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Remarks and Instructions

The complete manual, revision packages, and individual chapters can be accessed at www.wsdot.wa.gov/publications/manuals/m23-50.htm.

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

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Approved By

Signature

Washington State Department of Transportation

Bridge Design Manual (LRFD)

M 23-50.13

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Engineering and Regional Operations
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

Tom Baker, P.E.
Bridge and Structures Engineer

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1.2 Bridge and Structures Office Organization

1.2.1 General

The responsibilities of the Bridge and Structures Office are:

Provides structural engineering services for WSDOT. Provides technical advice and assistance to other governmental agencies on such matters.

The WSDOT *Design Manual* M 22-01 states the following:

Bridge design is the responsibility of the Bridge and Structures Office in Olympia. Any design authorized at the Region level is subject to review and approval by the Bridge and Structures Office.

1.2.2 Organizational Elements of the Bridge Office

A. **Bridge and Structures Engineer –** The Bridge and Structures Engineer is responsible for structural engineering services for the department and manages staff and programs for structural design, contract plan preparation, inspections and assessments of existing bridges.

B. **Bridge Design Engineer –** The Bridge Design Engineer is directly responsible to the Bridge and Structures Engineer for structural design and review, and advises other divisions and agencies on such matters.

   1. **Structural Design Units –** The Structural Design Units are responsible for the final design of bridges and other structures. Final design includes preparation of contract plans. The units provide special design studies, develop design criteria, check shop plans, and review designs submitted by consultants. Frequently, the Bridge Projects Engineer assigns the units the responsibility for preparing preliminary bridge plans and other unscheduled work.

      The Bridge Engineer Supervisor (Unit Supervisor) provides day-to-day leadership, project workforce planning, mentoring, and supervision for the design unit. Organization and job assignments within the unit are flexible and depend on projects underway at any particular time as well as the qualifications and experience level of individuals. The primary objective of the design units is to produce contract plans for bridges and structures within scope, schedule and budget. This involves designing, checking, reviewing, and detailing in an efficient and timely manner.

      Each specialist has a particular area of expertise which includes concrete, steel, seismic design and retrofit, expansion joints and bearings, and floating and movable bridges. The specialists act as a resource for the bridge office in their specialty and are responsible for keeping up-to-date on current AASHTO criteria, new design concepts and products, technical publications, construction and maintenance issues, and are the primary points of contact for industry representatives.

      The design units are also responsible for the design and preparation of contract plans for modifications to bridges in service. These include bridge rail replacement, deck repair, seismic retrofits, emergency repairs when bridges are damaged by vehicle or ship collision or natural phenomenon, and expansion joint and drainage retrofits. They review proposed plans of utility attachments to existing bridges.

   2. **Bridge Projects Unit –** The Bridge Projects Engineer directs preliminary design work, specification and cost estimates preparation, falsework review, project scoping, coordinates scheduling of bridge design projects and unscheduled work assignments with the Region Project Development Engineers, Bridge Design Engineer, and the Unit Supervisors.

      The Preliminary Plan Engineers are responsible for bridge project planning from initial scoping to design type, size, and location (TSL) studies and reports. They are responsible for preliminary plan preparation of bridge and walls including assembly and analysis of site data, preliminary structural analysis, cost analysis, determination of structure type, and drawing preparation. They also check preliminary plans prepared by others, review highway project environmental documents and design reports, and prepare U. S. Coast Guard Permits.

The Specifications and Estimate (S&E) Engineers develop and maintain construction specifications and cost estimates for bridge projects. They also develop specifications and cost estimates for bridge contracts prepared by consultants and other government agencies, which are administered by WSDOT. They assemble and review the completed bridge PS&E before submittal to the Regions. They also coordinate the PS&E preparation with the Regions and maintain bridge construction cost records.

The Construction Support Unit Engineers are responsible for checking the contractor's falsework, shoring, and forming plans. Shop plan review and approval are coordinated with the design units. Actual check of the shop plans is done in the design unit. Field requests for plan changes come through this office for a recommendation as to approval.

The Bridge Plans Engineer processes as-built plans in this unit. Region Project Engineers are responsible for preparing and submitting as-built plans at the completion of a contract.

The Scheduling Engineer monitors the design work schedule for the Bridge and Structures Office, updates the Bridge Design Schedule (BDS) and maintains records of bridge contract costs. Other duties include coordinating progress reports to Regions by the Unit Supervisors and S&E Engineers through the Project Delivery Information System (PDIS).

The Bridge Projects Unit dedicates one position to providing technical assistance for the design and detailing of expansion joint, bridge bearing and barrier/rail projects.

In addition, the unit is responsible for updating the *Bridge Design Manual* M 23-50. The unit coordinates changes to the WSDOT Standard Specifications and facilitates updates or revisions to WSDOT Bridge Office design standards.

3. **Mega Project Bridge Manager –** The Mega Project Bridge Manager provides leadership, guidance and project management responsibilities for various complex, unique and monumental bridge design and construction projects. Mega Bridge Projects are defined as suspension, cable-stayed, movable, segmental or a complex group of interchange/corridor bridges and include conventional and design-build project delivery methods. The Mega Project Bridge Manager represents the Bridge and Structures Office in Cost Estimate Validation Process activities, Value Engineering Studies and Research Projects regarding major bridge projects.

C. **Bridge Preservation Engineer –** The Bridge Preservation Engineer directs activities and develops programs to assure the structural and functional integrity of all state bridges in service. The Bridge Preservation Engineer directs emergency response activities when bridges are damaged.

1. **Bridge Preservation Office (BPO) –** The Bridge Preservation Office is responsible for planning and implementing an inspection program for the more than 3,200 fixed and movable state highway bridges. In addition, BPO provides inspection services on some local agency bridges and on the state's ferry terminals. All inspections are conducted in accordance with the National Bridge Inspection Standards (NBIS).

   BPO maintains the computerized Washington State Bridge Inventory System (WSBIS) of current information on more than 7,300 state, county, and city bridges in accordance with the NBIS. This includes load ratings for all bridges. BPO prepares a *Bridge List* of the state's bridges, which is published every two years, maintains the intranet-based Bridge Engineering Information System (BEIST), and prepares the annual Recommended Bridge Repair List (RBRL) based on the latest inspection reports.

   BPO is responsible for the bridge load rating and risk reduction (SCOUR) programs. It provides damage assessments and emergency response services when bridges are damaged because of vehicle or ship collision or natural phenomenon such as: floods, wind, or earthquakes.

D. **Bridge Management Engineer –** The Bridge Management Unit is responsible for the program development, planning and monitoring of all statewide bridge program activities. These include P2 funded bridge replacements and rehabilitation, bridge deck protection, major bridge repair, and bridge painting.

In addition, the Bridge Management Unit manages the bridge deck protection, deck testing and the bridge research programs. It is responsible for the planning, development, coordination, and implementation of new programs (e.g., Seismic Retrofit and Preventative Maintenance), experimental feature projects, new product evaluation, and technology transfer.

The Bridge Management Engineer is the Bridge and Structures Office's official Public Disclosure contact. (See Section 1.3.9 Public Disclosure Policy Regarding Bridge Plans).

E. **Computer Support Unit –** The Computer Support Unit is responsible for computer resource planning and implementation, computer user support, liaison with Management Information Systems (MIS), computer aided engineer operation support, and software development activities. In addition, the unit works closely with the Bridge Projects Unit in updating this manual and *Standard Plans*.

F. **Consultant Liaison Engineer –** The Consultant Liaison Engineer prepares bridge consultant agreements and coordinates consultant PS&E development activities with those of the Bridge Office. The Consultant Liaison Engineer negotiates bridge design contracts with consultants.

G. **State Bridge and Structures Architect –** The State Bridge and Structures Architect is responsible for reviewing and approving bridge preliminary plans, retaining walls, preparing renderings, coordinating aesthetic activities with Regions (i.e. suggesting corridor themes and approving public art), and other duties to improve the aesthetics of our bridges and structures. The State Bridge and Structures Architect works closely with bridge office and region staff. During the design phase, designers should get the Architect's approval for any changes to architectural details shown on the approved preliminary plan.

H. **Staff Support Unit –** The Staff Support Unit is responsible for many support functions, such as: typing, timekeeping, payroll, receptionist, vehicle management, mail, inventory management, and other duties requested by the Bridge and Structures Engineer. Other duties include: filing field data, plans for bridges under contract or constructed, and design calculations. This unit also maintains office supplies and provides other services.

I. **Office Administrator –** The Office Administrator is responsible for coordinating personnel actions, updating the organizational chart, ordering technical materials, and other duties requested by the Bridge and Structures Engineer. Staff development and training are coordinated through the Office Administrator. The Office Administrator also handles logistical support, office and building maintenance issues.

1.2.3 Design Unit Responsibilities and Expertise

The following is an updated summary of the structural design, review and plan preparation responsibilities/expertise within the Bridge Design Section. Contact the Unit Supervisor for the name of the appropriate staff expert for the needed specialty.

Unit Supervisor	Responsibility/Expertise
Richard Stoddard	Bridge Traffic Barriers and Rail Retrofits Concrete Design Technical Support Seismic Design Technical Support
TBD	Coast Guard Permits Special Provisions and Cost Estimates Preliminary Design Falsework, Forming and Temporary Structures *Bridge Design Manual* M 23-50 Bridge Projects Scheduling
Richard Zeldenrust	Overhead and Bridge-Mounted Sign Structures Light Standard & Traffic Signal Supports Repairs to Damaged Bridges Structural Steel Technical Support
	Emergency Slide Repairs Retaining Walls (including Structural Earth, Soldier Pile and Tie-Back, Geosynthetic, and Soil Nail) Pre-Approval of Retaining Wall Systems Noise Barrier Walls
DeWayne Wilson	Bridge Preservation Program (P2 Funds) – Establish needs and priorities (Seismic, Scour, Deck Overlay, Special Repairs, Painting, Replacement, Misc Structures Programs) Bridge Management System Bridge Engineering Software and CAD Consultant Liaison Bearings and Expansion Joints Floating Bridges Special Structures
Tim Moore	Mega Projects Manager
Paul Kinderman	Bridge Architect

Chapter 2 Preliminary Design — Contents

2.4 Selection of Structure Type

2.4.1 Bridge Types

See Appendix 2.4-A1-1 for a bar graph comparing structure type, span range and cost range.

The required superstructure depth is determined during the preliminary plan development process. The AASHTO LRFD Specifications in Section 2.5.2.6.3 show traditional minimum depths for constant depth superstructures. WSDOT has developed superstructure depth-to-span ratios based on past experience.

The AASHTO LRFD Specifications, Section 2.5.2.6.1, states that it is optional to check deflection criteria, except in a few specific cases. The WSDOT criteria is to check the live load deflection for all structures as specified in AASHTO LRFD Specifications, Section 3.6.1.3.2 and 2.5.2.6.2.

The superstructure depth is used to establish the vertical clearance that is available below the superstructure. For preliminary plans, the designer should use the more conservative depth determined from either the AASHTO LRFD criteria or the WSDOT criteria outlined below. In either case, the minimum depth includes the deck thickness. For both simple and continuous spans, the span length is the horizontal distance between centerlines of bearings.

Refer to Section 2.3.11 for inspection and maintenance access requirements. Superstructure depth may be influenced when inspection lighting and access is required for certain bridge types.

The superstructure depth may be refined during the final design phase. It is assumed that any refinement will result in a reduced superstructure depth so the vertical clearance is not reduced from that shown in the preliminary plan. However, when profile grade limitations restrict superstructure depth, the preliminary plan designer shall investigate and/or work with the structural designer to determine a superstructure type and depth that will fit the requirements.

A. **Reinforced Concrete Slab**

1. **Application –** Used for simple and continuous spans up to 60′.
2. **Characteristics –** Design details and falsework relatively simple. Shortest construction time for any cast-in-place structure. Correction for anticipated falsework settlement must be included in the dead load camber curve because of the single concrete placement sequence.
3. **Depth/Span Ratios**

   a. **Constant Depth**

Simple span	1⁄22
Continuous spans	1⁄25

   b. **Variable Depth –** Adjust ratios to account for change in relative stiffness of positive and negative moment sections.

B. **Reinforced Concrete Tee-Beam**

1. **Application –** This type of Super Structure is not recommended for new bridges. It could only be used for bridge widening and bridges with tight curvature or unusual geometry.

   Used for continuous spans 30′ to 60′. Has been used for longer spans with inclined leg piers.
2. **Characteristics –** Forming and falsework is more complicated than for a concrete slab. Construction time is longer than for a concrete slab.

3. **Depth/Span Ratios**

   a. **Constant Depth**

Simple spans	1⁄13
Continuous spans	1⁄15

   b. **Variable Depth –** Adjust ratios to account for change in relative stiffness of positive and negative moment sections.

C. **Reinforced Concrete Box Girder –** WSDOT restricts the use of cast-in-place reinforced concrete box girder for bridge superstructure. This type of superstructure may only be used for bridges with tight curvatures or irregular geometry upon Bridge Design Engineer approval.

1. **Application –** This type of super structure is not recommended for new bridges. It could only be used for bridge widening and bridges with tight curvature or unusual geometry.

   Used for continuous spans 50′ to 120′. Maximum simple span 100′ to limit excessive dead load deflections.

2. **Characteristics –** Forming and falsework is somewhat complicated. Construction time is approximately the same as for a tee-beam. High torsional resistance makes it desirable for curved alignments.

3. **Depth/Span Ratios***

   a. **Constant Depth**

Simple spans	1⁄18
Continuous spans	1⁄20

   b. **Variable Depth –** Adjust ratios to account for change in relative stiffness of positive and negative moment sections.

   *If the configuration of the exterior web is sloped and curved, a larger depth/span ratio may be necessary.

D. **Post-tensioned Concrete Box Girder**

1. **Application –** Normally used for continuous spans longer than 120′ or simple spans longer than 100′. Should be considered for shorter spans if a shallower structure depth is needed.

2. **Characteristics –** Construction time is somewhat longer due to post-tensioning operations. High torsional resistance makes it desirable for curved alignments.

3. **Depth/Span Ratios***

   a. **Constant Depth**

Simple spans	1⁄20.5
Continuous spans	1⁄25

   b. **Variable Depth –** Two span structures

At Center of span	1⁄25
At Intermediate pier	1⁄12.5
Multi-span structures	
At Center of span	1⁄36
At Intermediate pier	1⁄18

   *If the configuration of the exterior web is sloped and curved, a larger depth/span ratio may be necessary.

E. **Prestressed Concrete Sections**

1. **Application –** Local precast fabricators have several standard forms available for precast concrete sections based on the WSDOT standard girder series. These are versatile enough to cover a wide variety of span lengths.

   WSDOT standard girders are:

   a. WF100G, WF95G, WF83G, WF74G, WF58G, WF50G, WF42G, WF36G, W74G, W58G, W50G, and W42G precast, prestressed concrete I-girders requiring a cast-in-place concrete bridge deck used for spans less than 200′. The number (eg. 95) specifies the girder depth in inches.

      WF95PTG, WF83PTG and WF74PTG post-tensioned, precast segmental I-girders with cast-in-place concrete bridge deck use for simple span up to 230′, and continuous span up to 250′ with continuous post-tensioning over the intermediate piers.

   b. U**G* and UF**G* precast, prestressed concrete tub girders requiring a cast-in-place concrete bridge deck are used for spans less than 140′. "U" specifies webs without flanges, "UF" specifies webs with flanges, ** specifies the girder depth in inches, and * specifies the bottom flange width in feet. U**G* girders have been precast as shallow as 26″.

      Post-tensioned, precast, prestressed tub girders with cast-in-place concrete bridge deck are used for simple span up to 160′ and continuous span up to 200′.

   c. W65DG, W53DG, W41DG, and W35DG precast, prestressed concrete decked bulb tee girders requiring an HMA overlay roadway surface used for span less than 150′, with the Average Daily Truck limitation of 30,000 or less.

   d. W62BTG, W38BTG, and W32BTG precast, prestressed concrete bulb tee girders requiring a cast-in-place concrete deck for simple spans up to 130′.

   e. 12-inch, 18-inch, 26-inch, 30-inch, and 36-inch precast, prestressed slabs requiring 5″ minimum cast-in-place slab used for spans less than 100′.

   f. 26-inch precast, prestressed ribbed girder, deck double tee, used for span less than 60′, and double tee members requiring an HMA overlay roadway surface used for span less than 40′.

2. **Characteristics –** Superstructure design is quick for pretensioned girders with proven user-friendly software (PGSuper, PGSplice, and QConBridge)

   Construction details and forming are fairly simple. Construction time is less than for a cast-in-place bridge. Little or no falsework is required. Falsework over traffic is usually not required; construction time over existing traffic is reduced.

   Precast girders usually require that the bridge roadway superelevation transitions begin and end at or near piers; location of piers should consider this. The Region may be requested to adjust these transition points if possible.

   Fully reinforced, composite 8 inch cast-in-place deck slabs continuous over interior piers or reinforced 5 inch cast-in-place deck slabs continuous over interior piers have been used with e. and f.

F. **Composite Steel Plate Girder**

1. **Application –** Used for simple spans up to 260′ and for continuous spans from 120′ to 400′. Relatively low dead load when compared to a concrete superstructure makes this bridge type an asset in areas where foundation materials are poor.

2. **Characteristics –** Construction details and forming are fairly simple Construction time is comparatively short. Shipping and erecting of large sections must be reviewed. Cost of maintenance is higher than for concrete bridges. Current cost information should be considered because of changing steel market conditions.

3. **Depth/Span Ratios**

   a. **Constant Depth**

      Simple spans ⅟22
      Continuous spans ⅟25

   b. **Variable Depth**

      @ Center of span ⅟40
      @ Intermediate pier ⅟20

G. **Composite Steel Box Girder**

1. **Use –** Used for simple spans up to 260′ and for continuous spans from 120′ to 400′. Relatively low dead load when compared to a concrete superstructure makes this bridge type an asset in areas where foundation materials are poor.

   Inside clear height of less than 5 feet shall not be used because reasonable inspection access cannot be provided.

2. **Characteristics –** Construction details and forming are more difficult than for a steel plate girder. Shipping and erecting of large sections must be reviewed. Current cost information should be considered because of changing steel market conditions.

3. **Depth/Span Ratios**

   a. **Constant Depth**

      Simple spans ⅟22
      Continuous spans ⅟25

   b. **Variable Depth**

      At Center of span ⅟40
      At Intermediate pier ⅟20

      Note: Sloping webs are not used on box girders of variable depth.

H. **Steel Truss**

1. **Application –** Used for simple spans up to 300′ and for continuous spans up to 1,200′. Used where vertical clearance requirements dictate a shallow superstructure and long spans or where terrain dictates long spans and construction by cantilever method.

2. **Characteristics –** Construction details are numerous and can be complex. Cantilever construction method can facilitate construction over inaccessible areas. Through trusses are discouraged because of the resulting restricted horizontal and vertical clearances for the roadway.

3. **Depth/Span Ratios**

   a. Simple spans ⅙

   b. Continuous spans

      @ Center of span ⅟18
      @ Intermediate pier ⅑

I. **Segmental Concrete Box Girder**

1. **Application –** Used for continuous spans from 200′ to 700′. Used where site dictates long spans and construction by cantilever method.

2. **Characteristics –** Use of travelers for the form apparatus facilitates the cantilever construction method enabling long-span construction without falsework. Precast concrete segments may be used. Tight geometric control is required during construction to ensure proper alignment.

3. **Depth/Span Ratios**

Variable depth	
At Center of span	1⁄50
At Intermediate pier	1⁄20

J. **Railroad Bridges**

1. **Use –** For railway over highway grade separations, most railroad companies prefer simple span steel construction. This is to simplify repair and reconstruction in the event of derailment or some other damage to the structure.

2. **Characteristics –** The heavier loads of the railroad live load require deeper and stiffer members than for highway bridges. Through girders can be used to reduce overall structure depth if the railroad concurs. Piers should be normal to the railroad to eliminate skew loading effects.

3. **Depth/Span Ratios**

Constant depth	
Simple spans	1⁄12
Continuous two span	1⁄14
Continuous multi-span	1⁄15

K. **Timber**

1. **Use –** Generally used for spans under 40′. Usually used for detour bridges and other temporary structures. Timber bridges are not recommend for WSDOT Bridges.

2. **Characteristics –** Excellent for short-term duration as for a detour. Simple design and details.

3. **Depth/Span Ratios**

Constant depth	
Simple span – Timber beam	1⁄10
Simple span – Glulam beam	1⁄12
Continuous spans	1⁄14

L. **Other –** Bridge types such as cable-stayed, suspension, arch, tied arch, and floating bridges have special and limited applications. The use of these bridge types is generally dictated by site conditions. Preliminary design studies will generally be done when these types of structures are considered.

2.4.2 Wall Types

Retaining walls, wingwalls, curtain walls, and tall closed abutment walls may be used where required to shorten spans or superstructure length or to reduce the width of approach fills. The process of selecting a type of retaining wall should economically satisfy structural, functional, and aesthetic requirements and other considerations relevant to a specific site. A detailed listing of the common wall types and their characteristics can be found in Chapter 8.

2.5 Aesthetic Considerations

2.5.1 General Visual Impact

Bridge, retaining walls and noise walls have a strong visual impact in any landscape. Steps must be taken to assure that even the most basic structure will complement rather than detract from it's surroundings. The EIS and bridge site data submitted by the Region should each contain a discussion on the aesthetic importance of the project site. This commentary, together with submitted video and photographs, will help the designer determine the appropriate structure type.

The State Bridge and Structures Architect should be contacted early in the preliminary bridge plan process for input on aesthetics. Normally, a visit to the bridge site with the State Bridge and Structures Architect and Region design personnel should be made.

Aesthetics is a very subjective element that must be factored into the design process in the otherwise very quantitative field of structural engineering. Bridges that are structurally efficient using the least material possible are generally visually well proportioned. However, the details such as pier walls, columns, and crossbeams require special attention to ensure a structure that will enhance the general vicinity.

For large projects incorporating several to many bridges and retaining walls, an architectural theme is frequently developed to bring consistency in structure type, details, and architectural appointments. The preliminary plan designer shall work with the State Bridge and Structures Architect to implement the theme.

2.5.2 End Piers

A. **Wingwalls –** The size and exposure of the wingwall at the end pier should balance, visually, with the depth and type of superstructure used. For example, a prestressed girder structure fits best visually with a 15′ wingwall (or curtain wall/retaining wall). However, there are instances where a 20′ wingwall (or curtain wall/retaining wall) may be used with a prestressed girder (maximizing a span in a remote area, for example or with deep girders where they are proportionally better in appearance). The use of a 20′ wingwall shall be approved by the Bridge Design Engineer and the State Bridge and Structures Architect.

   It is less expensive for bridges of greater than 40′ of overall width to be designed with wingwalls (or curtain wall/retaining wall) than to use a longer superstructure.

B. **Retaining Walls –** For structures at sites where profile, right of way, and alignment dictate the use of high exposed wall-type abutments for the end piers, retaining walls that flank the approach roadway can be used to retain the roadway fill and reduce the overall structure length. Stepped walls are often used to break up the height, and allow for landscape planting. A curtain wall runs between the bridge abutment and the heel of the abutment footing. In this way, the joint in the retaining wall stem can coincide with the joint between the abutment footing and the retaining wall footing. This simplifies design and provides a convenient breaking point between design responsibilities if the retaining walls happen to be the responsibility of the Region. The length shown for the curtain wall dimension is an estimated dimension based on experience and preliminary foundation assumptions. It can be revised under design to satisfy the intent of having the wall joint coincide with the end of the abutment footing.

C. **Slope Protection –** The Region is responsible for making initial recommendations regarding slope protection. It should be compatible with the site and should match what has been used at other bridges in the vicinity. The type selected shall be shown on the Preliminary Plan. It shall be noted on the "Not Included in Bridge Quantities" list.

D. **Noise Walls –** Approval of the State Bridge and Structures Architect is required for the final selection of noise wall appearance, finish, materials and configuration.

2.5.3 Intermediate Piers

The size, shape, and spacing of the intermediate pier elements must satisfy two criteria. They must be correctly sized and detailed to efficiently handle the structural loads required by the design and shaped to enhance the aesthetics of the structure.

The primary view of the pier must be considered. For structures that cross over another roadway, the primary view will be a section normal to the roadway. This may not always be the same view as shown on the Preliminary Plan as with a skewed structure, for example. This primary view should be the focus of the aesthetic review.

Tapers and flares on columns should be kept simple and structurally functional. Fabrication and constructability of the formwork of the pier must be kept in mind. Crossbeam ends should be carefully reviewed. Skewed bridges and bridges with steep profile grades or those in sharp vertical curves will require special attention to detail.

Column spacing should not be so small as to create a cluttered look. Column spacing should be proportioned to maintain a reasonable crossbeam span balance.

2.5.4 Barrier and Wall Surface Treatments

A. **Plain Surface Finish –** This finish will normally be used on structures that do not have a high degree of visibility or where existing conditions warrant. A bridge in a remote area or a bridge among several existing bridges all having a plain finish would be examples.

B. **Formliner Finishes –** These finish are the most common and an easy way to add a decorative texture to a structure. Variations on this type of finish can be used for special cases. The specific areas to receive this finish should be reviewed with the State Bridge and Structures Architect.

C. **Pigmented Sealer –** The use of a pigmented sealer is used to control graffiti and can also be an aesthetic enhancement. Most commonly it is always used in urban areas. The selection should be reviewed with the Bridge Architect and the Region.

D. **Architectural Details –** Rustication grooves, relief panels, pilasters, and decorative finishes may visually improve appearance at transitions between different structure types such as cast-in-place abutments to structural earth retaining walls. Contact the State Bridge and Structures Architect for guidance.

   In special circumstances custom designs may be provided. Designs rising to the level of art shall be subject to the procedures outlined in the *Design Manual* M 22-01

2.5.5 Superstructure

The horizontal elements of the bridge are perhaps the strongest features. The sizing of the structure depth based on the span/depth ratios in Section 2.4.1, will generally produce a balanced relationship.

Designs rising to the level of "Art" shall be subject to the procedures outlined in the *Design Manual* M 22-01.

Haunches or rounding of girders at the piers can enhance the structure's appearance. The use of such features should be kept within reason considering fabrication of materials and construction of formwork. The amount of haunch should be carefully reviewed for overall balance from the primary viewing perspective. Haunches are not limited to cast-in-place superstructures, but may be used in special cases on precast, prestressed I girders. They require job-specific forms which increase cost, and standard design software is not directly applicable.

The slab overhang dimension should approach that used for the structure depth. This dimension should be balanced between what looks good for aesthetics and what is possible with a reasonable slab thickness and reinforcement.

For box girders, the exterior webs can be sloped, but vertical webs are preferred. The amount of slope should not exceed 1½: 1 for structural reasons, and should be limited to 4:1 if sloped webs are desired. Sloped webs should only be used in locations of high aesthetic impact.

When using precast, prestressed girders, all spans shall be the same series, unless approved otherwise by the Bridge and Structures Engineer.

2.7 WSDOT Standard Highway Bridge

2.7.1 Design Elements

The following are standard design elements for bridges carrying highway traffic. They are meant to provide a generic base for consistent, clean looking bridges, and to reduce design and construction costs. Modification of some elements may be required, depending on site conditions. This should be determined on a case-by-case basis during the preliminary plan stage of the design process.

A. **General –** Fractured Fin Finish shall be used on the exterior face of the traffic barrier. All other surfaces shall be Plain Surface Finish.

Exposed faces of wingwalls, columns, and abutments shall be vertical. The exterior face of the traffic barrier and the end of the intermediate pier crossbeam and diaphragm shall have a 1:12 backslope.

B. **Substructure –** End piers use the following details:

15′ wingwalls with prestressed girders up to 74″ in depth or a combination of curtain wall/retaining walls.

Stub abutment wall with vertical face. Footing elevation, pile type (if required), and setback dimension are determined from recommendations in the Materials Laboratory Geotechnical Services Branch Geotechnical Report.

Intermediate piers use the following details:

“Semi-raised” Crossbeams – The crossbeam below the girders is designed for the girder and slab dead load, and construction loads. The crossbeam and the diaphragm together are designed for all live loads and composite dead loads. The minimum depth of the crossbeam shall be 3′.

“Raised” Crossbeams – The crossbeam is at the same level as the girders are designed for all dead and live loads. “Raised” crossbeams are only used in conjunction with Prestressed Concrete Tub Girders.

Round Columns – Columns shall be 3′ to 6′ inch diameter. Dimensions are constant full height with no tapers. Bridges with roadway widths of 40′ or less will generally be single column piers. Bridges with roadway widths of greater the 40′ shall have two or more columns, following the criteria established in Section 2.3.1.H. Oval or rectangular column may be used if required for structural performance or bridge visual.

C. **Superstructure –** Concrete Slab – 7½ inch minimum thickness, with the top and bottom mat being epoxy coated steel reinforcing bars.

Prestressed Girders – Girder spacing will vary depending on roadway width and span length. The slab overhang dimension is approximately half of the girder spacing. Girder spacing typically ranges between 6′ and 12′.

Intermediate Diaphragms – Locate in accordance with Table 5.6.2-1 and Section 5.6.4.C. Provide full or partial depth in accordance with Section 5.6.4.C.4.

End Diaphragms – “End Wall on Girder” type.

Traffic Barrier – “F-shape” or Single-sloped barrier.

Fixed Diaphragm at Inter. Piers – Full or partial width of crossbeam between girders and outside of the exterior girders.

Hinged Diaphragm at Inter. Piers – Partial width of crossbeam between girders. Sloped curtain panel full width of crossbeam outside of exterior girders, fixed to ends of crossbeam.

BP Rail – 3′–6″ overall height for pedestrian traffic. 4′–6″ overall height for bicycle traffic.

Sidewalk – 6-inch height at curb line. Transverse slope of -0.02 feet per foot towards the curb line.

Sidewalk barrier – Inside face is vertical. Outside face slopes 1:12 outward.

The following table provides guidance regarding maximum bridge superstructure length beyond which the use of either intermediate expansion joints or modular expansion joints at the ends is required.

Superstructure Type	Maximum Length (Western WA)		Maximum Length (Eastern WA)	
	Stub Abutment	L-Abutment	Stub Abutment	L-Abutment
Concrete Superstructure				
Prestressed Girders*	450'	900'	450'	900'
PT Spliced Girder **	400'	700' ***	400'	700' ***
CIP-PT Box Girders **	400'	400'	400'	700' ***
Steel Superstructure				
Steel Plate Girder Steel Box Girder	300'	1000'	300'	800'

* Based upon 0.16" creep shortening per 100' of superstructure length, and 0.12" shrinkage shortening per 100' of superstructure length

** Based upon 0.31" creep shortening per 100' of superstructure length, and 0.19" shrinkage shortening per 100' of superstructure length

*** Can be increased to 800' if the joint opening at 64F at time of construction is specified in the expansion joint table to be less than the minimum installation width of 1½". This condition is acceptable if the gland is already installed when steel shapes are installed in the blockout. Otherwise (staged construction for example) the gland would need to be installed at temperatures less than 45ºF.

D. **Examples –** Appendices 2.3-A2-1 and 2.7-A1-1 detail the standard design elements of a standard highway bridge.

The following bridges are good examples of a standard highway bridge. However, they do have some modifications to the standard.

SR 17 Undercrossing 395/110	Contract 3785
Mullenix Road Overcrossing 16/203E&W	Contract 4143

2.7.2 Detailing the Preliminary Plan

The Bridge Preliminary Plan is used and reviewed by the Bridge and Structures Office or consultant who will do the structural design, Region designers and managers, Geotechnical engineers, Hydraulics engineers, Program managers, FHWA engineers and local agency designers and managers. It sometimes is used in public presentation of projects. With such visibility it is important that it's detailing is clear, complete, professional, and attractive. The designer, detailer, and checker shall strive for completeness and consistency in information, layout, line style, and fonts. Appendix B contains examples of Preliminary Plans following time-proven format that may be helpful. See also Chapter 11, Detailing Practice.

Typical sheet layout is as follows:

1. Plan and Elevation views. (This sheet ultimately becomes the Layout sheet of the design plan set)
2. Typical Section including details of stage construction.

   Superelevation diagrams, tables of existing elevations, Notes to Region, and other miscellaneous details as required shall go on Sheet 2, 3, or 4, as many as are required. See also the Preliminary Plan Checklist for details, dimensions, and notes typically required. The completed plan sheets shall be reviewed for consistency by the Preliminary Plans Detailing Specialist.

	STRUCTURE TYPES	SPAN RANGE, FT.	JAN 2014 COST RANGE $ / FT²
HYDAULIC STRUCTURES	PIPE	1 - 3	30 - 60
	CONCRETE CULVERT	3 - 20	100 - 120
	PLATE ARCH	12 - 20	65 - 80
STRUCTURES FOR CONVENTIONAL SITE CONDITIONS	REINF. CONCRETE SLAB	20 - 60	100 - 140
	REINF. CONCRETE TEE BEAM	30 - 60	100 - 150
	REINF. CONCRETE BOX GIRDER	50 - 120	200 - 275
	POST-TENSIONED CONC. BOX GIRDER	140 - 200	225 - 325
	SEGMENTAL P.T. BOX GIRDER	200 - 700	275 - 350
	PRESTRESSED CONC. SLAB	15 - 100	110 - 130
	PRESTRESSED CONC. DECK BULB TEE	40 - 160	110 - 150
	PRESTRESSED CONC. GIRDER	50 - 180	140 - 200
	PRESTRESSED TRAPEZOIDAL TUB GIRDER	40 - 140	175 - 225
	PRESTRESSED CONCRETE SPLICED GIRDER	140 - 230	175 - 225
	STEEL ROLLED GIRDER	20 - 70	150 - 175
	STEEL PLATE GIRDER	60 - 400	160 - 240
	STEEL BOX GIRDER	60 - 400	225 - 300
	STEEL TRUSS	300 - 1200	275 - 400
	TIMBER	10 - 20	130 - 150
	GLULAM TIMBER	14 - 40	130 - 150
STRUCTURES FOR SPECIAL SITE CONDITIONS	CABLE STAY BRIDGE	600 - 1200	550 - 650
	SUSPENSION BRIDGE	600 - 5000	900 - 1300
	FLOATING BRIDGE	600 +	900 - 1100
	ARCH BRIDGE	30 - 400	450 - 500
	MOVEABLE SPAN BRIDGE	200 - 350	2000 - 2500
	TUNNEL	30 +	2000 - 3000

SPAN RANGE, FT.

30 60 90 120 150 180 210 240 270 300 330 360 390 420 450 480 510 540 570 600 630 660 690+

THIS CHART IS INTENDED TO SHOW SOME OF THE MANY OPTIONS AVAILABLE FOR BRIDGE CONSTRUCTION AND THE WIDE RANGE OF COSTS ASSOCIATED WITH THEM. THE ACTUAL COST TO BE USED IN ANY COMPARISON FOR A SPECIFIC PROJECT IS VERY SENSITIVE TO THE FACTORS OUTLINED IN SECTION 2.2.3. ANY COMPARISON MADE FOR A PROJECT SHOULD BE DONE UNDER THE GUIDANCE OF THE PRELIMINARY DESIGN UNIT OF THE BRIDGE AND STRUCTURES OFFICE.

Last Revised on:1/27/14

Bridge Design Engr.	M:\BRIDGELIB\BDM\Chapter 2\window files\Bridge Selection Guide.wnd	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS	BRIDGE AND STRUCTURES OFFICE	Washington State Department of Transportation
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 SHEET NO.

SHEET OF SHEETS

SR JOB NO. SHEET

3.14 Earth Pressure

Earth Pressure loads see Chapter 7.

Chapter 4 Seismic Design and Retrofit Contents

4.2 WSDOT Modifications to *AASHTO Guide Specifications for LRFD Seismic Bridge Design*

WSDOT amendments to the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* are as follows:

4.2.1 Definitions

Guide Specifications Article 2.1 – Add the following definitions:

- **Oversized Pile Shaft –** A drilled shaft foundation that is larger in diameter than the supported column and has a reinforcing cage larger than and independent of the columns. The size of the shaft shall be in accordance with Section 7.8.2.
- **Owner –** Person or agency having jurisdiction over the bridge. For WSDOT projects, regardless of delivery method, the term "Owner" in these Guide Specifications shall be the WSDOT Bridge Design Engineer or/and the WSDOT Geotechnical Engineer.

4.2.2 Earthquake Resisting Systems (ERS) Requirements for SDCs C and D

Guide Specifications Article 3.3 – WSDOT Global Seismic Design Strategies:

- **Type 1** – Ductile Substructure with Essentially Elastic Superstructure. This category is permissible.
- **Type 2** – Essentially Elastic Substructure with a Ductile Superstructure. This category is not permissible.
- **Type 3** – Elastic Superstructure and Substructure With a Fusing Mechanism Between the Two. This category is permissible with WSDOT Bridge Design Engineer's approval.

With the approval of the Bridge Design Engineer, for Type 1 ERS for SDC C or D, if columns or pier walls are considered an integral part of the energy-dissipating system but remain elastic at the demand displacement, the forces to use for capacity design of other components are to be a minimum of 1.2 times the elastic forces resulting from the demand displacement in lieu of the forces obtained from overstrength plastic hinging analysis. Because maximum limiting inertial forces provided by yielding elements acting at a plastic mechanism level is not effective in the case of elastic design, the following constraints are imposed. These may be relaxed on a case-by-case basis with the approval of the Bridge Design Engineer.

1. Unless an analysis that considers redistribution of internal structure forces due to inelastic action is performed, all substructure units of the frame under consideration and of any adjacent frames that may transfer inertial forces to the frame in question must remain elastic at the design ground motion demand.
2. Effective member section properties must be consistent with the force levels ex-pected within the bridge system. Reinforced concrete columns and pier walls should be analyzed using cracked section properties. For this purpose, a moment of inertia equal to one-half that of the uncracked section shall be used.
3. Foundation modeling must be established such that uncertainties in modeling will not cause the internal forces of any elements under consideration to increase by more than 10 percent.

4. When site-specific ground response analysis is performed, the response spectrum ordinates must be selected such that uncertainties will not cause the internal forces of any elements under consideration to increase by more than 10 percent.
5. Thermal, shrinkage, prestress or other forces that may be present in the structure at the time of an earthquake must be considered to act in a sense that is least fa-vorable to the seismic load combination under investigation.
6. P-Delta effects must be assessed using the resistance of the frame in question at the deflection caused by the design ground motion.
7. Joint shear effects must be assessed with a minimum of the calculated elastic in-ternal forces applied to the joint.
8. Detailing as normally required in either SDC C or D, as appropriate, must be provided.

It is permitted to use expected material strengths for the determination of member strengths for elastic response of members.

