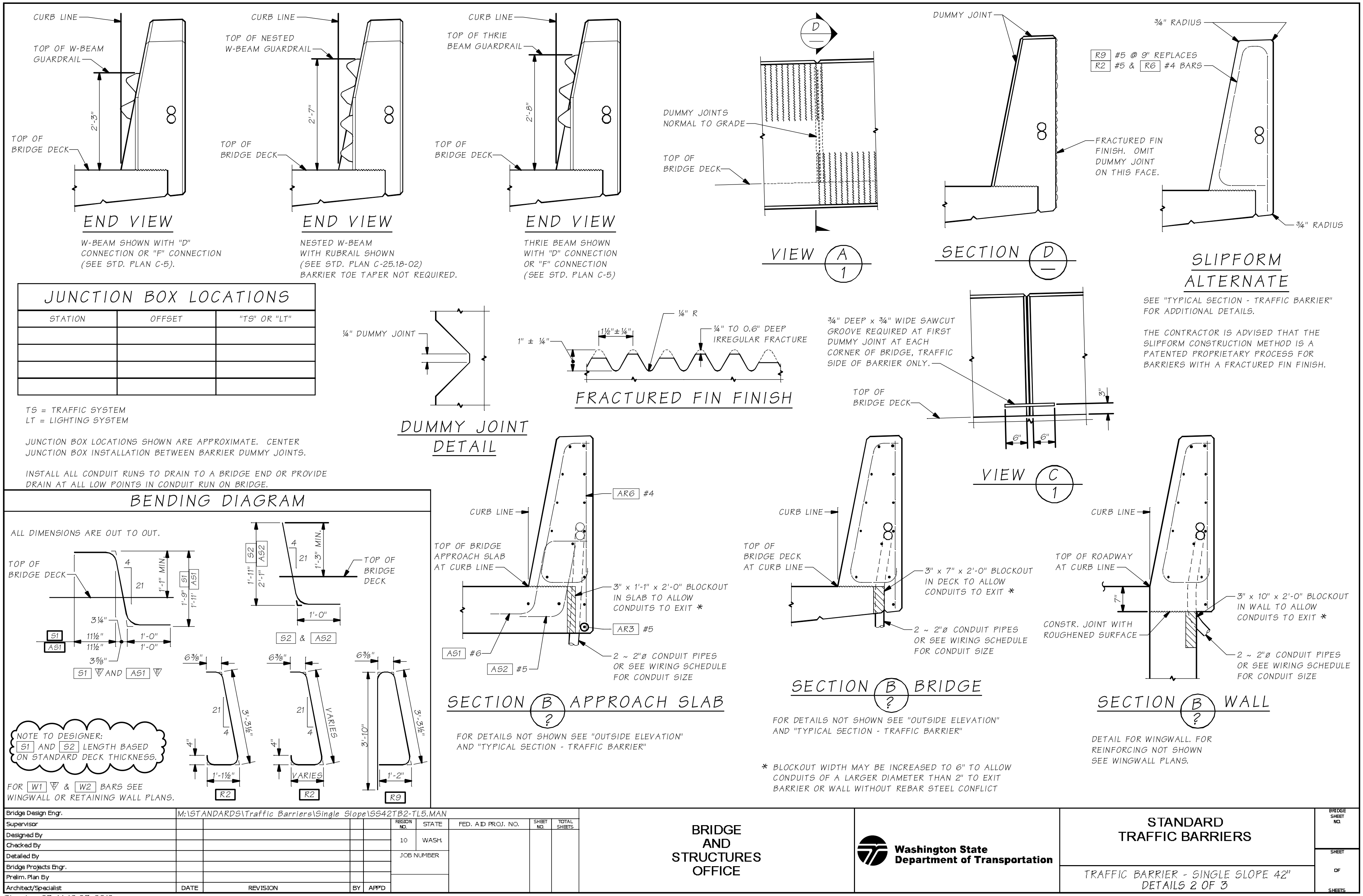
STANDARD TRAFFIC BARRIERS

TRAFFIC BARRIER - SINGLE SLOPE 42" DETAILS 2 OF 3

BRIDGE SHEET NO.
SHEET OF SHEETS

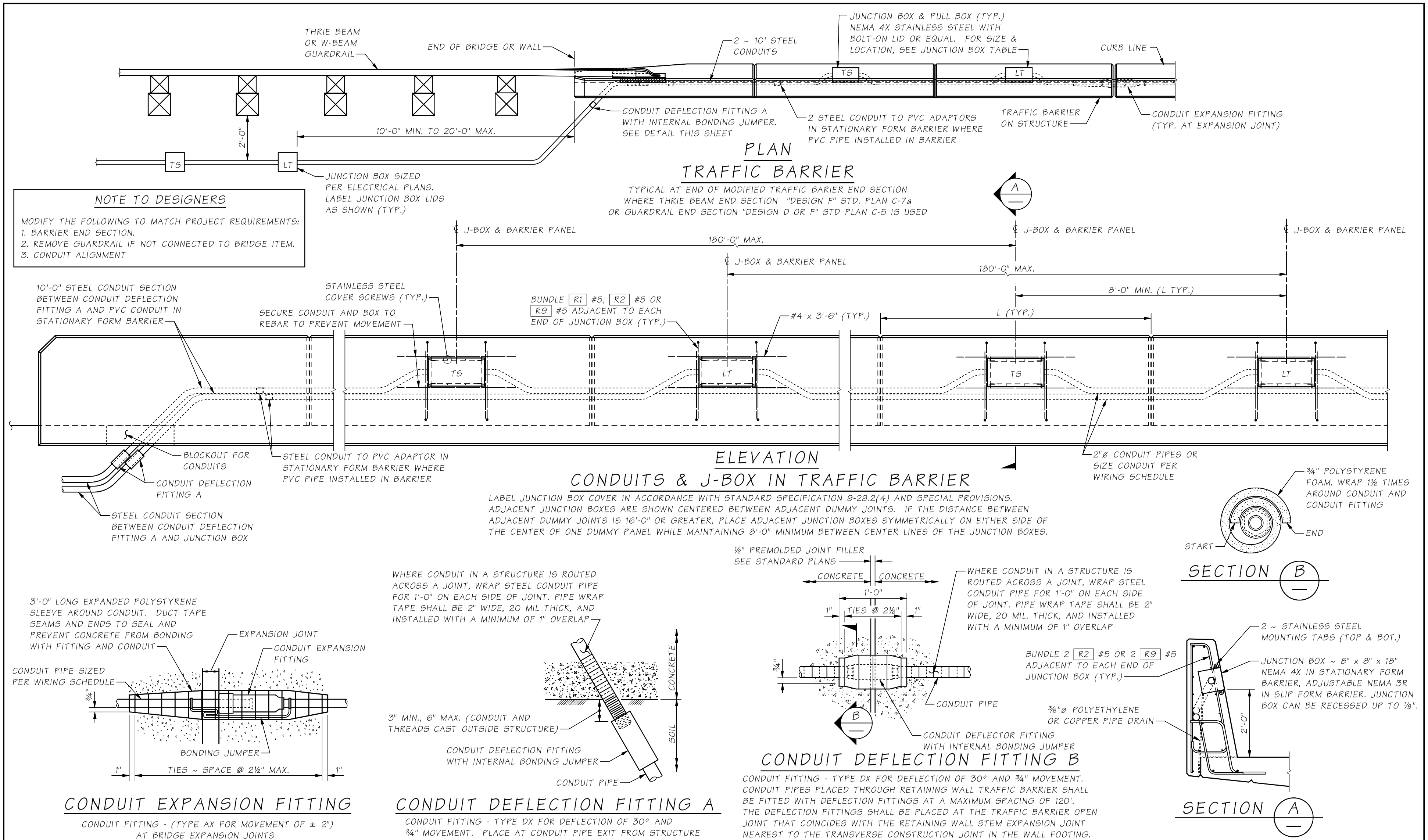
Thu Jun 07 14:19:06 2012

SR JOB NO. SHEET



JUNCTION BOX LOCATIONS

STATION	OFFSET	"TS" OR "LT"



Bridge Design Engr.	M:\STANDARDS\Traffic Barriers\Single Slope\SS42TB3.MAN			
Supervisor	REGION NO. 10	STATE WASH.	FED. AID PROJ. NO.	SHEET NO. / TOTAL SHEETS
Designed By	JOB NUMBER			
Checked By				
Detailed By				
Bridge Projects Engr.				
Prelim. Plan By				
Architect/Specialist	DATE	REVISION	BY	APP'D

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

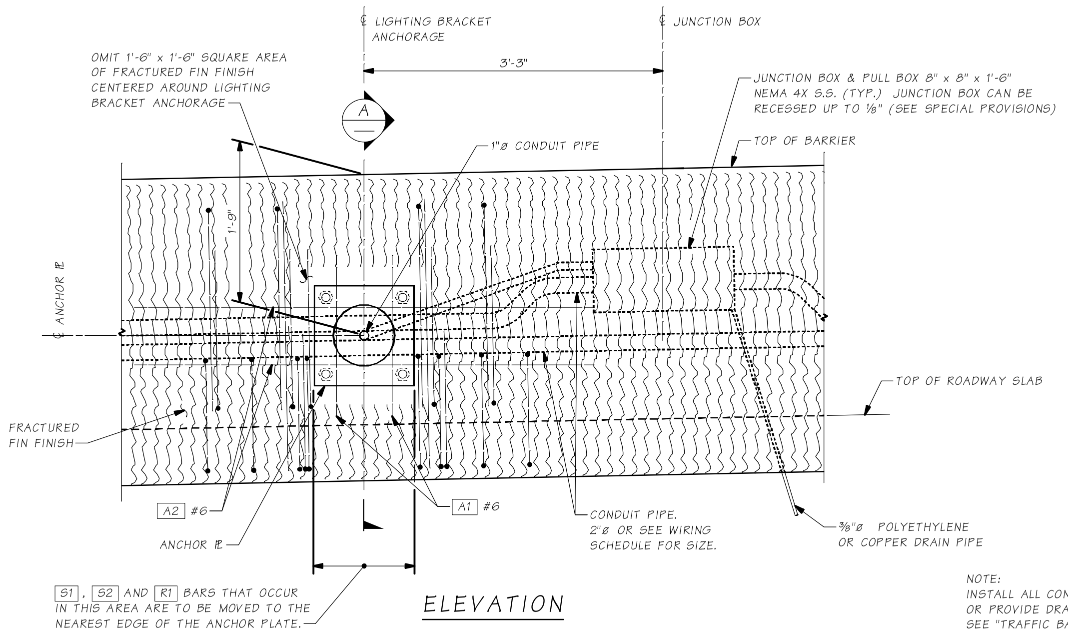
STANDARD TRAFFIC BARRIERS

TRAFFIC BARRIER - SINGLE SLOPE 42" DETAILS 3 OF 3

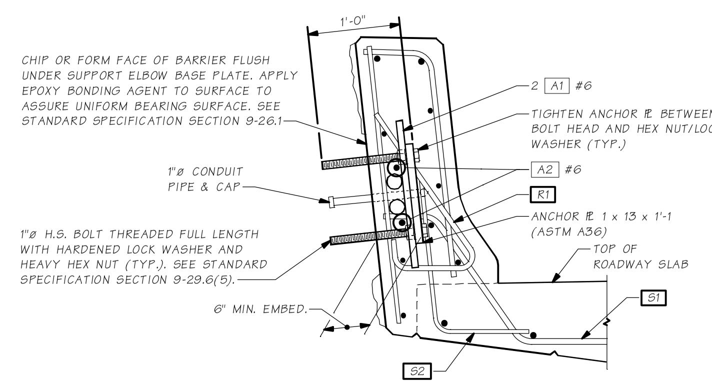
BRIDGE SHEET NO. / SHEET OF SHEETS

Tue Jul 17 14:32:33 2012

SR / JOB NO. / SHEET



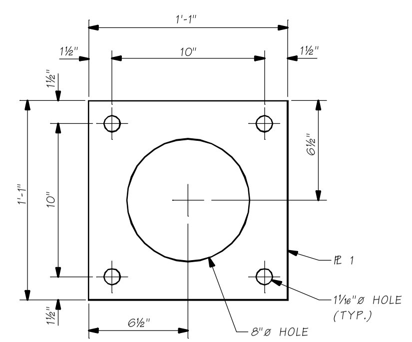
ELEVATION



SECTION A

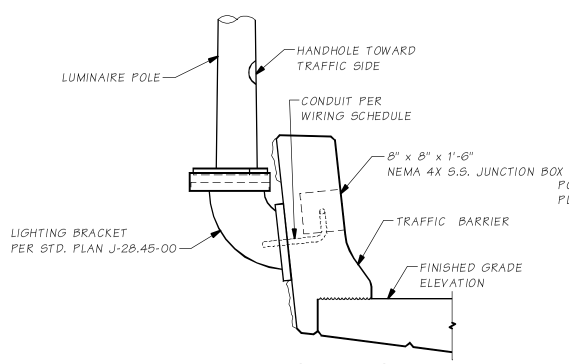
SEE TRAFFIC BARRIER SHEETS FOR INFORMATION NOT SHOWN.