The use of elastic design in lieu of overstrength plastic hinging forces for capacity protection described above shall only be considered if designer demonstrates that capacity design of Article 4.11 of the AASHTO Guide Specifications for LRFD Bridge Seismic Design is not feasible due to geotechnical or structural reasons.

Type 3 ERS may be considered only if Type 1 strategy is not suitable and Type 3 strategy has been deemed necessary for accommodating seismic loads. Isolation bearings shall be designed per the requirement of the *AASHTO Guide Specifications for Seismic Isolation*. Use of isolation bearings needs the approval of WSDOT Bridge Design Engineer.

The decision for using isolation bearings should be made at the early stage of project development based on the complexity of bridge geotechnical and structural design. A cost-benefit analysis comparing Type 1 design vs. Type 3 design with isolation bearings shall be performed and submitted for approval. The designer needs to perform two separate designs, one with and one without seismic isolation bearings. The cost-benefit analysis shall at least include:

- Higher initial design time and complexity of analysis.
- Impact of the initial and final design time on the project delivery schedule.
- Time required for preliminary investigation and correspondences with the isolation bearings suppliers.
- Life-cycle cost of additional and specialized bearing inspections.
- Potential cost impact for bearings and expansion joints replacements.
- Issues related to long-term performance and maintenance.
- Need for large movement expansion joints.

Seismic isolation bearings shall not be used between the top of the column and the bottom of the crossbeam in single or multi-column bents.

Once approval has been given for the use of seismic isolation bearing, the designer shall send a set of preliminary design and specification requirements to at least three seismic isolation bearing suppliers for evaluation to ensure that they can meet the design and specification requirements. Comments from isolation bearing suppliers should be incorporated before design of structure begins. Sole source isolation bearing supplier may be considered upon Bridge Design Office, and Project Engineer's office approval.

The designer shall submit to the isolation bearing suppliers maintenance and inspection requirements with design calculations. Isolation bearing suppliers shall provide maintenance and inspection requirements to ensure the isolators will function properly during the design life and after seismic events. The contract plans shall include bearing replacement methods and details.

Use of seismic isolation bearings are not recommended for conventional short and medium length bridges or bridges with geometrical complexities. Use of isolation bearings may not be beneficial for concrete bridges under 700 ft long, steel bridges under 800 ft long, bridges with skew angles exceeding 30 degrees, bridges with geometrical complexities, variable superstructure width, and bridges with drop-in spans.

The response modification factors (R-factors) of the *AASHTO Guide Specifications for Seismic Isolation Design* Article 6 shall not be used for structures if the provisions of *AASHTO Guide Specifications for LRFD Seismic Bridge Design* are being followed for the design of the bridge.

Suitability of isolation bearings for bridge projects should be carefully studied prior to approval. Isolation bearings may not be the effective solution for some bridges and sites since shifting the period to longer period may not reduce the force demand for the soft soils. Design shall consider the near fault effects and soil structure interaction of soft soil sites. The designer shall carefully study the effect of isolation bearings on the longitudinal bridge movement. The need for large movement expansion joints shall be investigated. Inspection, maintenance, and potential future bearing replacement should be considered when using the isolation bearings.

In order to have isolators fully effective, sufficient gap shall be provided to eliminate pounding between frames. Recommended bridge length and skew limitation are set to avoid using the modular joints. Most modular joints are not designed for seismic. Bridges are designed for extreme event which may or may not happen in the life span of the bridge. Introducing the modular joints to the bridge system could cause excessive maintenance issues. In estimation of life-cycle cost, specialized bearing inspections, potential cost impact for bearings, and expansion joints replacements the isolation bearing suppliers should be consulted.

If the columns or pier walls are designed for elastic forces, all other elements shall be designed for the lesser of the forces resulting from the overstrength plastic hinging moment capacity of columns or pier walls and the unreduced elastic seismic force in all SDCs. The minimum detailing according to the bridge seismic design category shall be provided. Shear design shall be based on 1.2 times elastic shear force and nominal material strengths shall be used for capacities. Limitations on the use of ERS and ERE are shown in Figures 3.3-1a, 3.3-1b, 3.3-2, and 3.3-3.

- Figure 3.3-1b Type 6, connection with moment reducing detail should only be used at column base if proved necessary for foundation design. Fixed connection at base of column remains the preferred option for WSDOT bridges.
- The design criteria for column base with moment reducing detail shall consider all applicable loads at service, strength, and extreme event limit states.
- Figure 3.3-2 Types 6 and 8 are not permissible for non-liquefied configuration and permissible with WSDOT Bridge Design Engineer's approval for liquefied configuration

For ERSs and EREs requiring approval, the WSDOT Bridge Design Engineer's approval is required regardless of contracting method (i.e., approval authority is not transferred to other entities).

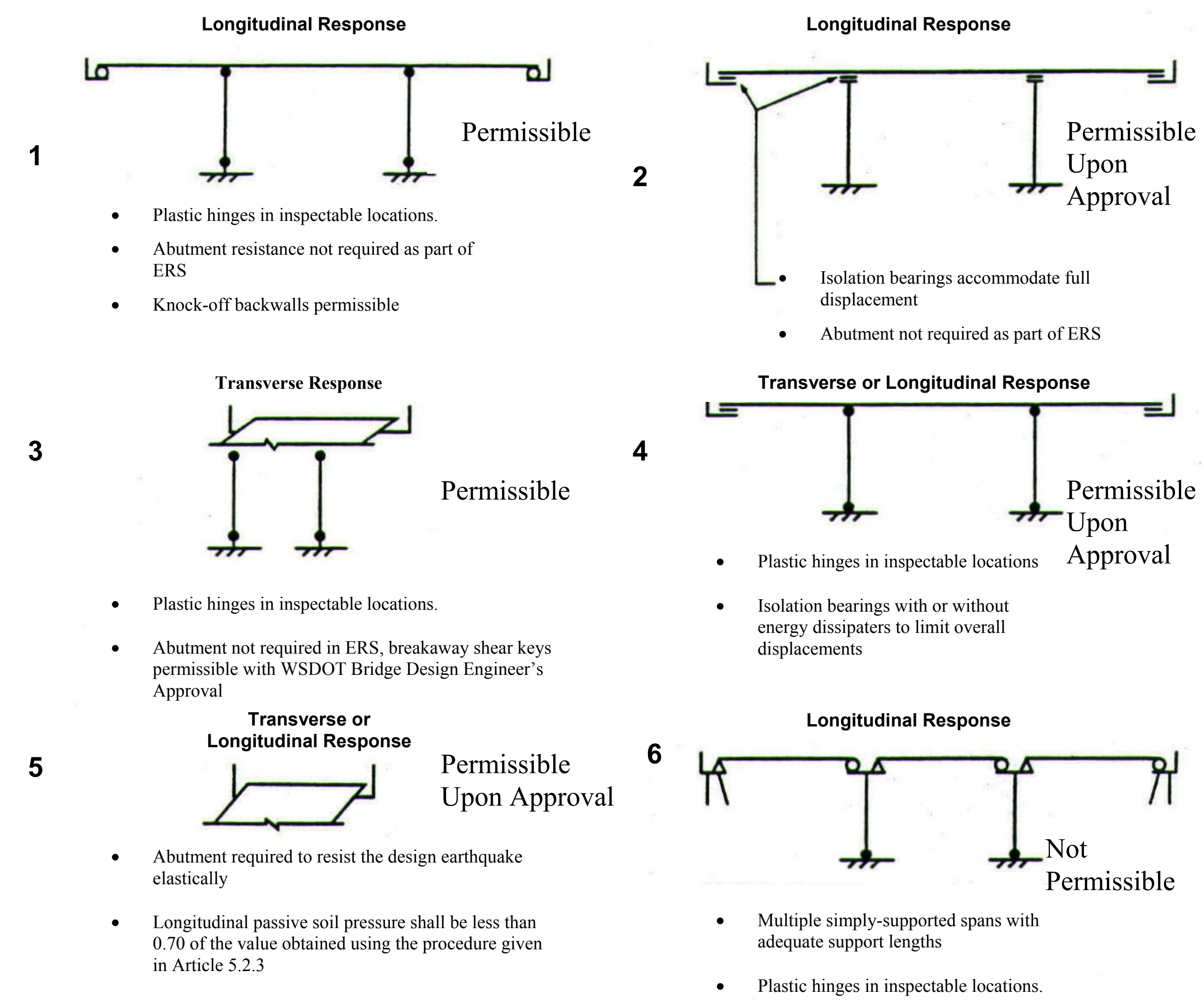


Figure 3.3-1a Permissible Earthquake-Resisting Systems (ERSs)
BDM Figure 4.2.2-1

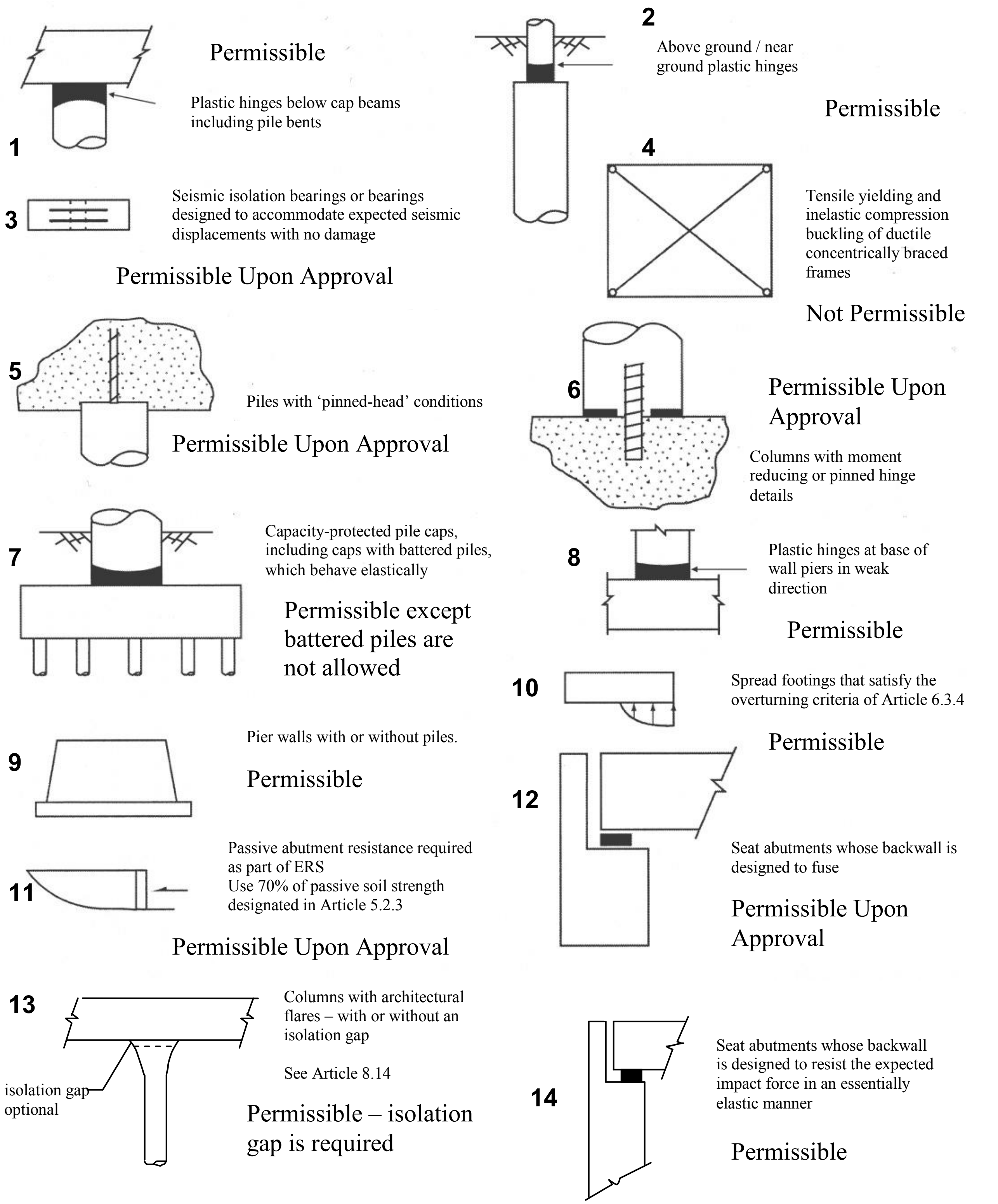


Figure 3.3-1b Permissible Earthquake-Resisting Elements (EREs)
BDM Figure 4.2.2-2

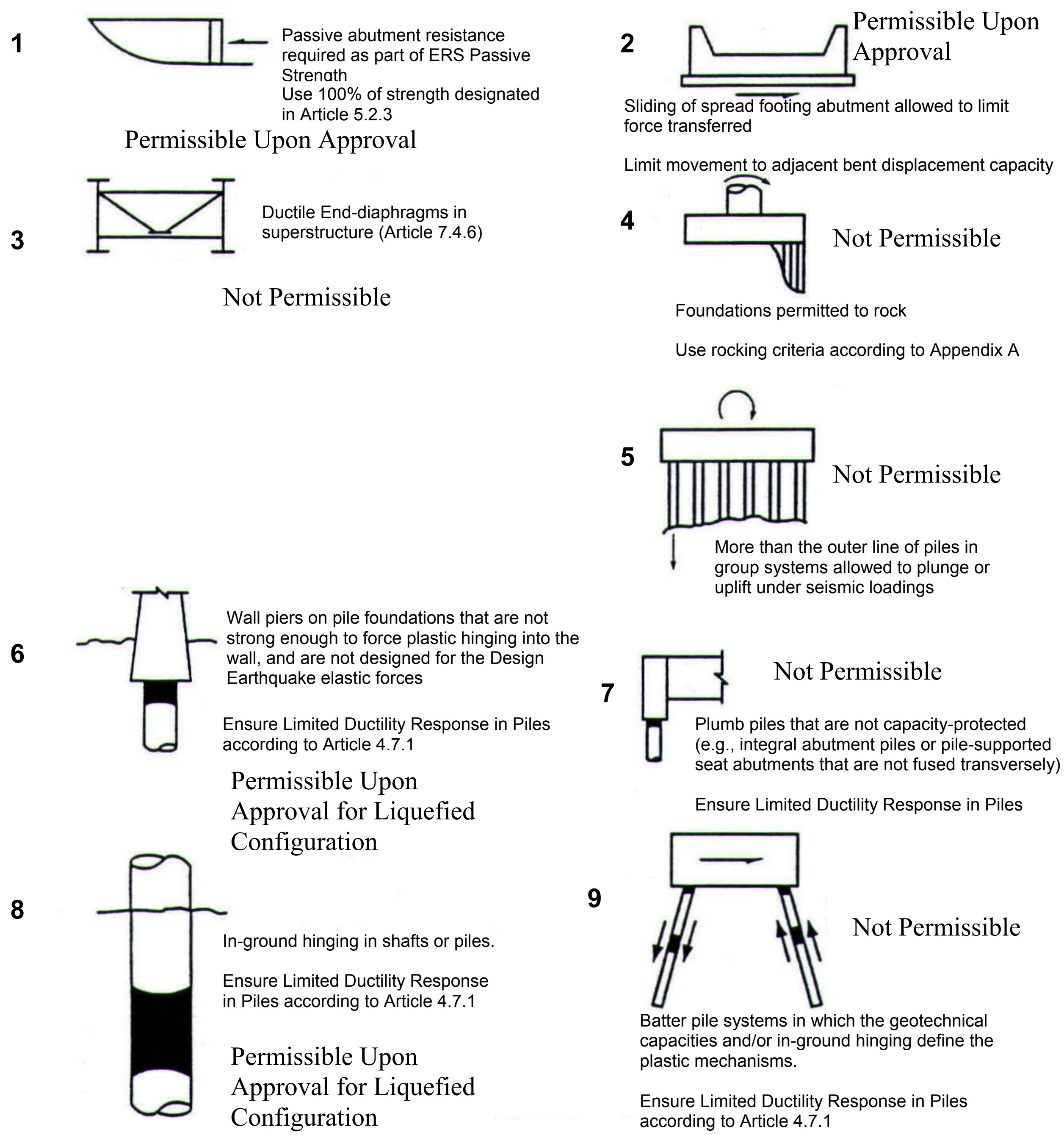


Figure 3.3-2 Permissible Earthquake-Resisting Elements That Require Owner's Approval
BDM Figure 4.2.2-3

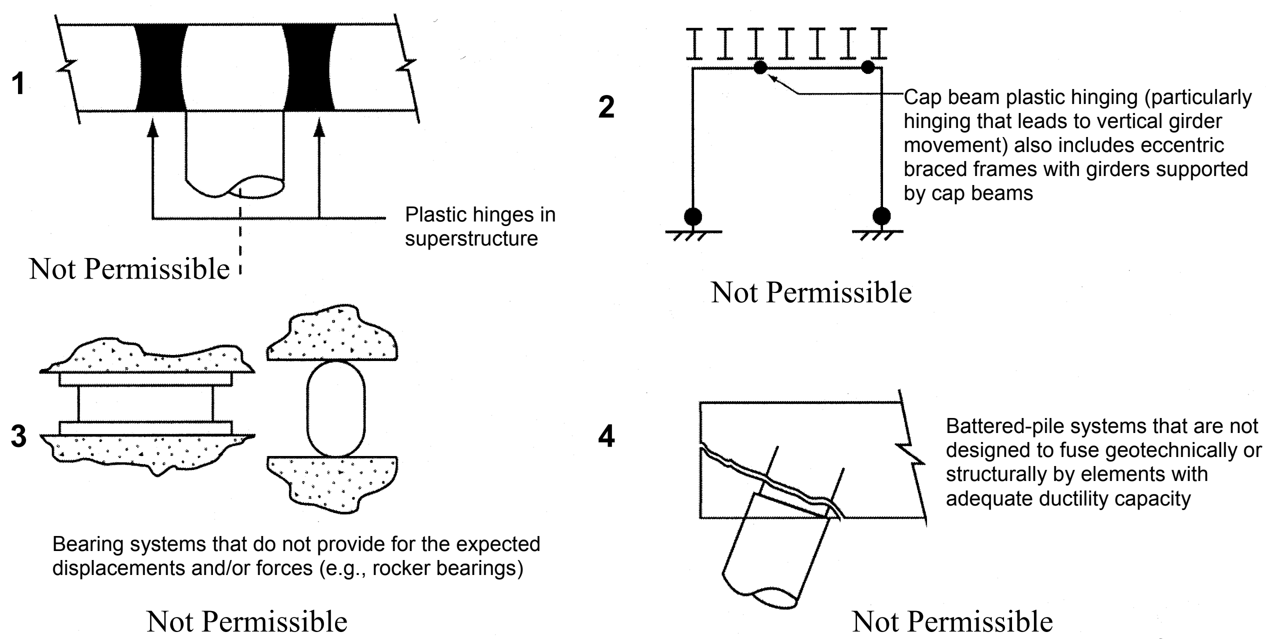


Figure 3.3-3 Earthquake-Resisting Elements that Are Not Recommended for New Bridges
BDM Figure 4.2.2-4

4.2.3 Seismic Ground Shaking Hazard

Guide Specifications Article 3.4 – For bridges that are considered critical or essential or normal bridges with a site Class F, the seismic ground shaking hazard shall be determined based on the WSDOT Geotechnical Engineer recommendations.

In cases where the site coefficients used to adjust mapped values of design ground motion for local conditions are inappropriate to determine the design spectra in accordance with general procedure of Article 3.4.1 (such as the period at the end of constant design spectral acceleration plateau (T_S) is greater than 1.0 second or the period at the beginning of constant design spectral acceleration plateau (T_o) is less than 0.2 second), a site-specific ground motion response analysis shall be performed.

4.2.4 Selection of Seismic Design Category (SDC)

Guide Specifications Article 3.5 – Pushover analysis shall be used to determine displacement capacity for both SDCs C and D.

4.2.5 Temporary and Staged Construction

Guide Specifications Article 3.6 – For bridges that are designed for a reduced seismic demand, the contract plans shall either include a statement that clearly indicates that the bridge was designed as temporary using a reduced seismic demand or show the Acceleration Response Spectrum (ARS) used for design.

4.2.6 Load and Resistance Factors

Guide Specifications Article 3.7 – Revise as follows:

Use load factors of 1.0 for all permanent loads. The load factor for live load shall be 0.0 when pushover analysis is used to determine the displacement capacity. Use live load factor of 0.5 for all other extreme event cases. Unless otherwise noted, all ϕ factors shall be taken as 1.0.

4.2.7 Balanced Stiffness Requirements and Balanced Frame Geometry Recommendation

Guide Specifications Articles 4.1.2 and 4.1.3 – Balanced stiffness between bents within a frame and between columns within a bent and balanced frame geometry for adjacent frames are required for bridges in both SDCs C and D. Deviations from balanced stiffness and balanced frame geometry requirements require approval from the WSDOT Bridge Design Engineer.

4.2.8 Selection of Analysis Procedure to Determine Seismic Demand

Guide Specifications Article 4.2 – Analysis Procedures:

- Procedure 1 (Equivalent Static Analysis) shall not be used.
- Procedure 2 (Elastic Dynamic Analysis) shall be used for all "regular" bridges with two through six spans and "not regular" bridges with two or more spans in SDCs B, C, or D.
- Procedure 3 (Nonlinear Time History) shall only be used with WSDOT Bridge Design Engineer's approval.

4.2.9 Member Ductility Requirement for SDCs C and D

Guide Specifications Article 4.9 – In-ground hinging for drilled shaft and pile foundations may be considered for the liquefied configuration with WSDOT Bridge Design Engineer approval.

4.2.10 Longitudinal Restrainers

Guide Specifications Article 4.13.1 – Longitudinal restrainers shall be provided at the expansion joints between superstructure segments. Restrainers shall be designed in accordance with the FHWA *Seismic Retrofitting Manual for Highway Structure* (FHWA-HRT-06-032) Article 8.4 The Iterative Method. See the earthquake restrainer design example in the Appendix of this chapter. Restrainers shall be detailed in accordance with the requirements of Guide Specifications Article 4.13.3 and Section 4.4.5. Restrainers may be omitted for SDCs C and D where the available seat width exceeds the calculated support length specified in Equation C4.13.1-1.

Omitting restrainers for liquefiable sites shall be approved by the WSDOT Bridge Design Engineer.

Longitudinal restrainers shall not be used at the end piers (abutments).

4.2.11 Abutments

Guide Specifications Article 5.2 – Diaphragm Abutment type shown in Figure 5.2.3.2-1 shall bridges.

Guide Specifications Article 5.2 – Abutments

Revise as follows:

5.2.1 - General

The participation of abutment walls in providing resistance to seismically induced inertial loads may be considered in the seismic design of bridges either to reduce column sizes or reduce the ductility demand on the columns. Damage to backwalls and wingwalls during earthquakes may be considered acceptable when considering no collapse criteria, provided that unseating or other damage to the superstructure does not occur. Abutment participation in the overall dynamic response of the bridge system shall reflect the structural configuration, the load transfer mechanism from the bridge to the abutment system, the effective stiffness and force capacity of the wall-soil system, and the level of acceptable abutment damage. The capacity of the abutments to resist the bridge inertial loads shall be compatible with the soil re-sistance that can be reliably mobilized, the structural design of the abutment wall, and whether the wall is permitted to be damaged by the design earthquake. The lateral load capacity of walls shall be evaluated on the basis of a rational passive earth-pressure theory.

The participation of the bridge approach slab in the overall dynamic response of bridge systems to earthquake loading and in providing resistance to seismically in-duced inertial loads may be considered permissible upon approval from both the WSDOT Bridge Design Engineer and the WSDOT Geotechnical Engineer.

The participation of the abutment in the ERS should be carefully evaluated with the Geotechnical Engineer and the Owner when the presence of the abutment backfill may be uncertain, as in the case of slumping or settlement due to liquefaction below or near the abutment.

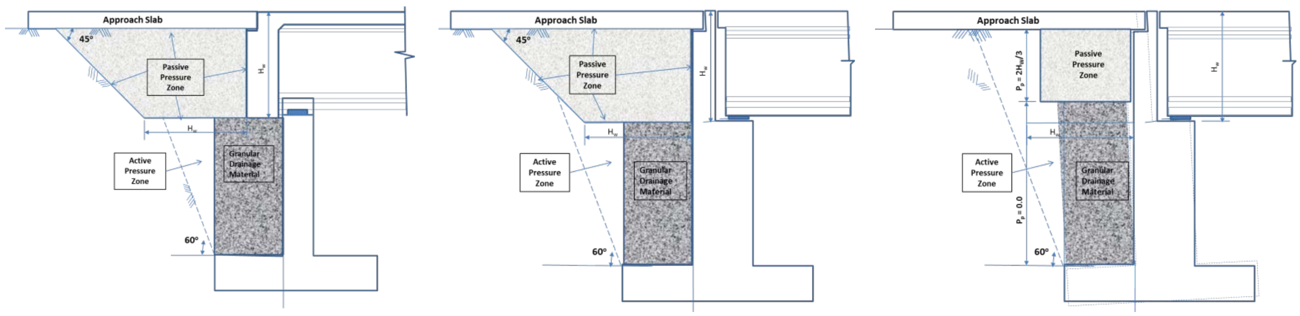
5.2.2 - Longitudinal Direction

Under earthquake loading, the earth-pressure action on abutment walls changes from a static condition to one of two possible conditions:

- The dynamic active pressure condition as the wall moves away from the backfill, or
- The passive pressure condition as the inertial load of the bridge pushes the wall into the backfill.

The governing earth-pressure condition depends on the magnitude of seismically induced movement of the abutment walls, the bridge superstructure, and the bridge/abutment configuration.

For semi-integral (Figure 5.2.2-a), L-shape abutment with backwall fuse (Figure 5.2.2-b), or without backwall fuse (Figure 5.2.2-c), for which the expansion joint is sufficiently large to accommodate both the cyclic movement between the abutment wall and the bridge superstructure (i.e., superstructure does not push against abutment wall), the seismically induced earth pressure on the abutment wall shall be considered to be the dynamic active pressure condition. However, when the gap at the expansion joint is not sufficient to accommodate the cyclic wall/bridge seismic movements, a transfer of forces will occur from the superstructure to the abutment wall. As a result, the active earth-pressure condition will not be valid and the earth pressure approaches a much larger passive pressure load condition behind the backwall. This larger load condition is the main cause for abutment damage, as demonstrated in past earthquakes. For semi-integral or L-shape abutments, the abutment stiffness and capacity under passive pressure loading are primary design concerns.



(a) Semi-integral Abutment (b) L-shape Abutment Backwall Fuses (c) L-shape Abutment Backwall Does Not Fuse

Figure 5.2.2 - Abutment Stiffness and Passive Pressure Estimate

Figure 4.2.11-1

Where the passive pressure resistance of soils behind semi-integral or L-shape abutments will be mobilized through large longitudinal superstructure displacements, the bridge may be designed with the abutments as key elements of the longitudinal ERS. Abutments shall be designed to sustain the design earthquake displacements. When abutment stiffness and capacity are included in the design, it should be recognized that the passive pressure zone mobilized by abutment displacement extends beyond the active pressure zone normally used for static service load design. This is illustrated schematically in Figures 1a and 1b. Dynamic active earth pressure acting on the abutment need not be considered in the dynamic analysis of the bridge. The passive abutment resistance shall be limited to 70% of the value obtained using the procedure given in Article 5.2.2.1.

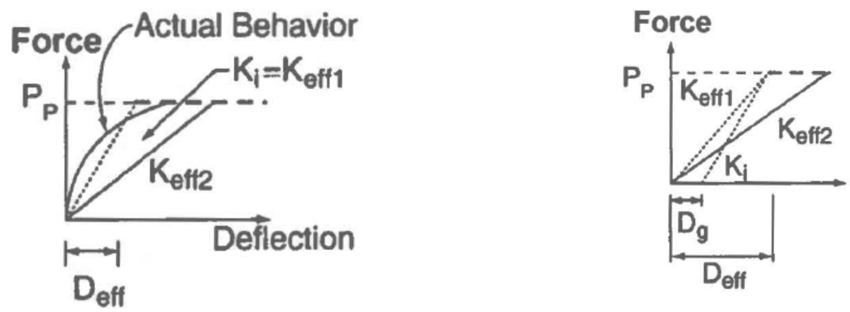
5.2.2.1 - Abutment Stiffness and Passive Pressure Estimate

Abutment stiffness, K_{eff} in kip/ft, and passive capacity, P_p in kips, should be charac-terized by a bilinear or other higher order nonlinear relationship as shown in Figure 5.2.2.1. When the motion of the back wall is primarily translation, passive pressures may be assumed uniformly distributed over the height (H_w) of the backwall or end diaphragm. The total passive force may be determined as:

$$P_p = {}_{pp} H_w W_w \tag{5.2.2.1-1}$$

Where:

- p_p = passive lateral earth pressure behind backwall or diaphragm (ksf)
- H_w = height of back wall or end diaphragm exposed to passive earth pressure (ft)
- W_w = width of back wall or diaphragm (ft)



(a) Semi-integral Abutment (b) L-shape Abutment

Figure 5.2.2.1- Characterization of Abutment Capacity and Stiffness
Figure 4.2.11-2

5.2.2.2 - Calculation of Best Estimate Passive Pressure Pp

If the strength characteristics of compacted or natural soils in the "passive pressure zone" are known, then the passive force for a given height, H_w, may be calculated using accepted analysis procedures. These procedures should account for the interface friction between the wall and the soil. The properties used shall be those indicative of the entire "passive pressure zone" as indicated in Figure 1. Therefore, the properties of backfill present immediately adjacent to the wall in the active pressure zone may not be appropriate as a weaker failure surface can develop elsewhere in the embankment.

For L-shape abutments where the backwall is not designed to fuse, H_w shall conservatively be taken as the depth of the superstructure, unless a more rational soil-structure interaction analysis is performed.

If presumptive passive pressures are to be used for design, then the following criteria shall apply:

- Soil in the "passive pressure zone" shall be compacted in accordance with WSDOT *Standard Specification* Section 2-03.3(14)I, which requires compaction to 95-percent maximum density for all “Bridge Approach Embankments”.
- For cohesionless, nonplastic backfill (fines content less than 30 percent), the passive pressure pp may be assumed equal to $2H_w/3$ ksf per foot of wall length.

For other cases, including abutments constructed in cuts, the passive pressures shall be developed by a geotechnical engineer.

5.2.2.3 - Calculation of Passive Soil Stiffness

Equivalent linear secant stiffness, K_{eff} in kip/ft, is required for analyses. For semi-integral or L-shape abutments initial secant stiffness may be determined as follows:

$$K_{eff1} = \frac{P_p}{\left(F_w H_w\right)} \qquad (5.2.2.3\text{-}1)$$

Where:

P_p = passive lateral earth pressure capacity (kip)
H_w = height of back wall (ft)
F_w = the value of Fw to use for a particular bridge may be found in Table C3.11.1-1 of the AASHTO LRFD Bridge Design Specifications.

For L-shape abutments, the expansion gap should be included in the initial estimate of the secant stiffness as specified in:

$$K_{eff1} = \frac{P_p}{\left(F_w H_w + D_g\right)} \quad (5.2.2.3\text{-}2)$$

Where:

D_g = width of gap between backwall and superstructure (ft)

For SDCs C and D, where pushover analyses are conducted, values of P_p and the ini-tial estimate of K_{eff1} should be used to define a bilinear load-displacement behavior of the abutment for the capacity assessment.

5.2.2.4 - Modeling Passive Pressure Stiffness in the Longitudinal Direction

In the longitudinal direction, when the bridge is moving toward the soil, the full pas-sive resistance of the soil may be mobilized, but when the bridge moves away from the soil no soil resistance is mobilized. Since passive pressure acts at only one abutment at a time, linear elastic dynamic models and frame pushover models should only include a passive pressure spring at one abutment in any given model. Secant stiffness values for passive pressure shall be developed independently for each abutment.

As an alternative, for straight or with horizontal curves up to 30-degrees single frame bridges, and compression models in straight multi-frame bridges where the passive pressure stiffness is similar between abutments, a spring may be used at each abutment concurrently. In this case, the assigned spring values at each end need to be reduced by half because they act in simultaneously, whereas the actual backfill passive resistance acts only in one direction and at one time. Correspondingly, the actual peak passive resistance force at either abutment will be equal to the sum of the peak forces developed in two springs. In this case, secant stiffness values for passive pressure shall be developed based on the sum of peak forces developed in each spring. If computed abutment forces exceed the soil capacity, the stiffness should be softened iteratively until abutment displacements are consistent (within 30 percent) with the assumed stiffness.

5.2.3 - Transverse Direction

Transverse stiffness of abutments may be considered in the overall dynamic response of bridge systems on a case-by-case basis upon Bridge Design Engineer approval. Upon approval, the transverse abutment stiffness used in the elastic demand models may be taken as 50-percent of the elastic transverse stiffness of the adjacent bent.

Girder stops are typically designed to transmit the lateral shear forces generated by small to moderate earthquakes and service loads and are expected to fuse at the de-sign event earthquake level of acceleration to limit the demand and control the dam-age in the abutments and supporting piles/shafts. Linear elastic analysis cannot cap-ture the inelastic response of the girder stops, wingwalls or piles/shafts. Therefore, the forces generated with elastic demand assessment models should not be used to size the abutment girder stops. Girder stops for abutments supported on a spread footing shall be designed to sustain the lesser of the acceleration coefficient, As, times the superstructure dead load reaction at the abutment plus the weight of abutment and its footing or sliding friction forces of spread footings. Girder stops for pile/shaft-supported foundations shall be designed to sustain the sum of 75% total lateral capacity of the piles/shafts and shear capacity of one wingwall.

The elastic resistance may be taken to include the use of bearings designed to accommodate the design displacements, soil frictional resistance acting against the base of a spread footing-supported abutment, or pile resistance provided by piles acting in their elastic range.

The stiffness of fusing or breakaway abutment elements such as wingwalls (yielding or non-yielding), elastomeric bearings, and sliding footings shall not be relied upon to reduce displacement demands at intermediate piers.

Unless fixed bearings are used, girder stops shall be provided between all girders regardless of the elastic seismic demand. The design of girder stops should consider that unequal forces that may develop in each stop.

When fusing girder stops, transverse shear keys, or other elements that potentially release the restraint of the superstructure are used, then adequate support length meeting the requirements of Article 4.12 of the Guide Specifications for LRFD Seismic Bridge Design must be provided. Additionally, the expected redistribution of internal forces in the superstructure and other bridge system element must be considered. Bounding analyses considering incremental release of transverse restraint at each end of the bridge should also be considered.

5.2.4 - Curved and Skewed Bridges

Passive earth pressure at abutments may be considered as a key element of the ERS of straight and curved bridges with abutment skews up to 20 degrees. For larger skews, due to a combination of longitudinal and transverse response, the span has a tendency to rotate in the direction of decreasing skew. Such motion will tend to cause binding in the obtuse corner and generate uneven passive earth pressure forces on the abutment, exceeding the passive pressure near one end of the backwall, and providing little or no resistance at other end. This requires a more refined analysis to determine the amount of expected movement. The passive pressure resistance in soils behind semi-integral or L-shape abutments shall be based on the projected width of the abutment wall normal to the centerline of the bridge. Abutment springs shall be included in the local coordinate system of the abutment wall.

4.2.12 Foundation – General

Guide Specifications Article 5.3.1 – The required foundation modeling method (FMM) and the requirements for estimation of foundation springs for spread footings, pile foundations, and drilled shafts shall be based on the WSDOT Geotechnical Engineer's recommendations.

4.2.13 Foundation – Spread Footing

Guide Specifications Article C5.3.2 – Foundation springs for spread footings shall be determined in accordance with Section 7.2.7, WSDOT *Geotechnical Design Manual* Section 6.5.1.1 and the WSDOT Geotechnical Engineer's recommendations.

4.2.14 Procedure 3: Nonlinear Time History Method

Guide Specifications Article 5.4.4 – The time histories of input acceleration used to describe the earthquake loads shall be selected in consultation with the WSDOT Geotechnical Engineer and the WSDOT Bridge Design Engineer.

4.2.15 I_{eff} for Box Girder Superstructure

Guide Specifications Article 5.6.3 – Gross moment of inertia shall be used for box girder superstructure modeling.

4.2.16 Foundation Rocking

Guide Specifications Article 6.3.9 – Foundation rocking shall not be used for the design of WSDOT bridges.

4.2.17 Drilled Shafts

Guide Specifications Article C6.5 – For WSDOT bridges, the scale factor for p-y curves or subgrade modulus for large diameter shafts shall not be used unless approved by the WSDOT Geotechnical Engineer and WSDOT Bridge Design Engineer.

4.2.18 Longitudinal Direction Requirements

Guide Specifications Article 6.7.1 – Case 2: Earthquake Resisting System (ERS) with abutment contribution may be used provided that the mobilized longitudinal passive pressure is not greater than 70 percent of the value obtained using procedure given in Article 5.2.2.1.

4.2.19 Liquefaction Design Requirements

Guide Specifications Article 6.8 – Soil liquefaction assessment shall be based on the WSDOT Geotechnical Engineer's recommendation and WSDOT *Geotechnical Design Manual* Section 6.4.2.8.

4.2.20 Reinforcing Steel

Guide Specifications Article 8.4.1 – Reinforcing bars, deformed wire, cold-draw wire, welded plain wire fabric and welded deformed wire fabric shall conform to the material standards as specified in AASHTO LRFD *Bridge Design specifications*.

ASTM A 615 reinforcement shall not be used in WSDOT Bridges. Only ASTM A 706 Grade 60 reinforcing steel shall be used in members where plastic hinging is expected for SDCs B, C, and D. ASTM A 706 Grade 80 reinforcing steels may be used for capacity-protected members as specified in Article 8.9. ASTM A 706 Grade 80 reinforcing steel shall not be used for oversized shafts where in ground plastic hinging is considered as a part of ERS.

Deformed welded wire fabric may be used with the WSDOT Bridge Design Engineer's approval.

Wire rope or strands for spirals and high strength bars with yield strength in excess of 75 ksi shall not be used.

Guide Specifications Article C8.4.1 – Add the following paragraph to Article C8.4.1.

The requirement for plastic hinging and capacity protected members do not apply to the structures in SDC A, therefore use of ASTM A706 Grade 80 reinforcing steel is permitted in SDC A.

For SDCs B, C, and D, the moment-curvature analyses based on strain compatibility and nonlinear stress strain relations are used to determine the plastic moment capacities of all ductile concrete members. Further research is required to establish the shape and model of the stress-strain curve, expected reinforcing strengths, strain limits, and the stress-strain relationships for concrete confined by lateral reinforcement made with ASTM A 706 Grade 80 reinforcing steel.

4.2.21 Concrete Modeling

Guide Specifications Article 8.4.4- Revise the last paragraph as follows:

Where in-ground plastic hinging approved by the WSDOT Bridge Design Engineer is part of the ERS, the confined concrete core shall be limited to a maximum compressive strain of 0.008. The clear spacing between the longitudinal reinforcements and between spirals and hoops in drilled shafts shall not be less than 6 inches or more than 8 inches when tremie placement of concrete is anticipated.

4.2.22 Expected Nominal Moment Capacity

Guide Specifications Article 8.5 – Add the following paragraphs after third paragraph.

The expected nominal capacity of capacity protected member using ASTM A 706 Grade 80 reinforcement shall be determined by strength design based on the expected concrete strength and yield strength of 80 ksi when the concrete reaches 0.003 or the reinforcing steel strain reaches 0.090 for #10 bars and smaller, 0.060 for #11 bars and larger.

Replace the definition of with the following:

λ_{mo} = overstrength factor
= 1.2 for ASTM A 706 Grade 60 reinforcement
= 1.4 for ASTM A 615 Grade 60 reinforcement

4.2.23 Interlocking Bar Size

Guide Specifications Article 8.6.7 – The longitudinal reinforcing bar inside the interlocking portion of column (interlocking bars) shall be the same size of bars used outside the interlocking portion.

4.2.24 Splicing of Longitudinal Reinforcement in Columns Subject to Ductility Demands for SDCs C and D

Guide Specifications Article 8.8.3 – The splicing of longitudinal column reinforcement outside the plastic hinging region shall be accomplished using mechanical couplers that are capable of developing the tensile strength of the spliced bar. Splices shall be staggered at least 2 ft. Lap splices shall not be used. The design engineer shall clearly identify the locations where splices in longitudinal column reinforcement are permitted on the plans. In general where the length of the rebar cage is less than 60 ft (72 ft for No. 14 and No. 18 bars), no splice in the longitudinal reinforcement shall be allowed.

4.2.25 Development Length for Column Bars Extended into Oversized Pile Shafts f or SDCs C and D

Guide Specifications Article 8.8.10 – Extending column bars into oversized shaft shall be per Section 7.4.4.C, based on TRAC Report WA-RD 417.1 "Non Contact Lap Splice in Bridge Column-Shaft Connections."

4.2.26 Lateral Confinement for Oversized Pile Shaft for SDCs C and D

Guide Specifications Article 8.8.12 – The requirement of this article for shaft lateral reinforcement in the column-shaft splice zone may be replaced with Section 7.8.2 K of this manual.

4.2.27 Lateral Confinement for Non-Oversized Strengthened Pile Shaft for SDCs C and D

Guide Specifications Article 8.8.13 – Non oversized column shaft (the cross section of the confined core is the same for both the column and the pile shaft) is not permissible unless approved by the WSDOT Bridge Design Engineer.

4.2.28 Requirements for Capacity Protected Members

Guide Specifications Article 8.9 – Add the following paragraphs:

For SDCs C and D where liquefaction is identified, with the WSDOT Bridge Design Engineer's approval, pile and drilled shaft in-ground hinging may be considered as an ERE. Where in-ground hinging is part of ERS, the confined concrete core should be limited to a maximum compressive strain of 0.008 and the member ductility demand shall be limited to 4.

Bridges shall be analyzed and designed for the nonliquefied condition and the liquefied condition in accordance with Article 6.8. The capacity protected members shall be designed in accordance with the requirements of Article 4.11. To ensure the formation of plastic hinges in columns, oversized pile shafts shall be designed for an expected nominal moment capacity, M_{ne}, at any location along the shaft, that is, equal to 1.25 times moment demand generated by the overstrength column plastic hinge moment and associated shear force at the base of the column. The safety factor of 1.25 may be reduced to 1.0 depending on the soil properties and upon the WSDOT Bridge Design Engineer's approval.

The design moments below ground for extended pile shaft may be determined using the nonlinear static procedure (pushover analysis) by pushing them laterally to the displacement demand obtained from an elastic response spectrum analysis. The point of maximum moment shall be identified based on the moment diagram. The expected plastic hinge zone shall extend 3D above and below the point of maximum moment. The plastic hinge zone shall be designated as the "no-splice" zone and the transverse steel for shear and confinement shall be provided accordingly.

4.2.29 Superstructure Capacity Design for Transverse Direction (Integral Bent Cap) for SDCs C and D

Guide Specifications Article 8.11 – Revise the last paragraph as follows:

For SDCs C and D, the longitudinal flexural bent cap beam reinforcement shall be continuous. Splicing of cap beam longitudinal flexural reinforcement shall be accomplished using mechanical couplers that are capable of developing a minimum tensile strength of 85 ksi. Splices shall be staggered at least 2 ft. Lap splices shall not be used.

4.2.30 Superstructure Design for Non Integral Bent Caps for SDCs B, C, and D

Guide Specifications Article 8.12 – Non integral bent caps shall not be used for continuous concrete bridges in SDC B, C, and D except at the expansion joints between superstructure segments.

4.2.31 Joint Proportioning

Guide Specifications Article 8.13.4.1.1 – Revise the last bullet as follows:

Exterior column joints for box girder superstructure and other superstructures if the cap beam extends the joint far enough to develop the longitudinal cap reinforcement.

4.2.32 Cast-in-Place and Precast Concrete Piles

Guide Specifications Article 8.16.2 – Minimum longitudinal reinforcement of 0.75 percent of A_g shall be provided for CIP piles in SDCs B, C, and D. Longitudinal reinforcement shall be provided for the full length of pile unless approved by the WSDOT Bridge Design Engineer.

4.3 Seismic Design Requirements for Bridge Widening Projects

4.3.1 Seismic Analysis and Retrofit Policy

Widening of existing bridges is often challenging, specifically when it comes to determining how to address elements of the existing structure that do not meet current design standards. The Seismic Analysis and Retrofit Policy for Bridge Widening Projects (Figure 4.3-1) has been established to give bridge design engineers guidance on how and when to address structural deficiencies in existing bridges that are being widened. This policy balances the engineers responsibility to "safeguard life, health, and property" (WAC 196-27A-020) with their responsibility to "achieve the goals and objectives agreed upon with their client or employer" (WAC 196-27A-020 (2)(a)). Current versions of bridge design specifications/codes do not provide guidance on how to treat existing structures that are being widened. This policy is based on and validated by the requirements of the International Building Code (2009 IBC Section 3403.4). The IBC is the code used throughout the nation for design of most structures other than bridges. Thus, the requirements of the IBC can be taken to provide an acceptable level of safety that meets the expectations of the public.

This "Do No Harm" policy requires the bridge engineer to compare existing bridge element seismic capacity/demand ratios for the before widening condition to those of the after widening condition. If the capacity/demand ratio is not decreased, the widening can be designed and constructed without retrofitting existing seismically deficient bridge elements. In this case retrofit of seismically deficient elements is recommended but not required. The decision to retrofit these elements is left to the region and is based on funding availability. If the widened capacity/demand ratios are decreased, the seismically deficient existing elements must be retrofitted as part of the widening project.

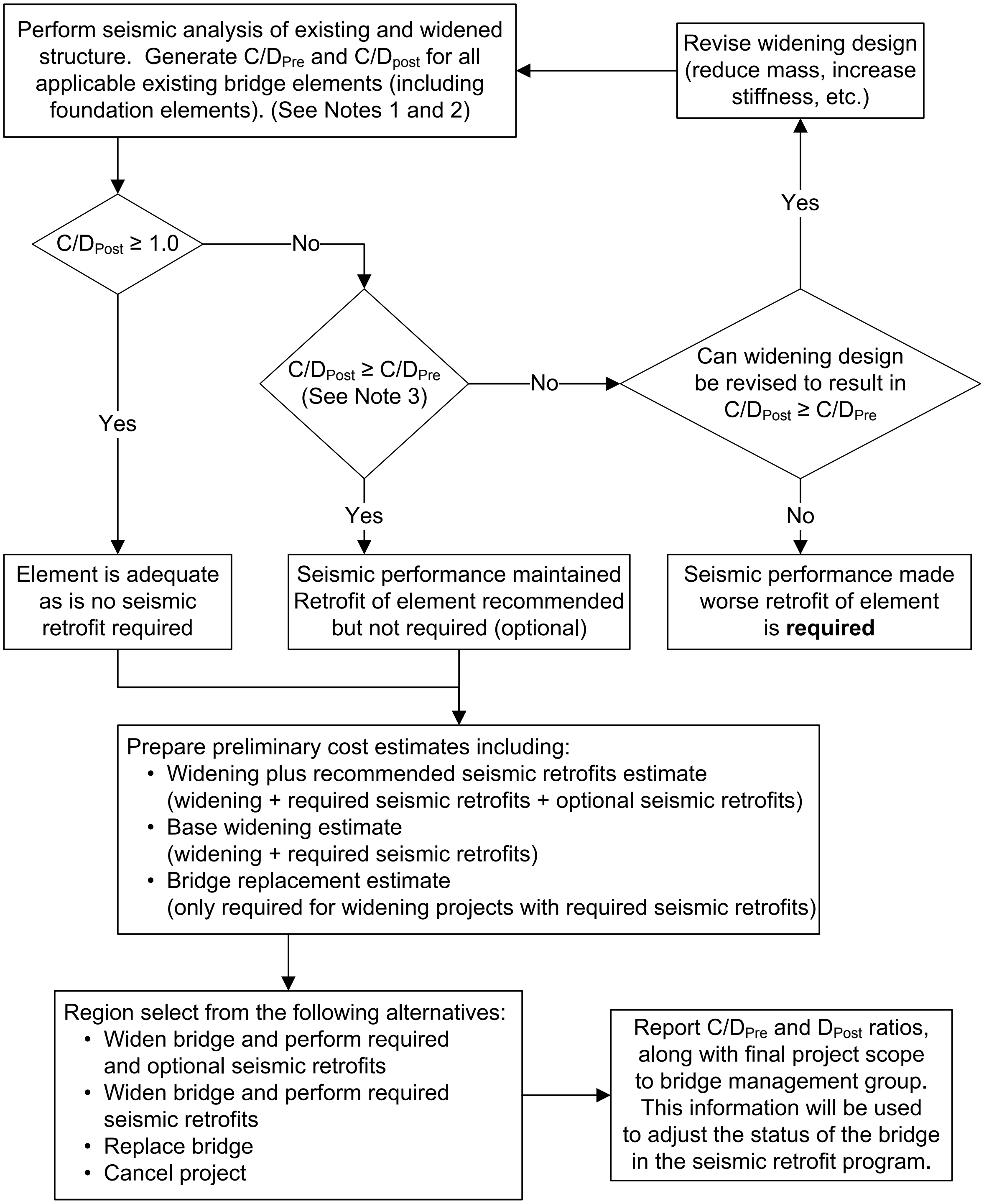
This policy allows bridge widening projects to be completed without addressing existing seismic risks, provided "No Harm" is done to the existing structure. The existing seismic risks are left to be addressed by a bridge seismic retrofit project. This approach maintains the priorities that have been set by the Washington State Legislature. Most widening projects are funded by the I1 - Mobility Program. The objective of the I1-Mobility Program is to improve mobility… not to address seismic risks. Bridge seismic risks are addressed through bridge seismic retrofit projects that are funded as part of the P2 - Structures Preservation Program. The Legislature has established the priority of these and other programs and set funding levels accordingly. This policy upholds the priorities established by the Legislature, by accomplishing widening (mobility) projects without requiring that retrofit (preservation/risk reduction) work be added to the scope, provided the existing structure is not made worse.

Widening elements (new structure) shall be designed to meet current WSDOT standards for new bridges.

A seismic analysis is not required for single-span bridges. However, existing elements of single span bridges shall meet the requirements of *AASHTO Guide Specifications for LRFD Seismic Bridge Design* Section 4.5.

A seismic analysis is not required for bridges in SDC A. However, existing elements of bridges in SDC A shall meet the requirements of *AASHTO Guide Specifications for LRFD Seismic Bridge Design* Section 4.6.

When the addition of the widening has insignificant effects on the existing structure elements, the seismic analysis may be waived with the WSDOT Bridge Design Engineer's approval. In many cases, adding less than 10 percent mass without new substructure could be considered insignificant.



Legend:

C/D_{Pre} = Existing bridge element seismic capacity demand ratio before widening

C/D_{Post} = Existing bridge element seismic capacity demand ratio after widening

Notes:

1. Widening elements (new structure) shall be designed to meet current WSDOT standards for New Bridges.
2. Seismic analysis shall account for substandard details of the existing bridge.
3. C/D ratios are evaluated for each existing bridge element.

WSDOT Seismic Analysis and Retrofit Policy for Bridge Widening Projects
Figure 4.3.1-1

4.3.2 Design and Detailing Considerations

Support Length – The support length at existing abutments, piers, in-span hinges, and pavement seats shall be checked. If there is a need for longitudinal restrainers, transverse restrainers, or additional support length on the existing structure, they shall be included in the widening design.

Connections Between Existing and New Elements – Connections between the new elements and existing elements should be designed for maximum over-strength forces. Where yielding is expected in the crossbeam connection at the extreme event limit state, the new structure shall be designed to carry live loads independently at the Strength I limit state. In cases where large differential settlement and/or a liquefaction-induced loss of bearing strength are expected, the connections may be designed to deflect or hinge in order to isolate the two parts of the structure. Elements subject to inelastic behavior shall be designed and detailed to sustain the expected deformations.

Longitudinal joints between the existing and new structure are not permitted.

Differential Settlement – The geotechnical designer should evaluate the potential for differential settlement between the existing structure and widening structure. Additional geotechnical measures may be required to limit differential settlements to tolerable levels for both static and seismic conditions. The bridge designer shall evaluate, design, and detail all elements of new and existing portions of the widened structure for the differential settlement warranted by the Geotechnical Engineer. Angular distortions between adjacent foundations greater than 0.008 (RAD) in simple spans and 0.004 (RAD) in continuous spans should not be permitted in settlement criteria.

The horizontal displacement of pile and shaft foundations shall be estimated using procedures that consider soil-structure interaction (see *Geotechnical Design Manual* M 46-03 Section 8.12.2.3). Horizontal movement criteria should be established at the top of the foundation based on the tolerance of the structure to lateral movement with consideration of the column length and stiffness. Tolerance of the superstructure to lateral movement will depend on bridge seat widths, bearing type(s), structure type, and load distribution effects.

Foundation Types – The foundation type of the new structure should match that of the existing structure. However, a different type of foundation may be used for the new structure due to geotechnical recommendations or the limited space available between existing and new structures. For example, a shaft foundation may be used in lieu of spread footing.

Existing Strutted Columns – The horizontal strut between existing columns may be removed. The existing columns shall then be analyzed with the new unbraced length and retrofitted if necessary.

Non Structural Element Stiffness – Median barrier and other potentially stiffening elements shall be isolated from the columns to avoid any additional stiffness to the system.

Deformation capacities of existing bridge members that do not meet current detailing standards shall be determined using the provisions of Section 7.8 of the *Retrofitting Manual for Highway Structures: Part 1 – Bridges*, FHWA-HRT-06-032. Deformation capacities of existing bridge members that meet current detailing standards shall be determined using the latest edition of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*.

Joint shear capacities of existing structures shall be checked using Caltrans *Bridge Design Aid*, 14-4 Joint Shear Modeling Guidelines for Existing Structures.

In lieu of specific data, the reinforcement properties provided in Table 4.3.2-1 should be used.

Property	Notation	Bar Size	ASTM A706	ASTM A615 Grade 60	ASTM A615 Grade 40*
Specified minimum yield stress (ksi)	f_y	No. 3 - No. 18	60	60	40
Expected yield stress (ksi)	f_{ye}	No. 3 - No. 18	68	68	48
Expected tensile strength (ksi)	f_{ue}	No. 3 - No. 18	95	95	81
Expected yield strain	ε_{ye}	No. 3 - No. 18	0.0023	0.0023	0.00166
Onset of strain hardening	ε_{sh}	No. 3 - No. 8	0.0150	0.0150	0.0193
		No. 9	0.0125	0.0125	
		No. 10 & No. 11	0.0115	0.0115	
		No. 14	0.0075	0.0075	
		No. 18	0.0050	0.0050	
Reduced ultimate tensile strain	ε_{su}^{R}	No. 4 - No. 10	0.090	0.060	0.090
		No. 11 - No. 18	0.060	0.040	0.060
Ultimate tensile strain	ε_{su}	No. 4 - No. 10	0.120	0.090	0.120
		No. 11 - No. 18	0.090	0.060	0.090

* ASTM A615 Grade 40 is for existing bridges in widening projects.

Stress Properties of Reinforcing Steel Bars
Table 4.3.2-1

Isolation Bearings – Isolation bearings may be used for bridge widening projects to reduce the seismic demand through modification of the dynamic properties of the bridge. These bearings are a viable alternative to strengthening weak elements or non-ductile bridge substructure members of the existing bridge. Isolation bearings shall be designed per the requirement of the AASHTO *Guide Specifications for Seismic Isolation*. Use of isolation bearings needs the approval of WSDOT Bridge Design Engineer.

The decision for using isolation bearings should be made at the early stage of project development based on the complexity of bridge geotechnical and structural design. A cost-benefit analysis comparing design with strengthening weak elements vs. design with isolation bearings shall be performed and submitted for approval. The designer needs to perform two separate designs, one with and one without seismic isolation bearings. The cost-benefit analysis shall at least include:

- Higher initial design time and complexity of analysis.
- Impact of the initial and final design time on the project delivery schedule.
- Time required for preliminary investigation and correspondences with the isolation bearing suppliers.
- Life-cycle cost of additional and specialized and bearing inspections.
- Potential cost impact for bearing and expansion joints replacements.
- Issues related to long-term performance and maintenance.
- Need for large movement expansion joints.

Once approval has been given for the use of seismic isolation bearings, the designer shall send a set of preliminary design and specification requirements to at least three seismic isolation bearing suppliers for evaluation to ensure that they can meet the design and specification requirements. Comments from isolation bearing suppliers should be incorporated before design of structure begins. Sole source isolation bearing supplier may be considered upon Bridge Design Office and Project Engineer's office approval.

The designer shall submit to the isolation bearing suppliers maintenance and inspection requirements with design calculations. Isolation bearing suppliers shall provide maintenance and inspection requirements to ensure the isolators will function properly during the design life and after seismic events. The contract plans shall include bearing replacement methods and details.

4.4 Seismic Retrofitting of Existing Bridges

Seismic retrofitting of existing bridges shall be performed in accordance with the FHWA publication FHWA HRT 06 032, *Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges* and WSDOT amendments as follows:

- Article 7.4.2 Seismic Loading in Two or Three Orthogonal Directions

Revise the first paragraph as follows:

When combining the response of two or three orthogonal directions the design value of any quantity of interest (displacement, bending moment, shear or axial force) shall be obtained by the 100-30 percent combination rule as described in AASHTO *Guide Specifications* Article 4.4.

- Delete Eq. 7.49 and replace with the following:

$$\phi_p = \left(5\left(\frac{V_i - V_m}{V_i - V_f}\right) + 2\right)\phi_y \tag{7.49}$$

- Delete Eq. 7.51 and replace with the following:

$$\phi_p = \left(4\left(\frac{V_{ji} - V_{jh}}{V_{ji} - V_{jf}}\right) + 2\right)\phi_y \tag{7.51}$$

4.4.1 Seismic Analysis Requirements

The first step in retrofitting a bridge is to analyze the existing structure to identify seismically deficient elements. The initial analysis consists of generating capacity/demand ratios for all relevant bridge components. Seismic displacement and force demands shall be determined using the multi-mode spectral analysis of Section 5.4.2.2 (at a minimum). Prescriptive requirements, such as support length, shall be considered a demand and shall be included in the analysis. Seismic capacities shall be determined in accordance with the requirements of the *Seismic Retrofitting Manual*. Displacement capacities shall be determined by the Method D2 – Structure Capacity/Demand (Pushover) Method of Section 5.6. For most WSDOT bridges, the seismic analysis need only be performed for the upper level (1,000 year return period) ground motions with a life safety seismic performance level.

4.4.2 Seismic Retrofit Design

Once seismically deficient bridge elements have been identified, appropriate retrofit measures shall be selected and designed. Table 1-11, Chapters 8, 9, 10, 11, and Appendices D thru F of the *Seismic Retrofitting Manual* shall be used in selecting and designing the seismic retrofit measures. The WSDOT Bridge and Structure Office Seismic Specialist will be consulted in the selection and design of the retrofit measures.

4.4.3 Computer Analysis Verification

The computer results will be verified to ensure accuracy and correctness. The designer should use the following procedures for model verification:

- Using graphics to check the orientation of all nodes, members, supports, joint, and member releases. Make sure that all the structural components and connections correctly model the actual structure.
- Check dead load reactions with hand calculations. The difference should be less than 5 percent.
- Calculate fundamental and subsequent modes by hand and compare results with computer results.
- Check the mode shapes and verify that structure movements are reasonable.
- Increase the number of modes to obtain 90 percent or more mass participation in each direction. GTSTRUDL/SAP2000 directly calculates the percentage of mass participation.
- Check the distribution of lateral forces. Are they consistent with column stiffness? Do small changes in stiffness of certain columns give predictable results?

4.4.4 Earthquake Restrainers

Longitudinal restrainers shall be high strength steel rods conform to ASTM F 1554 Grade 105, including Supplement Requirements S2, S3 and S5. Nuts, and couplers if required, shall conform to ASTM A 563 Grade DH. Washers shall conform to AASHTO M 293. High strength steel rods and associated couplers, nuts and washers shall be galvanized after fabrication in accordance with AASHTO M 232. The length of longitudinal restrainers shall be less than 24 feet.

4.4.5 Isolation Bearings

Isolation bearings may be used for seismic retrofit projects to reduce the demands through modification of the dynamic properties of the bridge as a viable alternative to strengthening weak elements of non ductile bridge substructure members of existing bridge. Use of isolation bearings needs the approval of WSDOT Bridge Design Engineer. Isolation bearings shall be designed per the requirement of the *AASHTO Guide Specifications for Seismic Isolation*.

The decision for using isolation bearings should be made at the early stage of project development based on the complexity of bridge geotechnical and structural design. A cost-benefit analysis comparing design with strengthening weak elements vs. design with isolation bearings shall be performed and submitted for approval. The designer needs to perform two separate designs, one with and one without seismic isolation bearings. The cost-benefit analysis shall at least include:

- Higher initial design time and complexity of analysis.
- Impact of the initial and final design time on the project delivery schedule.
- Time required for preliminary investigation and correspondences with the isolation bearing suppliers.

- Life-cycle cost of additional and specialized bearing inspection.
- Potential cost impact for bearings and expansion joints replacements.
- Issues related to long-term performance and maintenance.
- Need for large movement expansion joints.

Once approval has been given for the use of seismic isolation bearing, the designer shall send a set of preliminary design and specification requirements to at least three seismic isolation bearing suppliers for evaluation to ensure that they can meet the design and specification requirements. Comments from isolation bearing suppliers should be incorporated before design of structure begins. Sole source isolation bearing supplier may be considered upon Bridge Design Office and Project Engineer's office approval.

The designer shall submit to the isolation bearing suppliers maintenance and inspection requirements with design calculations. Isolation bearing suppliers shall provide maintenance and inspection requirements to ensure the isolators will function properly during the design life and after seismic events. The contract plans shall include bearing replacement methods and details.

4.99 References

AASHTO *LRFD Bridge Design Specifications*, 6th Edition, 2012

AASHTO *Guide Specifications for LRFD Seismic Bridge Design*, 2nd Edition, 2011

AASHTO *Gudie Specifications for Seismic Isolation Design*, 3rd Edition, 2010

Caltrans *Bridge Design Aids* 14 4 Joint Shear Modeling Guidelines for Existing Structures, California Department of Transportation, August 2008

FHWA *Seismic Retrofitting Manual for Highway Structures: Part 1 Bridges*, Publication No. FHWA HRT 06 032, January 2006

McLean, D.I. and Smith, C.L., *Noncontact Lap Splices in Bridge Column-Shaft Connections*, Report Nunber WA-RD 417.1, Washington State University

WSDOT *Geotechnical Design Manual* M 46-03, Environmental and Engineering Program, Geotechnical Services, Washington State Department of Transportation

Chapter 5 Concrete Structures — Contents

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

$$\frac{x}{70} = \frac{100}{64} \quad Therefore, x = 110\% \qquad (5.1.1\text{-}1)$$

   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.

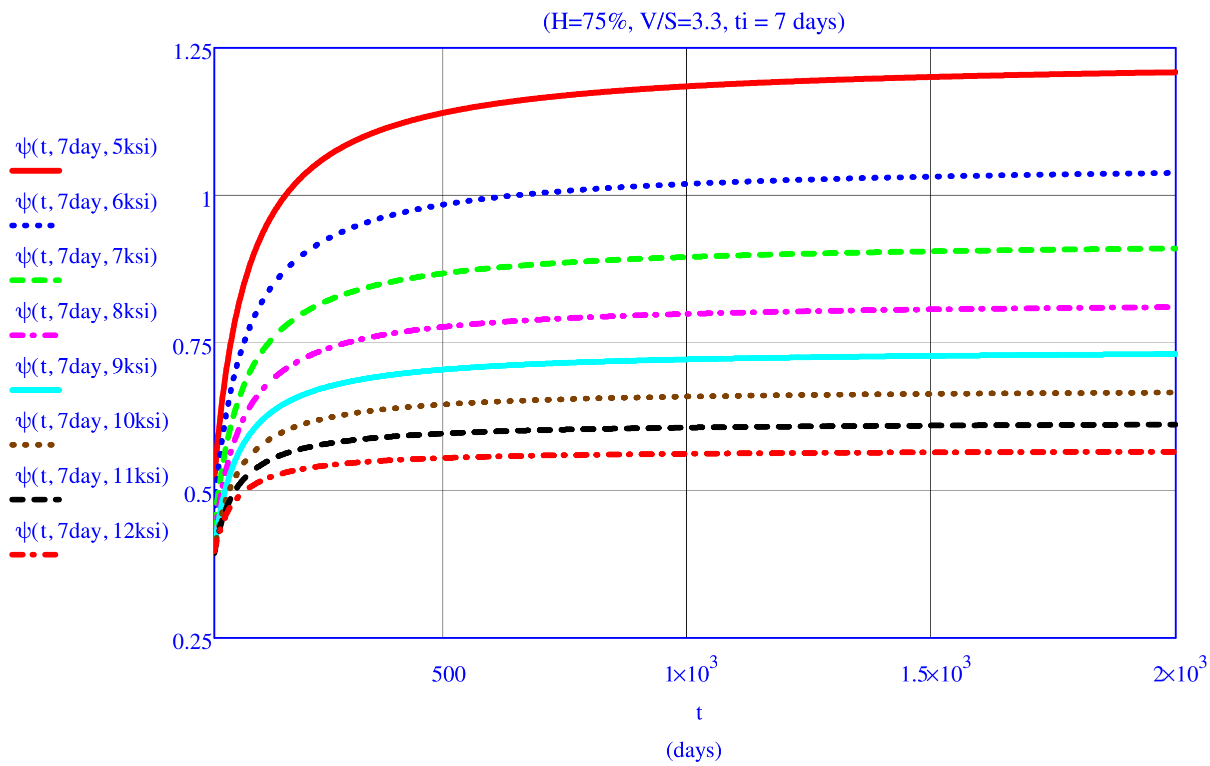
E. **Creep** – The creep coefficient shall be calculated per AASHTO LRFD 5.4.2.3.2. The relative humidity, *H*, may be taken as 75 percent for standard conditions. The maturity of concrete, *t*, may be taken as 2,000 days for standard conditions. The volume-to-surface ratio, *V/S*, is given in Table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

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

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) may be used in structural members such as precast noise wall panels, barriers, three-sided structures, etc. as described in *Standard Specifications* Section 6-02.3(27).

SCC may be used in prestressed concrete girders.

SCC may be specified for cast-in-place applications where the use of conventional concrete could be challenging and problematic. Examples are where new concrete is being cast up against an existing soffit, or in members with very dense/congested reinforcing steel. Use of SCC for primary structural components such as columns, crossbeams, slabs, etc. requires the approval of the WSDOT Bridge Design Engineer.

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:

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

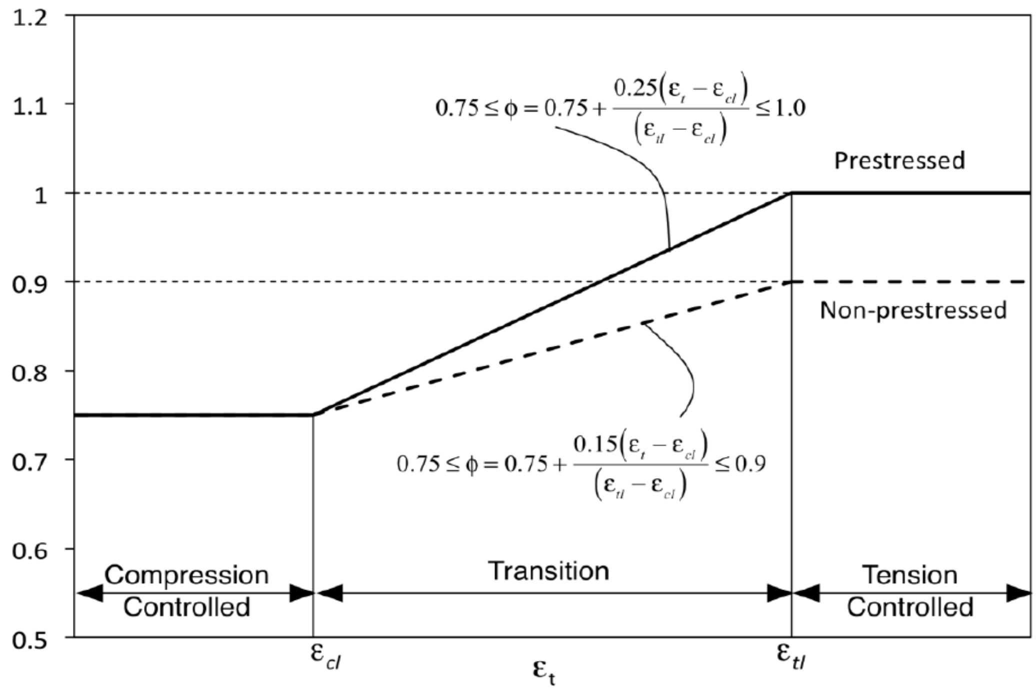
      and for nonprestressed members such that:

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

      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 ε_{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 ε_{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.

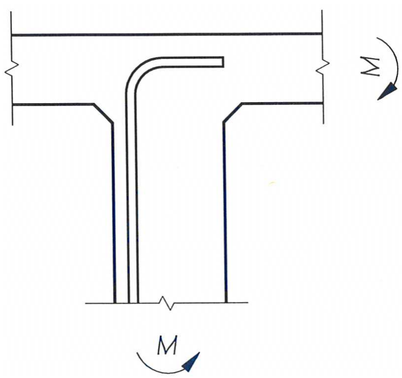
G. **Placement** – Placement of reinforcing bars can be a problem during construction. Sometimes it may be necessary to make a large scale drawing of reinforcement to look for interference and placement problems in confined areas. If interference is expected, additional details are required in the contract 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**

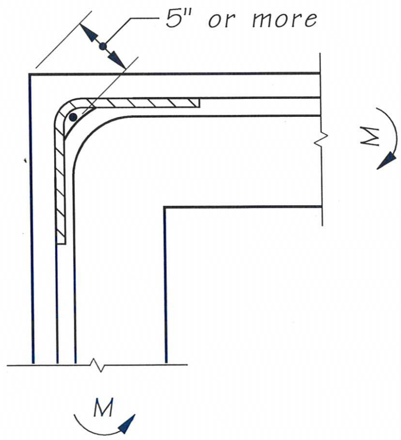
1. **T-Joint** – The forces form a tension crack at 45° in the joint. Reinforcement as shown in Figure 5.1.2-2 is more than twice as effective in developing the strength of the corner than if the reinforcement was turned 180°.
2. **"Normal" Right Corners** – Corners subjected to bending as shown in Figure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

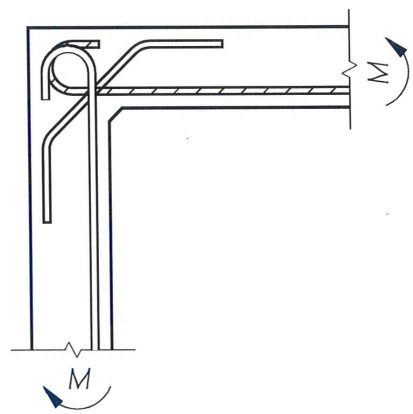
   Adding diagonal reinforcing steel across the corner, approximately equal to 50 percent of the main reinforcing steel, will develop the corner strength to fully resist the applied moment. Extend the diagonal reinforcement past the corner each direction for anchorage. Since this bar arrangement will fully 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 designer shall calculate the number of extended straight strands needed to develop the required capacity at the end of the girder. <u>The number of extended strands shall not be less than four.</u>

   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, where some girders are outside the effective widths and the effective widths for each column do not overlap, 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

For precast, prestressed girders with cast-in-place deck slabs, where all girders are within the effective width or the effective widths for each column overlap, the plastic hinging moment at the c.g. of the superstructure shall be resisted by all girders within the effective width. The plastic hinging moment per girder is calculated using the following:

$$M_{sei} = \frac{M_{po}^{CG}}{N_g^{int}} \tag{5.1.3-7}$$

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

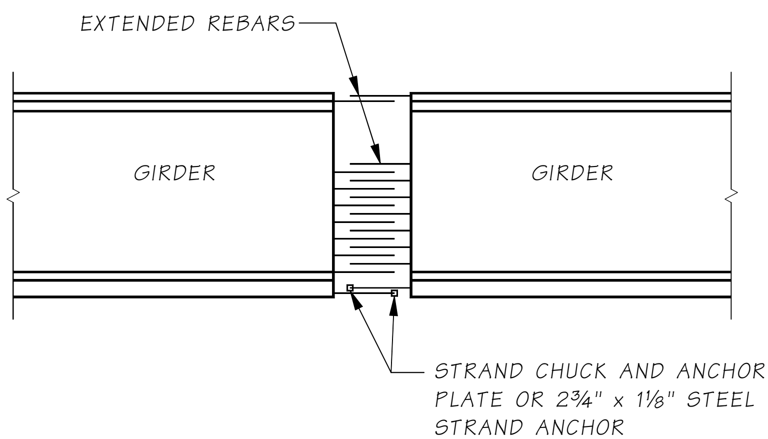
$$B_{eff} = D_c + D_s \tag{5.1.3-8}$$

Where:

D_c = Diameter or width of column, see Figure 5.1.3-3

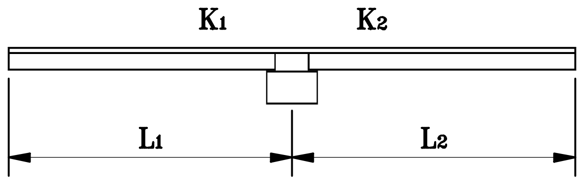
D_s = Depth of superstructure from top of column to top of deck slab, see Figure 5.1.3-3

See Appendix 5-B10 for a design example.

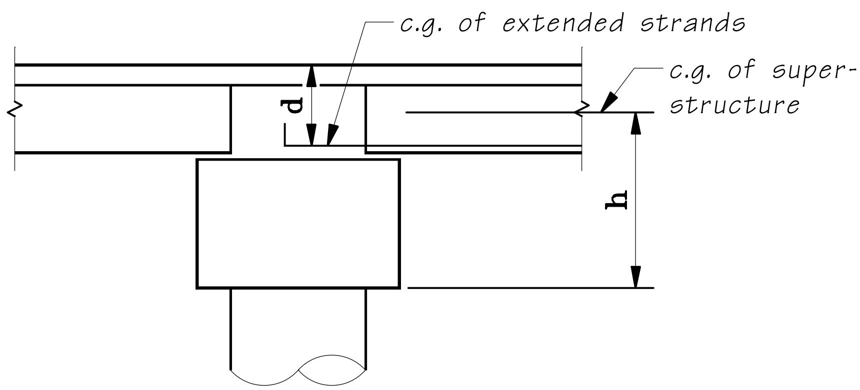


Strand Development

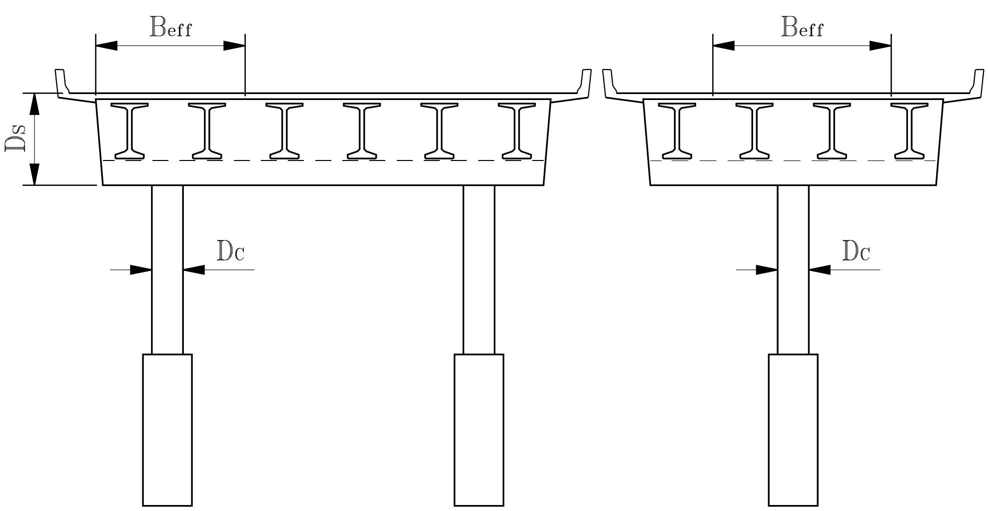
Figure 5.1.3-1



$$K_1 = \frac{L_2}{L_1 + L_2} \qquad K_2 = \frac{L_1}{L_1 + L_2}$$



Extended Strand Design
Figure 5.1.3-2



Effective Superstructure Width for Extended Strand Design
Figure 5.1.3-3

Continuity of extended strands is essential for all prestressed girders bridges with fixed diaphragms at intermediate piers. Strand continuity may be achieved by directly overlapping extended strands as shown in Figure 5.1.3-4, by use of strand ties as shown in Figure 5.1.3-5, by the use of the crossbeam ties as shown in Figure 5.1.3-6 along with strand ties, or by a combination of all three methods. The following methods in order of hierarchy shall be used for all precast girders for creating continuity of extended strands:

Method 1 – Direct extended strands overlapping shall be used at intermediate piers without any angle point due to horizontal curvature and for any crossbeam width. This is the preferred method of achieving extended strand continuity. Congestion of reinforcement and girder setting constructability shall be considered when large numbers of extended strands are required. In these cases, strand ties may be used in conjunction with extended strands.