NOTE:
INSTALL ALL CONDUIT RUNS TO DRAIN TO A BRIDGE END OR PROVIDE DRAIN AT ALL LOW POINTS WITHIN CONDUIT RUN.
SEE "TRAFFIC BARRIER SHEETS" FOR BARRIER DIMENSIONS.



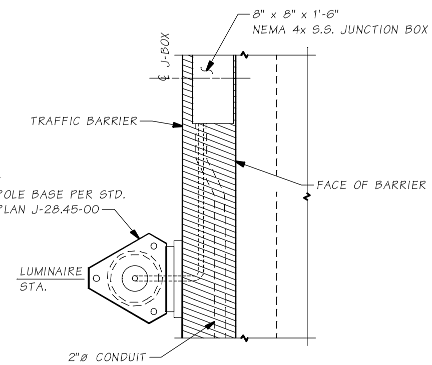
ANCHOR PLATE

GALVANIZE PER AASHTO M 111



ELEVATION PROFILE
LUMINAIRE SUPPORT

SHOWN FOR BRIDGE, WALLS ARE SIMILAR



PLAN

LUMINAIRE POLE LOCATIONS ON BRIDGE & WALLS	
STATION	OFFSET
XXX 0+000.000	0.000 RT.
XXX 0+000.000	0.000 RT.

ANCHORAGE BARLIST			
MARK #	SIZE	LENGTH	BEND TYPE
A1	6	1'-9"	STRAIGHT
A2	6	5'-0"	STRAIGHT

Bridge Design Engr.
Supervisor
Designed By
Checked By
Detailed By
Bridge Projects Engr.
Prelim. Plan By
Architect/Specialist

M:\STANDARDS\Traffic Barriers\Luminare\LUMANCHORAGE-TBF.man

DATE	REVISION	BY	APP'D

REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
10	WASH.			

JOB NUMBER

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

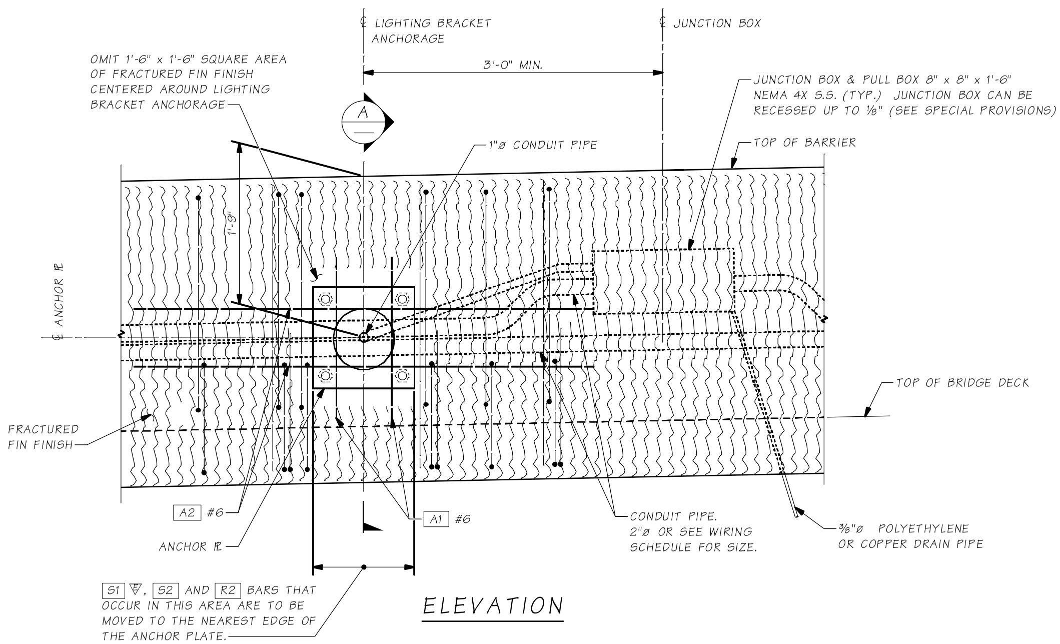
STANDARD TRAFFIC BARRIERS

TRAFFIC BARRIER - SHAPE F
LUMINAIRE ANCHORAGE DEATILS

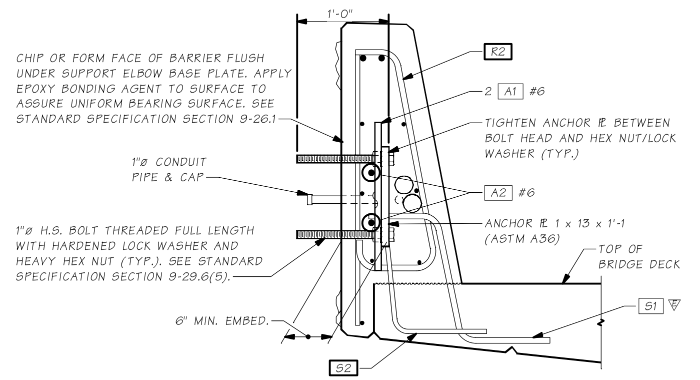
BRIDGE SHEET NO.
SHEET
OF
SHEETS

SR
JOB NO.
SHEET

Fri Jun 15 14:19:16 2012



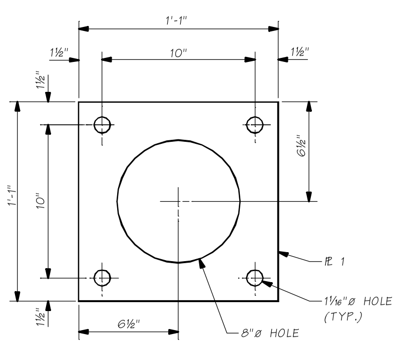
ELEVATION



SECTION A

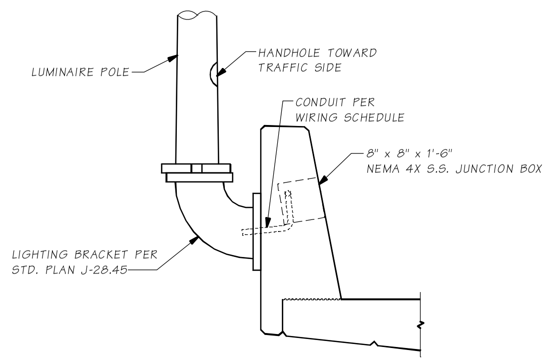
SEE TRAFFIC BARRIER SHEETS FOR INFORMATION NOT SHOWN.

NOTE:
INSTALL ALL CONDUIT RUNS TO DRAIN TO A BRIDGE END OR PROVIDE DRAIN AT ALL LOW POINTS WITHIN CONDUIT RUN. SEE "TRAFFIC BARRIER SHEETS" FOR BARRIER DIMENSIONS.



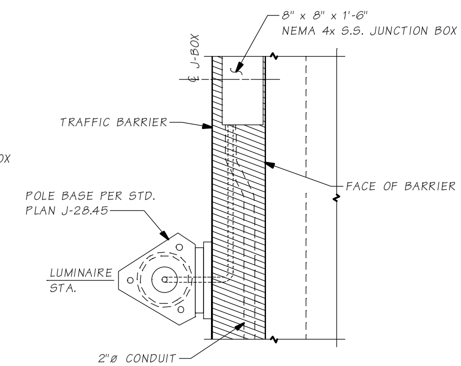
ANCHOR PLATE

GALVANIZE PER AASHTO M 111



ELEVATION PROFILE
LUMINAIRE SUPPORT

SHOWN FOR BRIDGE, WALLS ARE SIMILAR



PLAN

LUMINAIRE POLE LOCATIONS ON BRIDGE & WALLS

STATION	OFFSET
XXX 0+000.000	0.000 RT.
XXX 0+000.000	0.000 RT.

ANCHORAGE BARLIST

MARK #	SIZE	LENGTH	BEND TYPE
A1	6	1'-9"	STRAIGHT
A2	6	5'-0"	STRAIGHT

Bridge Design Engr.	M:\STANDARDS\Traffic Barriers\Luminare\LUMANCHORAGE-SSTB.MAN
Supervisor	
Designed By	
Checked By	
Detailed By	
Bridge Projects Engr.	
Prelim. Plan By	
Architect/Specialist	

DATE | REVISION | BY | APP'D

REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
10	WASH.			

JOB NUMBER

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

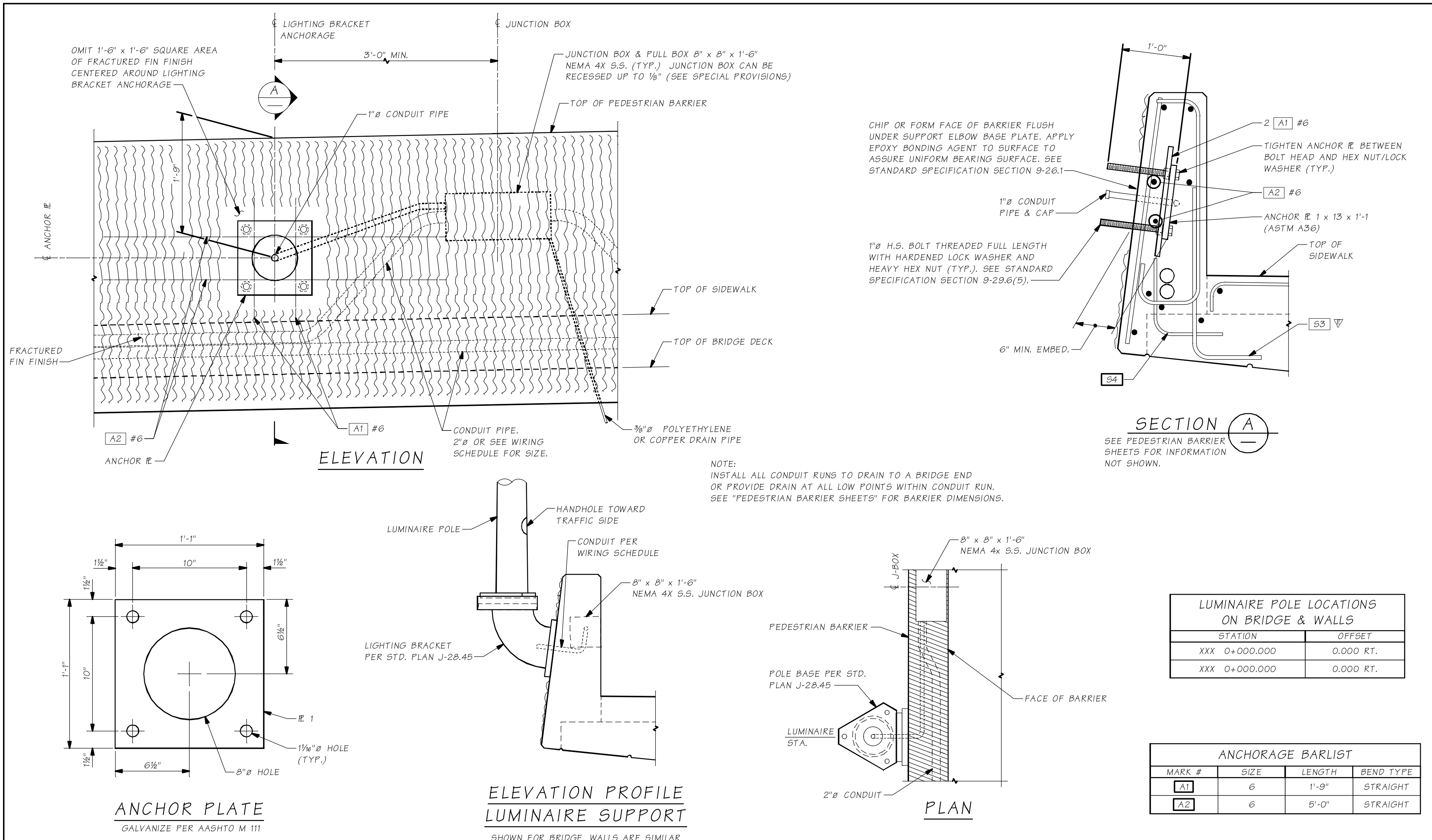
STANDARD TRAFFIC BARRIERS

TRAFFIC BARRIER - SINGLE SLOPE
LUMINAIRE ANCHORAGE DEATILS

BRIDGE SHEET NO.
SHEET OF SHEETS

Fri Jun 15 14:19:16 2012

JOB NO. SR SHEET



LUMINAIRE POLE LOCATIONS ON BRIDGE & WALLS

STATION	OFFSET
XXX 0+000.000	0.000 RT.
XXX 0+000.000	0.000 RT.