Method 2 – Strand ties shall be used at intermediate piers with a girder angle point due to horizontal curvature where extended strands are not parallel and would cross during girder placement. Crossbeam widths shall be greater than or equal to 6 ft measured along the skew. It is preferable that strand ties be used for all extended strands, however if the region becomes too congested for rebar placement and concrete consolidation, additional forces may be carried by crossbeam ties up to a maximum limit as specified in equation 5.1.3-8.

Method 3 – For crossbeams with widths less than 6' and a girder angle point due to horizontal curvature, strand ties shall be used if a minimum of 8" of lap can be provided between the extended strand and strand tie. In this case the strand ties shall be considered fully effective. For cases where less than 8" of lap is provided, the effectiveness of the strand tie shall be reduced proportional to the reduction in lap. All additional forces not taken by strand ties must be carried by crossbeam ties up to the maximum limit as specified in equation 5.1.3-8. If this limit is exceeded, the geometry of the width of the crossbeam shall be increased to provide sufficient lap for the strand ties.

The area of transverse ties considered effective for strand ties development in the lower crossbeam (A_s) shall not exceed:

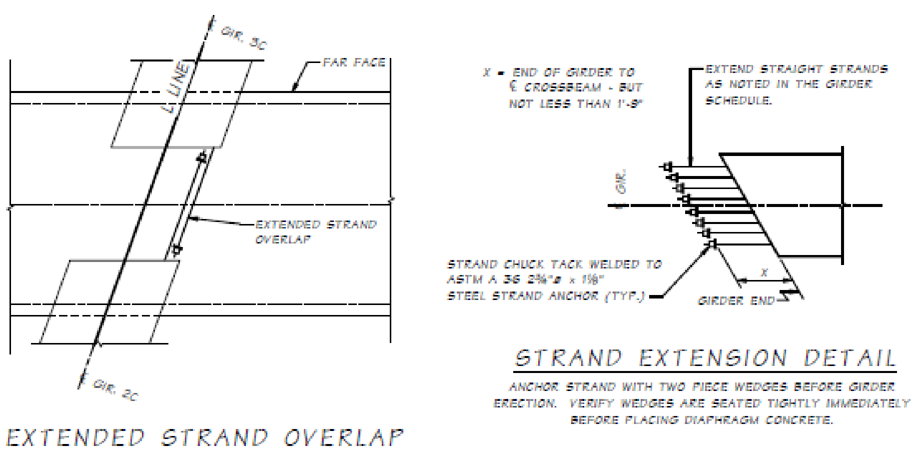
$$A_s = \frac{1}{2}\left(\frac{A_{ps} f_{py} n_s}{f_{ye}}\right) \tag{5.1.3-9}$$

Where:

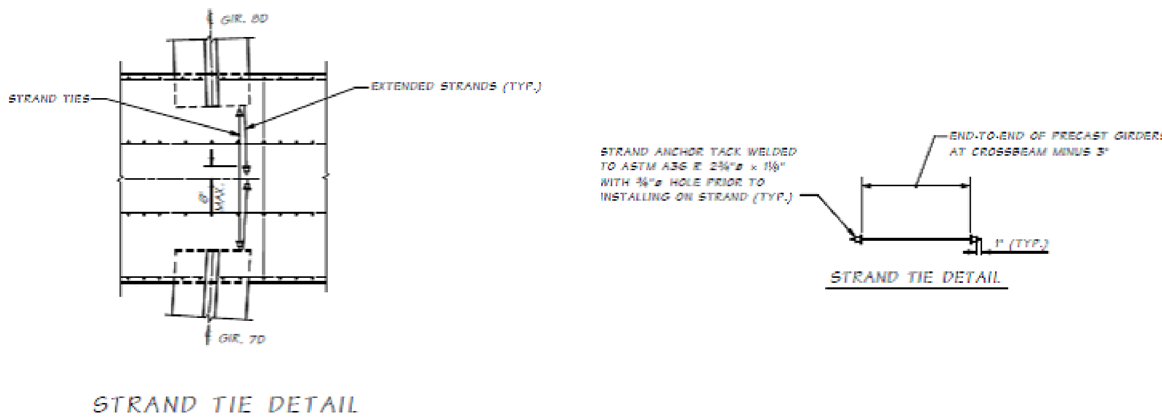
A_{ps} = Area of strand ties, in^2
f_{py} = Yield strength of extended strands, ksi
n_s = Number of extended strands that are spliced with strand and crossbeam ties
f_{ye} = Expected yield strength of transverse tie reinforcement, ksi

Two-thirds of A_s shall be placed directly below the girder and the remainder of A_s shall be placed outside the bottom flange width as shown in Figure 5.1.3-6.

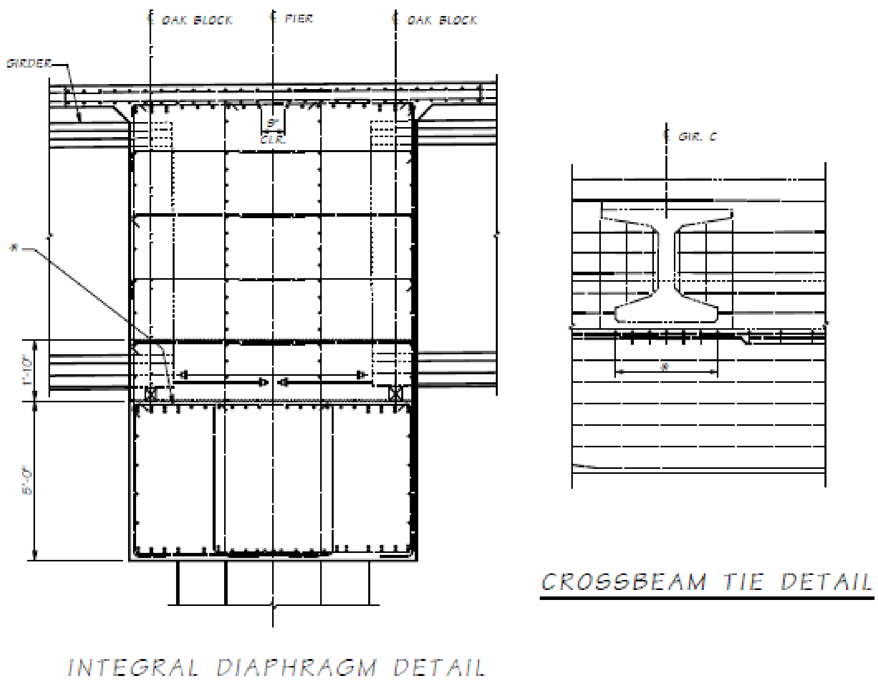
The size of strand ties shall be the same as the extended strands, and shall be placed at the same level and proximity of the extended strands.



Overlapping Extended Strand
Figure 5.1.3-4



Strand Ties
Figure 5.1.3-5



Lower Crossbeam Ties
Figure 5.1.3-6

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**

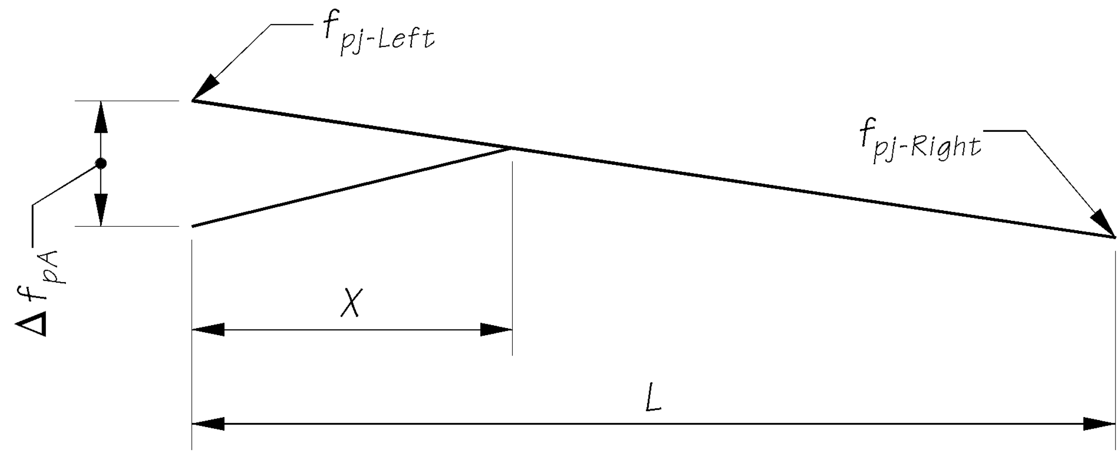
1. **Elastic Shortening of Concrete** – Transfer of prestress forces into the girder ends results in an instantaneous elastic loss. The prestress loss due to elastic shortening shall be added to the time dependent losses to determine the total losses. The loss due to elastic shortening shall be taken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

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

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



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.

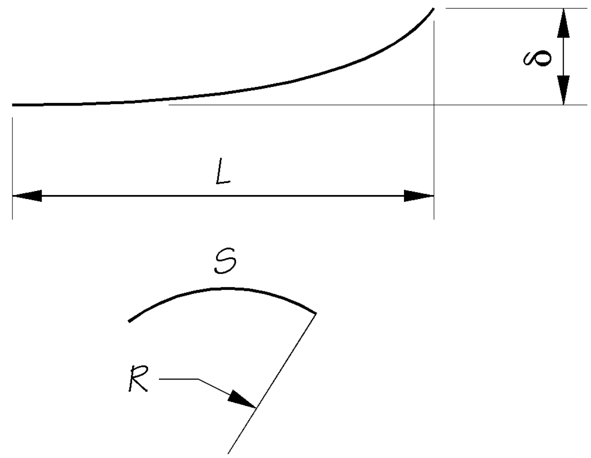
$$\Delta f_{pF} = f_{pj}\left(1 - e^{-(kx+\mu\alpha)}\right) \tag{5.1.4-3}$$

   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.

$$\alpha = \sqrt{(\alpha_H)^2 + (\alpha_V)^2}$$

where: $$\alpha_V = \frac{2\delta}{L}$$

$$\alpha_H = \frac{S}{R}$$



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

$$f_{pe} = f_{pj} - \Delta f_{pT} - \Delta f_{pES} - \Delta f_{pED} - \Delta f_{pSS} \quad (5.1.4\text{-}4)$$

The total prestress loss may be estimated as:

$$\Delta f_{pT} = \Delta f_{pRO} + \Delta f_{pLT} \quad (5.1.4\text{-}5)$$

Initial relaxation that occurs between the time of strand stressing and prestress transfer may be estimated as:

$$\Delta f_{pRO} = \frac{\log(24t)}{40}\left(\frac{f_{pj}}{f_{py}} - 0.55\right) f_{pj} \quad (5.1.4\text{-}6)$$

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

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. Elastic gain due to deck shrinkage shall be considered separately.

Elastic shortening, Δf_{pES}, is computed in accordance with AASHTO LRFD 5.9.5.2.3a.

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

$$\Delta f_{pED} = \frac{E_p}{E_c}\left[-\frac{(M_{slab}+M_{diaphragms})e_{ps}}{I_g} - \frac{(M_{sidl}+\gamma_{LL}M_{LL+IM})(Y_{bc}-Y_{bg}+e_{ps})}{I_c}\right] \quad (5.1.4\text{-}7)$$

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
M_{LL+IM} = Moment caused by live load and dynamic load allowance
γ_{LL} = Live load factor (1.0 for Service I and 0.8 for Service III)
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

The elastic gain due to slab shrinkage, Δf_{pSS}, shall be computed in accordance with AASHTO LRFD 5.9.5.4.3d. Deck shrinkage shall be considered as an external force applied to the composite section for the Service I, Service III, and Fatigue I limit states. This force is applied at the center of the deck with an eccentricity from the center of the deck to the center of gravity of the composite section. This force causes compression in the top of the girder, tension in the bottom of the girder, and an increase in the effective prestress force (an elastic gain). The deck shrinkage strain shall be computed as 50% of the strain determined by AASHTO LRFD Equation 5.4.2.3.3-1.

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:

$$\Delta f_{pTH} = \Delta f_{pR0} + \Delta f_{pES} + \Delta f_{pH} \quad (5.1.4\text{-}8)$$

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.

5.6 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

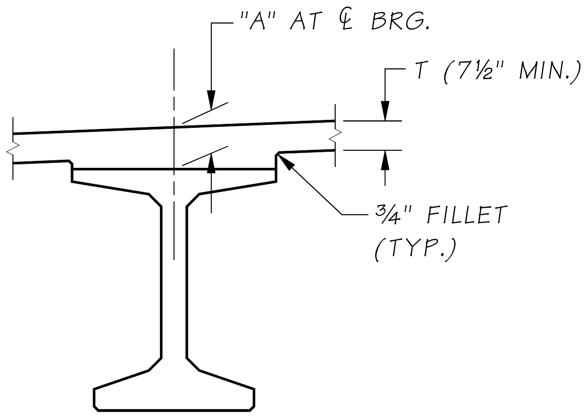
WSDOT design criteria for precast prestressed girder superstructures are given in Table 5.6.2-1.

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.

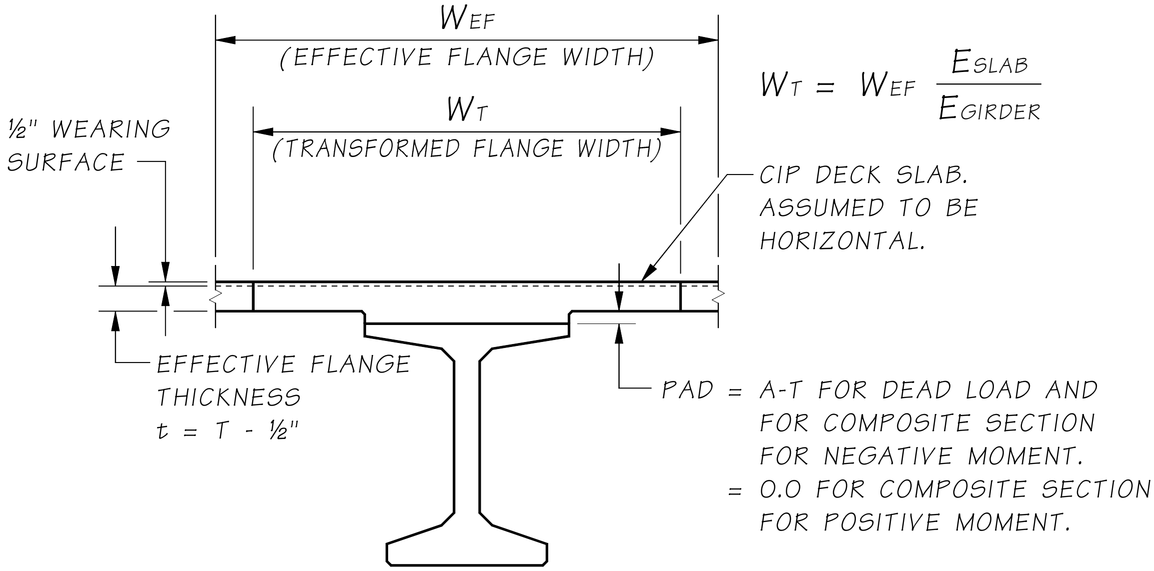
Design Specifications	AASHTO LRFD Specifications and WSDOT *Bridge Design Manual* M 23-50
Design Method	Precast, prestressed members shall be designed for service limit state for allowable stresses and checked for strength limit state for ultimate capacity.
Superstructure Continuity	Precast, prestressed 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.



SECTION AS DETAILED



SECTION FOR COMPUTATION OF COMPOSITE SECTION PROPERTIES

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**

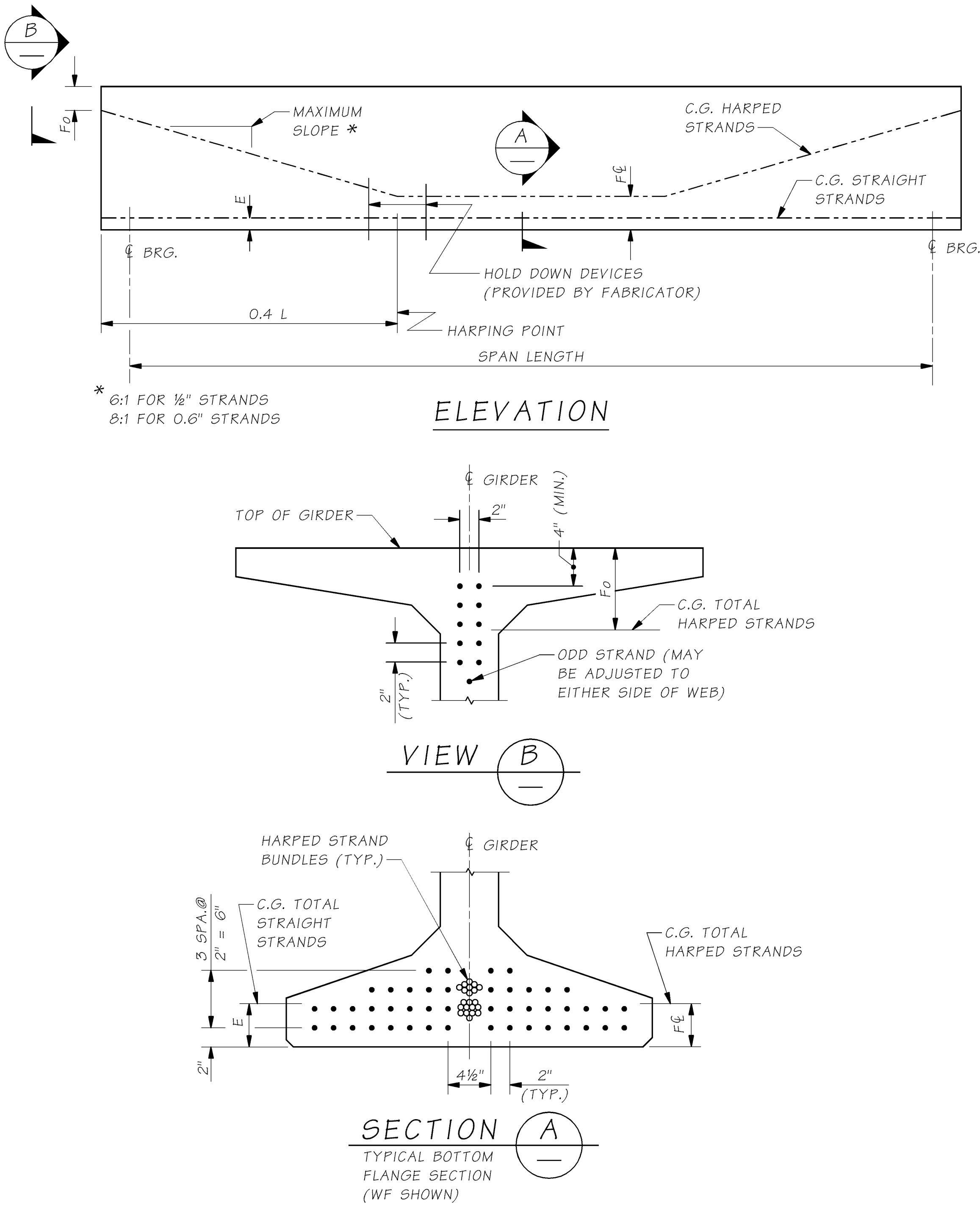
1. **General –** The WSDOT Prestressed Girder Design computer program PGSuper is the preferred method for final design.

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.



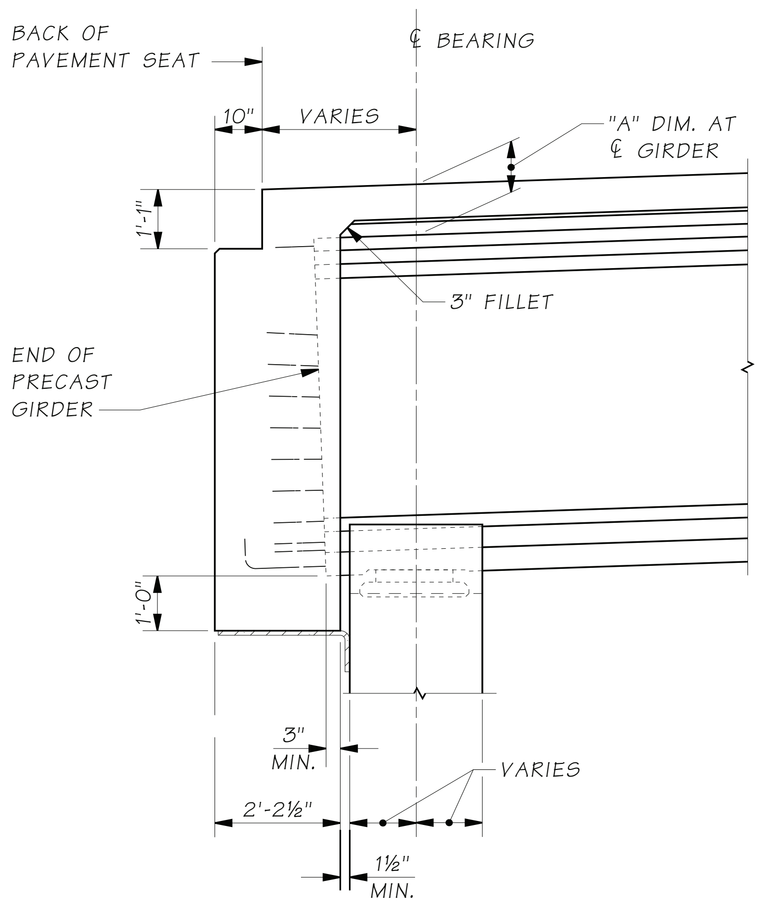
Typical Prestressed Girder Configuration
Figure 5.6.2-2

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.

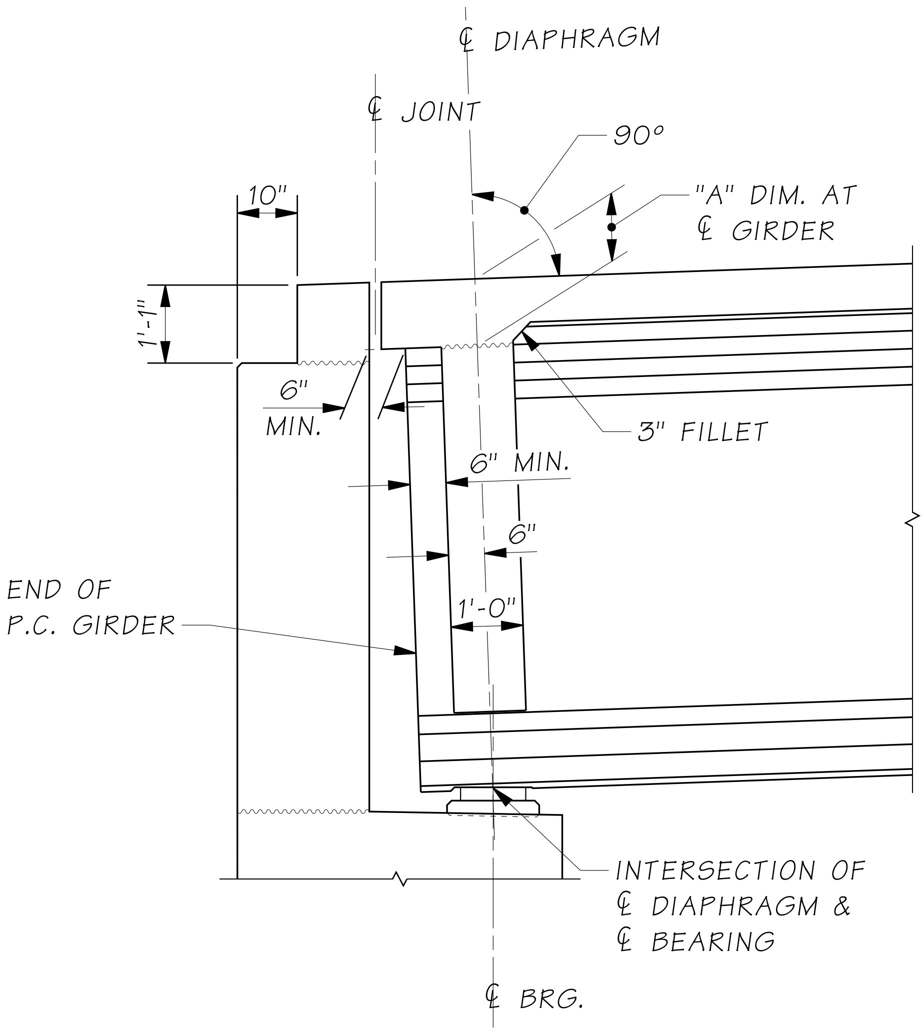
   The gap between the end diaphragm and the stem wall shall be a minimum of 1½″ or ½″ greater than required for longitudinal bridge movement.



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

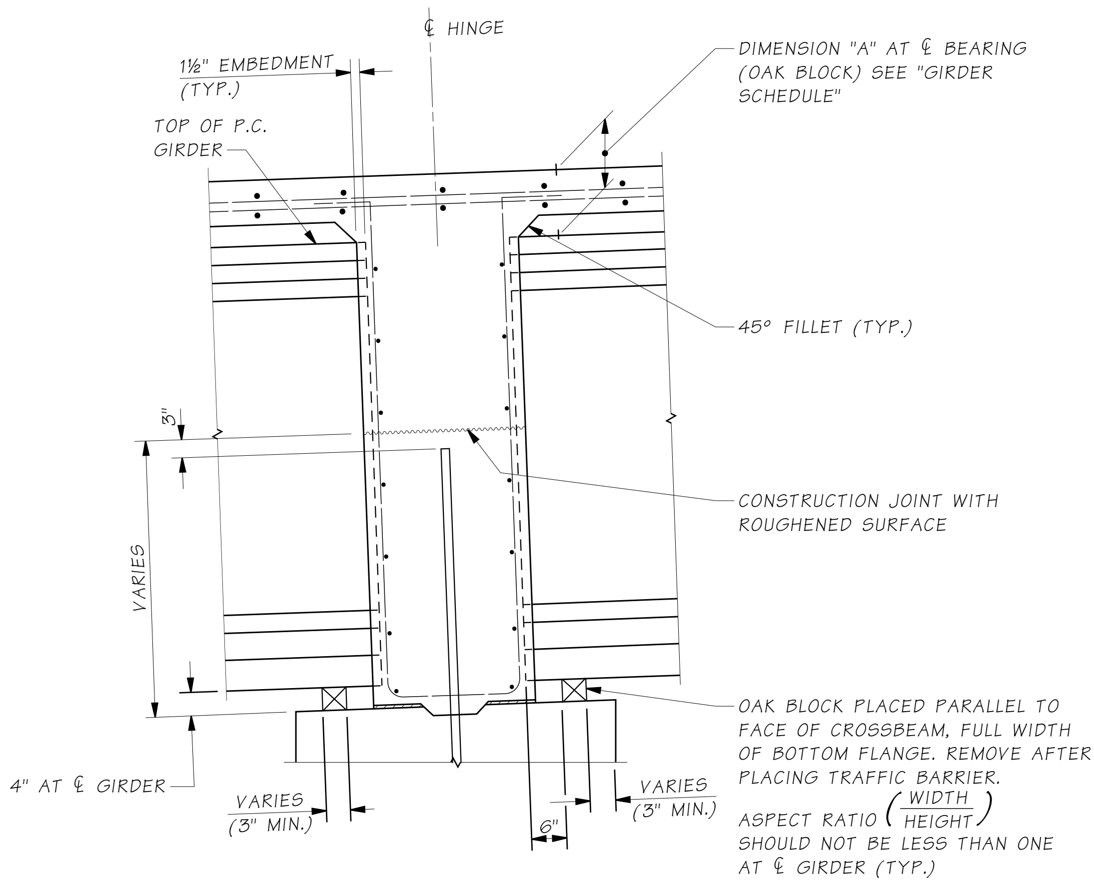
3. **End Type C –** End Type C as shown in Figure 5.6.2-5 is for continuous spans and an intermediate hinge diaphragm at an intermediate pier. There is no bearing recess and the girder is temporarily supported on oak blocks. This detail is generally used only in low seismic areas such as east of the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

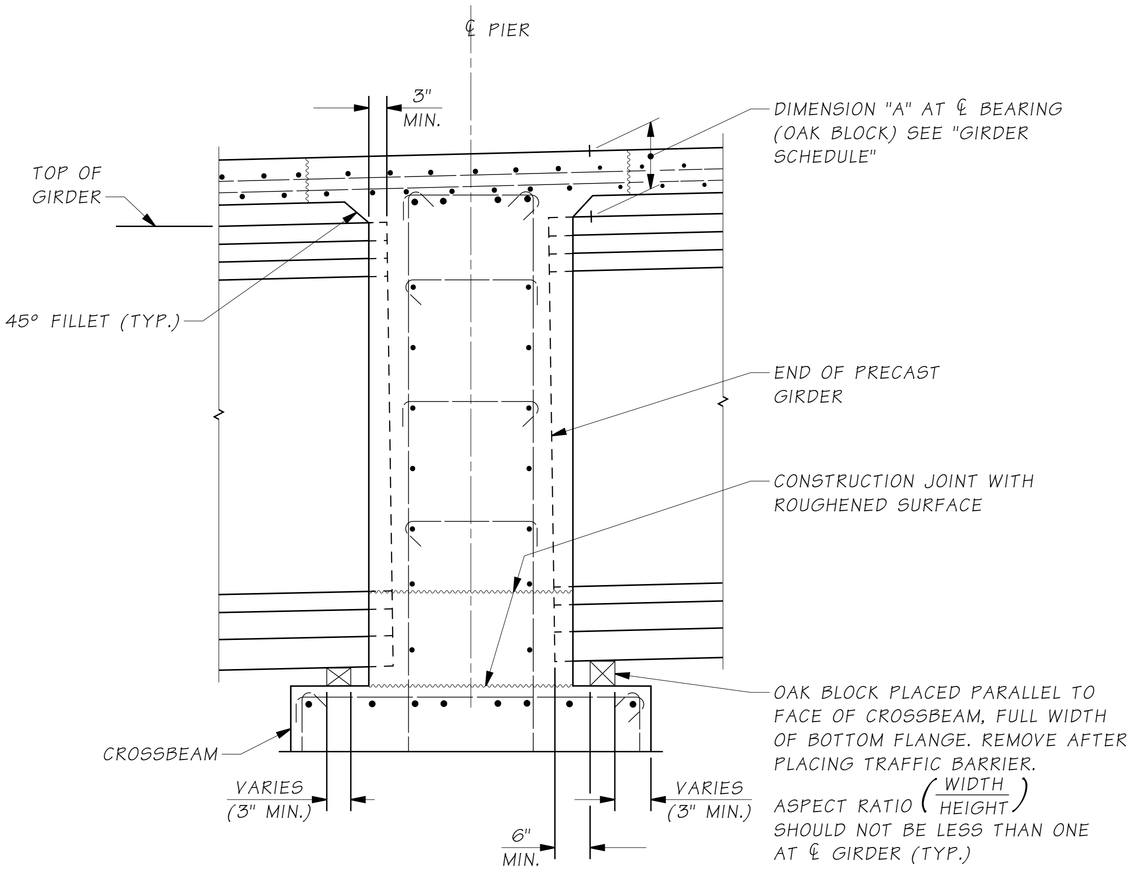
   b. Check interface shear friction at girder end (see Section 5.2.2.C.2).



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:

- $\gamma_{LL} = 0.8$ when gross section properties are used
- $\gamma_{LL} = 1.0$ 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.

   Inserts may be used to accommodate the construction of intermediate diaphragms for connections between the diaphragm and the web of precast girders. The designer shall investigate the adequacy of the insert and the connection to develop the tensile capacity of diaphragm reinforcement. The designer shall also investigate the interface shear capacity of the diaphragm-to-web connections for construction and deck placement loads.

   Open holes should be provided for interior webs so through reinforcement can be placed.

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-shaped (including WF, deck bulb tee, etc.) 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.
5. **Tub Girder Intermediate Diaphragms** – Intermediate diaphragms shall be provided both inside and between prestressed concrete tub girders.

   The diaphragms inside the tub may be cast in the field or at the fabrication plant. The bottom of the diaphragm inside the tub shall be at least 3 inches above the top of the bottom flange.

   The diaphragms between the tubs shall be cast in the field. For diaphragms between the tubs, the roughened surface or shear keys on the sloped web faces may not be effective in resisting interface shear. All diaphragm and construction loads on the diaphragm before the deck cures and gains strength will then be resisted by the reinforcement or inserts alone.

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

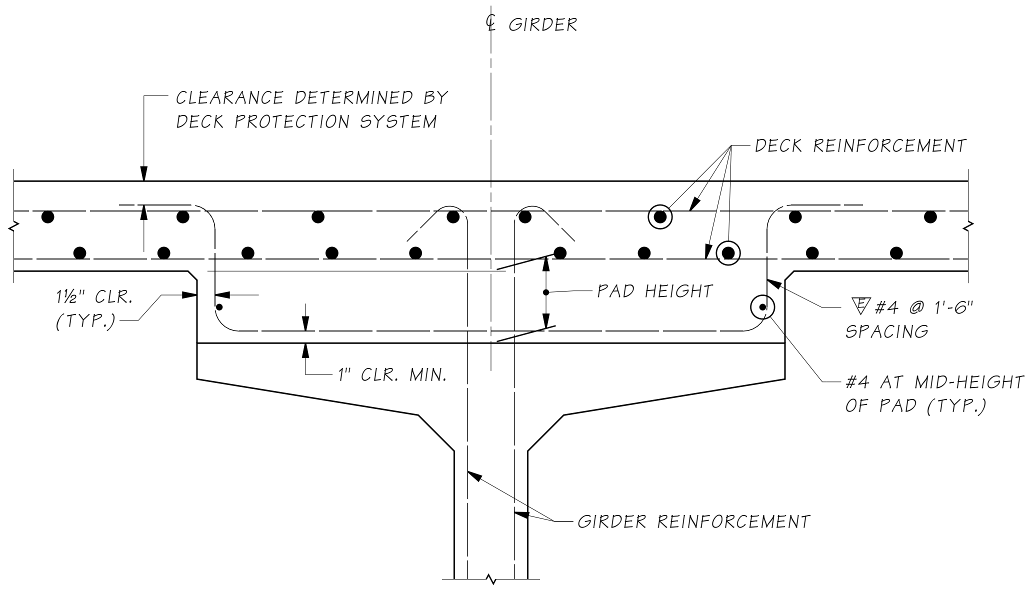
1. **Analysis** – Normally, the effect of skew on girder analysis is ignored. It is assumed that skew has little structural effect on normal spans and normal skews. For short, wide spans and for extreme skews (values over 45°), the effect of the skew on structural action shall be investigated. All trapezoidal 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.

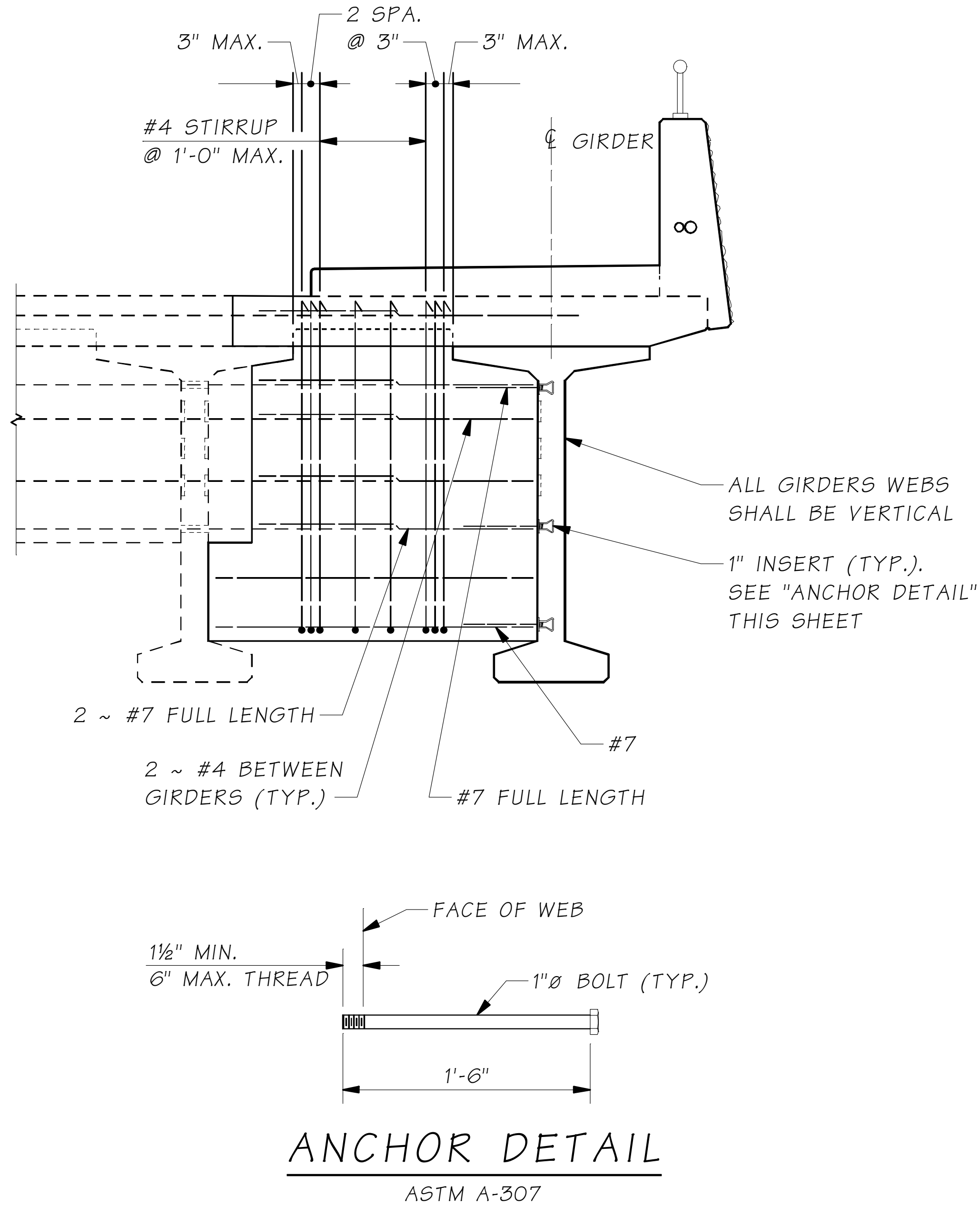
Methods of construction shall be specified in the plans that will minimize inconvenience and dangers to the public while achieving a satisfactory structural result. High early strength grouts and concretes should be considered.

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.

Drain holes shall be provided at the low point of the tub girders at the centerline of the bottom flange.