ANCHORAGE BARLIST

MARK #	SIZE	LENGTH	BEND TYPE
A1	6	1'-9"	STRAIGHT
A2	6	5'-0"	STRAIGHT

Bridge Design Engr. Khaleghi, B
Supervisor Lewis, RA
Designed By
Checked By
Detailed By
Bridge Projects Engr.
Prelim. Plan By
Architect/Specialist
DATE REVISION BY APP'D
M:\STANDARDS\Traffic Barriers\Luminare\LUMANCHORAGE-PED.MAN
REGION NO. 10 | STATE WASH. | FED. AID PROJ. NO. | SHEET NO. | TOTAL SHEETS
JOB NUMBER 10C502
Fri Jun 15 14:19:16 2012

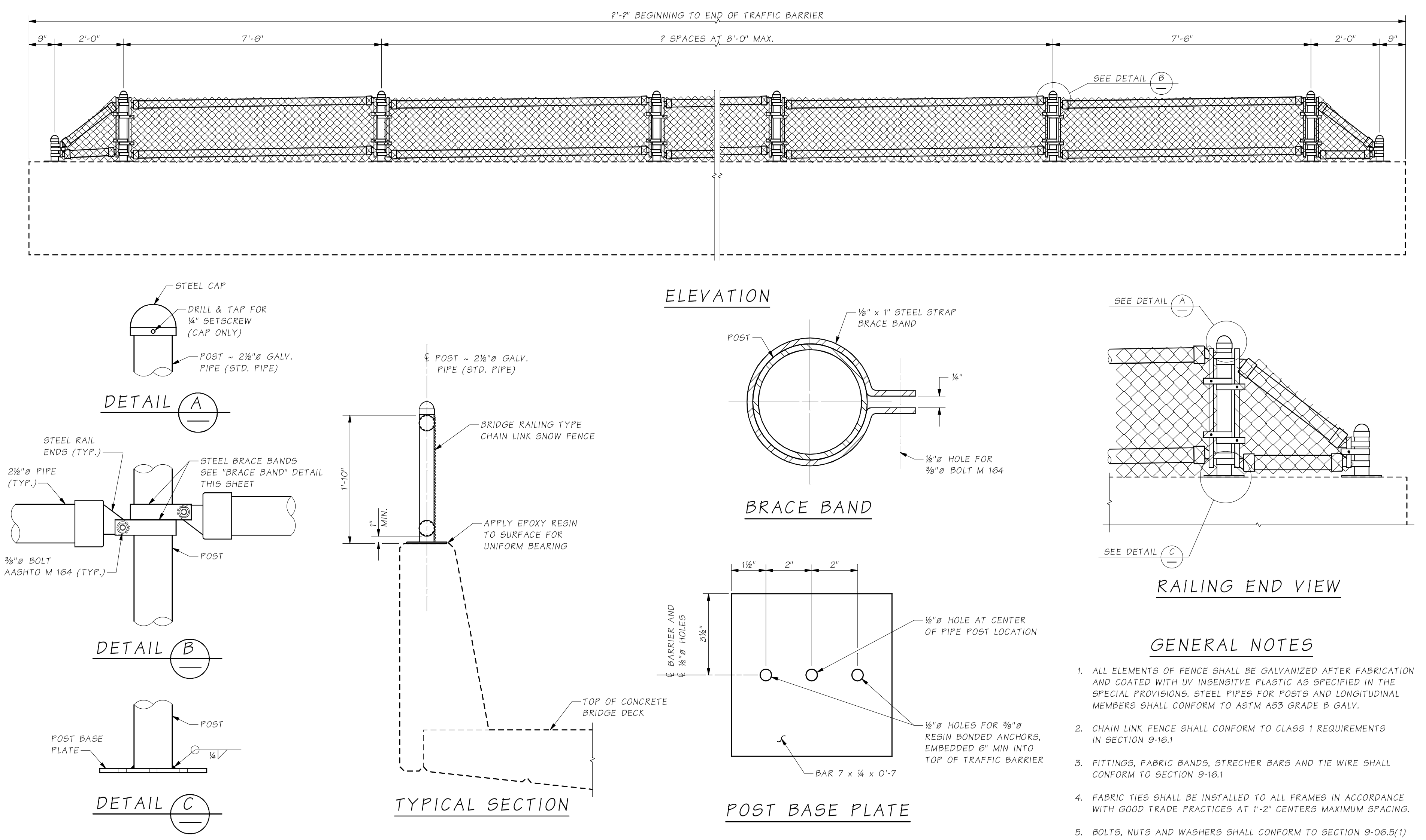
BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

I-5
GRAND MOUND TO MAYTOWN
STAGE TWO - REPLACE INTERCHANGE
US 12 OVER I-5 12/118 REPLACEMENT
BRIDGE MOUNTED ELBOW LUMINAIRE

BRIDGE SHEET NO. 0
SHEET OF SHEETS

SR 12 FILE NO. SHEET 0



GENERAL NOTES

1. ALL ELEMENTS OF FENCE SHALL BE GALVANIZED AFTER FABRICATION AND COATED WITH UV INSENSITVE PLASTIC AS SPECIFIED IN THE SPECIAL PROVISIONS. STEEL PIPES FOR POSTS AND LONGITUDINAL MEMBERS SHALL CONFORM TO ASTM A53 GRADE B GALV.
2. CHAIN LINK FENCE SHALL CONFORM TO CLASS 1 REQUIREMENTS IN SECTION 9-16.1
3. FITTINGS, FABRIC BANDS, STRECHER BARS AND TIE WIRE SHALL CONFORM TO SECTION 9-16.1
4. FABRIC TIES SHALL BE INSTALLED TO ALL FRAMES IN ACCORDANCE WITH GOOD TRADE PRACTICES AT 1'-2" CENTERS MAXIMUM SPACING.
5. BOLTS, NUTS AND WASHERS SHALL CONFORM TO SECTION 9-06.5(1) AND GALVANIZED IN ACCORDANCE WITH AASHTO M 232.

Bridge Design Engr.	M:\STANDARDS\BP Rails\SNOW FENCE.MAN				
Supervisor					
Designed By					
Checked By					
Detailed By					
Bridge Projects Engr.					
Prelim. Plan By					
Architect/Specialist	DATE	REVISION	BY	APP'D	

REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
10	WASH.			
JOB NUMBER				

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

STANDARD RAILINGS

BRIDGE RAILING TYPE CHAIN LINK SNOW FENCE

BRIDGE SHEET NO.

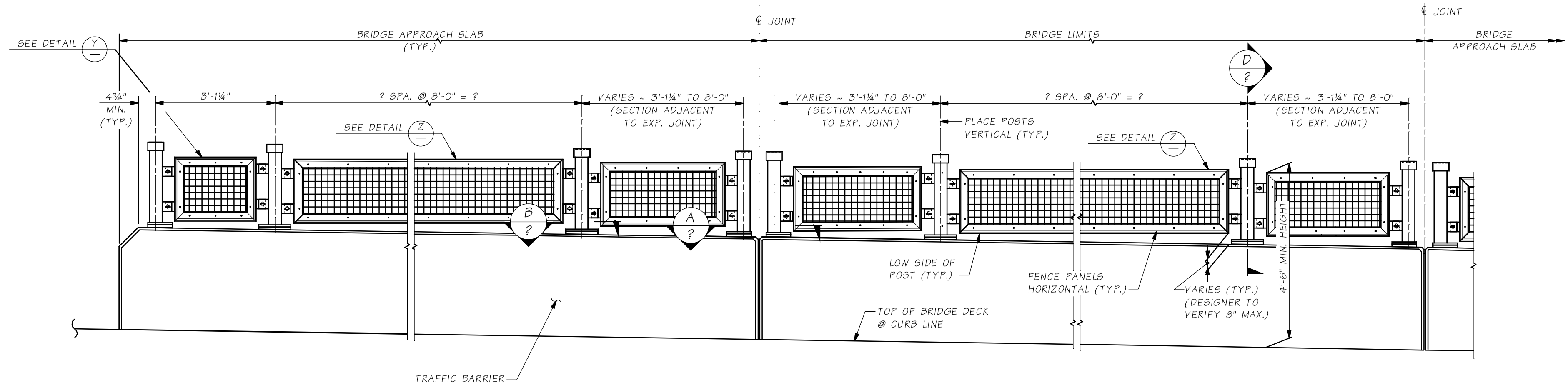
SHEET OF SHEETS

Thu Sep 02 07:40:04 2010

SHEET 1

JOB NO.

SR



TRAFFIC BARRIER OUTSIDE ELEVATION

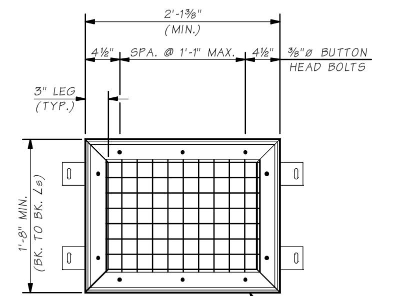
ARCHITECTURAL FINISH OMITTED FOR CLARITY.

NOTES:

1. SLOTS ON FENCE PANEL ARE VERTICAL. SLOTS ON POST ARE HORIZONTAL. BUTTON HEAD BOLTS TO BE INSTALLED IN THE CENTER OF EACH SLOT.
2. POST SPACING AND ADJUSTMENT IN FENCE PANELS SHALL BE SUCH THAT AN 8"ø SPHERE WILL NOT PASS THROUGH PANEL ABOVE 2'-3" FROM FINISHED GRADE, 6"ø SPHERE SHALL NOT PASS THROUGH OTHERWISE.

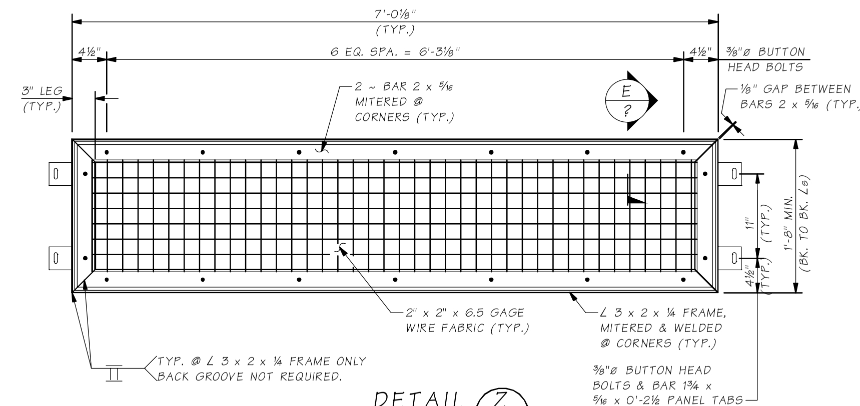
NOTE TO DESIGNER:

1. ADJUST FENCE PANEL HEIGHT AND DIMENSION BETWEEN TABS ON THIS SHEET AND ON SECTION D TO ENSURE THE TOP OF THE SNOW FENCE IS 4'-6" ABOVE THE DECK ELEVATION. ENSURE THE TOP OF THE POST IS ADJUSTED TO PROVIDE A POST HEIGHT PLEASING TO THE EYE.
2. ADJUST DUMMY JOINT SPACING ON TRAFFIC BARRIER SHEETS TO BE CENTERED BETWEEN POSTS.
3. FOR FENCE HEIGHTS GREATER THAN 6'-0" THE DESIGNER SHALL RE-EVALUATE ALL STRUCTURAL STEEL COMPONENTS AND POST ANCHORAGES.



DETAIL Y

VIEW FROM OUTSIDE ELEVATION FOR DETAILS NOT SHOWN SEE DETAIL Z



DETAIL Z

VIEW FROM OUTSIDE ELEVATION

Bridge Design Engr.	Jugesh Kapur	M:\STANDARDS\BP Rails\BRIDGE RAILING SNOW FENCE 1.MAN	
Supervisor	Sup		
Designed By	Designer	Date	
Checked By	Checker	Date	
Detailed By	Detailer	Date	
Bridge Projects Engr.			
Prelim. Plan By			
Architect/Specialist		DATE	REVISION / BY / APP'D

REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
10	WASH.			
JOB NUMBER				

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

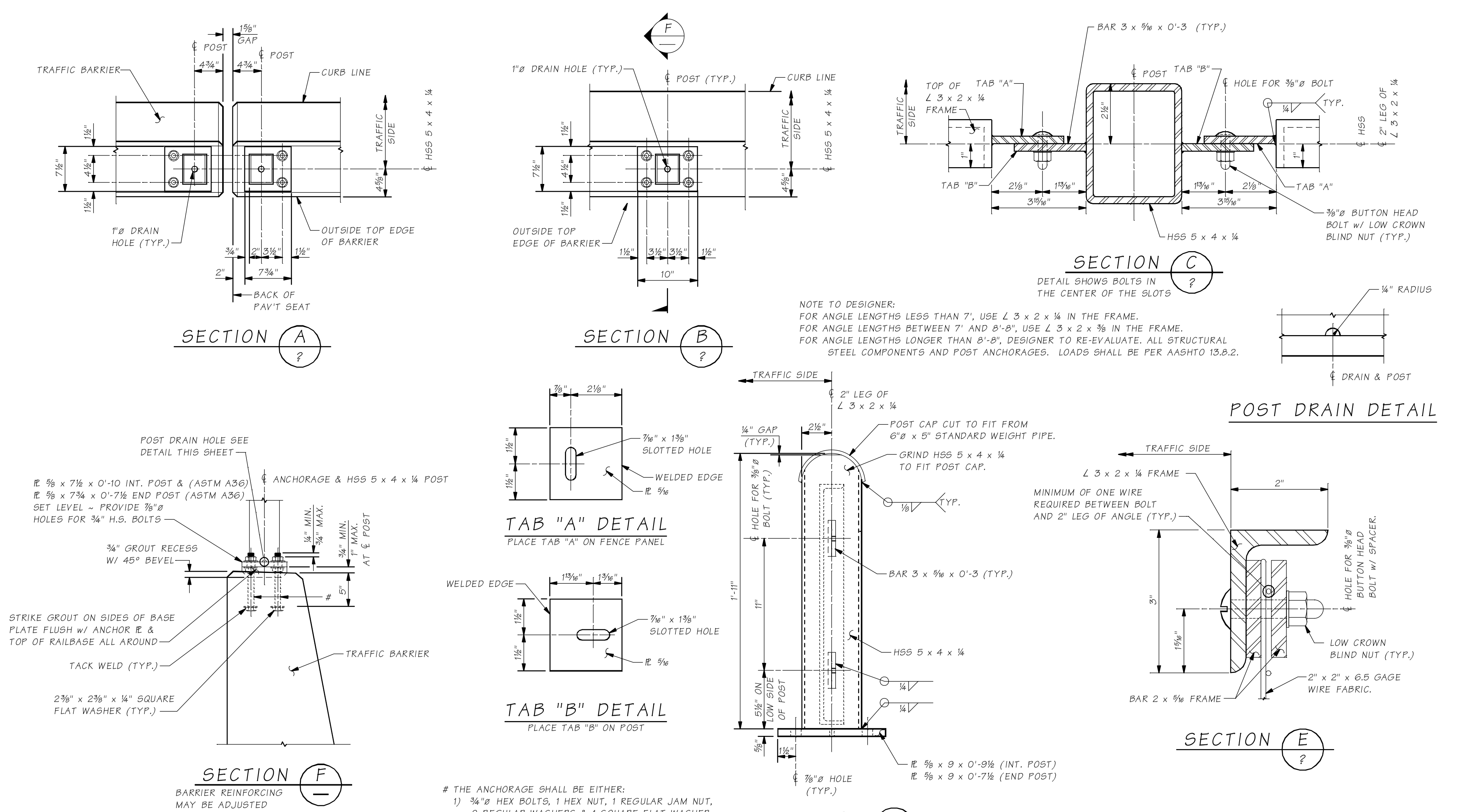
STANDARD RAILINGS

BRIDGE RAILING TYPE SNOW FENCE DETAILS 1 OF 2

BRIDGE SHEET NO. / SHEET / OF / SHEETS

Mon Jun 18 10:36:29 2012

SR 5 FILE NO. Job SHEET 1



NOTE TO DESIGNER:
FOR ANGLE LENGTHS LESS THAN 7', USE ∠ 3 x 2 x ¼ IN THE FRAME.
FOR ANGLE LENGTHS BETWEEN 7' AND 8'-8", USE ∠ 3 x 2 x ⅜ IN THE FRAME.
FOR ANGLE LENGTHS LONGER THAN 8'-8", DESIGNER TO RE-EVALUATE. ALL STRUCTURAL STEEL COMPONENTS AND POST ANCHORAGES. LOADS SHALL BE PER AASHTO 13.8.2.

THE ANCHORAGE SHALL BE EITHER:
1) ¾"ø HEX BOLTS, 1 HEX NUT, 1 REGULAR JAM NUT, 2 REGULAR WASHERS & 1 SQUARE FLAT WASHER (2⅜ x 2⅜ x ¼) REQUIRED PER BOLT,
2) ¾"ø RESIN BONDED ANCHORS. USE MANUFACTURER'S RECOMMENDED EMBEDMENT DEPTH FOR RESIN BONDED ANCHORS. RESIN BONDED ANCHORS REQUIRE JAMB NUT AND AN ADDITIONAL WASHER.

NOTE:

BUTTON HEAD BOLTS SHALL INCLUDE A SLOT IN THE DOMED BOLT HEAD. THE THREADS SHALL BE FULLY COATED WITH THREAD LOCKING AGENT JUST PRIOR TO CONNECTING THE RAILING PANELS TO THE POSTS. THE BUTTON HEAD BOLTS SHALL BE TIGHTENED IN ACCORDANCE WITH SECTION 6-06.3(2) AS SUPPLIMENTED IN THE SPECIAL PROVISIONS.

Bridge Design Engr.		
Supervisor	Sup	
Designed By	Designer	Date
Checked By	Checker	Date
Detailed By	Detailer	Date
Bridge Projects Engr.		
Prelim. Plan By		
Architect/Specialist		

M:\STANDARDS\BP Rails\BRIDGE RAILING SNOW FENCE 2.MAN

DATE	REVISION	BY	APP'D

REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
10	WASH.			

JOB NUMBER

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

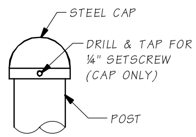
STANDARD RAILINGS

BRIDGE RAILING TYPE SNOW FENCE DETAILS 2 OF 2

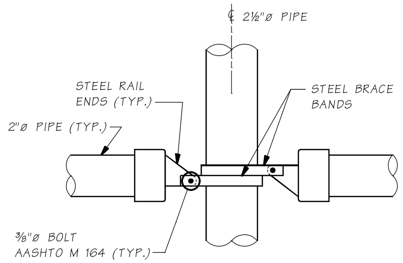
BRIDGE SHEET NO.
SHEET OF SHEETS

SR FILE NO. Job SHEET 1

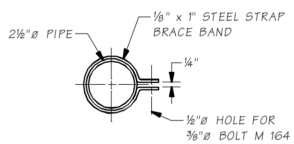
Mon Jun 18 10:36:24 2012



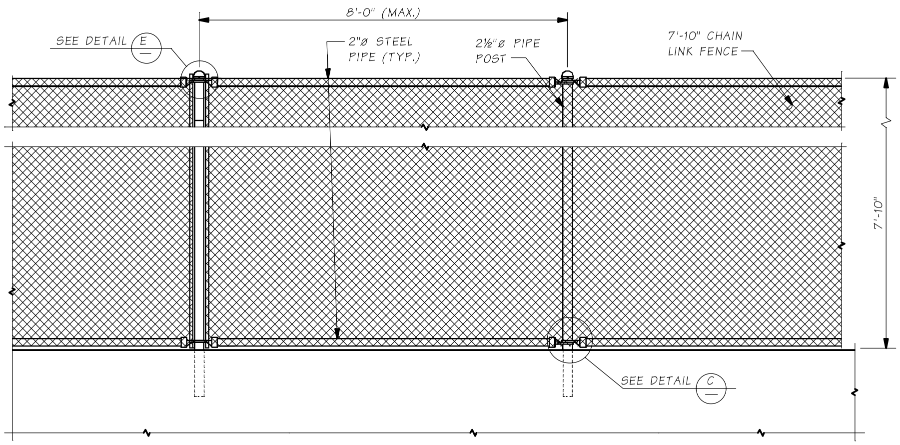
DETAIL E



DETAIL C

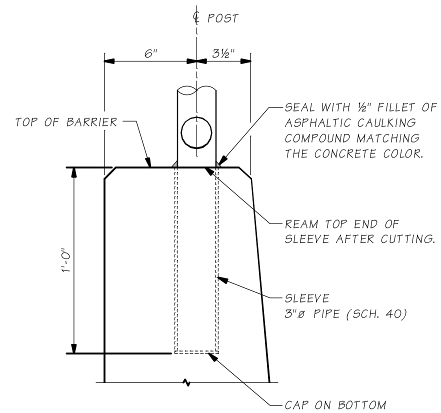


BRACE BAND



ELEVATION

Please coordinate approval with the Bridge Architect before providing designs with this fence type.



DETAIL B

NOTE: PLUG SLEEVE TO PREVENT INFILTRATION DURING CASTING OF CONRETE.

PROTECTIVE SCREENING NOTES

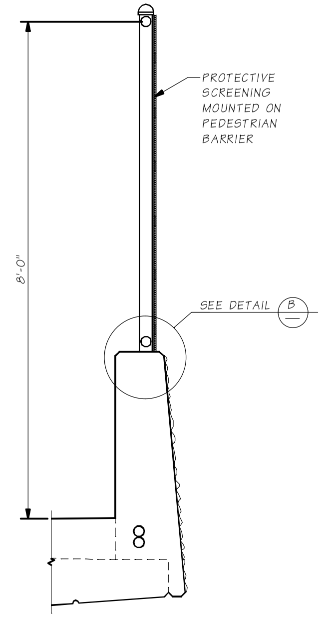
ALL ELEMENTS OF FENCE SHALL BE HOT DIPPED GALVANIZED AFTER FABRICATION.

STEEL PIPES FOR POSTS AND LONGITUDINAL MEMBERS SHALL CONFORM TO ASTM SPECIFICATION A53 GRADE B GALV. PER AASHTO M 111.

ALL HARDWARE SHALL CONFORM TO AASHTO SPECIFICATION M 183 GALV. PER AASHTO M 111 & M232 UNLESS NOTED OTHERWISE.

FABRIC SHALL BE HEAVY DUTY ALUMINUM OF #9 GAGE WIRE WOVEN IN A 50 CHAIN LINK DIAMOND MESH.

FABRIC TIES SHALL BE INSTALLED TO ALL FRAMES IN ACCORDANCE WITH GOOD TRADE PRACTICES AT 355 CENTERS MAXIMUM SPACING.