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.

5.7 Deck Slabs

Concrete deck slabs shall be designed using the Traditional Design of AASHTO LRFD 9.7.3 as modified by this section.

The following information is intended to provide guidance for deck slab thickness and transverse and longitudinal reinforcement of deck slabs. Information on deck protection systems is given in Section 5.7.4.

5.7.1 Deck Slab Requirements

A. **Minimum Deck Slab Thickness** – The minimum deck slab thickness (including 0.5″ wearing surface) shall be 7.5″ for concrete bridges, 8.0″ for steel girder bridges, and 8.5″ for concrete girder bridges with SIP deck panels. This minimum deck slab thickness may be reduced by 0.5″ for bridges with Deck Protection Systems 2, 3 and 5.

   The minimum CIP deck slab thickness for precast slab girders is 5″.

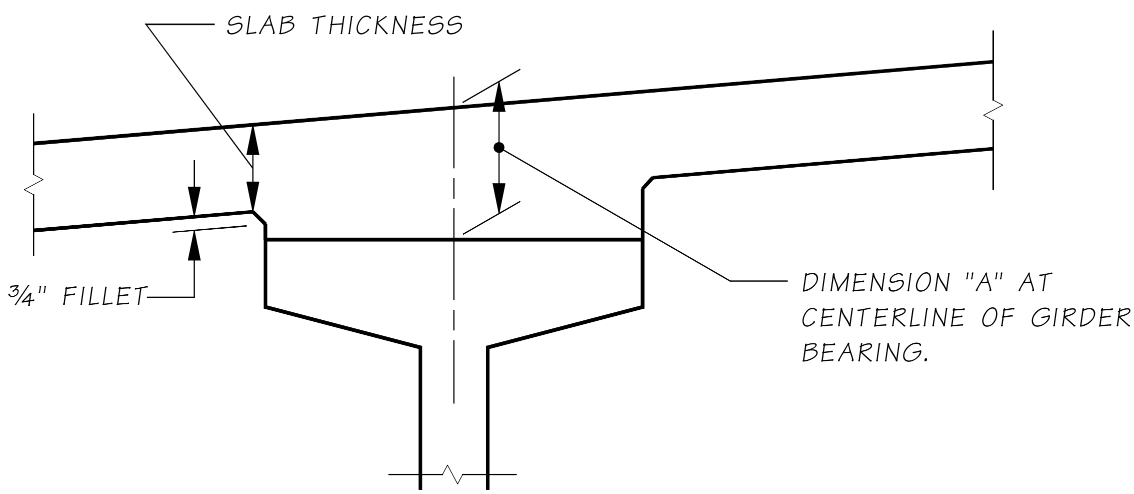
   Minimum slab thicknesses are established in order to ensure that overloads will not result in premature deck slab cracking.

   The minimum clearance between top and bottom reinforcing mats shall be 1″.

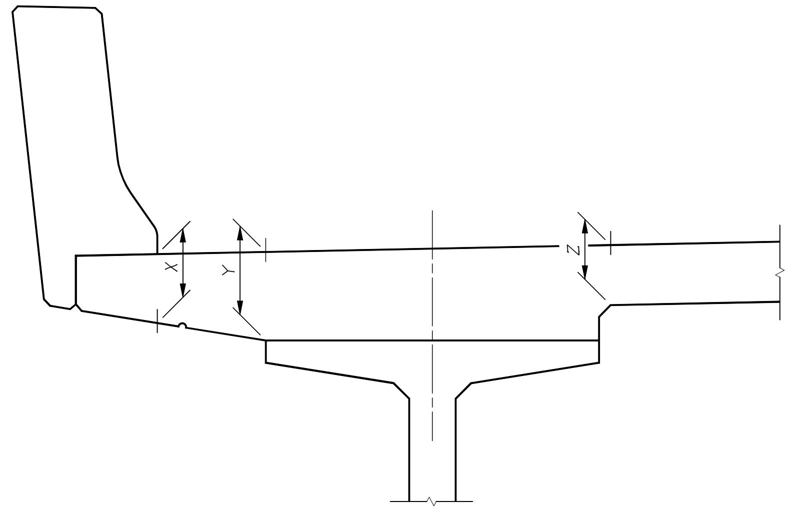
B. **Computation of Deck Slab Strength** – The design thickness for usual deck slabs are shown in Figures 5.7.1-1 & 2.

   The thickness of the deck slab and reinforcement in the area of the cantilever may be governed by traffic barrier loading. Wheel loads plus dead load shall be resisted by the sections shown in Figure 5.7.1-2.

   Design of the cantilever is normally based on the expected depth of deck slab at centerline of girder span. This is usually less than the dimensions at the girder ends.



Depths for Deck Slab Design at Interior Girder
Figure 5.7.1-1



Depths for Deck Slab Design at Overhang
Figure 5.7.1-2

C. **Computation of "A" Dimension** – The distance from the top of the deck slab to the top of the girder at centerline bearing at centerline of girder is represented by the "A" Dimension. It is calculated in accordance with the guidance of Appendix 5-B1. This ensures that adequate allowance will be made for excess camber, transverse deck slopes, vertical and horizontal curvatures. Where temporary prestress strands at top of girder are used to control the girder stresses due to shipping and handling, the "A" dimension must be adjusted accordingly.

The note in the left margin of the layout sheet shall read: "A" Dimension = X" (not for design).

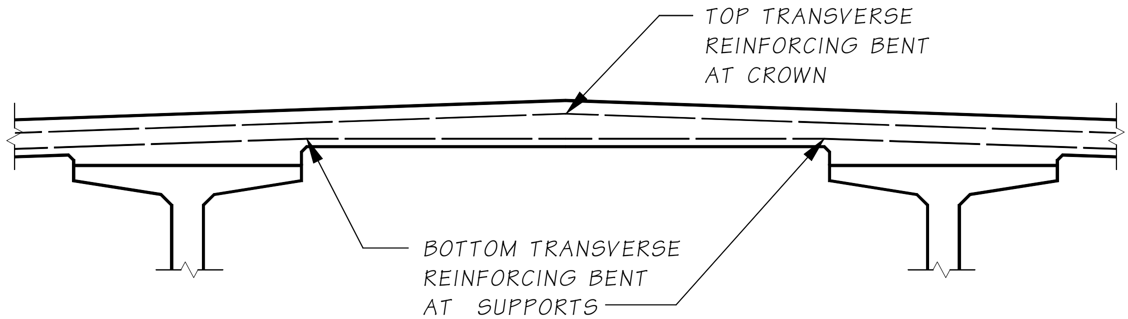
5.7.2 Deck Slab Reinforcement

A. **Transverse Reinforcement** – The size and spacing of transverse reinforcement may be governed by interior deck slab span design and cantilever design. Where cantilever design governs, short hooked bars may be added at the deck slab edge to increase the reinforcement available in that area. Top transverse reinforcement is always hooked at the deck slab edge unless a traffic barrier is not used. Top transverse reinforcement is preferably spliced at some point between girders in order to allow the clearance of the hooks to the deck slab edge forms to be properly adjusted in the field. Usually, the deck slab edge hooks will need to be tilted in order to place them. On larger bars, the clearance for the longitudinal bar through the hooks shall be checked. Appendices 5.3-A5 through 5.3-A8 can be used to aid in selection of bar size and spacing.

For skewed spans, the transverse bars are placed normal to bridge centerline and the areas near the expansion joints and bridge ends are reinforced by partial length bars. For raised crossbeam bridges, the bottom transverse deck slab reinforcement is discontinued at the crossbeam.

The spacing of bars over the crossbeam must be detailed to be large enough to allow concrete to be poured into the crossbeam. For typical requirements, see Section 5.3.3.D.

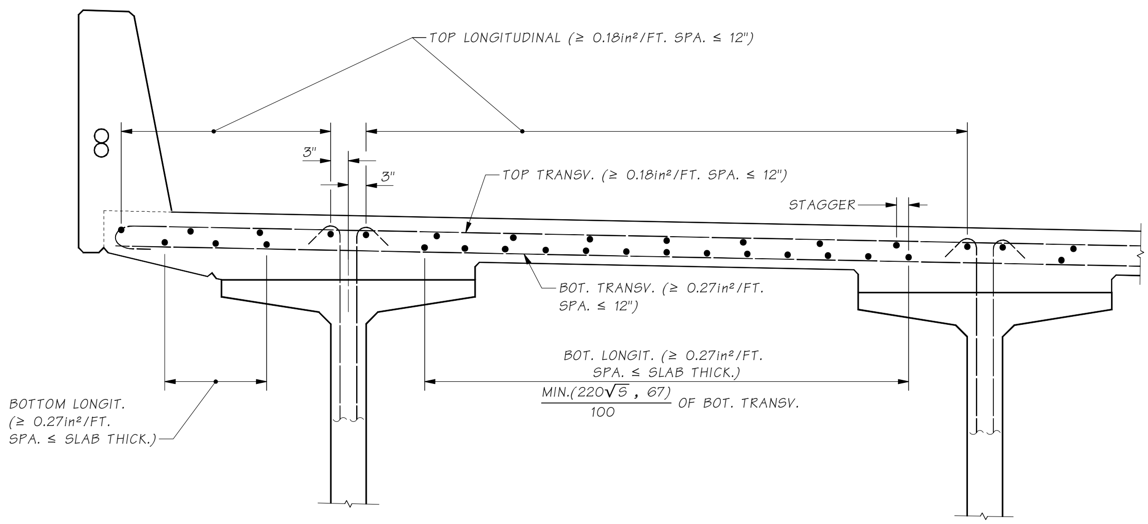
For deck slabs with a crowned roadway, the bottom surface and rebar shall be flat, as shown in Figure 5.7.2-1.



Bottom of Deck Slab at Crown Point
Figure 5.7.2-1

B. **Longitudinal Reinforcement** – This section discusses reinforcement requirements for resistance of longitudinal moments in continuous multi-span precast girder bridges and is limited to reinforcement in the deck slab since capacity for resisting positive moment is provided by the girder reinforcement.

1. **Simple Spans** – For simple span bridges, longitudinal deck slab reinforcement is not required to resist negative moments and therefore the reinforcement requirements are nominal. Figure 5.7.2-2 defines longitudinal reinforcement requirements for these slabs. The bottom longitudinal reinforcement is defined by AASHTO LRFD 9.7.3.2 requirements for distribution reinforcement. The top longitudinal reinforcement is based on current office practice.



Nominal Longitudinal Deck Slab Reinforcement
Figure 5.7.2-2

2. **Continuous Spans** – Continuity reinforcement shall be provided at supports for loads applied after establishing continuity. The longitudinal reinforcement in the deck slab at intermediate piers is dominated by the negative moment requirement. Where these bars are cut off, they are lapped by the nominal top longitudinal reinforcement described in Section 5.7.2.D. The required deck slab thickness for various bar combinations is shown in Table 5.7.2-1.

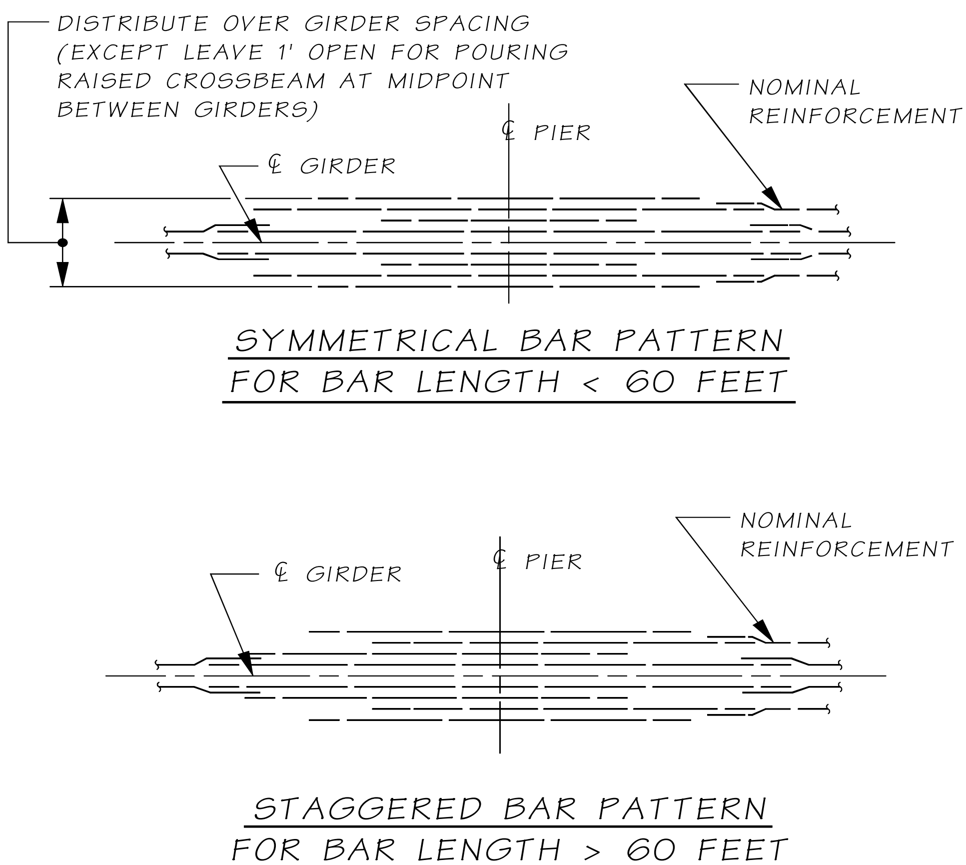
C. **Distribution of Flexural Reinforcement** – The provision of AASHTO LRFD 5.7.3.4 for class 2 exposure condition shall be satisfied for both the top and bottom faces of the deck slab.

	Minimum Deck Slab Thickness (Inches) Transverse Bar		
Longitudinal Bar	**#5**	**#6**	**#7**
#4	7½	--	--
#5	7½	7½	7¾
#6	7½	7¾	8
#7	7¾	8	8¼
#8	8	8½	8¾
#9	8½	8¾	9
#10	8¾	--	--

Note:
Deduct ½″ from minimum deck slab thickness shown in table when an overlay is used.

Minimum Deck Slab Thickness for Various Bar Sizes
Table 5.7.2-1

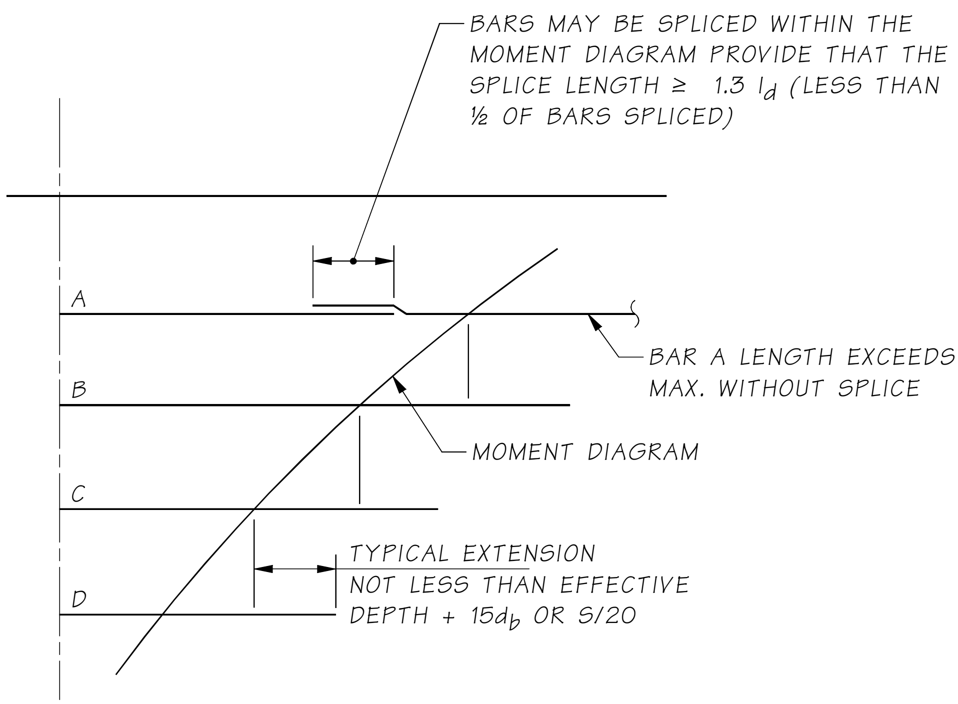
D. **Bar Patterns** – Figure 5.7.2-3 shows two typical top longitudinal reinforcing bar patterns. Care must be taken that bar lengths conform to the requirements of Section 5.1.2.



Longitudinal Reinforcing Bar Patterns
Figure 5.7.2-3

The symmetrical bar pattern shown should normally not be used when required bar lengths exceed 60 feet. If the staggered bar pattern will not result in bar lengths within the limits specified in Section 5.1.2, the method shown in Figure 5.7.2-4 may be used to provide an adequate splice. All bars shall be extended by their development length beyond the point where the bar is required.

Normally, no more than 33% of the total area of main reinforcing bars at a support (negative moment) or at midspan (positive moment) shall be cut off at one point. Where limiting this value to 33% leads to severe restrictions on the reinforcement pattern, an increase in figure may be considered. Two reinforcement bars shall be used as stirrup hangers.



Bar Splice Within Moment Envelope
Figure 5.7.2-4

E. **Concrete Deck Slab Design and Detailing** – These requirements are primarily for beam-slab bridges with main reinforcement perpendicular to traffic:

- Minimum cover over the top layer of reinforcement shall be 2.5″ including 0.5″ wearing surface (Deck Protection Systems 1 and 4). The minimum cover over the bottom layer reinforcement shall be 1.0″.
- The minimum clearance between top and bottom reinforcing mats shall be 1″.
- A maximum bar size of #5 is preferred for longitudinal and transverse reinforcement in the deck slab except that a maximum bar size of #7 is preferred for longitudinal reinforcement at intermediate piers.
- The minimum amount of reinforcement in each direction shall be 0.18 in.^{2}/ft for the top layer and 0.27 in.^{2}/ft for the bottom layer. The amount of longitudinal reinforcement in the bottom of deck slabs shall not be less than $\frac{220}{\sqrt{S}} \leq 67$ percent of the positive moment as specified in AASHTO LRFD 9.7.3.2.
- Top and bottom reinforcement in longitudinal direction of deck slab shall be staggered to allow better flow of concrete between the reinforcing bars.

- The maximum bar spacing in transverse and longitudinal directions for the top mat, and transverse direction of the bottom mat shall not exceed 12″. The maximum bar spacing for bottom longitudinal within the effective length, as specified in AASHTO LRFD 9.7.2.3, shall not exceed the deck thickness.
- Allow the Contractor the option of either a roughened surface or a shear key at the intermediate pier diaphragm construction joint.
- Both, top and bottom layer reinforcement shall be considered when designing for negative moment at the intermediate piers.
- Reduce lap splices if possible. Use staggered lap splices for both top and bottom in longitudinal and transverse directions.

5.7.3 Stay-in-place Deck Panels

A. **General** – The use of precast, prestressed stay-in-place (SIP) deck panels for bridge decks may be investigated at the preliminary design stage. The acceptance evaluation will consider such items as extra weight for seismic design and the resulting substructure impacts.

The composite deck system consisting of precast prestressed concrete deck panels with a CIP topping has advantages in minimizing traffic disruption, speeding up construction and solving constructability issues on certain projects. Contractors, in most cases, prefer this composite deck panel system for bridge decks in traffic congested areas and other specific cases.

SIP deck panels may be used on WSDOT bridges with WSDOT Bridge and Structures Office approval. Details for SIP deck panels are shown in Appendix 5.6-A10-1.

Steel deck forms are not permitted in order to allow inspection of slab soffits and to avoid maintenance of a corrosion protection system.

B. **Design Criteria** – The design of SIP deck panels follows the AASHTO LRFD Specifications and the PCI Bridge Design Manual. The design philosophy of SIP deck panels is identical to simple span prestressed girders. They are designed for Service Limit State and checked for Strength Limit State. The precast panels support the dead load of deck panels and CIP topping, and the composite SIP deck panel and CIP cross-section resists the live load and superimposed dead loads. The tensile stress at the bottom of the panel is limited to zero per WSDOT design practice.

C. **Limitations on SIP Deck Panels** – The conventional full-depth CIP deck slab shall be used for most applications. However, the WSDOT Bridge and Structures Office may allow the use of SIP deck panels with the following limitations:

1. SIP deck panels shall not be used in negative moment regions of continuous conventionally reinforced bridges. SIP deck panels may be used in post-tensioned continuous bridges.
2. Bridge widening. SIP deck panels are not allowed in the bay adjacent to the existing structure because it is difficult to set the panels properly on the existing structure, and the requirement for a CIP closure. SIP deck panels can be used on the other girders when the widening involves multiple girders.
3. Phased construction. SIP deck panels are not allowed in the bay adjacent to the previously placed deck because of the requirement for a CIP closure.
4. Prestressed girders with narrow flanges. Placement of SIP deck panels on girders with flanges less than 12″ wide is difficult.
5. A minimum deck slab thickness of 8.5″, including 3.5″ precast deck panel and 5″ CIP concrete topping shall be specified.
6. SIP deck panels are not allowed for steel girder bridges.

5.7.4 Bridge Deck Protection

All bridge decks, precast or cast-in-place slabs, or deck girder structures shall use a deck protection system as described in this section to reduce the deterioration of the bridge deck and superstructure. The WSDOT Bridge Management Unit shall determine the type of protection system during the preliminary plan or Request For Proposal (RFP) stage for structures not described in this section. Special conditions (i.e. a widening) where it may be desirable to deviate from the standard deck protection systems require approval of the WSDOT Bridge Management Unit.

Preliminary plans shall indicate the protection system in the left margin per BDM Section 2.3.8.

Saw cutting or grinding pavement items are not allowed on the bridge decks. Rumble strips and recessed pavement markers shall not be placed on bridge decks, or approach slab surfaces whether they are concrete or asphalted as stated in Section 8-08 and 8-09 of the *Standard Specifications*, respectively.

Traffic detection loops shall not be located in an existing bridge surface. They may be installed during the construction of bridge decks prior to placing the deck concrete in accordance with Std. Plan J-50.16.

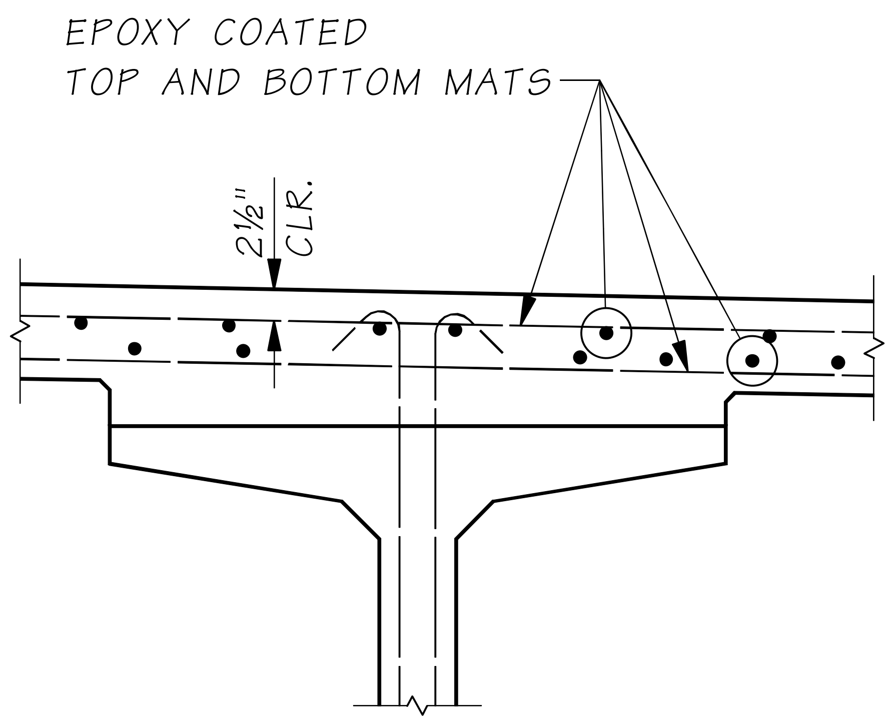
A. **Deck Protection Systems** – The following paragraphs describe five WSDOT protective systems used to protect a traditional concrete bridge deck, deck-girder, or slab design.

1. **Type 1 Protection System** – This is the minimum default protection system for cases where a protection system has not been specified on a structure. Type 1 protection system shall be used for cast-in-place bridge decks with two layers of reinforcement, see Figure 5.7.4-1. This also applies to CIP slab bridges, deck replacements and the widening of existing decks. System 1 consists of the following:

   a. A minimum 2½″ of concrete cover over top bar of deck reinforcing. The cover includes a ½″ wearing surface and ¼″ tolerance for the placement of the reinforcing steel.

   b. Both the top and bottom mat of deck reinforcing shall be epoxy-coated.

   c. Girder stirrups and horizontal shear reinforcement do not require epoxy-coating.



Type 1 Protection System
Figure 5.7.4-1

2. **Type 2 Protection System** – This protection system consists of concrete overlays, see Figure 5.7.4-2. Concrete overlays are generally described as a 1.5″ unreinforced layer of modified concrete used to rehabilitate an existing deck. Overlay concrete is modified to provide a low permeability that slows or prevents the penetration of water into the bridge deck, but also has a high resistance to rutting.

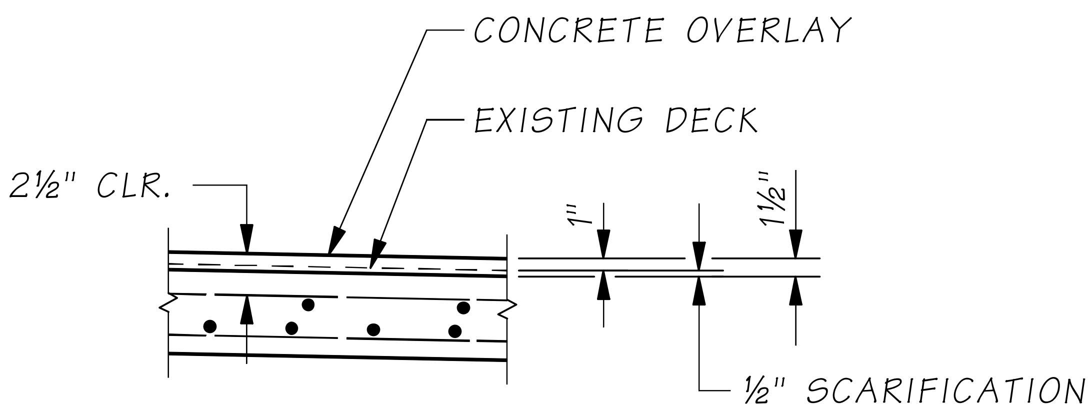
   WSDOT Bridge Management Unit shall determine the type of concrete overlay placed on all new or existing decks; and may specify similar overlays such as a polyester or RSLMC in special cases when rapid construction is cost effective. Brief descriptions of common overlays are as follows.

   a. **1½″ Modified Concrete Overlay** – These overlays were first used by WSDOT in 1979 and have an expected life between 20-40 years. There are more than 600 bridges with concrete overlays as of 2010. This is the preferred overlay system for deck rehabilitation that provides long-term deck protection and a durable wearing surface. In construction, the existing bridge deck is hydromilled ½″ prior to placing the 1.5″ overlay. This requires the grade to be raised 1″. The modified concrete overlay specifications allow a contractor to choose between a Latex, Microsilica or Fly ash mix design. Construction requires a deck temperature between 45°F - 75°F with a wind speed less than 10 mph. Traffic control can be significant since the time to construct and cure is 42 hours.

   b. **¾″ Polyester Modified Concrete Overlay** – These overlays were first used by WSDOT in 1989 and have an expected life between 20-40 years with more than 20 overlay as of 2010. This type of overlay uses specialized polyester equipment and materials. Construction requires dry weather with temperatures above 50°F and normally cures in 4 hours. A polyester concrete overlay may be specified in special cases when rapid construction is needed.

   c. **1½″ Rapid Set Latex Modified Concrete Overlay** – A rapid set latex modified concrete (RSLMC) overlay uses special cement manufactured by the CTS Company based in California. RSLMC is mixed in a mobile mixing truck and applied like a regular concrete overlay. The first RSLMC overlay was applied to bridge 162/20 South Prairie Creek in 2002 under contract 016395. Like polyester, this overlay cures in 4 hours and may be specified in special cases when rapid construction is needed.

   d. **½″ Thin Polymer Overlay** – Thin polymer overlays are built up layers of a polymer material with aggregate broad cast by hand. The first thin overlay was placed in 1986 and after placing 25 overlays, they were discontinued in the late 1998 due to poor performance.



Type 2 Protection System
Figure 5.7.4-2

3. **Type 3 Protection System** – This protection system consists of a Hot Mixed Asphalt (HMA) overlay wearing surface and requires the use of a waterproofing membrane, see Figure 5.7.4-3. HMA overlays provide a lower level of deck protection and introduce the risk of damage by planing equipment during resurfacing. Asphalt overlays with a membrane were first used on a WSDOT bridges in 1971 and about ⅓ of WSDOT structures have HMA. The bridge HMA has an expected life equal to the roadway HMA when properly constructed.

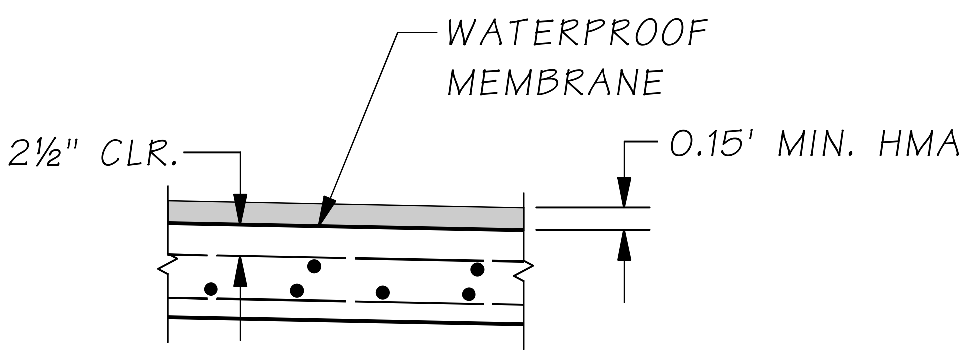
   Waterproof membranes are required with the HMA overlay. Unlike roadway surfaces, the HMA material collects and traps water carrying salts and oxygen at the concrete surface deck. This is additional stress to an epoxy protection system or a bare deck and requires a membrane to mitigate the penetration of salts and oxygen to the structural reinforcement and cement paste. See Standard Specifications for more information on waterproof membranes.

   HMA overlays may be used in addition to the Type 1 Protection System for new bridges where it is desired to match roadway pavement materials. New bridge designs using HMA shall have a depth of overlay between 0.15′ (1.8″) and 0.25′ (3″). Designers should consider designing for a maximum depth of 0.25′ to allow future overlays to remove and replace 0.15′ HMA without damaging the concrete cover or the waterproof membrane. Plan sheet references to the depth of HMA shall be in feet, since this is customary for the paving industry. WSDOT roadway resurfacing operations will normally plane and pave 0.15′ of HMA which encourages the following design criteria.

   Existing structures may apply an HMA overlay in accordance with the Bridge Paving Policies, Section 5.7.5.

   Standard Plan A-40.20.00, Bridge Transverse Joints Seals for HMA provides some standard details for saw cutting small relief joints in HMA paving. Saw cut joints can have a longer life, better ride, and help seal the joint at a location known to crack and may be used for small bridge expansion joints less than 1 inch.

   WSDOT prohibits the use of a Type 3 Protection System for prestressed slab or deck girder bridges managed by WSDOT except for pedestrian bridges or for widening existing similar structures with an HMA overlay. The HMA with membrane provides some protection to the connections between girder or slab units, but can be prone to reflective cracking at the joints. It is not uncommon for voided slabs to fill with water and aggressively corrode the reinforcement. Precast prestressed members with a Type 3 Protection System shall have a minimum cover of 2″ over an epoxy coated top mat.

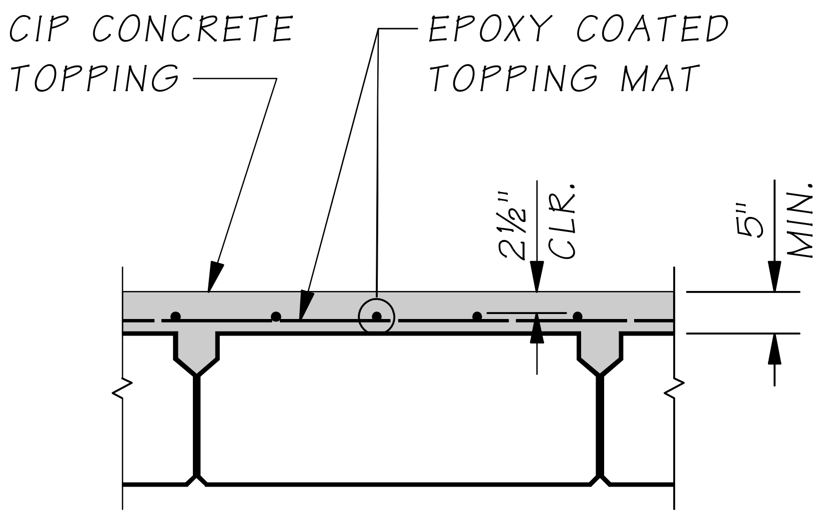


Type 3 Protection System
Figure 5.7.4-3

4. **Type 4 Protection System** – This system is a minimum 5″ cast-in-place (CIP) topping with one mat of epoxy coated reinforcement and placed on prestressed slab or deck girder members, see Figure 5.7.4-4. This system eliminates girder wheel distribution problems, provides a quality protection system and provides a durable wearing surface.

   a. A minimum concrete cover of 1″ applies to the top mat of the top flange of the prestressed member.

   b. Epoxy coating the prestressed member top mat reinforcement is not required.



Type 4 Protection System
Figure 5.7.4-4

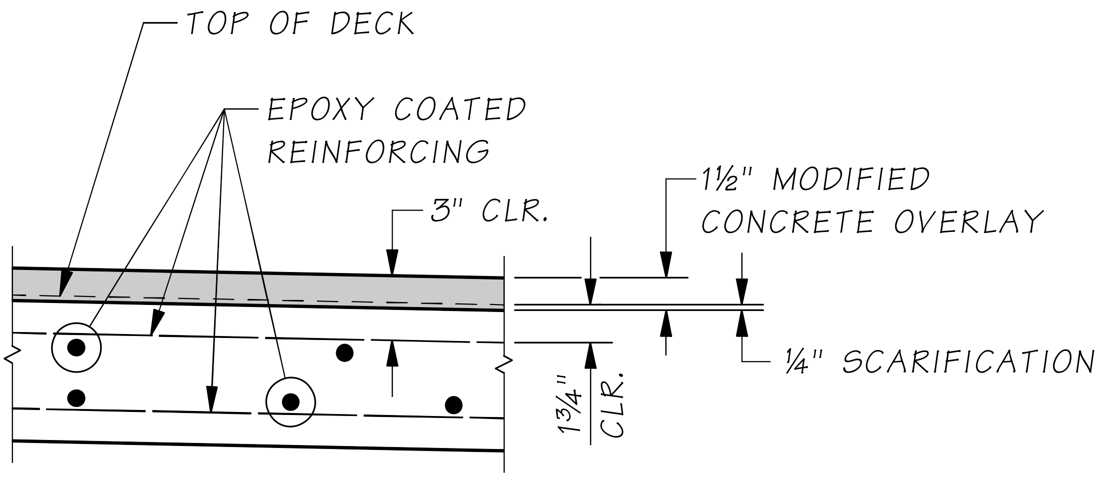
5. **Type 5 System** – This system requires a layered, 3″ concrete cover for double protection, see Figure 5.7.4-5. All segmentally constructed bridges shall use this system to protect construction joints and provide minor grade adjustments during construction. Bridge decks with transverse or longitudinal post-tensioning in the deck shall use this system since deck rehabilitation due to premature deterioration is very costly. The 3″ cover consists of the following:

   a. The deck is constructed with a 1¾″ concrete cover.

   b. Both the top and bottom mat of deck reinforcing are epoxy-coated. Girder/web stirrups and horizontal shear reinforcement does not require epoxy-coating.

   c. The deck is then scarified ¼″ prior to the placement of a modified concrete overlay. Scarification shall be diamond grinding to preserve the integrity of the segmental deck and joints.

   d. A Type 2a, 1½″ Modified Concrete Overlay is placed as a wearing surface.



Type 5 Protection System
Figure 5.7.4-5

B. **Existing Bridge Deck Widening** – New deck rebar shall match the existing top layer. This provides steel at a uniform depth which is important when removing concrete during future rehab work. Bridges prior to the mid 1980's used 1½″ concrete cover. New and widened decks using a Type 1 Protection System have 2½″ cover.

When an existing bridge is widened, the existing concrete or asphalt deck may require resurfacing. WSDOT is forced to rehab concrete decks based on the condition of the existing deck or concrete overlay. If a deck or overlay warrants rehabilitation, then the existing structure shall be resurfaced and included in the widening project.

By applying the stated design criteria, the following policies shall apply to bridge widening projects which may require special traffic closures for the bridge work.

1. **Rebar** – The deck or cast-in-place slab of the new widened portion shall use the Type 1 Protection System, even though the existing structure has bare rebar. The top mat of new rebar shall match the height of existing rebar. Variations in deck thickness are to be obtained by lowering the bottom of the deck or slab.

2. **Concrete Decks** – If the existing deck is original concrete without a concrete overlay, the new deck shall have a Type 1 Protection System and the existing deck shall have a 1½″ concrete overlay or Type 2 Protection System. This matches the rebar height and provides a concrete cover of 2.5″ on both the new and old structure.

   If the existing deck has a concrete overlay, the new deck shall have a Type 1 Protection System and the existing overlay shall be replaced if the deck deterioration is greater than 1% of the deck area.

3. **Concrete Overlays** – It is preferred to place a concrete overlay from curb to curb. If this is problematic for traffic control, then Plans shall provide at least a 6″ offset lap where the overlay construction joint will not match the deck construction joint.

4. **HMA Overlays** – The depth of existing asphalt must be field measured and shown on the bridge plans. This mitigates damage of the existing structure due to removal operations and reveals other design problems such as: improper joint height, buried construction problems, excessive weight, or roadway grade transitions adjustments due to drainage.

   The new deck must meet the rebar and cover criteria stated above for Concrete Decks and deck tinning is not required. Type 3 Protection system shall be used and HMA shall be placed to provide a minimum 0.15′ or the optimum 0.25′.

5. **Small Width Widening –** With approval of the WSDOT Bridge Management Unit, smaller width widening design that has traffic on the new construction can match existing 1½″ concrete cover for the widened portion, if the existing deck deterioration is greater than 1% of the deck area.
6. **Expansion Joints –** All joints shall be in good condition and water tight for the existing bridge and the newly constructed widened portion. The following joint criteria applies:
   a. The existing expansion joint shall be replaced if:
      1. More than 10% of the length of a joint has repairs within 1′-0″ of the joint.
      2. Part of a joint is missing.
      3. The joint is a non-standard joint system placed by maintenance.
   b. All existing joint seals shall be replaced.
   c. When existing steel joints are not replaced in the project, the new joint shall be the same type and manufacturer as the existing steel joint.
   d. Steel joints shall have no more than one splice and the splice shall be at a lane line. Modular joints shall not have any splices.

5.7.5 Bridge Deck HMA Paving Design Policies

This Section of the BDM establishes the criteria used to provide bridge paving design options for paving projects. Bridge paving design options are customized for each bridge based on the existing conditions and previous paving. Paving designers including paving consultants are required to request a Bridge deck Condition Report (BCR) for each bridge which contains the paving design options and other relevant bridge information for each bridge within the project limits.

An asphalt wearing surface is a recognized method to manage concrete rutting, improve the ride on HMA roadways, and is a form of deck protection. Bridges may or may not have the capacity to carry the additional dead load of an asphalt wearing surface. The design options are documented for the paving project in a BCR. The Bridge Office will provide bridge sheets for structural items and the required Special Provisions for the WSDOT projects. The Bridge Office Projects and Design Engineer should be notified early in the paving design to allow time to complete engineering and plan sheets.

All WSDOT structures within the defined project limits must be evaluated for paving or Bituminous Surface Treatment (BST or chip seal). All bridges shall be identified in the Plans as “INCLUDED IN PROJECT” or “NOT INCLUDED” per WSDOT Plan Preparation Manual, Section 4 “Vicinity Map”, paragraph (n). This includes all state bridges and not limited to:

1. Off the main line. Typical locations include bridges on ramps, frontage roads, or bridges out of right-of-way.
2. Bridges where the main line route crosses under the structure.
3. Bridges at the beginning and ending stations of the project. It is not necessary to include the bridge when it was recently resurfaced, but it should be included if incidental joint maintenance repairs are necessary.

Region is responsible for field evaluation of paving condition and the depth of asphalt provided by the last paving contract. Asphalt depths can vary on the concrete deck and from bridge to bridge. In most cases, asphalt depth measurements at the fog line on the four corners of the deck are sufficient to establish a design depth for contracts. The Bridge Asset Manager shall be informed of the measurements. Paving shown in the Plans would use an approximate or averaged value of the measurements. Some situations may require a Plan Detail showing how the depth varies for the Planing contractor.

A standard Microstation detail is available to simplify detailing of bridge paving in the Plans, see “SH_DT_RDSECBridgeDeckOverlay_Detail”. The table format is copied from the BCR and allows the bridge paving design requirements to be listed in the table. All bridges within the limits of the project must be listed in the table to clarify which structures do not have paving and facilitate data logging for the Washington State Pavement Management System and the Bridge Office.

The following bridge paving policies have been developed with the concurrence of WSDOT Pavement Managers to establish bridge HMA Design options available for state managed structures.

1. **Maximum HMA Depth** – Bridge decks shall be 0.25′ or 3″. A greater depth may be allowed if the structure is specifically designed for more than 0.25′, such as structures with ballast or as approved by the WSDOT Load Rating Engineer. Paving designs that increase the HMA more than 3″ require a new Load Rating analysis and shall be submitted to the WSDOT Load Rating engineer.
    a. Concrete bridge decks with more than 0.21′ HMA may be exempted from paving restrictions for mill/fill HMA design.
    b. Deck girders and slabs with less than 0.25′ HMA require paving restrictions to avoid planing the supporting structure.
    c. A paving grade change will be required when more than 0.25′ of asphalt exists on a structure in order to reduce the weight on the structure and meet acceptable rail height standards.
2. **Grade limited/0.15′** – For bridge decks with 0.15′ HMA and the grade is limited by bridge joint height or other considerations, resurfacing must provide full depth removal of HMA or mill/fill the minimum 0.12′.
3. **Grade Transitions** – When raising or lowering the HMA grade profile on/off or under the bridge, the maximum rate of change or slope shall be 1″/40′ (1′/500′) as shown in Standard Plan A60.30.00, even if this means extending the project limits. Incorrect transitions are the cause of many “bumps at the bridge” and create an undesired increase in truck loading. The following items should be considered when transitioning a roadway grade:
    a. Previous HMA overlays that raised the grade can significantly increase the minimum transition length.
    b. Drainage considerations may require longer transitions or should plane to existing catch basins.
    c. Mainline paving that raises the grade under a bridge must verify Vertical Clearance remains in conformance to current Vertical Clearance requirements. Mill/Fill of the roadway at the bridge is generally desired unless lowering the grade is required.
        i. Design Manual Reference: Section 720.04 Bridge Site Design Elements, (5) Vertical Clearances, (c) Minimum Clearance for Existing Structures, 1. Bridge Over a Roadway.
4. **Full Removal** – Full depth removal and replacement of the HMA is always an alternate resurfacing design option. Full depth removal may be required by the Region Pavement Manager or the Bridge Office due to poor condition of the HMA or bridge deck. Bridge Deck Repair and Membrane Waterproofing (Deck Seal) standard pay items are required for this option and the Bridge Office will provide engineering estimates of the quantity (SF) and cost for both.
    a. Bridge Deck Repair will be required when the HMA is removed and the concrete is exposed for deck inspection. Chain Drag Testing is completed and based on the results, the contractor is directed to fix the quantity of deck repairs. The Chain Drag results are sent to the Bridge Asset Manager and used by the Bridge Office to monitor the condition of the concrete deck and determine when the deck needs rehabilitation or replacement.

   b. Membrane Waterproofing (Deck Seal) is Std. Item 4455 and will be required for all HMA bridge decks, except when the following conditions are met.

      i. HMA placed on a deck that has a Modified Concrete Overlay which acts like a membrane.

      ii. The bridge is on the P2 replacement list or deck rehabilitation scheduled within the next 4 years or two bienniums.

5. **Bare Deck HMA** – Paving projects may place HMA on a bare concrete deck, with concurrence of the WSDOT Bridge Asset Manager, if the bridge is on an HMA route and one of the following conditions apply.

   a. Rutting on the concrete deck is ½″ or more.

   b. The Region prefers to simplify paving construction or improve the smoothness at the bridge.

   When the concrete bridge deck does not have asphalt on the surface, Region Design should contact the Region Materials lab and have a Chain Drag Report completed and forwarded to the Bridge Asset Manager during design to establish the Bridge Deck Repair quantities for the project. Pavement Design should then contact Region Bridge Maintenance to request the repairs be completed prior to contract; or the repairs may be included in the paving contract. Small amounts of Bridge Deck Repair have an expensive unit cost by contract during paving operations.

6. **Bridge Transverse Joint Seals** – Saw cut pavement joints shown in Std. Plan A-40.20.00 perform better and help prevent water problems at the abutment or in the roadway. Typical cracking locations where pavement joint seals are required: End of the bridge; End of the approach slab; or HMA joints on the deck. Std. Plan A-40.20.00, Detail 8 shall be used at all truss panel joint locations. However, if Pavement Designers do not see cracking at the ends of the bridge, then sawcut joints may be omitted for these locations. HQ Program management has determined this work is “incidental” to P1 by definition and should be included in a P1 paving project and use Std. Item 6517. The following summarizes the intended application of the Details in Std. Plan A-40.20.00.

   a. **Detail 1** – Applies where HMA on the bridge surface butts to the HMA roadway.

   b. **Detail 2, 3, & 4 –** Applies where concrete bridge surface butts to the HMA roadway.

   c. **Detail 8** – Applies at truss panel joints or generic open concrete joints.

   d. **Detail 5, 6 &7** – For larger 1″ sawcut joints instead of ½″ joints provided in details 2, 3, & 4.

7. **BST (chip seal)** – Bituminous Surface Treatments ½″ thick may be applied to bridge decks with HMA under the following conditions.

   a. Plans must identify or list all structures bridges included or excepted within project limits and identify bridge expansion joint systems to be protected.

   b. BST is not allowed on weight restricted or posted bridges.

   c. Planing will be required for structures at the maximum asphalt design depth or the grade is limited.

   It is true that BSTs are not generally a problem but only if the structure is not grade limited by for structural reasons. BCRs will specify a ½″ chip seal paving depth of 0.03′ for BST Design to be consistent with Washington State Pavement Management System. Plans should indicate ½″ chip seal to be consistent with Standard Specifications and standard pay items.

8. **Culverts and Other Structures** – Culverts or structures with significant fill and do not have rail posts attached to the structure generally will not have paving limitations. Culverts and structures with HMA pavement applied directly to the structure have bridge paving design limits.

5.8 Cast-in-place Post-tensioned Bridges

5.8.1 Design Parameters

A. **General** – Post-tensioning is generally used for CIP construction and spliced precast girders since pretensioning is generally practical only for fabricator-produced structural members. The Post-tensioned Box Girder Bridge Manual[17] published by the Post-tensioning Institute in 1978 is recommended as the guide for design. This manual discusses longitudinal post-tensioning of box girder webs and transverse post-tensioning of box girder slabs, but the methods apply equally well to other types of bridges. The following recommendations are intended to augment the PTI Manual and the AASHTO LRFD Specifications and point out where current WSDOT practice departs from practices followed elsewhere.

The AASHTO criteria for reinforced concrete apply equally to bridges with or without post- tensioning steel. However, designers should note certain requirements unique to prestressed concrete such as special ϕ-factors, load factors and shear provisions.

Post-tensioning consists of installing steel tendons into a hollow duct in a structure after the concrete sections are cast. These tendons are usually anchored at each end of the structure and stressed to a design strength using a hydraulic jacking system. After the tendon has been stressed, the duct is filled with grout which bonds the tendon to the concrete section and prevents corrosion of the strand. The anchor heads are then encased in concrete to provide corrosion protection.

B. **Bridge Types –** Post-tensioning has been used in various types of CIP bridges in Washington State with box girders predominating. See Appendix 5-B4 for a comprehensive list of box girder designs. The following are some examples of other bridge types:

Kitsap County, Contract 9788, Multi-Span Slab

Peninsula Drive, Contract 5898, Two- Span Box Girder

Covington Way to 180th Avenue SE, Contract 4919, Two-Span Box Girder Longitudinal Post-tensioning

Snohomish River Bridge, Contract 4444, Multi-Span Box Girder Longitudinal Post-tensioning

See Section 2.4.1 of this manual for structure type comparison of post-tensioned concrete box girder bridges to other structures. In general, a post-tensioned CIP bridge can have a smaller depth-to-span ratio than the same bridge with conventional reinforcement. This is an important advantage where minimum structure depth is desirable.

1. **Slab Bridge –** Structure depth can be quite shallow in the positive moment region when post-tensioning is combined with haunching in the negative moment region. However, post-tensioned CIP slabs are usually more expensive than when reinforced conventionally. Designers should proceed with caution when considering post-tensioned slab bridges because severe cracking in the decks of bridges of this type has occurred [21, 22, 23].

   The Olalla Bridge (Contract 9202) could be reviewed as an example. This bridge has spans of 41.5′ - 50′ - 41.5′, a midspan structure depth of 15 inches, and some haunching at the piers.

2. **T-Beam Bridge –** This type of bridge, combined with tapered columns, can be structurally efficient and aesthetically pleasing, particularly when the spacing of the beams and the columns are the same. A T-Beam bridge can also be a good choice for a single-span simply-supported structure.

   When equally spaced beams and columns are used in the design, the width of beam webs should generally be equal to the width of the supporting columns. See SR 16, Union Avenue O’Xings, for an example. Since longitudinal structural frame action predominates in this type of design, crossbeams at intermediate piers can be relatively small and the post-tensioning tendons can be placed side-by-side in the webs, resulting in an efficient center of gravity of steel line throughout. For other types of T-Beam bridges, the preferred solution may be smaller, more closely spaced beams and fewer, but larger pier elements. If this type of construction is used in a multispan, continuous bridge, the beam cross-section properties in the negative moment regions need to be considerably larger than the properties in the positive moment regions to resist compression.

Larger section properties can be obtained by gradually increasing the web thickness in the vicinity of intermediate piers or, if possible, by adding a fillet or haunch. The deck slab overhang over exterior webs should be roughly half the web spacing.

3. **Box Girder Bridge –** This type of bridge has been a popular choice in this state. The cost of a prestressed box girder bridge is practically the same as a conventionally-reinforced box girder bridge, however, longer spans and shallower depths are possible with prestressing.

   The superstructure of multi-cell box girders shall be designed as a unit. The entire superstructure section (traffic barrier excluded) shall be considered when computing the section properties.

   For criteria on distribution of live loads, see Section 3.9.4. All slender members subjected to compression must satisfy buckling criteria.

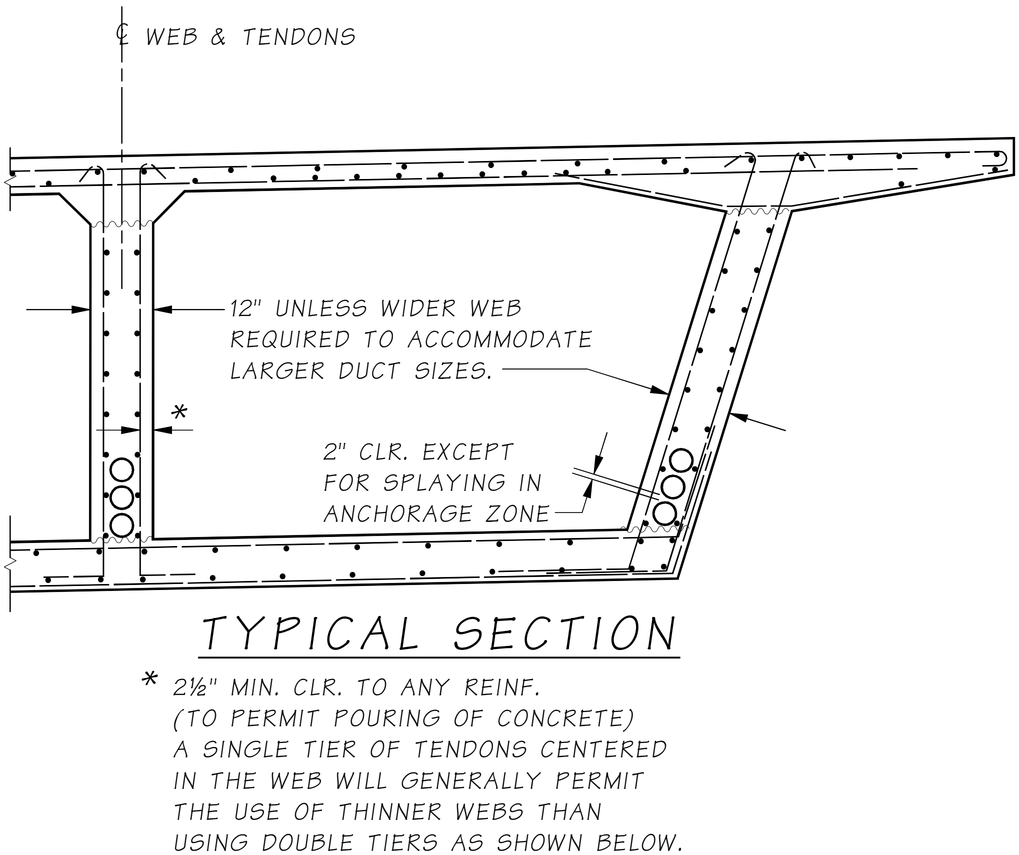
   Web spacing should normally be 8 to 11 feet and the top slab overhang over exterior girders should be approximately half the girder spacing unless transverse post-tensioning is used. The apparent visual depth of box girder bridges can be reduced by sloping all or the lower portion of the exterior web. If the latter is done, the overall structure depth may have to be increased. Web thickness should be 12 inches minimum, but not less than required for shear and for concrete placing clearance. Providing 2½″ of clear cover expedites concrete placement and consolidation in the heavily congested regions adjacent to the post-tensioning ducts. Webs should be flared at anchorages. Top and bottom slab thickness should normally meet the requirements of Section 5.3.1.B, but not less than required by stress and specifications. Generally, the bottom slab would require thickening at the interior piers of continuous spans. This thickening should be accomplished by raising the top surface of the bottom slab at the maximum rate of ½″ per foot.

C. **Strand and Tendon Arrangements –** The total number of strands selected should be the minimum required to carry the service loads at all points. Duct sizes and the number of strands they contain vary slightly, depending on the supplier. Chapter 2 of the PTI Post-tensioned Box Girder Bridge Manual, and shop drawings of the recent post-tensioned bridges kept on file in the Construction Plans Section offer guidance to strand selection. In general, a supplier will offer several duct sizes and associated end anchors, each of which will accommodate a range of strand numbers up to a maximum in the range. Present WSDOT practice is to indicate only the design force and cable path on the contract plans and allow the post-tensioning supplier to satisfy these requirements with tendons and anchors. The most economical tendon selection will generally be the maximum size within the range. Commonly-stocked tendons for ½″ diameter strands include 9, 12, 19, 27, 31, and 37 strands, and the design should utilize a combination of these commonly-stocked items. For example, a design requiring 72 strands per web would be most economically satisfied by two standard 37-strand tendons. A less economical choice would be three standard 27-strand tendons containing 24 strands each. Tendons shall not be larger than (37) ½″ strand units or (27) 0.6″ strand units, unless specifically approved by the WSDOT Bridge Design Engineer. The duct area shall be at least 2.5 times the net area of the prestressing steel. In the regions away from the end anchorages, the duct placement patterns indicated in Figures **5.8.1-1** through 5.8.1-3 shall be used.

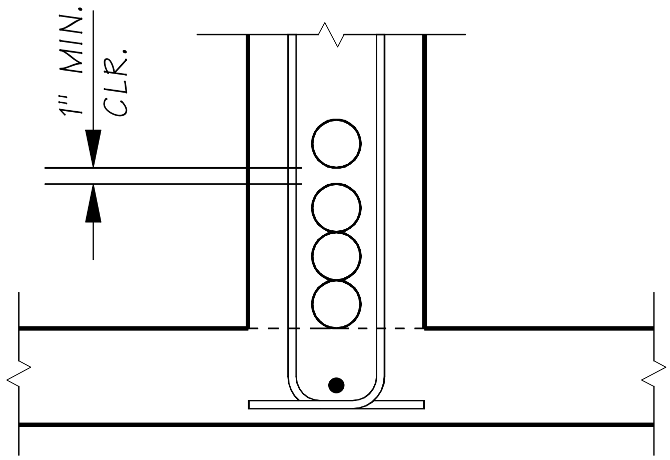
   Although post-tensioning steel normally takes precedence in a member, sufficient room must be provided for other essential mild steel and placement of concrete, in particular near diaphragms and cross-beams.

More prestress may be needed in certain portions of a continuous superstructure than elsewhere, and the designer may consider using separate short tendons in those portions of the spans only. However, the savings on prestressing steel possible with such an arrangement should be balanced against the difficulty involved in providing suitable anchoring points and sufficient room for jacking equipment at intermediate locations in the structure. For example, torsion in continuous, multigirder bridges on a curve can be counter-balanced by applying more prestress in the girders on the outside of the curve than in those on the inside of the curve.

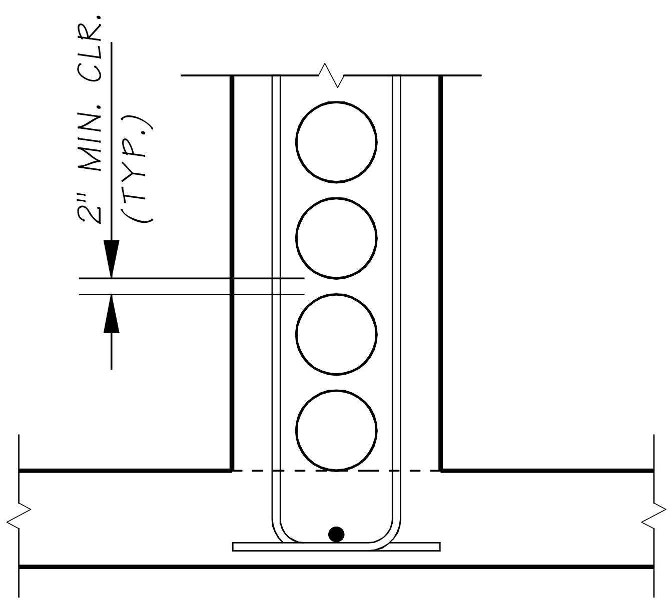
Some systems offer couplers which make possible stage construction of long bridges. With such systems, forms can be constructed and concrete cast and stressed in a number of spans during stage 1, as determined by the designer. After stage 1 stressing, couplers can be added, steel installed, concrete cast and stressed in additional spans. To avoid local crushing of concrete and/or grout, the stress existing in the steel at the coupled end after stage 1 stressing shall not be exceeded during stage 2 stressing.



Tendon Placement Pattern for Box Girder Bridges
***Figure* 5.8.1-1**

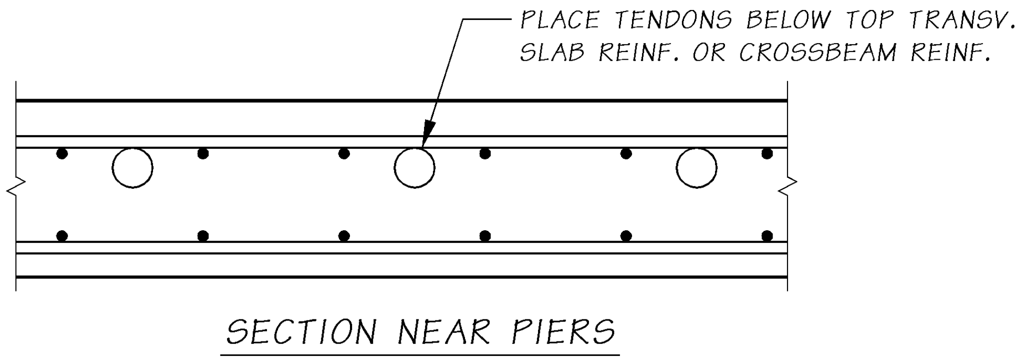


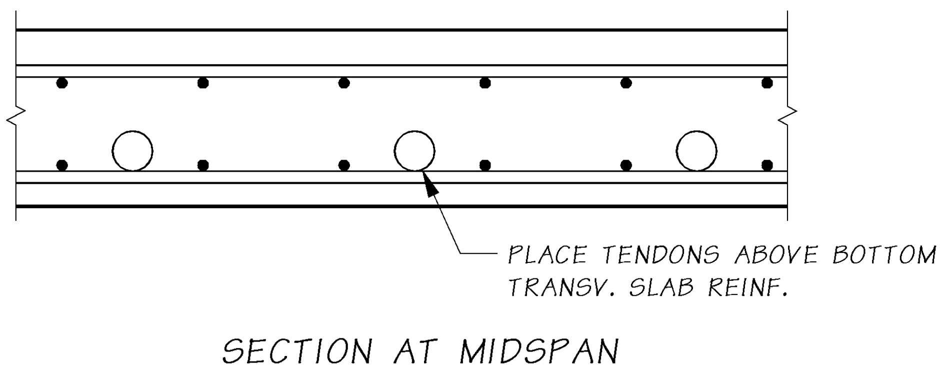
DUCTS 2 " O.D. TO 3" O.D.



DUCTS OVER 3" O.D.

Tendon Placement Pattern for Box Girder Bridges
Figure 5.8.1-2



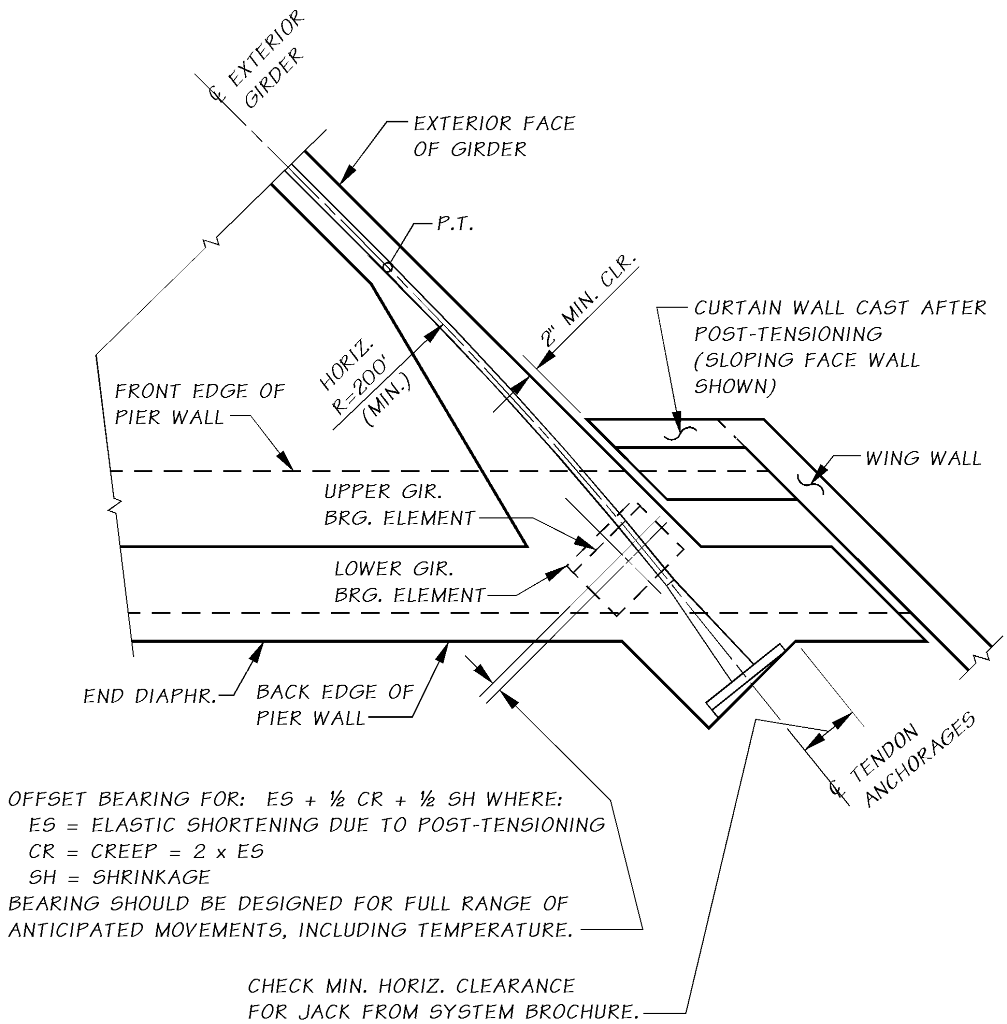


Tendon Placement Pattern for Flat Slab Bridges
Figure 5.8.1-3

D. **Layout of Anchorages and End Blocks –** Consult industry brochures and shop plans for recent bridges before laying out end blocks. To encourage bids from a wider range of suppliers, try to accommodate the large square bearing plate sizes common to several systems.

Sufficient room must be allowed inside the member for mild steel and concrete placement and outside the member for jacking equipment. The size of the anchorage block in the plane of the anchor plates shall be large enough to provide a minimum of 1″ clearance from the plates to any free edge.

The end block dimensions shall meet the requirements of the AASHTO LRFD Specifications. Note that in long-span box girder superstructures requiring large bearing pads, the end block should be somewhat wider than the bearing pad beneath to avoid subjecting the relatively thin bottom slab to high bearing stresses. When the piers of box girder or T-beam bridges are severely skewed, the layout of end blocks, bearing pads, and curtain walls at exterior girders become extremely difficult as shown in Figure 5.8.1-4. Note that if the exterior face of the exterior girder is in the same plane throughout its entire length, all the end block widening must be on the inside. To lessen the risk of tendon break-out through the side of a thin web, the end block shall be long enough to accommodate a horizontal tendon curve of 200 feet minimum radius. The radial component of force in a curved cable is discussed in AASHTO LRFD 5.10.4.3.



Layout of Anchorages and End Blocks
Figure 5.8.1-4

All post-tensioning anchorages in webs of box girder or multi stem superstructures shall be vertically aligned. Multi plane anchor systems may be used to avoid a staggered anchorage layout. If a staggered layout must be used, the plans shall be reviewed and approved by the WSDOT Bridge Design Engineer.

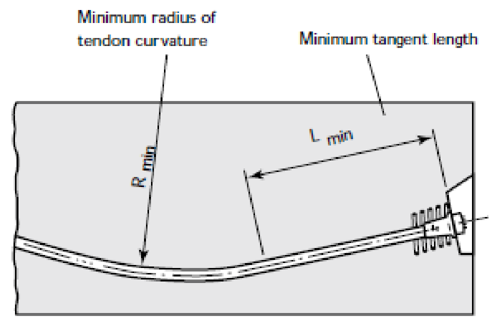
To ensure maximum anchorage efficiency, maximum fatigue life and prevention of strand breakage, a minimum tangent length at the anchorage is required to ensure that the strands enter the anchorage without excessive kinking.

To prevent excessive friction loss and damage to the prestressing sheathings, adherence to the minimum tendon radii is required.

Table 5.8.1-1 and Figure 5.8.1-5 present the required minimum radius of curvature along with the required minimum tangent lengths at stressing anchorages. Deviation from these requirements needs the approval of the WSDOT Bridge Design Engineer.

Anchor Types	Radii, ft.	Tangent Length, ft.
½" Diameter Strand Tendons		
5-4	7.5	2.6
5-7	9.8	2.6
5-12	13.5	3.3
5-19	17.7	3.3
5-27	21.0	3.3
5-31	22.3	4.9
5-37	24.0	4.9
0.6" Diameter Strand Tendons		
6-4	10.6	3.3
6-7	12.8	3.3
6-12	16.4	3.3
6-19	20.7	4.9
6-22	22.6	4.9
6-31	26.4	4.9

Minimum Tendon Radii and Tangent Length
Table 5.8.1-1



Tangent Length and Tendon Radii
Figure 5.8.1-5

E. **Superstructure Shortening –** Whenever members such as columns, crossbeams, and diaphragms are appreciably affected by post-tensioning of the main girders, those effects shall be included in the design. This will generally be true in structures containing rigid frame elements. For further discussion, see Chapter 2.6 of reference [17].

Past practice in the state of Washington regarding control of superstructure shortening in post-tensioned bridges with rigid piers can be illustrated by a few examples. Single-span bridges have been provided with a hinge at one pier and longitudinal slide bearings at the other pier. Two-span bridges have been detailed with longitudinal slide bearings at the end piers and a monolithic middle pier. On the six-span Evergreen Parkway Undercrossing (Bridge Number 101/510), the center pier (pier 4) was built monolithic with the superstructure, and all the other piers were constructed with slide bearings. After post-tensioning, the bearings at piers 3 and 5 were converted into fixed bearings to help resist large horizontal loads such as earthquakes.

Superstructures which are allowed to move longitudinally at certain piers are typically restrained against motion in the transverse direction at those piers. This can be accomplished with suitable transverse shear corbels or bearings allowing motion parallel to the bridge only. The casting length for box girder bridges shall be slightly longer than the actual bridge layout length to account for the elastic shortening of the concrete due to prestress.

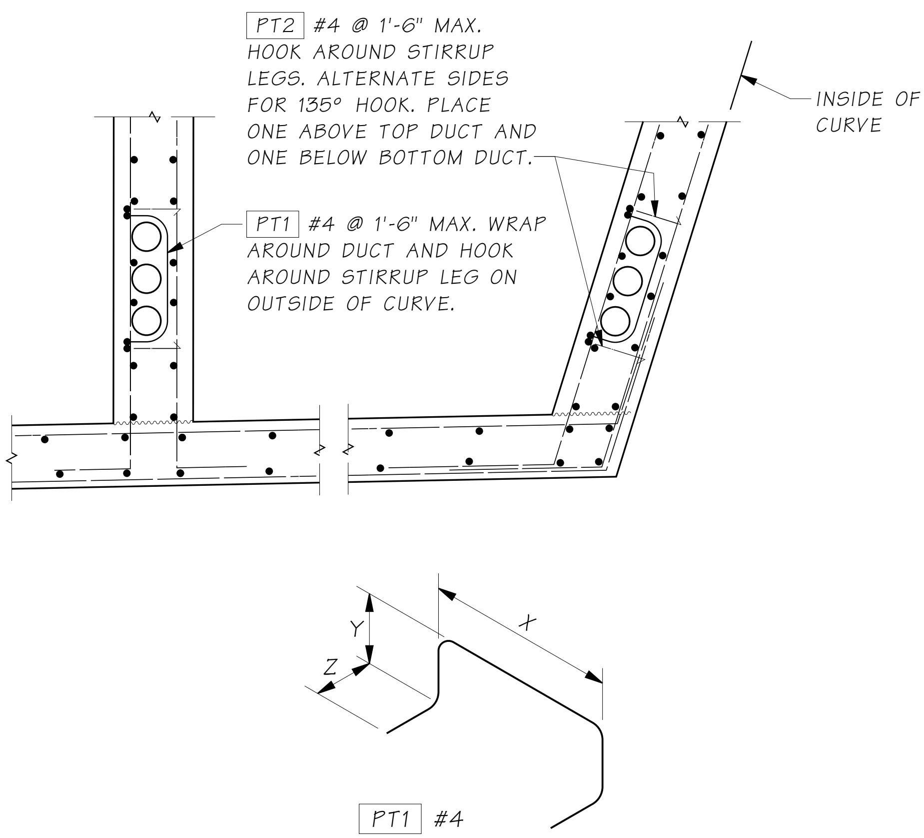
F. **Effects of Curved Tendons –** AASHTO LRFD 5.10.4.3 shall be used to consider the effects of curved tendons. In addition, confinement reinforcement shall be provided to confine the PT tendons when R_{in} is less than 800 ft or the effect of in-plane plus out-of-plane forces is greater than or equal to 10 k/ft:

$$\frac{P_u}{R_{in}} + \frac{P_u}{\pi R_{out}} \geq 10\frac{k}{ft} \tag{5.8.1-1}$$

Where:

P_u = Factored tendon force = 1.2 P_{jack} (kips)
R_{in} = Radius of curvature of the tendon at the considered location causing in-plane force effects (typically horizontal) (ft)
R_{out} = Radius of curvature of the tendon at the considered location causing out-of-plane force effects (typically vertical) (ft)

Curved tendon confinement reinforcement, when required, shall be as shown in Figure 5.8.1-6. Spacing of the confinement reinforcement shall not exceed either 3.0 times the outside diameter of the duct or 18.0 in.



Curved Tendon Confinement Reinforcement
Figure 5.8.1-6

G. **Edge Tension Forces –** If the centroid of all tendons is located outside of the kern of the section, longitudinal edge tension force is induced. The longitudinal edge tension force may be determined from an analysis of a section located at one-half the depth of the section away from the loaded surface taken as a beam subjected to combined flexural and axial load.

5.8.2 Analysis

A. **General –** The procedures outlined in Section 2.1 through 2.5 of reference [17] for computation of stress in single and multispan box girders can be followed for the analysis of T-beams and slab bridges as well.

The BDS program available on the WSDOT system will quickly perform a complete stress analysis of a box girder, T-beam, or slab bridge, provided the structure can be idealized as a plane frame. For further information, see the program user instructions.

STRUDL or SAP is recommended for complex structures which are more accurately idealized as space frames. Examples are bridges with sharp curvature, varying superstructure width, severe skew, or slope-leg intermediate piers. An analysis method in Chapter 10 of reference[18] for continuous prestressed beams is particularly well adapted to the loading input format in STRUDL. In the method, the forces exerted by cables of parabolic or other configurations are converted into equivalent vertical linear or concentrated loads applied to members and joints of the superstructure. The vertical loads are considered positive when acting up toward the center of tendon curvature and negative when acting down toward the center of tendon curvature. Forces exerted by anchor plates at the cable ends are coded in as axial and vertical concentrated forces combined with a concentrated moment if the anchor plate group is eccentric. Since the prestress force varies along the spans due to the effects of friction, the difference between the external forces applied at the end anchors at opposite ends of the bridge must be coded in at various points along the spans in order for the summation of horizontal forces to equal zero. With correct input (check thoroughly before submitting for computation), the effects of elastic shortening and secondary moments are properly reflected in all output listings, and the prestress moments printed out are the actual resultant (total) moments acting on the structure. For examples of the application of STRUDL to post-tensioning design, see the calculations for I-90 West Sunset Way Ramp (simple), I-5 Nalley Valley Viaduct (complex), and the STRUDL manuals.

B. **Section Properties –** As in other types of bridges, the design normally begins with a preliminary estimate of the superstructure cross-section and the amount of prestress needed at points of maximum stress and at points of cross-section change. For box girders, see Figures 2-0 through 2-5 of Reference[17]. For T-beam and slab bridges, previous designs are a useful guide in making a good first choice.

For frame analysis, use the properties of the entire superstructure regardless of the type of bridge being designed. For stress analysis of slab bridges, calculate loads and steel requirements for a 1′ wide strip. For stress analysis of T-beam bridges, use the procedures outlined in the AASHTO LRFD Specifications.

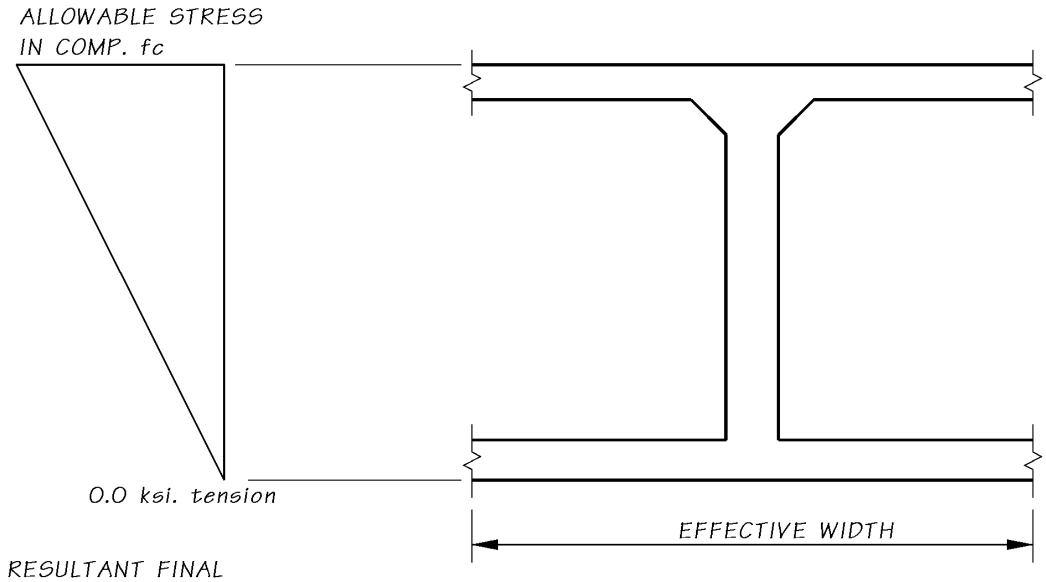
Note that when different concrete strengths are used in different portions of the same member, the equivalent section properties shall be calculated in terms of either the stronger or weaker material. In general, the concrete strength shall be limited to the values indicated in Section 5.1.1 of this manual.