TYPICAL SECTION
PROTECTIVE SCREEN

Bridge Design Engr.	M:\STANDARDS\BP Rails\BRIDGE RAILING CHAIN LINK FENCE.MAN							
Supervisor				REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
Designed By				10	WASH.			
Checked By								
Detailed By				JOB NUMBER				
Bridge Projects Engr.								
Prelim. Plan By								
Architect/Specialist	DATE	REVISION	BY / APP'D					

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

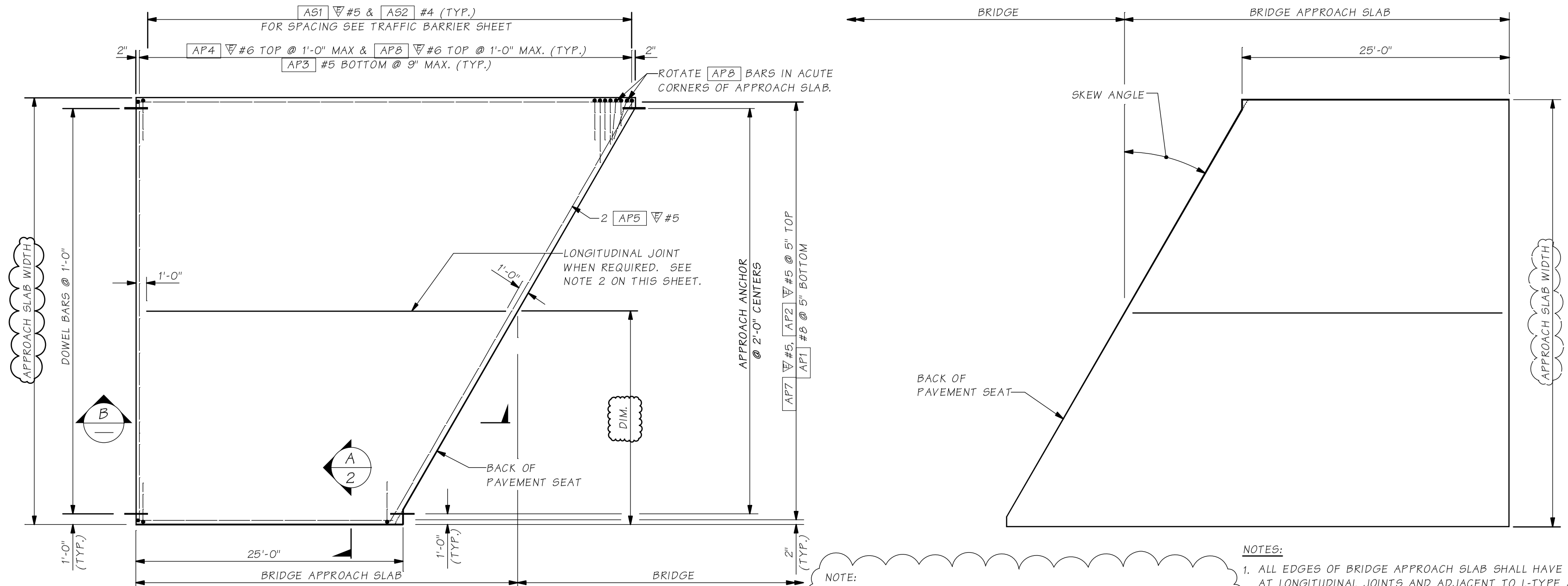
STANDARD RAILINGS

BRIDGE RAILING
TYPE CHAIN LINK FENCE

BRIDGE SHEET NO. | SHEET OF SHEETS

Tue Jul 10 11:50:09 2012

SR | JOB NO. | SHEET



NOTE:
(A) Bridge approach slabs less than 40' wide - no joint required.
(B) Bridge approach slabs wider than 40' - one or more joints are required to divide the slab into approximately 24' wide sections.

NOTE:
Designer to consult with Bridge Design Engineer for skews greater than 30 degrees .

PLAN

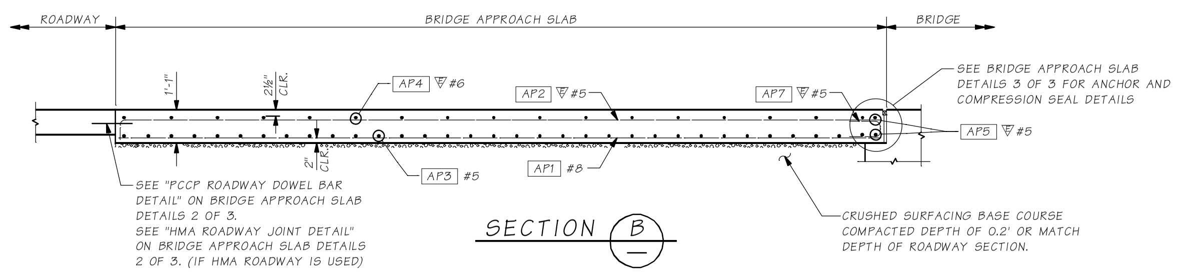
NOTE:
Designer to remove call outs that are not applicable in note 3.

NOTES:

1. ALL EDGES OF BRIDGE APPROACH SLAB SHALL HAVE ½" RADIUS EXCEPT AT LONGITUDINAL JOINTS AND ADJACENT TO L-TYPE ABUTMENTS.
2. LONGITUDINAL JOINTS SHALL BE PLACED ON LANE LINES AND SHALL BE CONSTRUCTED AND SEALED IN ACCORDANCE WITH STD. SPEC. SECTION 5-05.3(8). JOINTS MAY BE EITHER A SAWCUT CRACK CONTROL JOINT OR A CONSTRUCTION JOINT. SAWCUT JOINTS SHALL TERMINATE 1'-0" BEFORE REACHING EDGE OF SLAB AND MUST BE SAW CUT AS SOON AS POSSIBLE AFTER PLACEMENT OF CONCRETE. SEE "LONGITUDINAL JOINT DETAIL" ON BRIDGE APPROACH SLAB DETAILS 2 OF 3.
3. THE MINIMUM LAP SPLICE OF #5 IS 2'-0", ∀#5 IS 2'-6", ∀#6 IS 3'-0", AND #8 IS 3'-3". ALL LAP SPLICES SHALL BE STAGGERED SO THAT NO MORE THAN 50% OF REBAR IS SPLICED AT THE SAME LOCATION. LAP SPLICES SHALL BE LOCATED WITHIN THE MIDDLE HALF OF THE BRIDGE APPROACH SLAB. OPTIONAL SPLICES ARE ALLOWED FOR AP4 ∀#6.

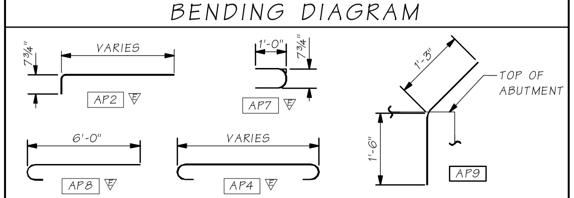
NOTE:
AP9 bars are only applicable in L abutments or approach retrofits. Remove when not used.

NOTE:
Designer to remove AP8, AS1, and AS2 bars when there is no traffic barrier. For 42" barriers designed for 124k impact load (TL-5 Loading) designer shall compute additional reinforcement required.



SECTION B

BENDING DIAGRAM



∀ = EPOXY COATED REINFORCING STEEL NOTE: ALL DIMENSIONS ARE OUT TO OUT

Bridge Design Engr.	M:\STANDARDS\Approach Slabs\Approach Slab 1.MAN					
Supervisor				REGION NO.	STATE	FED. AID PROJ. NO.
Designed By				10	WASH.	
Checked By				JOB NUMBER		
Detailed By						
Bridge Projects Engr.						
Prelim. Plan By						
Architect/Specialist	DATE	REVISION	BY	APP'D		

SHEET NO. | TOTAL SHEETS

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

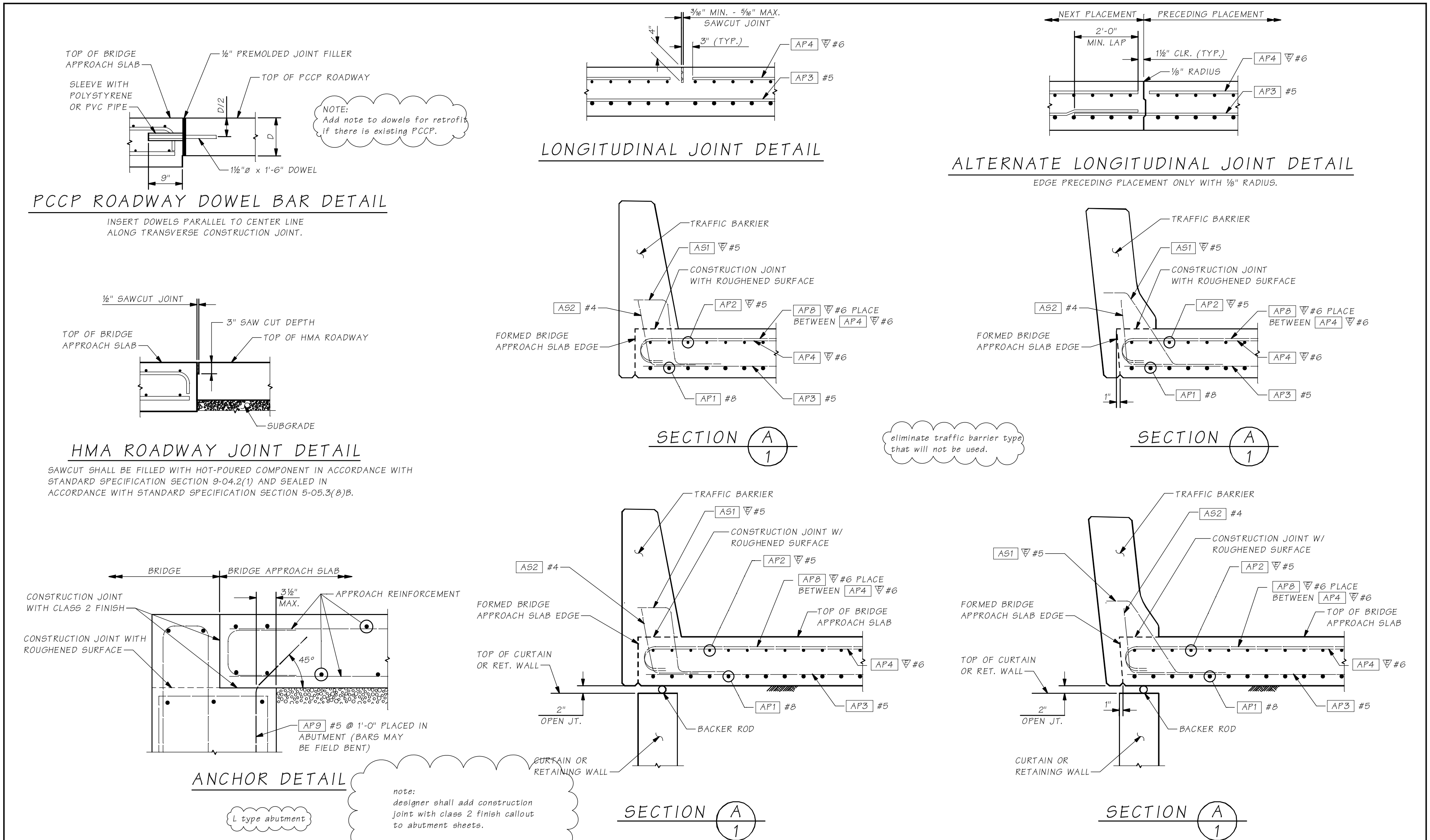
APPROACH SLABS

BRIDGE APPROACH SLAB DETAILS 1 OF 3

BRIDGE SHEET NO.
SHEET OF SHEETS

Mon Jun 18 10:37:57 2012

SR
JOB NO.
SHEET 1



Bridge Design Engr.	M:\STANDARDS\Approach Slabs\Approach Slab 2.man					
Supervisor						
Designed By						
Checked By						
Detailed By						
Bridge Projects Engr.						
Prelim. Plan By						
Architect/Specialist	DATE	REVISION	BY	APP'D		

REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
10	WASH.			
JOB NUMBER				

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

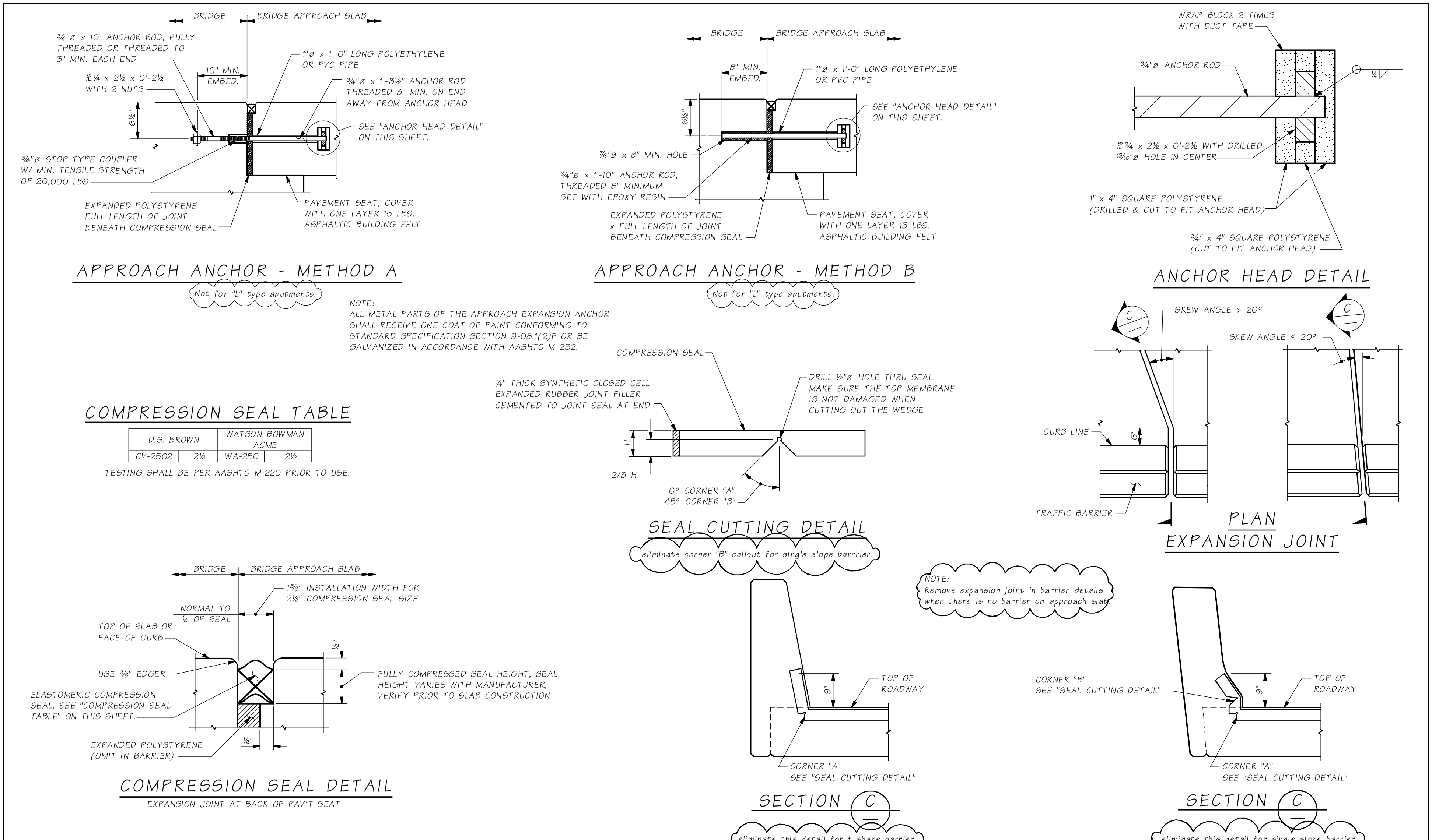
APPROACH SLABS

BRIDGE APPROACH SLAB DETAILS 2 OF 3

BRIDGE SHEET NO.
SHEET OF SHEETS

Mon Jun 18 10:38:04 2012

SR ____ JOB NO. ____ SHEET ____



NOTE:
ALL METAL PARTS OF THE APPROACH EXPANSION ANCHOR SHALL RECEIVE ONE COAT OF PAINT CONFORMING TO STANDARD SPECIFICATION SECTION 9-08.1(2)F OR BE GALVANIZED IN ACCORDANCE WITH AASHTO M 232.

COMPRESSION SEAL TABLE

D.S. BROWN		WATSON BOWMAN ACME	
CV-2502	2½	WA-250	2½

TESTING SHALL BE PER AASHTO M-220 PRIOR TO USE.

Bridge Design Engr.	M:\STANDARDS\Approach Slabs\Approach Slab 3.man									
Supervisor					REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS	
Designed By					10	WASH.				
Checked By					JOB NUMBER					
Detailed By										
Bridge Projects Engr.										
Prelim. Plan By										
Architect/Specialist	DATE	REVISION	BY	APP'D						

Mon Jun 18 10:38:10 2012

BRIDGE AND STRUCTURES OFFICE

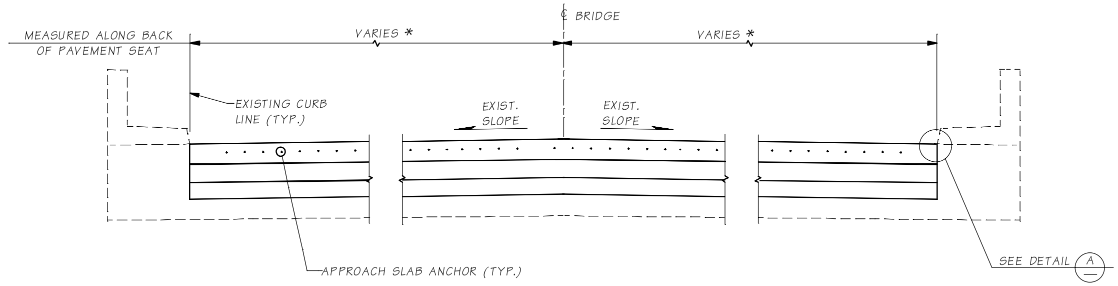
Washington State Department of Transportation

APPROACH SLABS

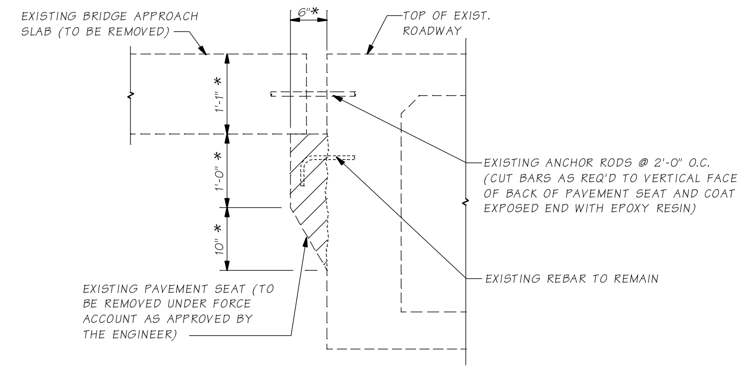
BRIDGE APPROACH SLAB DETAILS 3 OF 3

BRIDGE SHEET NO.
SHEET OF SHEETS

SR
FILE NO.
SHEET



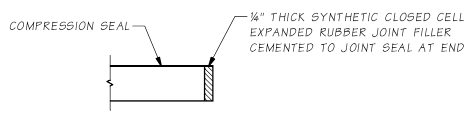
PAVEMANT SEAT END VIEW



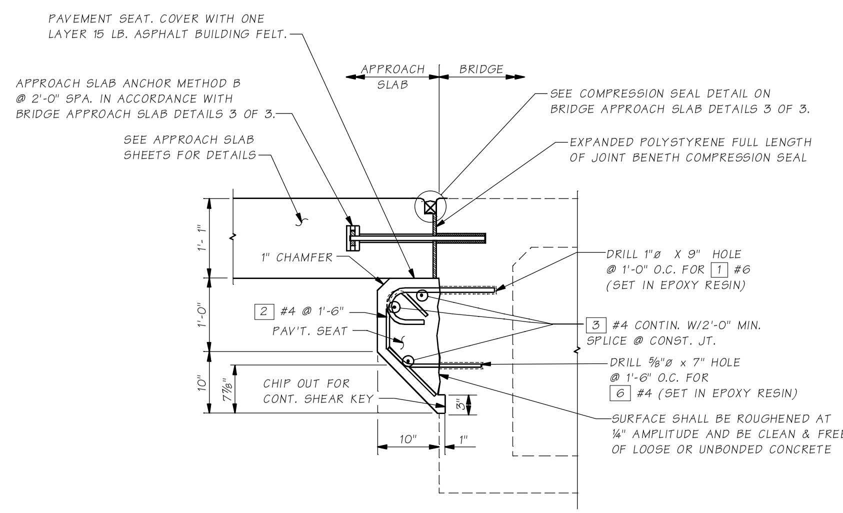
PAVEMENT SEAT

EXISTING CONDITION

* THE DIMENSIONS SHOWN IN THE PLANS ARE BASED ON ORIGINAL CONSTRUCTION RECORDS TOGETHER WITH SURVEY DATA. THESE DIMENSION SHALL BE MEASURED IN THE FIELD BY THE CONTRACTOR PRIOR TO FABRICATION OF ANY COMPONENTS.



DETAIL A

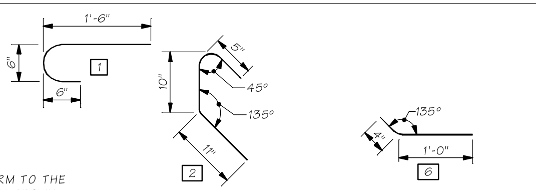


PAVEMENT SEAT

RETROFIT CONDITION

BAR LIST

MARK	SIZE	LENGTH		BENDING DIAGRAM(ALL DIMENSIONS ARE OUT TO OUT TO POINTS OF INTERSECTION)
1	6	2'-4"		
2	4	2'-4'		
3	4	(A)	STR.	
6	4	1'-3"		



(A) DETERMINE FROM PLANS
BEND BARS AS REQ'D TO CONFORM TO THE CONFIGURATION OF THE ROADWAY CROWN.

Bridge Design Engr.	M:\STANDARDS\Approach Slabs\CONCRETE PAVEMENT SEAT REPAIR.man				
Supervisor				REGION NO. 10	STATE WASH.
Designed By				FED. AID PROJ. NO.	
Checked By				JOB NUMBER	
Detailed By				SHEET NO.	TOTAL SHEETS
Bridge Projects Engr.					
Prelim. Plan By					
Architect/Specialist	DATE	REVISION	BY	APP'D	

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

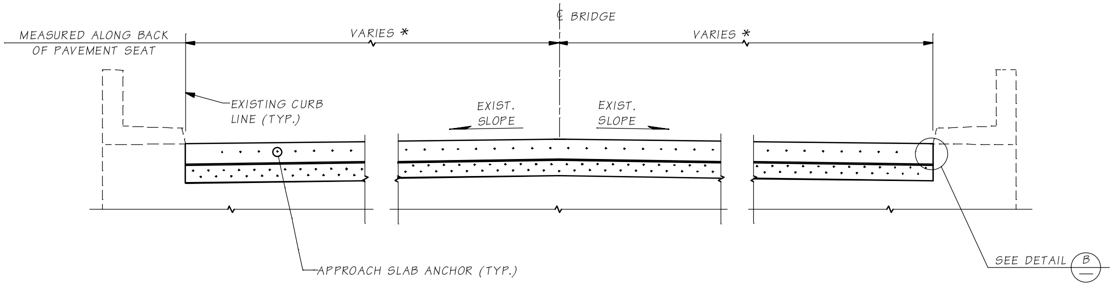
PAVEMENT SEAT REPAIR

PAVEMENT SEAT REPAIR DETAILS

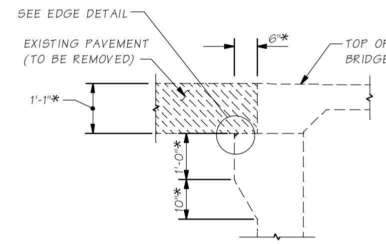
BRIDGE SHEET NO. | SHEET | OF | SHEETS

SR | JOB NO. | 1000 SHEET 3

Tue Jun 19 14:43:23 2012



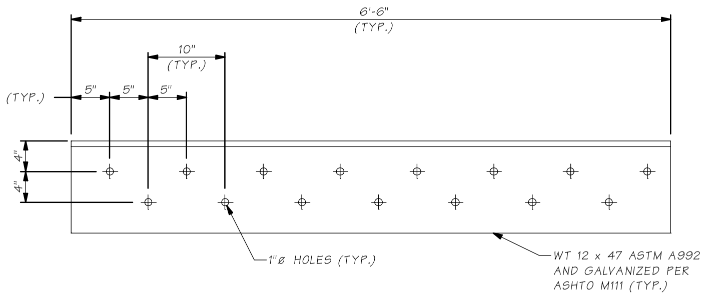
SECTION A



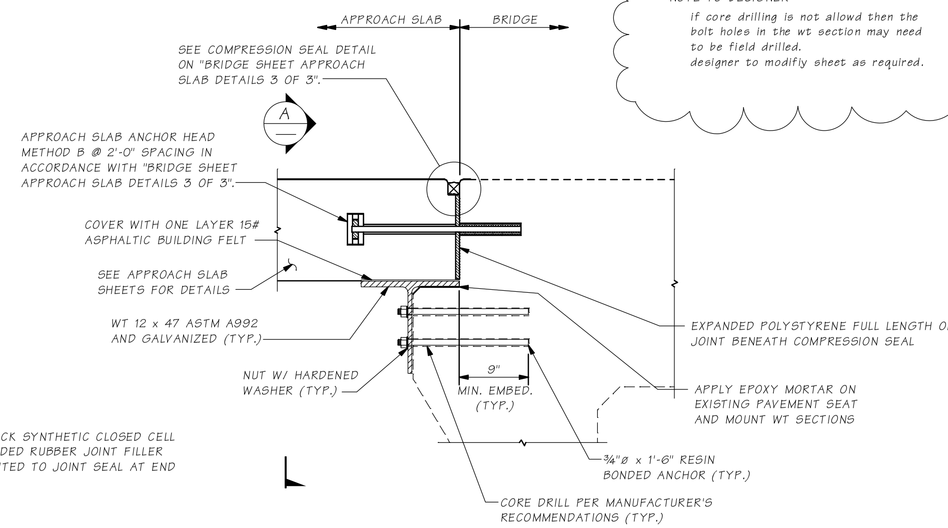
PAVEMENT REMOVAL DETAIL

* THE DIMENSIONS SHOWN IN THE PLANS ARE BASED ON ORIGINAL CONSTRUCTION RECORDS TOGETHER WITH SURVEY DATA. THESE DIMENSION SHALL BE MEASURED IN THE FIELD BY THE CONTRACTOR PRIOR TO FABRICATION OF ANY COMPONENTS.