C. **Preliminary Stress Check –** In accordance with AASHTO, flexural stresses in prestressed members are calculated at service load levels. Shear stresses, stirrups, moment capacities vs. applied moments are calculated at ultimate load levels.

During preliminary design, the first objective should be to satisfy the allowable flexural stresses in the concrete at the critical points in the structure with the chosen cross-section and amount of prestressing steel, then the requirements for shear stress, stirrups, and ultimate moment capacity can be readily met with minor or no modifications in the cross-section. For example, girder webs can be thickened locally near piers to reduce excessive shear stress.

In the AASHTO formulas for allowable tensile stress in concrete, bonded reinforcement should be interpreted to mean bonded auxiliary (nonprestressed) reinforcement in conformity with Article 8.6 of the 2002 ACI Code for Analysis and Design of Reinforced Concrete Bridge Structures. The refined estimate for computing time-dependent losses in steel stress given in the code shall be used. To minimize concrete cracking and protect reinforcing steel against corrosion for bridges, the allowable concrete stress under final conditions in the precompressed tensile zone shall be limited to zero in the top and bottom fibers as shown in Figure 5.8.2-1.

In all cases where tension is allowed in the concrete under initial or final conditions, extra mild steel (auxiliary reinforcement) shall be added to carry the total tension present. This steel can be computed as described in Chapter 9-5 of Reference[18].



$$\frac{P}{A} \pm \frac{Pe}{S} \pm \frac{M_{DL}}{S} \pm \frac{M_{LL+I}}{S}$$

Box Girder Stresses
Figure 5.8.2-1

In case of overstress, try one or more of the following remedies: adjust tendon profiles, add or subtract prestress steel, thicken slabs, revise strength of concrete of top slab, add more short tendons locally, etc.

D. **Camber –**The camber to be shown on the plans shall include the effect of both dead load and final prestress and may be taken as given in Table 5.2.4-1.

E. **Expansion Bearing Offsets –** Figure 5.8.1-4 indicates expansion bearing offsets for the partial effects of elastic shortening, creep, and shrinkage. The initial offset shown is intended to result in minimal bearing eccentricity for the majority of the life of the structure. The bearing shall be designed for the full range of anticipated movements: *ES+CR+SH+TEMP*.

5.8.3 Post-tensioning

A. **Tendon Layout –** After a preliminary estimate has been made of the concrete section and the amount of prestressing needed at points of maximum applied load, it may be advantageous in multispan bridges to draw a tendon profile to a convenient scale superimposed on a plot of the center of gravity of concrete (c.g.c.) line. The most efficient tendon profile from the standpoint of steel stress loss will normally be a series of rather long interconnected parabolas, but other configurations are possible. For continuous bridges with unequal span lengths, the tendon profile (eccentricity) shall be based on the span requirement. This results in an efficient post-tensioning design. The tendon profile and c.g.c. line plot is strongly recommended for superstructures of variable cross-section and/or multiple unsymmetrical span arrangements, but is not necessary for superstructures having constant cross- section and symmetrical spans. The main advantages of the tendon profile and c.g.c. plot are:

1. The primary prestress moment curves (prestress force times distance from c.g.c. line to center of gravity of steel (c.g.s.) lines) at all points throughout all spans are quickly obtained from this plot and will be used to develop the secondary moment curves (if present) and, ultimately, to develop the resultant total prestress moment curve.

2. Possible conflicts between prestressing steel and mild steel near end regions, crossbeams, and diaphragms may become apparent.
3. Possible design revisions may be indicated. For example, camber in bridges with unequal spans can be balanced by adjusting tendon profiles.

   The tendon profile and c.g.c. line diagram shall also contain a sketch of how the end bearing plates or anchors are to be arranged at the ends of the bridge. Such a sketch can be useful in determining how large the end block in a girder bridge will have to be and how much space will be required for mild steel in the end region. In general, the arrangement of anchor plates should be the same as the arrangement of the ducts to which they belong to avoid problems with duct cross-overs and to keep end blocks of reasonable width.

B. **Prestress Losses –** Prestress losses shall be as indicated in Section 5.1.4.

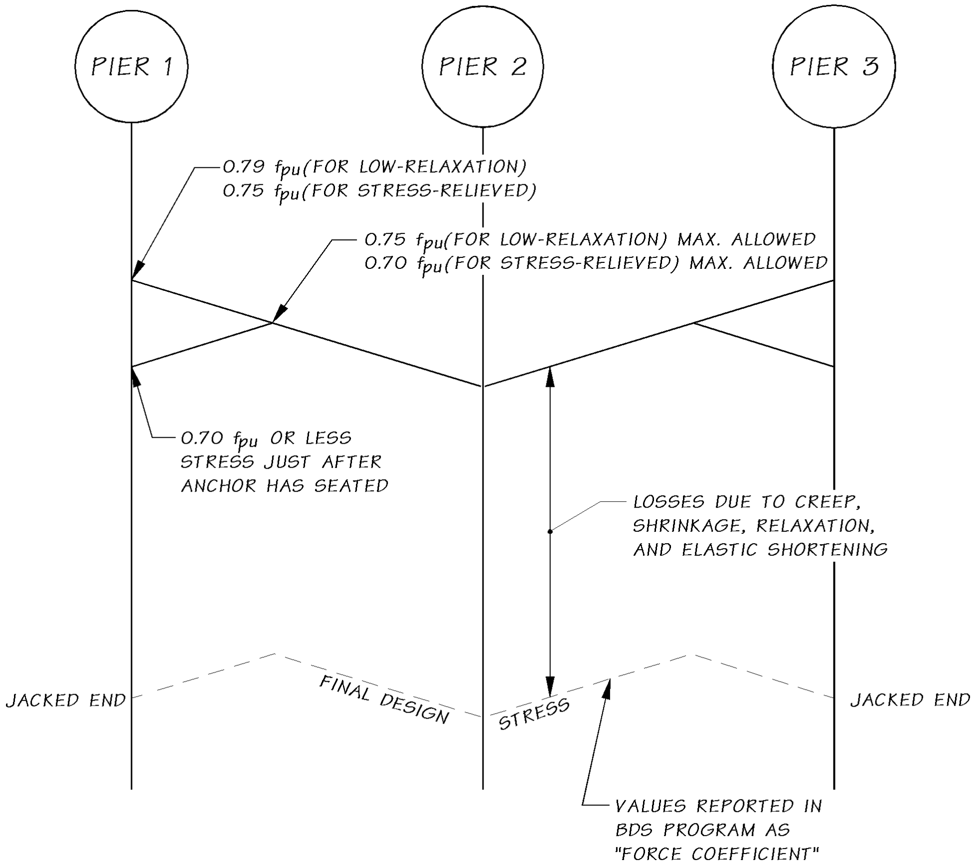
C. **Jacking End –** Effective prestressing force in design of post-tensioned bridges depends on the accumulation of friction losses due to the horizontal and vertical curvature of the tendons as well as the curvature of the bridge. Although jacking ends of post-tensioned bridges is important to achieve more effective design, consideration shall be given to the practicality of jacking during construction. The following general stressing guidelines shall be considered in specifying jacking end of post-tensioned bridges.

   - All simple or multiple span CIP or precast concrete bridges with total length of less than 350′ shall be stressed from one end only.
   - All CIP or precast concrete post tensioned bridges with total length between 350′ to 600′. may be stressed from one end or both ends if greater friction losses due to vertical or horizontal curvature are justified by the designer.
   - All CIP or precast concrete bridges with total length of greater than 600′ shall be stressed from both ends.

   When stressing tendons from both ends or when alternating a single pull from both ends (half tendons pulled from one end with the other half pulled from the other end), all tendons shall be stressed on one end before all tendons are stressed on the opposite end.

   Stressing at both ends shall preferably be done on alternate tendons, and need not be done simultaneously on the same tendon. In rare cases, tendons can be stressed from both ends to reduce large tendon losses but is undesirable due to worker safety issues and a reduction in stressing redundancy.

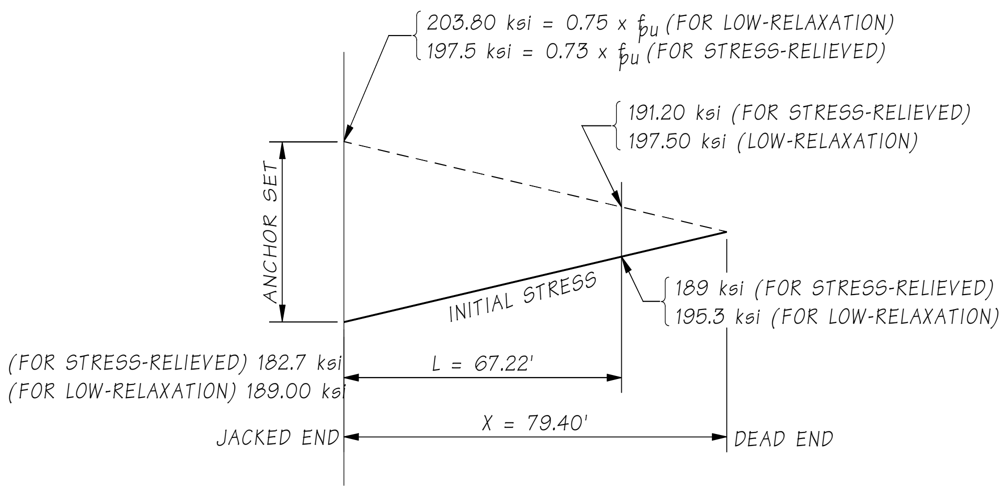
D. **Steel Stress Curve –** Steel stresses may be plotted either as the actual values or as a percentage of the jacking stresses. A steel stress diagram for a typical two-span bridge is shown in Figure 5.8.3-1. Spans are symmetrical about pier 2 and the bridge is jacked from both ends.



Stress Diagram for a 2-Span PT Bridge
Figure 5.8.3-1

Accurate plotting of steel stress variation due to local curvature is normally not necessary, and straight lines between intersection points on the diagram as shown in Figure 5.8.3-1 are usually sufficient. When tendons are continuous through the length of the bridge, the stress for design purposes at the jacked end shall be limited to $0.75f_{pu}$ or 202 ksi for 270 ksi stress relieved strands or $0.79f_{pu}$ or 213 ksi for 270 ksi low relaxation strands. This would permit the post-tensioning contractor to jack to the slightly higher value of $0.77f_{pu}$ for stress relieved strands or $0.81f_{pu}$ for low relaxation strands as allowed by the AASHTO LRFD Specifications in case friction values encountered in the field turn out somewhat greater than the standard values used in design. Stress loss at jacked end shall be calculated from the assumed anchor set of ⅜″, the normal slippage during anchoring in most systems. At the high points on the initial stress curve, the stress shall not exceed $0.70f_{pu}$ for stress relieved strands or $0.75f_{pu}$ low relaxation strands after sealing of anchorage. If these values are exceeded, the jacking stress can be lowered or alternately the specified amount of anchor set can be increased.

When the total tendon length (L) is less than the length of cable influenced by anchor set (x) and the friction loss is small, as in short straight tendons, the $0.70f_{pu}$ value governs. In these cases, the maximum allowable jacking stress value of $0.75f_{pu}$ for stress relieved or $0.78f_{pu}$ for low relaxation strands cannot be used and a slightly lower value shall be specified as shown in Figure 5.8.3-2.



Stress Diagram at Jacking End
Figure 5.8.3-2

In single-span, simply supported superstructures friction losses are so small that jacking from both ends is normally not warranted. In the longer multispan bridges where the tendons experience greater friction losses, jacking from both ends will usually be necessary. Jacking at both ends need not be done simultaneously, since final results are virtually the same whether or not the jacking is simultaneous. If unsymmetrical two-span structures are to be jacked from one end only, the jacking must be done from the end of the longest span.

The friction coefficient for post-tensioning tendons in rigid and semi-rigid galvanized metal sheathing shall be taken as shown in Table 5.8.3-1.

Tendon Length	μ
500 ft or less	0.15
Over 500 ft to 750 ft	0.20
Over 750 ft to 1,000 ft	0.25

Friction Coefficients for Post-tensioning Tendons
Table 5.8.3-1

For tendon lengths greater than 1,000 feet, investigation is warranted on current field data of similar length bridges for appropriate values of μ.

E. **Flexural Stress in Concrete –** Stress at service load levels in the top and bottom fibers of prestressed members shall be checked for at least two conditions that will occur in the lifetime of the members. The initial condition occurs just after the transfer of prestress when the concrete is relatively fresh and the member is carrying its own dead load. The final condition occurs after all the prestress losses when the concrete has gained its full ultimate strength and the member is carrying dead load and live load. For certain bridges, other intermediate loading conditions may have to be checked, such as when prestressing and falsework release are done in stages and when special construction loads have to be carried, etc. The concrete stresses shall be within the AASHTO LRFD Specification allowable except as amended in Section 5.2.1.

In single-span simply supported superstructures with parabolic tendon paths, flexural stresses at service load levels need to be investigated at the span midpoint where moments are maximum, at points where the cross-section changes, and near the span ends where shear stress is likely to be maximum (see Section 5.8.4 Shear). For tendon paths other than parabolic, flexural stress shall be investigated at other points in the span as well.

In multispan continuous superstructures, investigate flexural stress at points of maximum moment (in the negative moment region of box girders, check at the quarter point of the crossbeam), at points where the cross section changes, and at points where shear is likely to be maximum. Normally, mild steel should not be used to supplement the ultimate moment capacity. It may be necessary, however, to determine the partial temperature and shrinkage stresses that occur prior to post-tensioning and supply mild steel reinforcing for this condition.

In addition, maximum and minimum steel percentages and cracking moment shall be checked. See Section 2.3.8 of Reference[17].

F. **Prestress Moment Curves**

1. **Single-Span Bridges, Simply Supported –** The primary prestress moment curve is developed by multiplying the initial steel stress curve ordinates by the area of prestressing steel times the eccentricity of steel from the center of gravity of the concrete section at every tenth point in the span. The primary prestress moment curve is not necessary for calculating concrete stresses in single-span simply supported bridges. Since there is no secondary prestress moment developed in the span of a single span, simply supported bridge which is free to shorten, the primary prestress moment curve is equal to the total prestress moment curve in the span. However, if the single span is rigidly framed to supporting piers, the effect of elastic shortening shall be calculated. The same would be true when unexpected high friction is developed in bearings during or after construction.

2. **Multispan Continuous Bridges –** With the exception of T.Y. Lin's equivalent vertical load method used in conjunction with the STRUDL program, none of the methods described in the following section take into account the elastic shortening of the superstructure due to prestressing. To obtain the total prestress moment curve used to check concrete stresses, the primary and secondary prestress moment curves must be added algebraically at all points in the spans. As the secondary moment can have a large absolute value in some structures, it is very important to obtain the proper sign for this moment, or a serious error could result.

   A discussion of methods for calculating secondary prestress moments follows:

3. **WSDOT BEAMDEF Program –** If the primary prestress moment values at tenth points are coded into this program, span stiffness factors, carry-overs, and fixed-end moments will be obtained. Distribution of the fixed-end moments in all spans will yield the secondary moments at all piers. The secondary moments will be zero at simply supported span ends and cantilevers.

   a. **Equivalent Vertical Load –** See discussion in Section 5.8.2 of this manual.

   b. **Table of Influence Lines –** See Appendix A.1 of Reference[17] for a discussion. This method is similar to T. Y. Lin's equivalent vertical load method and is a relatively quick way to manually compute prestress moments in bridges of up to five spans. Since the secondary moment effect due to vertical support reactions is included in the coefficients listed in the tables, the support moment computed is the total moment at that point.

   c. **Slope Deflection –** See Section 2.5 of Reference[17] for a discussion. The method, though straightforward, is time consuming.

G. **Partial prestressing –** Partial prestressing is not allowed in WSDOT bridge designs. However, mild reinforcement could be added to satisfy the ultimate flexural capacity under factored loads if following requirements are satisfied:

1. Allowable stresses, as specified in this manual for Service-I and Service-III limit states, shall be satisfied with post-tensioning only. The zero-tension policy remains unchanged.
2. Additional mild reinforcement could be used if the ultimate flexural capacity cannot be met with the prestressing provided for service load combinations. The mild reinforcement is filling the gap between the service load and ultimate load requirements. This should be a very small amount of mild reinforcement since adequate post-tensioning is already provided to satisfy the service load requirement for dead load and live loads.
3. If mild reinforcement added, the resistance factor for flexural design shall be adjusted per AASHTO LRFD 5.5.4.2.1 to account for the effect of partial prestressing.
4. If mild reinforcement added, the section will still be considered uncracked and requirements for crack control, and side skin reinforcement do not apply.

5.8.4 Shear and Anchorages

A. **Shear Capacity –** Concrete box girder and T-beam bridges with horizontal construction joints (which result from webs and slabs being cast at different times) shall be checked for both vertical and horizontal shear capacity. Generally, horizontal shear requirements will control the stirrup design.

Vertical concrete shear capacity for prestressed or post-tensioned structural members is calculated per AASHTO LRFD 5.8.3. Minimum stirrup area and maximum stirrup spacing are subject to the limitations presented in AASHTO LRFD 5.8.2.5 and 5.8.2.7. For further explanation, refer to Section 11.4 of the ACI 318-02 Building Code Requirements for Reinforced Concrete and Commentary. Chapter 27 of Notes on ACI 318-02 Building Code Requirements for Reinforced Concrete with Design Applications presents two excellent example problems for vertical shear design.

B. **Horizontal Shear –** Horizontal shear stress acts over the contact area between two interconnected surfaces of a composite structural member. AASHTO LRFD 5.8.4 shall be used for shear-friction design.

C. **End Block Stresses –** The highly concentrated forces at the end anchorages cause bursting and spalling stresses in the concrete which must be resisted by reinforcement. For a better understanding of this subject, see Chapter 7 of Reference[18 and 19], and Section 2.82 of Reference[17].

Note that the procedures for computing horizontal bursting and spalling steel in the slabs of box girders and T-beams are similar to those required for computing vertical steel in girder webs, except that the slab steel is figured in a horizontal instead of a vertical plane. In box girders, this slab steel should be placed half in the top slab and half in the bottom slab. The anchorage zones of slab bridges will require vertical stirrups as well as additional horizontal transverse bars extending across the width of the bridge. The horizontal spalling and bursting steel in slab bridges shall be placed half in a top layer and half in a bottom layer.

D. **Anchorage Stresses –** The average bearing stress on the concrete behind the anchor plate and the bending stress in the plate material shall satisfy the requirements of the AASHTO LRFD Specification. In all sizes up to the 31-strand tendons, the square anchor plates used by three suppliers (DSI, VSL, AVAR, Stronghold) meet the AASHTO requirements, and detailing end blocks to accommodate these plates is the recommended procedure. In the cases where nonstandard (rectangular) anchor plates must be specified because of space limitations, assume that the trumpet associated with the equivalent size square plate will be used. In order to calculate the net bearing plate area pressing on the concrete behind it, the trumpet size can be scaled from photos in supplier brochures. Assume for simplicity that the concrete bearing stress is uniform. Bending stress in the steel should be checked assuming bending can occur across a corner of the plate or across a line parallel to its narrow edge. See Appendix 5-B2 for preapproved anchorages for post-tensioning.

E. **Anchorage Plate Design –** The design and detailing of anchorage block in CIP post-tensioned box girders shall be based on single plane anchorage device. Multi-plane anchorage, however, could be used if stacking of single plane anchorage plates within the depth of girder is geometrically not possible. Anchorage plates shall not extend to top and bottom slab of box girders. If multi-plane anchorage is used, it shall be specified in the contract plans and bridge special provisions.

5.8.5 Temperature Effects

Most specifications for massive bridges call for a verification of stresses under uniform temperature changes of the total bridge superstructure. Stresses due to temperature unevenly distributed within the cross-section are not generally verified. In reality, however, considerable temperature gradients are set up within the cross-section of superstructures. Such temperature differences are mostly of a very complex nature, depending on the type of cross-section and direction of solar radiation [20].

Solar radiation produces uniform heating of the upper surface of a bridge superstructure which is greater than that of the lower surface. An inverse temperature gradient with higher temperatures at the lower surface occurs rarely and involves much smaller temperature differences. In statically indeterminate continuous bridge beams, a temperature rise at the upper surface produces positive flexural moments which cause tensile stresses in the bottom fibers. When the temperature gradient is constant over the entire length of a continuous beam superstructure, positive flexural moments are induced in all spans. These moments are of equal constant magnitude in the interior spans and decrease linearly to zero in the end spans. The most critical zones are those which have the lowest compressive stress reserve in the bottom fibers under prestress plus dead load. Normally, these are the zones near the interior supports where additional tensile stresses develop in the bottom fibers due to

- A concentrated support reaction, and
- Insufficient curvature of prestressed reinforcement.

Studies have shown that temperature is the most important tension-producing factor, especially in two-span continuous beams in the vicinity of intermediate supports, even when the temperature difference is only 10°C between the deck and bottom of the beam. In practice, a box girder can exhibit a ΔT=30°C. The zone at a distance of about 0.3 to 2.0d on either side of the intermediate support proved to be particularly crack-prone.

Computation of stresses induced by vertical temperature gradients within prestressed concrete bridges can become quite complex and are ignored in typical designs done by WSDOT. It is assumed that movements at the expansion devices will generally relieve any induced stresses. However, such stresses can be substantial in massive, deep bridge members in localities with large temperature fluctuations. If the structure being designed falls within this category, a thermal stress investigation shall be considered. See Reference[17] and the following temperature criteria for further guidance.

1. A mean temperature 50°F with rise 45°F and fall 45°F for longitudinal analysis using one-half of the modulus of elasticity. (Maximum Seasonal Variation.)
2. The superstructure box girder shall be designed transversely for a temperature differential between inside and outside surfaces of ±15°F with no reduction in modulus of elasticity (Maximum Daily Variation).
3. The superstructure box girder shall be designed longitudinally for a top slab temperature increase of 20°F with no reduction in modulus of elasticity. (In accordance with Post-tensioning Institute Manual, Precast Segmental Box Girder Bridge Manual, Section 3.3.4.)

5.8.6 Construction

A. **General –** Construction plans for conventional post-tensioned box girder bridges include two different sets of drawings. The first set (contract plans) is prepared by the design engineer and the second set (shop plans) is prepared by the post-tensioning materials supplier (contractor).

B. **Contract Plans –** The contract plans shall be prepared to accommodate any post-tensioning system, so only prestressing forces and eccentricity should be detailed. The concrete sections shall be detailed so that available systems can be installed. Design the thickness of webs and flanges to facilitate concrete placement. Generally, web thickness for post-tensioned bridges shall be at least 12″.

C. **Shop Plans –** The shop plans are used to detail, install, and stress the post-tensioning system selected by the Contractor. These plans must contain sufficient information to allow the engineer to check their compliance with the contract plans. These plans must also contain the location of anchorages, stressing data, and arrangement of tendons.

D. **Review of Shop Plans for Post-tensioned Girder –** Post-tensioning shop drawings shall be reviewed by the designer (or Bridge Technical Advisor for non-Bridge Office projects) and consulted with the Concrete Specialist if needed. Review of shop drawing shall include:

1. All post-tensioning strands shall be of ½″ or 0.6″ diameter grade 270 low relaxation uncoated strands.
2. Tendon profile and tendon placement patterns.
3. Duct size shall be based on the duct area at least 2.5 times the total area of prestressing strands.
4. Anchor set shall conform to the contract plans. The post-tensioning design is typically based on an anchor set of ⅜″.
5. Maximum number of strands per tendon shall not exceed (37) ½″ diameter strands or (27) 0.6″ diameter strands per *Standard Specifications* 6-02.3(26)F.
6. Jacking force per web.
7. Prestress force after anchor set (lift-off force).
8. Number of strands per web.
9. Anchorage system shall conform to pre-approved list of post-tensioning system per Appendix 5-B. The anchorage assembly dimensions and reinforcement detailing shall conform to the corresponding post-tensioning catalog.
10. The curvature friction coefficient and wobble friction coefficient. The curvature friction coefficient of μ= 0.15 for bridges less than 400 feet, μ= 0.2 for bridges between 400 feet and 800 feet, and μ= 0.25 for bridges longer than 800 feet. The wobble friction coefficient of k = 0.0002/ft is often used. These coefficients may be revised by the post-tensioning supplier if approved by the design engineer and conform to the *Standard Specifications* 6.02.3(26)G.
11. Post-tensioning stressing sequence.
12. Tendon stresses shall not exceed as specified per Figure 5.8.3-2:
    1. $0.80f_{pu}$ at anchor ends immediately before seating.
    2. $0.70f_{pu}$ at anchor ends immediately after seating.
    3. $0.74f_{pu}$ at the end point of length influenced by anchor set.

13. Elongation calculations for each jacking operation shall be verified. If the difference in tendon elongation exceeds 2%, the elongation calculations shall be separated for each tendon per *Standard Specifications* 6-02.3(26) A.
14. Vent points shall be provided at all high points along tendon.
15. Drain holes shall be provided at all low points along tendon.
16. The concrete strength at the time of post-tensioning, f'_{ci} shall not be less than 4,000 psi per *Standard Specifications* 6-02.3(26)G. Different concrete strength may be used if specified in the contract plans.
17. Concrete stresses at the anchorage shall be checked per *Standard Specifications* 6-02.3(26)C for bearing type anchorage. For other type of anchorage assemblies, if not covered in the Appendix 5-B2 for pre-approved list of post-tensioning system, testing per *Standard Specifications* 6-02.3(26)D is required.

E. **During Construction**

1. If the measured elongation of each strand tendon is within ± 7% of the approved calculated elongation, the stressed tendon is acceptable.
2. If the measured elongation is greater than 7%, force verification after seating (lift-off force) is required. The lift-off force shall not be less than 99% of the approved calculated force nor more than 70% $f_{pu} A_s$.
3. If the measured elongation is less than 7%, the bridge construction office will instruct the force verification.
4. One broken strand per tendon is usually acceptable. (Post-tensioning design shall preferably allow one broken strand). If more than one strand per tendon is broken, the group of tendon per web should be considered. If the group of tendons in a web is under-stressed, then the adequacy of the entire structure shall be investigated by the designer and consulted with the Bridge Construction Office.
5. Failed anchorage is usually taken care of by the Bridge Construction Office.
6. Over or under elongation is usually taken care of by the Bridge Construction Office.
7. In case of low concrete strength the design engineer shall investigate the adequacy of design with lower strength.
8. Other problems such as unbalanced and out of sequence post-tensioning, strands surface condition, strand subjected to corrosion and exposure, delayed post-tensioning due to mechanical problems, Jack calibration, etc. should be evaluated per case-by-case basis and are usually taken care by Bridge Construction Office.

5.8.7 Post-tensioning Notes — Cast-in-place Girders

A. **General –** The design plans shall contain the following information for use by the post-tensioned and state inspector: Tendon jacking sequence, friction coefficients, duct type, elastic and time-dependent losses, anchor set, prestress forces, strand elongations, falsework construction and removal. If jacking is done at both ends of the bridge, the minimum strand elongation due to the specified jacking load for the end jacked first as well as the end jacked last shall be indicated. The calculated strand elongations at the ends of the bridge are compared with the measured field values to ensure that the friction coefficients (and hence the levels of prestressing throughout the structure) agree with the values assumed by the designer.

The tendons shall be jacked in a sequence that avoids causing overstress or tension in the bridge.

The standard post-tensioning notes for the sequence of stressing of longitudinal tendons shall be shown in the Contract Plans.

5.9 Spliced Precast Girders

5.9.1 Definitions

The provisions herein apply to precast girders fabricated in segments that are spliced longitudinally to form the girders in the final structure. The cross-section for this type of bridge is typically composed of bulb tee girders or trapezoidal tub girders with a composite CIP deck. WSDOT standard drawings for spliced I-girders are shown in Appendices 5.9-A1 through 5.9-A3, and for spliced-tub girders are shown in Appendices 5.9-A4 and 5.9-A5. Span capabilities of precast spliced girders are shown in Appendices 5.6-A1-8 for I-girders and 5.6-A1-9 for trapezoidal tub girders.

Precast deck bulb tee girder bridges may also be fabricated in segments and spliced longitudinally. Splicing in this type of girder may be beneficial because the significant weight of the cross-section may exceed usual limits for handling and transportation. Spliced structures of this type, which have longitudinal joints in the deck between each deck girder, shall comply with the additional requirements of AASHTO LRFD 5.14.4.3.

Spliced precast girder bridges may be distinguished from what is referred to as “segmental construction” in bridge specifications by several features which typically include:

- The lengths of some or all segments in a bridge are a significant fraction of the span length rather than having a number of segments in each span.
- Design of joints between girder segments at the service limit state does not typically govern the design for the entire length of the bridge for either construction or for the completed structure.
- Wet-cast closure joints are usually used to join girder segments rather than match-cast joints.
- The bridge cross-section is composed of girders with a CIP concrete composite deck rather than precasting the full width and depth of the superstructure as one piece. In some cases, the deck may be integrally cast with each girder. Connecting the girders across the longitudinal joints completes a bridge of this type.
- Girder sections are used, such as bulb tee, deck bulb tee or trapezoidal tub girders, rather than closed cell boxes with wide monolithic flanges.
- Provisional ducts are required for segmental construction to provide for possible adjustment of prestress force during construction. Similar requirements are not given for spliced precast girder bridges because of the redundancy provided by a greater number of webs and tendons, and typically lower friction losses because of fewer joint locations.
- The method of construction and any required temporary support is of paramount importance in the design of spliced precast girder bridges. Such considerations often govern final conditions in the selection of section dimensions and reinforcing and/or prestressing.

All supports required prior to the splicing of the girder shall be shown on the contract documents, including elevations and reactions. The stage of construction during which the temporary supports are removed shall also be shown on the contract documents. Stresses due to changes in the structural system, in particular the effects of the application of load to one structural system and its removal from a different structural system, shall be accounted for. Redistribution of such stresses by creep shall be taken into account and allowance shall be made for possible variations in the creep rate and magnitude.

Prestress losses in spliced precast girder bridges shall be estimated using the provisions of Section 5.1.4. The effects of combined pretensioning and post-tensioning and staged post-tensioning shall be considered. When required, the effects of creep and shrinkage in spliced precast girder bridges shall be estimated using the provisions of Section 5.1.1.

5.9.2 WSDOT Criteria for Use of Spliced Girders

See Section 5.6.3.D.3 for criteria on providing an alternate spliced-girder design for long span one-piece pretensioned girders.

5.9.3 Girder Segment Design

A. **Design Considerations –** Stress limits for temporary concrete stresses in girder segments before losses and stress limits for concrete stresses in girder segments at the service limit state after losses specified in Section 5.2.1.C shall apply at each stage of pretensioning or post-tensioning with due consideration for all applicable loads during construction. The concrete strength at the time the stage of prestressing is applied shall be substituted for f'_{ci} in the stress limits.

The designer shall consider requirements for bracing of the girder segments once they have been erected on the substructure. Any requirements for temporary or permanent bracing during subsequent stages of construction, along with the contractor's responsibilities for designing and placing them, shall be specified in the contract documents.

Effects of curved tendons shall be considered per Section 5.8.1.F.

B. **Post-tensioning –** Post-tensioning may be applied either before and/or after placement of deck concrete. Part of the post-tensioning may be applied prior to placement of the deck concrete, with the remainder placed after deck concrete placement. In the case of multi-stage post-tensioning, ducts for tendons to be tensioned before the deck slab concrete shall not be located in the deck slab.

All post-tensioning tendons shall be fully grouted after stressing. Prior to grouting of post-tensioning ducts, gross cross-section properties shall be reduced by deducting the area of ducts and void areas around tendon couplers.

Where some or all post-tensioning is applied after the deck concrete is placed, fewer post-tensioning tendons and a lower concrete strength in the closure joint may be required. However, deck replacement, if necessary, is difficult to accommodate with this construction sequence. Where all of the post-tensioning is applied before the deck concrete is placed, a greater number of post-tensioning tendons and a higher concrete strength in the closure joint may be required. However, in this case, the deck can be replaced if necessary.

5.9.4 Joints Between Segments

A. **General –** Cast-in-place closure joints are typically used in spliced girder construction. The sequence of placing concrete for the closure joints and deck shall be specified in the contract documents. Match-cast joints shall not be specified for spliced girder bridges unless approved by the Bridge Design Engineer. Prestress, dead load, and creep effects may cause rotation of the faces of the match-cast joints prior to splicing. If match cast joint is specified, the procedures for splicing the girder segments that overcome this rotation to close the match-cast joint shall be shown on the contract plans.

B. **Location of Closure Joints –** The location of intermediate diaphragms shall be offset by at least 2′-0″ from the edge of cast-in-place closure joints.

In horizontally curved spliced girder bridges, intermediate diaphragms could be located at the CIP closure joints if straight segments are spliced with deflection points at closures. In this case, the diaphragm could be extended beyond the face of the exterior girder for improved development of diaphragm reinforcement.

The final configuration of the closures shall be coordinated with the State Bridge and Structures Architect on all highly visible bridges, such as bridges over vehicular or pedestrian traffic.

C. **Details of Closure Joints –** The length of a closure joint between precast concrete segments shall allow for the splicing of steel whose continuity is required by design considerations and the accommodation of the splicing of post-tensioning ducts. The length of a closure joint shall not be less than 2′-0″. A longer closure joint may be used to provide more room to accommodate tolerances for potential misalignment of ducts within girder segments and misalignment of girder segments at erection.

Web reinforcement within the joint shall be the larger of that in the adjacent girders. The face of the precast segments at closure joints shall be specified as intentionally roughened surface.

Concrete cover to web stirrups at the CIP closures of pier diaphragms shall not be less than 2½″. If intermediate diaphragm locations coincide with CIP closures between precast segments, then the concrete cover at the CIP closures shall not be less than 2½″. This increase in concrete cover is not necessary if intermediate diaphragm locations are away from the CIP closures. See Figures 5.9.4-1 to 5.9.4-3 for details of closure joints.

Adequate reinforcement shall be provided to confine tendons at CIP closures and at intermediate pier diaphragms. The reinforcement shall be proportioned to ensure that the steel stress during the jacking operation does not exceed $0.6f_y$.

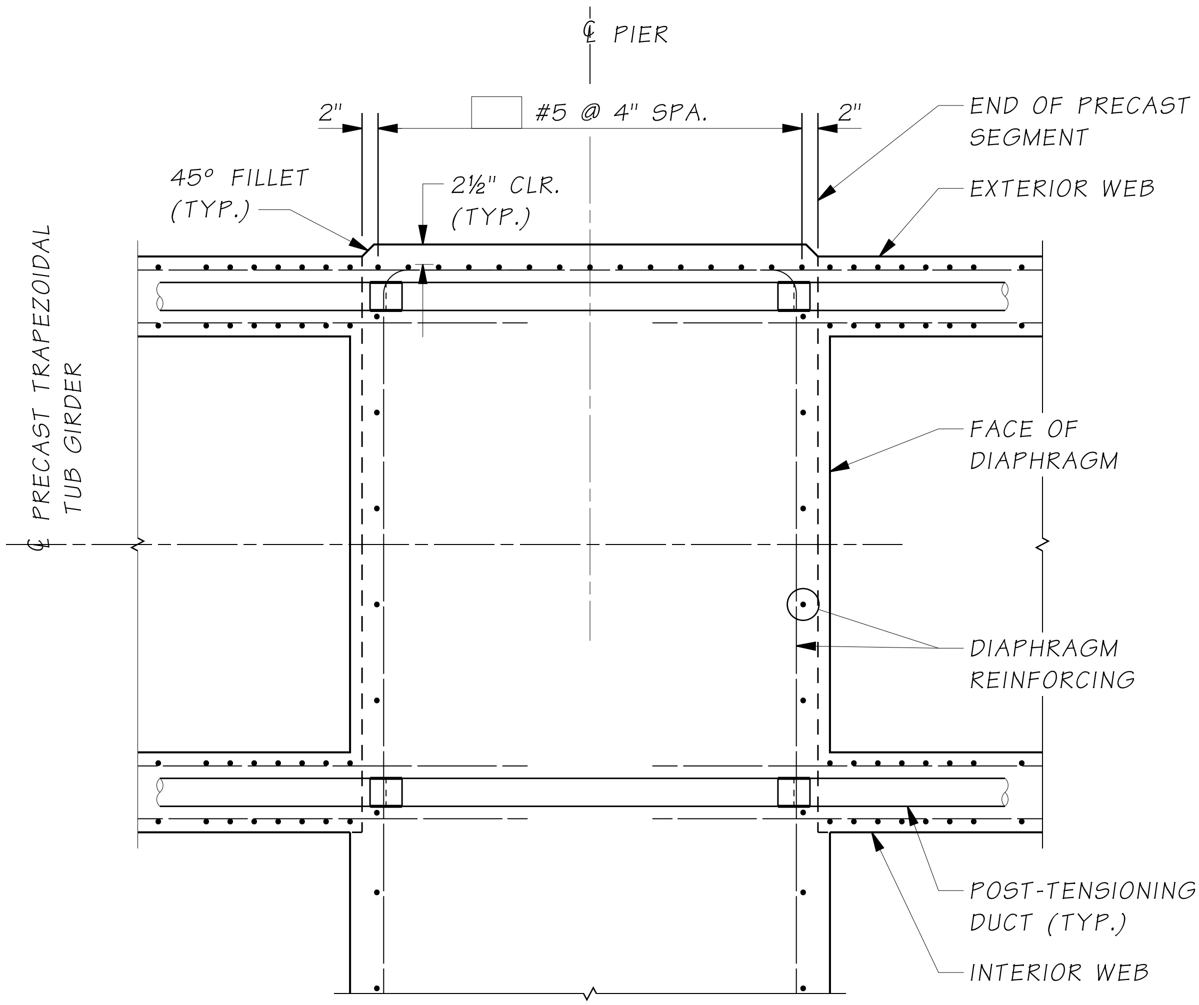
The clear spacing between ducts at CIP closures of pier diaphragms shall be 2.0″ minimum. The duct diameter for WSDOT standard spliced girders shall not exceed 4.0″ for spliced I-girders and 4½″ for spliced tub girders.