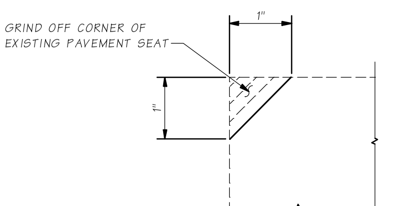
NOTE TO DESIGNER
if core drilling is not allowd then the bolt holes in the wt section may need to be field drilled.
designer to modify sheet as required.



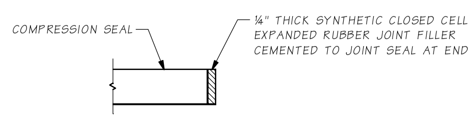
TYPICAL WT 12 SECTION DETAIL



PAVEMENT SEAT REPLACEMENT



EDGE DETAIL



DETAIL B

NOTE:
REPAIR EXISTING PAVEMENT SEAT CONCRETE PRIOR TO INSTALLING WT SECTIONS.

Bridge Design Engr.
Supervisor
Designed By
Checked By
Detailed By
Bridge Projects Engr.
Prelim. Plan By
Architect/Specialist

M:\STANDARDS\Approach Slabs\T-SECTION PAVEMENT SEAT REPAIR.MAN

DATE	REVISION	BY	APP'D

REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
10	WASH.			

JOB NUMBER

BRIDGE
AND
STRUCTURES
OFFICE

Washington State
Department of Transportation

PAVEMENT SEAT REPAIR

PAVEMENT SEAT REPAIR DETAILS

BRIDGE SHEET NO.
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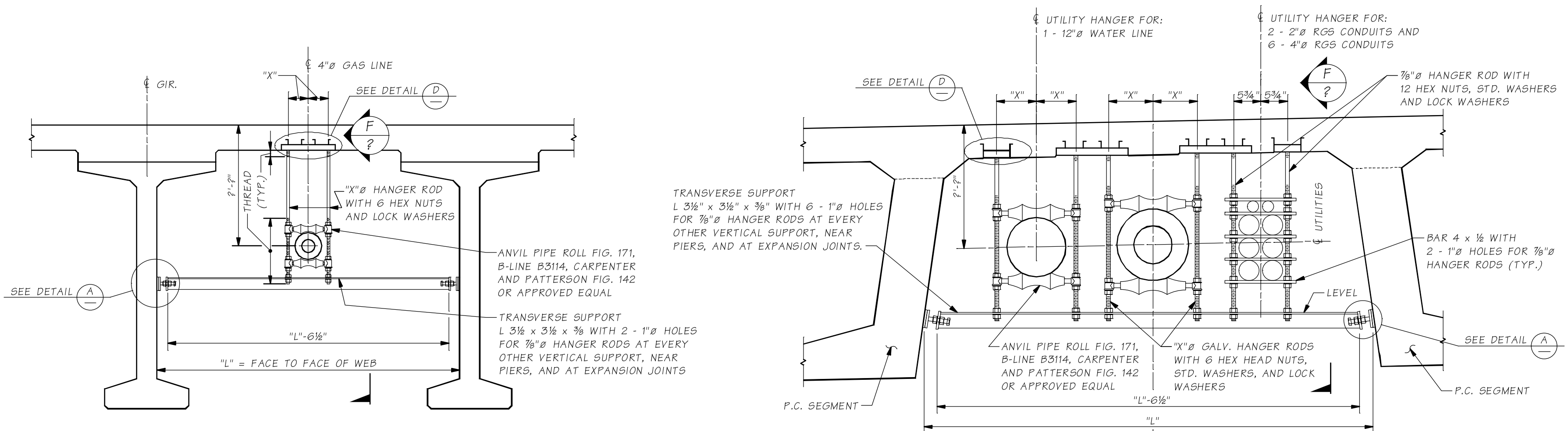
OF
SHEETS

Tue Jun 19 14:39:52 2012

SR
JOB NO. 1000 SHEET 3



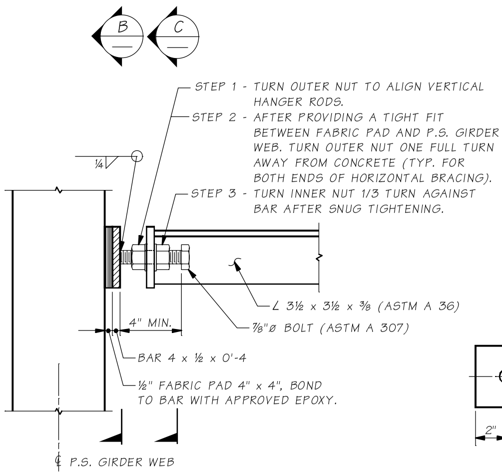
HANGER UTILITY SUPPORT

"X" DETERMINED FROM MANUFACTURER

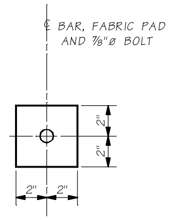
NOTES:

1. ALL MATERIALS SHALL BE GALVANIZED AFTER FABRICATION PER AASHTO M 111 OR AASHTO M 232 EXCEPT PIPE ROLLERS.
2. PAINT ROLLERS WITH THREE COATS OF GALVANIZING REPAIR PAINT. SEE STD. SPEC. 9-08.2.

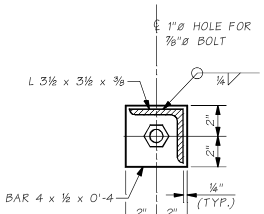
NOTES TO DESIGNERS:
- Verify that the insert does not interfere with reinforcement.
- Verify that the load on the insert and rod is acceptable.
- See BDM chapter 10, section 10.8.6 for insert design.
- "DETAIL A" is provided for prestressed girders only. For steel girders use angle iron bolted connections.



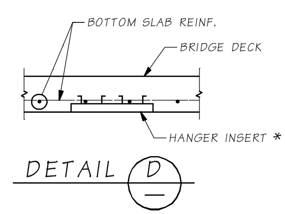
DETAIL A



SECTION B

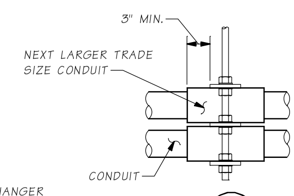


SECTION C

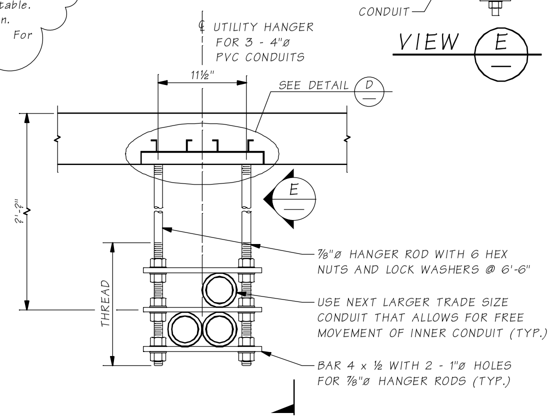


DETAIL D

* COOPER B-LINE B22-I-??, POWERSTRUT PS 349-??, UNISTRUT P32??, OR APPROVED EQUAL (TYP.) WITH SPRING NUT. CHASE THREADS ON HANGER ROD FOR THREAD COMPATIBILITY WITH SPRING NUT. INSERT TO BE INSTALLED LEVEL LONGITUDINALLY AND TRANSVERSELY. PLACE INSERT TO PROVIDE FOR TRANSVERSE ADJUSTMENT OF HANGER RODS. HANGER RODS SHALL NOT BE WITHIN 2" OF THE END OF THE INSERT. TWO INSERTS MAY BE USED TO ACCOMMODATE SUPERELEVATION.



VIEW E



PVC CONDUIT SUPPORT DETAIL

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Detailed By								
Bridge Projects Engr.								
Prelim. Plan By								
Architect/Specialist	DATE	REVISION	BY APP'D					

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

UTILITY INSTALLATION GUIDELINE DETAILS FOR EXISTING BRIDGES

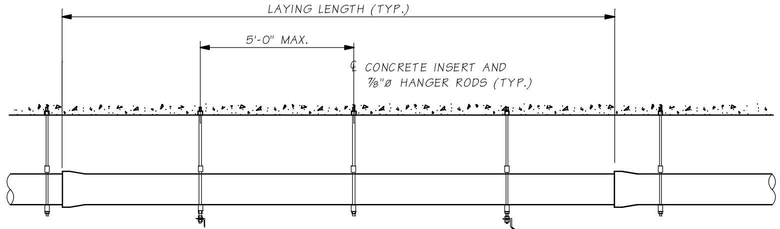
UTILITY HANGER DETAILS

BRIDGE SHEET NO.

SHEET OF SHEETS

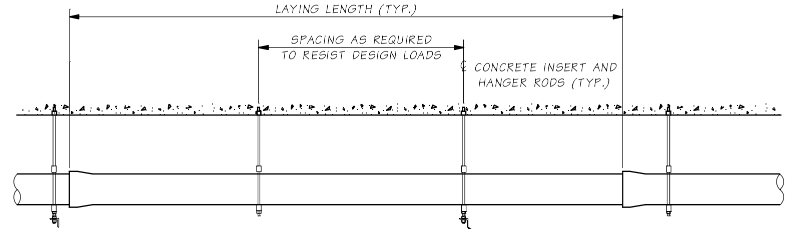
SR JOB NO. SHEET 1

Tue Jun 19 09:01:07 2012



SECTION F/?

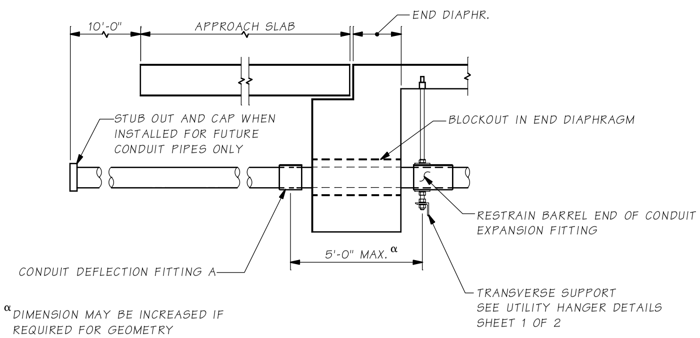
RGS OR PVC CONDUIT



SECTION F/?

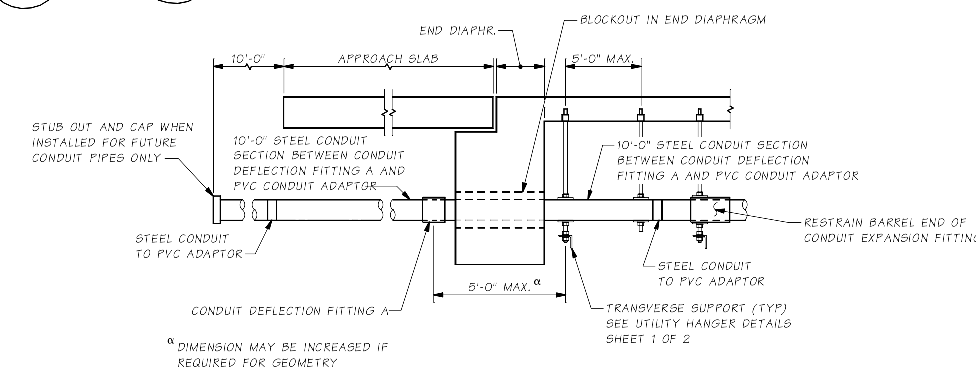
UTILITIES OTHER THAN RGS OR PVC CONDUITS

NOTE TO DESIGNER:
COORDINATE WITH UTILITY ENGINEER FOR CONDUIT PLACEMENT DETAILS AT ABUTMENTS FOR UTILITIES OTHER THAN RGS OR PVC.



UTILITY CONDUIT PLACEMENT DETAIL ~ RGS

SEE BARRIER SHEETS FOR CONDUIT DEFLECTION FITTING A DETAIL.



UTILITY CONDUIT PLACEMENT DETAIL ~ PVC

NOTES:

1. SET POSITION OF EXPANSION FITTING BASED ON MANUFACTURER RECOMMENDATIONS AND TEMPERATURE AT TIME OF INSTALLATION.
2. EXPANSION FITTINGS SHALL BE INSTALLED EVERY 100'-0" MAX, AND SHALL ACCOMMODATE 5.1 INCHES OF MOVEMENT. THE DESIGN TEMPERATURE RANGE IS 125 DEGREES (-15° TO 110°).
3. SEE BARRIER SHEETS FOR CONDUIT DEFLECTION FITTING A DETAIL.

Bridge Design Engr.	M:\STANDARDS\Utilities\Utility Hanger Details 2.MAN			
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Prelim. Plan By				
Architect/Specialist	DATE / REVISION / BY / APP'D			

SHEET NO. / TOTAL SHEETS

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

UTILITY INSTALLATION GUIDELINE DETAILS FOR EXISTING BRIDGES

UTILITY HANGER DETAILS

BRIDGE SHEET NO. / SHEET / OF / SHEETS

SR / JOB NO. / SHEET 1

Tue Jun 19 09:01:00 2012

Chapter 13 Bridge Load Rating — Contents

Chapter 13 Bridge Load Rating

13.1 General

Bridge load rating is a procedure to evaluate the adequacy of various structural components to carry predetermined live loads. The Bridge Load Rating Engineer in the WSDOT Bridge Preservation Office is responsible for the bridge inventory and load rating of existing and new bridges in accordance with the NBIS and the AASHTO Manual for Bridge Evaluation (MBE), latest edition. Currently, only elements of the superstructure will be rated, however, if conditions warrant, substructure elements can be rated. The superstructure shall be defined as all structural elements above the column tops including drop crossbeams.

Load ratings are required for all new, widened, or rehabilitated bridges where the rehabilitation alters the load carrying capacity of the structure. Load ratings shall be done immediately after the design is completed and rating calculations shall be filed separately per Section 13.4 and files shall be forwarded to WSDOT's Load Rating Engineer.

The Bridge Preservation Office is responsible for maintaining an updated bridge load rating throughout the life of the bridge based on the current condition of the bridge. Conditions of existing bridges change over time, resulting in the need for reevaluation of the load rating. Such changes may be caused by damage to structural elements, extensive maintenance or rehabilitative work, or any other deterioration identified by the Bridge Preservation Office through their regular inspection program.

New bridges that have designs completed after October 1, 2010 shall be rated based on the Load and Resistance Factor Rating (LRFR) method per the MBE and this chapter. NBI ratings shall be based on the HL-93 truck and shall be reported as a rating factor. For new bridges designed prior to October 1, 2010, partially reconstructed or rehabilitated bridges where part of the existing structure is designed by the allowable stress method or by the load factor method (LFR), and existing structures, NBI ratings can be based on either the LFR or LRFR methods. The rating factors shall be based on HS loading and reported in tons when using the LFR method. Verify with WSDOT's Load Rating Engineer regarding which load rating method to use for existing bridges and new bridges designed prior to October 1, 2010.

By definition, the adequacy or inadequacy of a structural element to carry a specified truck load will be indicated by the value of its rating factor (RF); that is, whether it is greater or smaller than 1.0.

13.1.1 LRFR Method per the MBE

Rating Equation

$$RF = \frac{(C - \gamma_{DC}\, DC - \gamma_{DW}\, DW \pm \gamma_p\, P)}{\gamma_{LL}\, LL\, (1+IM)} \qquad (13.1.1A\text{-}1)$$

Where:

RF = Rating factor
C = $\phi c\ \phi s\ \phi n\ R_n$, where $\phi c\ \phi s \geq 0.85$ for strength limit state
C = f_R for service limit state
R_n = Nominal Capacity of member
f_R = Allowable Stress per LRFD specs
DC = Dead load due to structural components and attachments
DW = Dead load due to wearing surface and utilities
P = Permanent loads other than dead loads
LL = Live load effect
IM = Dynamic load allowance (Impact)
γ_{DC} = Dead load factor for structural components and attachments
γ_{DW} = Dead load factor for wearing surface and utilities
γ_P = Load factor for permanent load
γ_{LL} = Live load factor
ϕ_c = Condition factor
ϕ_s = System factor
ϕ_n = Resistance factor based on construction material

When rating the full section of a bridge, like a box girder or 3D truss, or crossbeams, with two or more lanes, the following formula applies when rating overload trucks.

$$RF = \frac{C - \gamma_{DC}\, DC - \gamma_{DW}\, DW \pm \gamma_p\, P - \gamma_{LL}\, LL_{lgl}(1+IM)}{\gamma_{LL}\, LL\, (1+IM)} \qquad (13.1.1A\text{-}2)$$

The formula above assumes that there is one overload truck occupying one lane, and one of the legal trucks occupying each of the remaining lanes. Trucks shall be placed in the lanes in a manner that produces the maximum forces. The live load factor for both of the legal truck and permit truck shall be equal and are dependent on the permit truck. The LL_{lgl} shown in the equation above corresponds to the maximum effect of the legal truck(s).

Condition Factor (ϕc_c)

Condition factor is based on the BMS condition state of the element per the latest inspection report.

Structural Condition of Member	ϕ_c
Good or Satisfactory, BMS Condition 1 or 2	1.00
Fair, BMS Condition 3	0.90
Poor, BMS Condition 4	0.85

Dead and Live Load Factors

Dead load factor = 1.30
Live load factor = 2.17 (Inventory)
= 1.30 (Operating)

Impact (IM)

Truck	IM	NBI Element 681	BMS Flag 322
Design and Legal loads (Inventory & Operating)	Span dependant	N/A	N/A
Permit Loads:			
Smooth Riding Surface Along Approach onto the Bridge, Bridge Deck and Expansion Joints	10%	8	1, 2, or none
Minor Surface Deviations and Depressions	20%	6	3
Severe Impact to the Bridge	30%	3	4

If the inspection report has no NBI Code 681 or BMS Flag 322, then assume smooth approaches.

Impact (*IM*) for design and legal loads is span dependent:

$$IM = \frac{50}{(125+L)} \quad (13.1.2\text{-}2)$$

Where:
L is equal to span length

When rating the full section of a bridge, like a box girder or 3D truss, or crossbeams, which have two or more lanes, the following formula applies when rating overload trucks.

$$RF = \frac{C-\gamma_{DL}\, D \pm S - \gamma_{LL}\, LL_{lgl}(1+IM)}{\gamma_{LL}\, LL(1+IM)} \quad (13.1.2\text{-}3)$$

The formula above assumes that there is one overload truck occupying one lane, and one of the legal trucks occupying each of the remaining lanes. Trucks shall be placed in the lanes in a manner that produces the maximum forces. The LL_{lgl} shown in the equation above corresponds to the maximum effect of the legal trucks(s). The γ_{LL} corresponds to the live load factor for the overload truck and is the same for both legal and overload trucks.

Resistance Factors (LFR)

The resistance factors for NBI ratings shall be per the latest AASHTO Standard Specifications. Following are the NBI resistance factors assuming the member is in good condition:

Steel members:	1.00 (Flexure) 1.00 (Shear)
Prestressed concrete	1.00 (Flexure, positive moment) 0.90 (Shear)
Post-tensioned, cast-in-place:	0.95 (Flexure, positive moment) 0.90 (Shear)
Reinforced concrete:	0.90 (Flexure) 0.85 (Shear)

For prestressed and post-tensioned members, where reinforcing steel is used to resist negative moment, the resistance factors for reinforced concrete section shall be used in the ratings.

In cases where there is deterioration in a member, the cross section shall be reduced based on the inspection report. For cases where deterioration in members is described in general terms, reduce resistance factors of member by 0.10 for BMS Condition State of 3, and reduce resistance factors by 0.20 for BMS Condition State of 4.

Service Method (LFR)

Prestressed and post-tensioned members in positive moment regions, and where post-tensioning is continuous over the supports, shall also be rated based on allowable stresses at service loads. The lowest rating factor between service and ultimate methods shall be the governing inventory rating.

Inventory Rating

Concrete Tension:

$$RF = \frac{6\sqrt{f'_c} - (F_d + F_p + F_s)}{F_l\,(1+IM)} \qquad (13.1.2\text{-}4)$$

Concrete Compression:

$$RF = \frac{0.60 f'_c - (F_d + F_p + F_s)}{F_l\,(1+IM)} \qquad (13.1.2\text{-}5)$$

$$RF = \frac{0.40 f'_c - 1/2\,(F_d + F_p + F_s)}{F_l\,(1+IM)} \qquad (13.1.2\text{-}6)$$

Prestressing Steel Tension:

$$RF = \frac{0.80 f_y^* - (F_d + F_p + F_s)}{F_l\,(1+IM)} \qquad (13.1.2\text{-}7)$$

Operating Rating

Prestressing Steel Tension:

$$RF = \frac{0.90 f_y^* - (F_d + F_p + F_s)}{F_l\,(1+IM)} \qquad (13.1.2\text{-}8)$$

Where:

RF = Rating factor
f'_c = Compressive strength of concrete
F_d = Dead load stress
F_p = Prestressing stress
F_s = Stress due to secondary prestress forces
F_l = Live load stress
IM = Dynamic load allowance (Impact)
f_y* = Prestressing steel yield stress

Allowable concrete stress shall be increased by 15 percent for overload vehicles. Impact is calculated same as ultimate method.

13.1.3 Allowable Stress Method (ASD)

The allowable stress method is applicable to timber structures. Impact is not applied to timber structures.

Rating Equation:

$$RF = \frac{(F_a + F_d)}{F_l} \qquad (13.1.3\text{-}1)$$

Where:

RF = Rating factor
*F_a = Allowable stress
F_d = Dead load stress
F_l = Live load stress

*F_a, for inventory rating, shall be per AASHTO Standard Specifications. For operating rating, F_A shall be increased by 33%

13.4 Load Rating Reports

Rating reports shall be organized in such a manner that it is easy to follow and all assumptions are clearly stated. For complex large structures, include a table of contents and number the pages in the report.

The report shall consist of:

1. A Bridge Rating Summary sheet, as shown on Appendix 13.4-A1 (LFR) and 13.4-A2 (LRFR) reflecting the lowest rating factor. The summary sheet shall be stamped, signed and dated by a professional engineer licensed in the state of Washington.
2. A brief report of any anomalies in the ratings and an explanation of the cause of any rating factor below 1.0.
3. Hard copy of computer output files (RPT files) used for rating, and any other calculations or special analysis required.
4. A complete set of plans for the bridge (applies to new designed bridges).
5. One compact disk which contains the final versions of all input and output files, and other calculations created in performing the load rating.
6. A minimum of 30 days is required for the Bridge Preservation Office review of any load rating submitted as part of a Design Build Contract.

All reports shall be bound in Accopress-type binders.

When the load rating calculations are produced as part of a design project (new, widening, or rehabilitation,) the load rating report and design calculations shall be bound separately.