On the construction sequence sheet indicate that the side forms at the CIP closures and intermediate pier diaphragms shall be removed to inspect for concrete consolidation prior to post-tensioning and grouting.

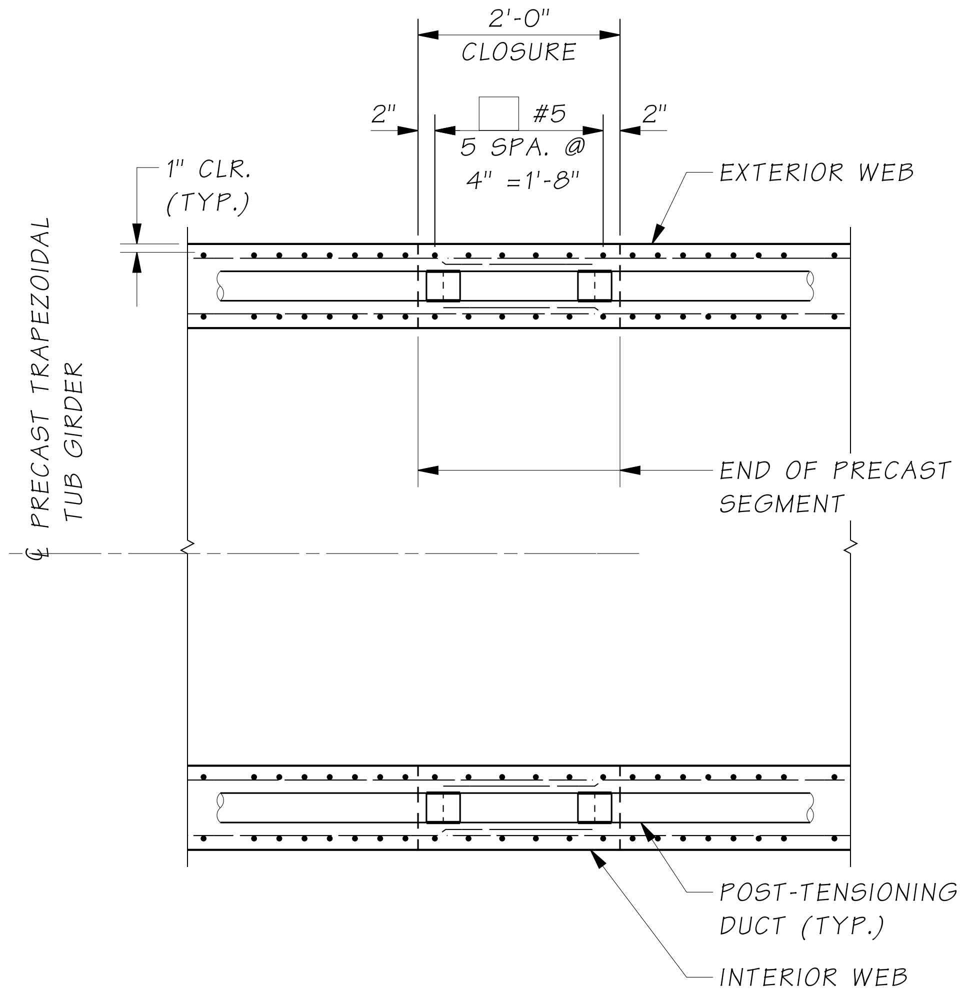
Self-consolidating concrete (SCC) may be used for CIP closures.

D. **Joint Design –** Stress limits for temporary concrete stresses in joints before losses specified in Section 5.2.1.C shall apply at each stage of post-tensioning. The concrete strength at the time the stage of post-tensioning is applied shall be substituted for f'_{ci} in the stress limits.

Stress limits for concrete stresses in joints at the service limit state after losses specified in Section 5.2.1.C shall apply. These stress limits shall also apply for intermediate load stages, with the concrete strength at the time of loading substituted for f'_c in the stress limits. The compressive strength of the closure joint concrete at a specified age shall be compatible with design stress limitations.

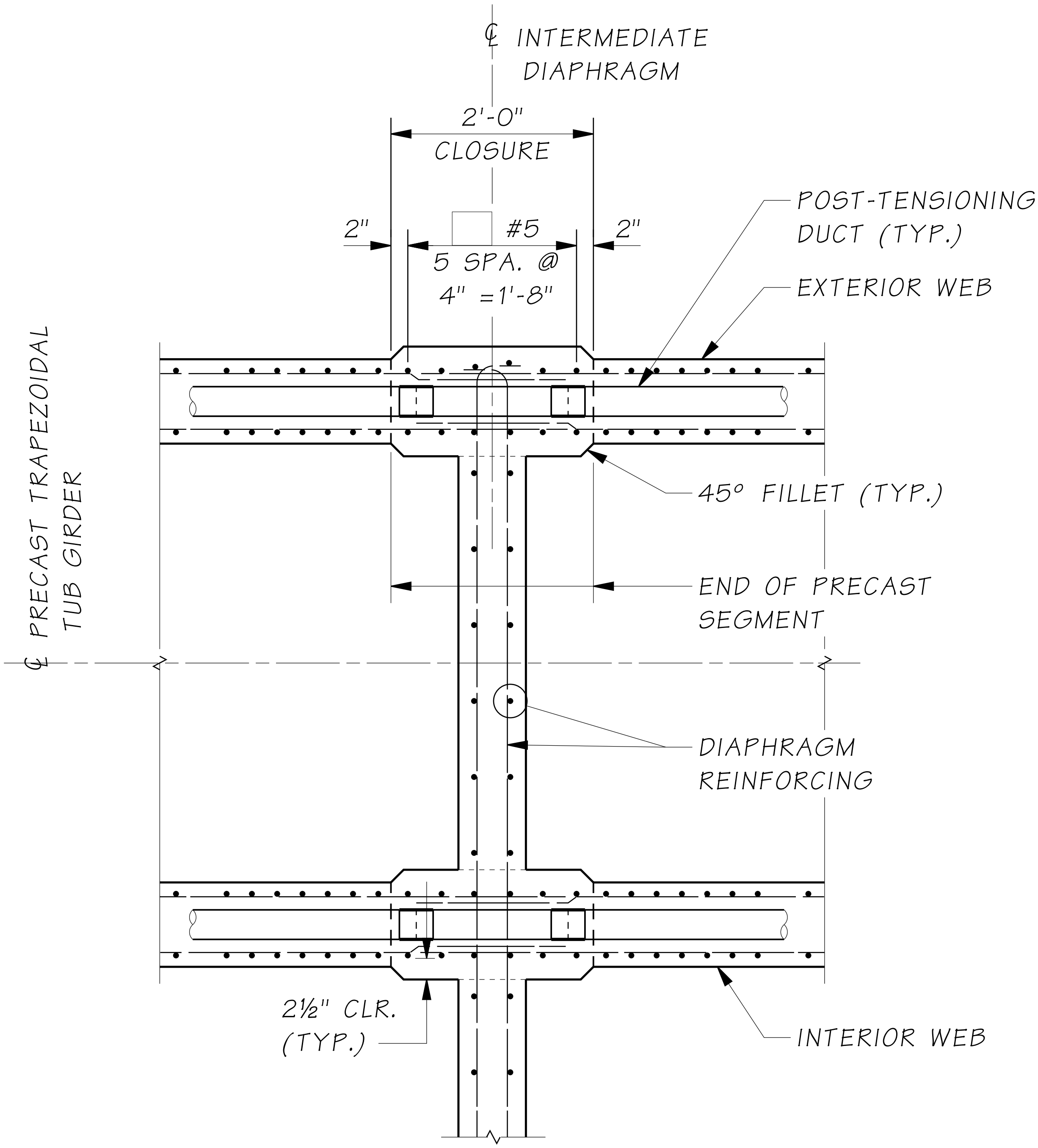


CIP Closure at Pier Diaphragm
Figure 5.9.4-1



CIP Closure Away from Intermediate Diaphragm

Figure 5.9.4-2



CIP Closure at Intermediate Diaphragm
Figure 5.9.4-3

5.9.5 Review of Shop Plans for Precast Post-tensioned Spliced-girders

Shop drawings for precast post-tensioned spliced-girders shall be reviewed by the designer (or Bridge Technical Advisor for non-Bridge Office projects) and consulted with the Concrete Specialist if needed. Review of shop drawing shall include:

1. All prestressing strands shall be of ½″ or 0.6″ diameter grade 270 low relaxation uncoated strands.
2. Number of strands per segment.
3. Pretensioning strands jacking stresses shall not exceed $0.75 f_{pu}$.
4. Strand placement patterns.
5. Temporary strand placement patterns, location and size of blockouts for cutting strands.
6. Procedure for cutting temporary strands and patching the blockouts shall be specified.
7. Number and length of extended strands and rebars at girder ends.
8. Location of holes and shear keys for intermediate and end diaphragms.
9. Location and size of bearing recesses.
10. Saw tooth at girder ends.
11. Location and size of lifting loops or lifting bars.
12. Number and size of horizontal and vertical reinforcement.
13. Segment length and end skew.
14. Tendon profile and tendon placement pattern.
15. Duct size shall be based on the duct area at least 2.5 times the total area of prestressing strands.
16. Anchor set. The post-tensioning design is typically based on an anchor set of ⅜″.
17. Maximum number of strands per tendon shall not exceed (37) ½″ diameter strands or (27) 0.6″ diameter strands per *Standard Specifications* 6-02.3(26)F.
18. Jacking force per girder.
19. Prestress force after anchor set (lift-off force).
20. Number of strands per web.
21. Anchorage system shall conform to pre-approved list of post-tensioning system per Appendix 5-B4 of this manual. The anchorage assembly dimensions and reinforcement detailing shall conform to the corresponding post-tensioning catalog.
22. The curvature friction coefficient and wobble friction coefficient. The curvature friction coefficient of μ= 0.15 for bridges less than 400 feet, μ= 0.2 for bridges between 400 feet and 800 feet, and μ= 0.25 for bridges longer than 800 feet. The wobble friction coefficient of k = 0.0002/ft is often used. These coefficients may be revised by the post-tensioning supplier if approved by the design engineer and conform to the *Standard Specifications* 6.02.3(26)G.
23. Post-tensioning stressing sequence.
24. Tendon stresses shall not exceed as specified per Figure 5.8.3-2:
    $0.80 f_{pu}$ at anchor ends immediately before seating.
    $0.70 f_{pu}$ at anchor ends immediately after seating.
    $0.74 f_{pu}$ at the end point of length influenced by anchor set.

25. Elongation calculations for each jacking operation shall be verified. If the difference in tendon elongation exceeds 2%, the elongation calculations shall be separated for each tendon per *Standard Specifications* 6-02.3(26)A.
26. Vent points shall be provided at all high points along tendon.
27. Drain holes shall be provided at all low points along tendon.
28. The concrete strength at the time of post-tensioning, f'_{ci} shall not be less than 4,000 psi per *Standard Specifications* 6-02.3(26)G. Different concrete strength may be used if specified in the contract plans.
29. Concrete stresses at the anchorage shall be checked per *Standard Specifications* 6-02.3(26)C for bearing type anchorage. For other type of anchorage assemblies, if not covered in the Appendix 5-B2 for pre-approved list of post-tensioning system, testing per *Standard Specifications* 6-02.3(26)D is required.
30. Concrete stresses at CIP closures shall conform to allowable stresses of Table 5.2.1-1.

5.9.6 Post-tensioning Notes — Precast Post-tensioning Spliced-Girders

1. The CIP concrete in deck slab shall be Class 4000D. The minimum compressive strength of the CIP concrete at the wet joint at the time of post-tensioning shall be …. ksi.
2. The minimum prestressing load after seating and the minimum number of prestressing strands for each girder shall be as shown in post-tensioning table.
3. The design is based on ….. inch diameter low relaxation strands with a jacking load for each girder as shown in post-tensioning table, an anchor set of ⅜″ a curvature friction coefficient, μ= 0.20 and a wobble friction coefficient, k = 0.0002/ft. The actual anchor set used by the contractor shall be specified in the shop plans and included in the transfer force calculations.
4. The design is based on the estimated prestress loss of post-tensioned prestressing strands as shown in post-tensioning table due to steel relaxation, elastic shortening, creep and shrinkage of concrete.
5. The contractor shall submit the stressing sequence and elongation calculations to the engineer for approval. All losses due to tendon vertical and horizontal curvature must be included in elongation calculations. The stressing sequence shall meet the following criteria:
   A. The prestressing force shall be distributed with an approximately equal amount in each web and shall be placed symmetrically about the centerline of the bridge.
   B. No more than one-half of the prestressing force in any web may be stressed before an equal force is stressed in the adjacent webs. At no time during stressing operation will more than one-sixth of the total prestressing force is applied eccentrically about the centerline of bridge.
6. The maximum outside diameter of the duct shall be …. inches. The area of the duct shall be at least 2.5 times the net area of the prestressing steel in the duct.
7. All tendons shall be stressed from pier ….

AASHTO M203 Grade 270 Uncoated Prestressing Strands Properties and Development Length

Strand Diameter (in)	Weight (lbs/ft)	Nominal Diameter (in)	Area (in^2)	Transfer length (in)	Develop. Length *k* = 1.0 (ft)	Develop. Length *k* = 1.6 (ft)
⅜	0.290	0.375	0.085	22.5	5.05	8.08
7⁄16	0.390	0.438	0.115	26.3	5.90	9.44
½	0.520	0.500	0.153	30.0	6.74	10.78
½ S	0.568	0.520	0.167	31.2	7.01	11.21
9⁄16	0.651	0.563	0.192	33.8	7.58	12.14
0.60	0.740	0.600	0.217	36.0	8.08	12.93
0.62	0.788	0.620	0.231	37.2	8.35	13.36
0.70	1.000	0.700	0.294	42.0	9.43	15.09

Assumptions for determining development length:

f_{ps} = f_{pu} = 270 ksi

f_{pe} = (270 ksi x 0.75) - 40 ksi = 162.5 ksi

Chapter 7 Substructure Design — Contents

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.

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

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.

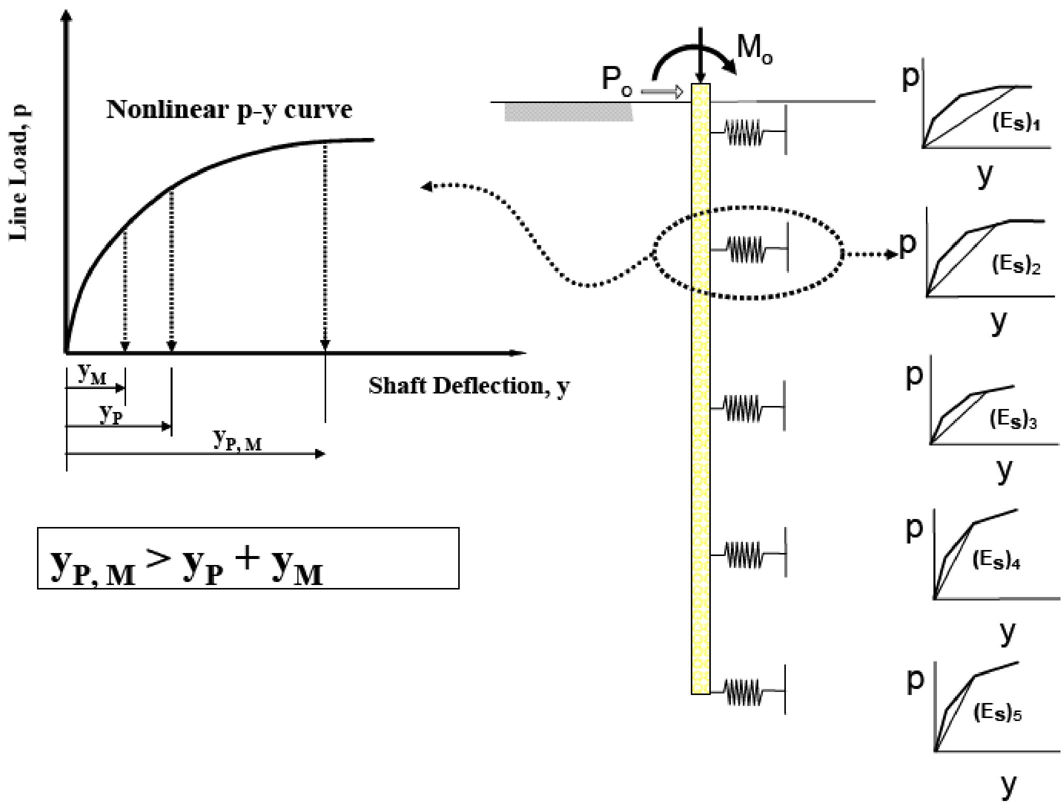
   Technique I – The matrix is generated, using superposition, to reproduce the non-linear behavior of the soil and foundation at the maximum loading. With Technique I, 10 terms are produced, 4 of these terms are "cross couples." Soil response programs, such as Lpile or DFSAP, analyze the non-linear soil 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.

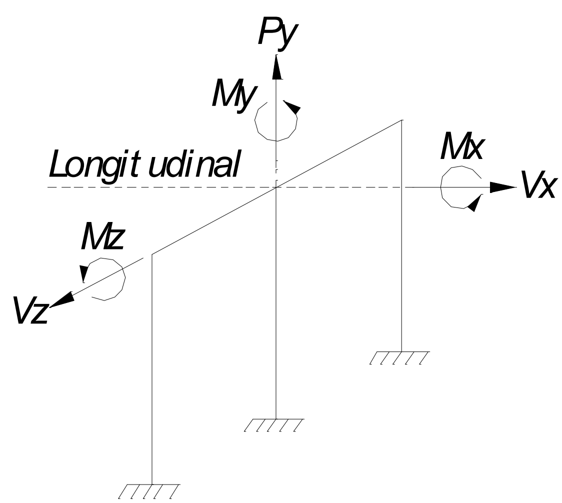
The key to determine the correct boundary condition is to resolve the correct sign of the moment and shear at the top of the shaft (or point of interest for the spring location). Since multi-mode results are always positive (CQC), this can be worked out by observing the seismic moment and shear diagrams 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.

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

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

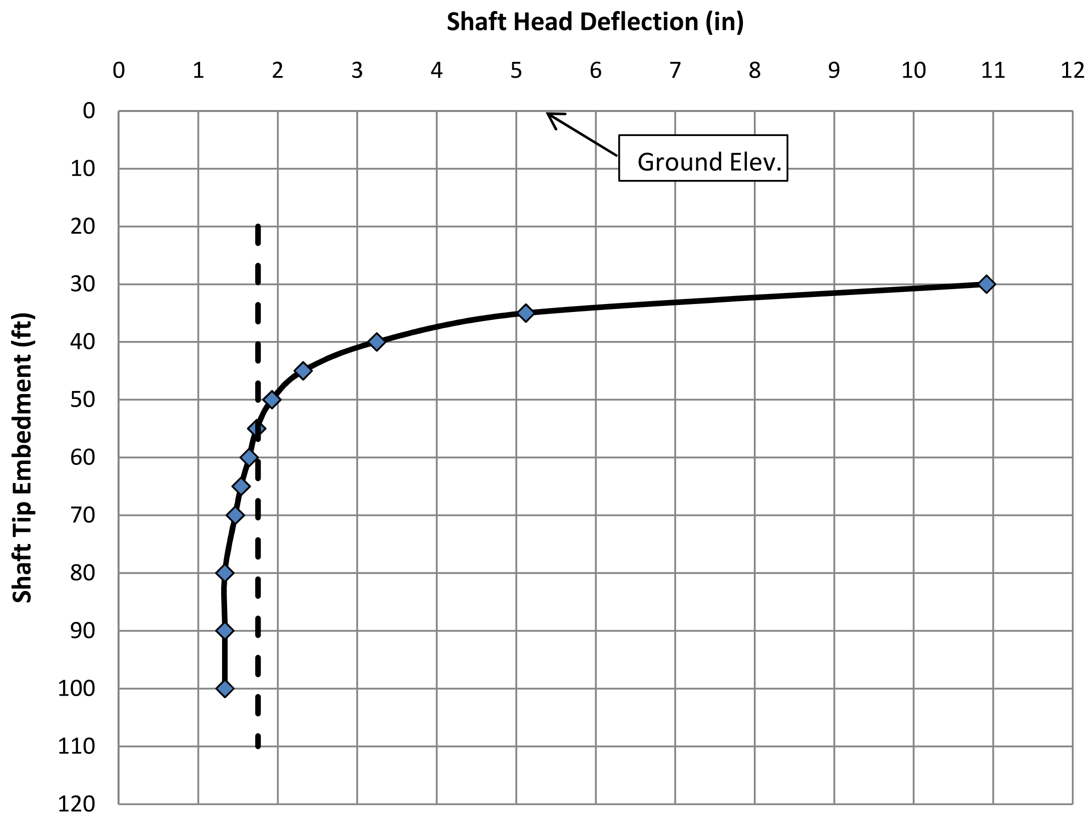
Lateral analysis of piles and shafts involves determination of a shaft or pile tip location sufficient to resist lateral loads in both orthogonal directions. In many cases, the shaft or pile tip depth required to resist lateral loads may be deeper than that required for bearing or uplift. However, a 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.

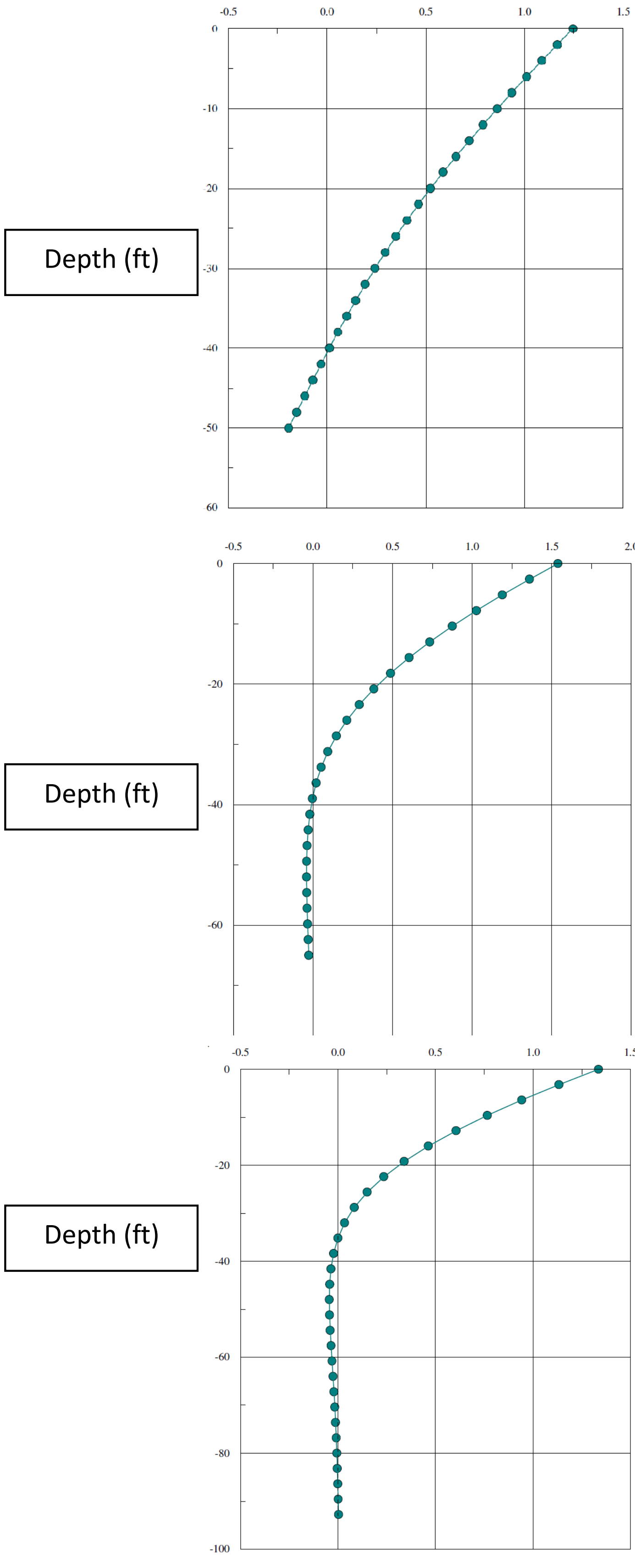
1. Size columns and determine column reinforcement requirements for Strength and Service load cases.
2. Determine the column plastic over-strength moment and shear at the base of the column using the axial dead load and expected column material properties. A program such as Xtract or SAP2000 may be used 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.

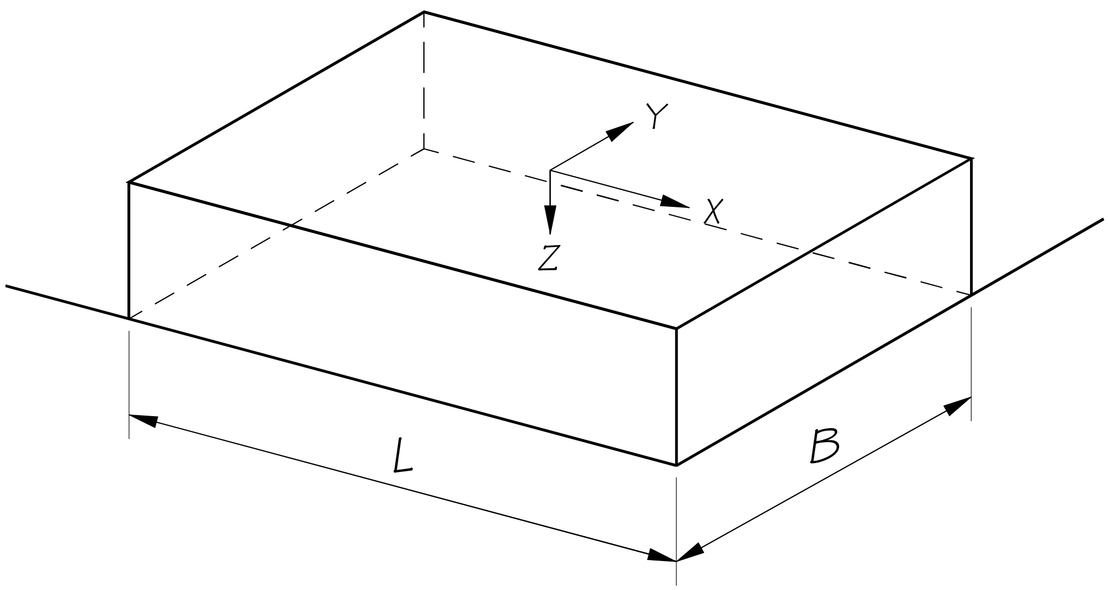
Note: Often, the highest acceleration the bridge sees is in the first cycles of the earthquake, and degradation and/or liquefaction of the soil tends to occur toward the middle or end of the earthquake. Therefore, early in the earthquake, loads are high, soil-structure stiffness is high, and deflections 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 *Geotechnical Design Manual* M 46-03 Chapter 6 for further discussion and consult the geotechnical engineer for guidance.

Refer to Section7.8 Shafts and Chapter 4 for additional guidance/requirements on design and detailing of 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.)



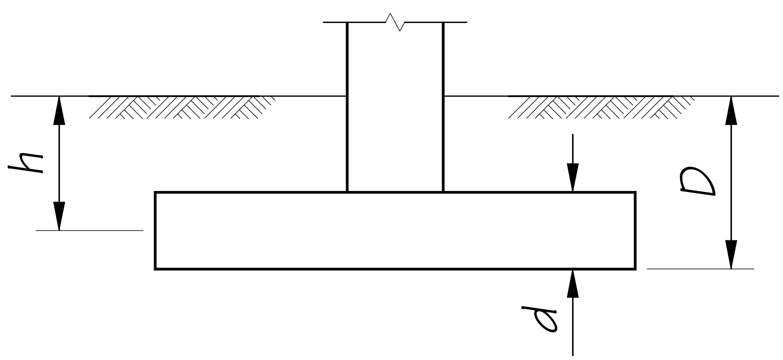
Orient axes such that $L > B$.
If $L = B$ use x-axis equations for both x-axis and y-axis.
Figure 7.2.7-1

Where:

$K = \beta K_{sur}$
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

7.4 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.

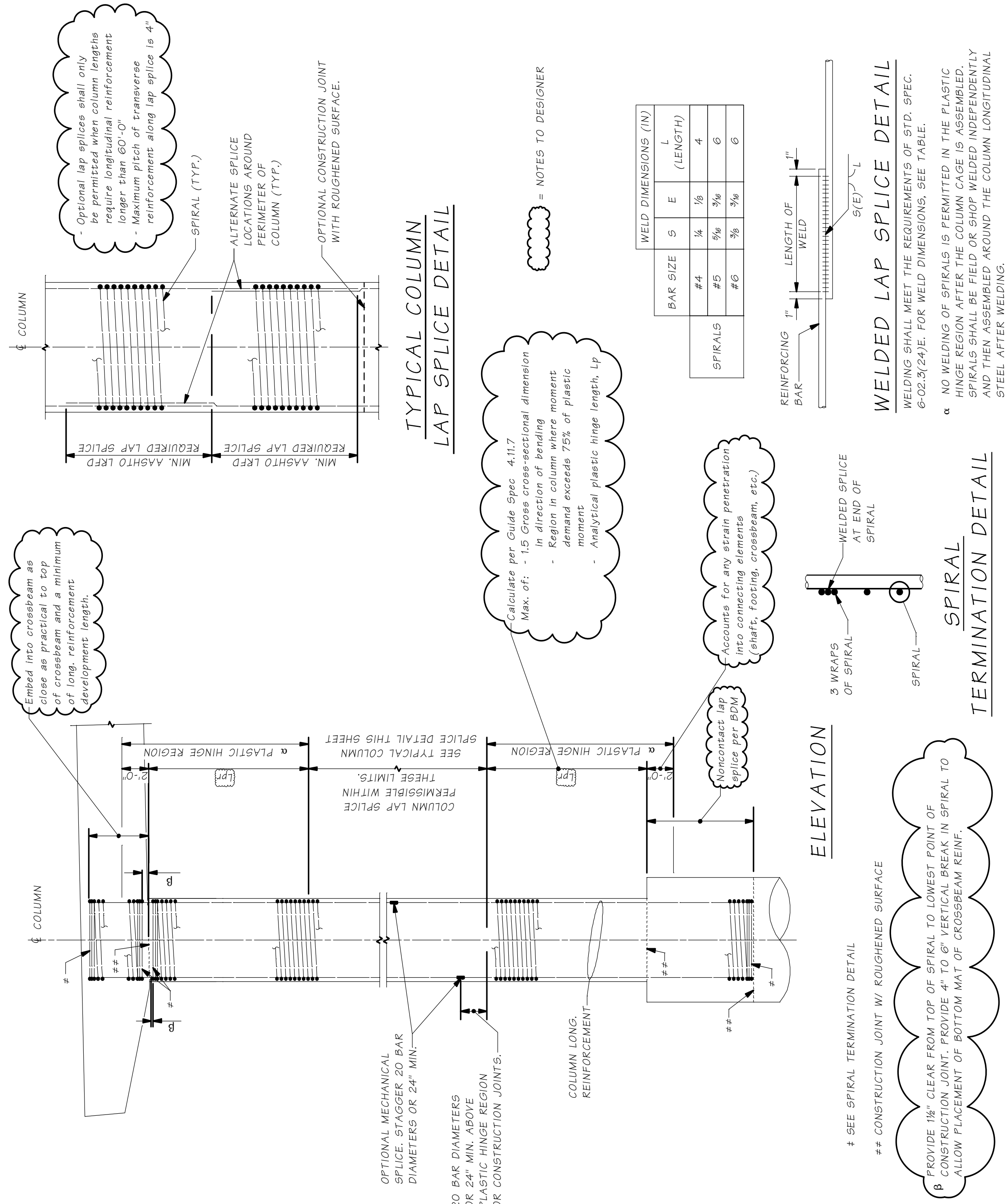
7.4.3 Longitudinal Splices

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. Longitudinal reinforcing shall be extended into the crossbeam as required for seismic joint design.

For precast prestressed concrete girder bridges in SDC A and B with fixed diaphragms at intermediate piers, column longitudinal reinforcement may be terminated at top of lower crossbeam, provided that adequate transfer of column forces is provided.

For precast prestressed concrete girder bridges in SDC C and D with two-stage fixed diaphragms at intermediate piers, all column longitudinal reinforcement should extend to the top of the cast-in-place concrete diaphragm (upper crossbeam) above the lower crossbeam. Careful attention should be given that column reinforcement does not interfere with extended strands projecting from the end of the prestressed concrete girders. In case of interference, column longitudinal reinforcement obstructing the extended strands may be terminated at top of the lower crossbeam, and shall be replaced with equivalent full-height stirrups extending from the lower to upper crossbeam within the effective width as shown in Figure 7.4.4-1. All stirrups within the effective zone, based on an approximate strut-and-tie model, may be used for this purpose. The effective zone shall be taken as column diameter plus depth of lower crossbeam provided that straight column bars are adequately developed into the lower crossbeam. The effective zone may be increased to the column diameter plus two times depth of lower crossbeam if headed bars are used for column longitudinal reinforcement.

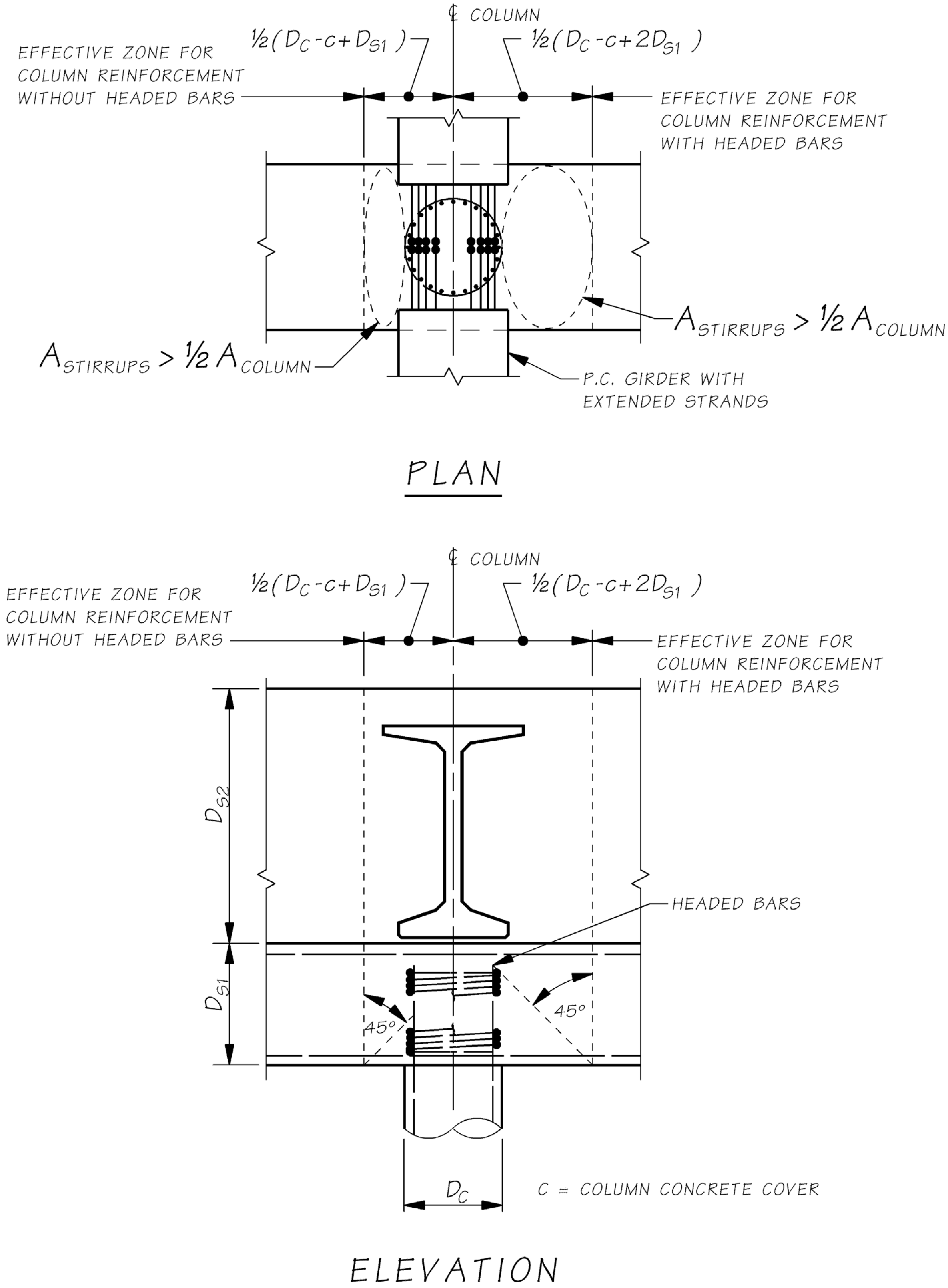
If the depth of lower crossbeam is less than 1.25 times the tension development length required for column reinforcement, headed bars shall be used. Heads on column bars terminated in the lower crossbeam are preferable from a structural perspective. However, extra care in detailing during design and extra care in placement of the column reinforcement during construction is required. Typically the heads on the column bars will be placed below the lower crossbeam top mat of reinforcement. Headed reinforcement shall conform to the requirements of ASTM 970 Class HA.

Transverse column reinforcement only needs to extend to the top of the lower crossbeam just below the top longitudinal steel. However, when the joint shear principal tension is less than $0.11\sqrt{f'_c}$, minimum cross tie reinforcement shall be placed acting across the upper cross beam in accordance with the AASHTO Seismic 8.13.3. The minimum cross tie reinforcement shall provide at least as much confining pressure at yield as the column spiral can provide at yield. This pressure may be calculated assuming hydrostatic conditions. If the joint shear principal tension exceeds $0.11\sqrt{f'_c}$, then additional joint reinforcement as outlined by AASHTO Seismic 8.13.3 shall be provided. With the exception of J-bars, the additional reinforcement shall be placed in the upper and lower crossbeam. The cross tie reinforcement may be placed with a lap splice in the center of the joint.

Large columns or columns with high longitudinal reinforcement ratios may result in closely spaced stirrups with little clear space left for proper concrete consolidation outside the reinforcement. In such cases, either hanger reinforcement comprised of larger bars with headed anchors may be used in the effective zone shown in Figure 7.4.4-1or supplemental stirrups may be placed beyond the effective zone. Hanger reinforcement in the effective zone is preferred.

The designer is encouraged to include interference detail/plan views of the crossbeam reinforcement in relation to the column steel in the contract drawings. Suggested plans include the views at the lower stage crossbeam top reinforcement and the upper crossbeam top reinforcement.

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.



Longitudinal Development Into Crossbeams
Figure 7.4.4-1

C. **Shafts –** Column longitudinal reinforcement in 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.

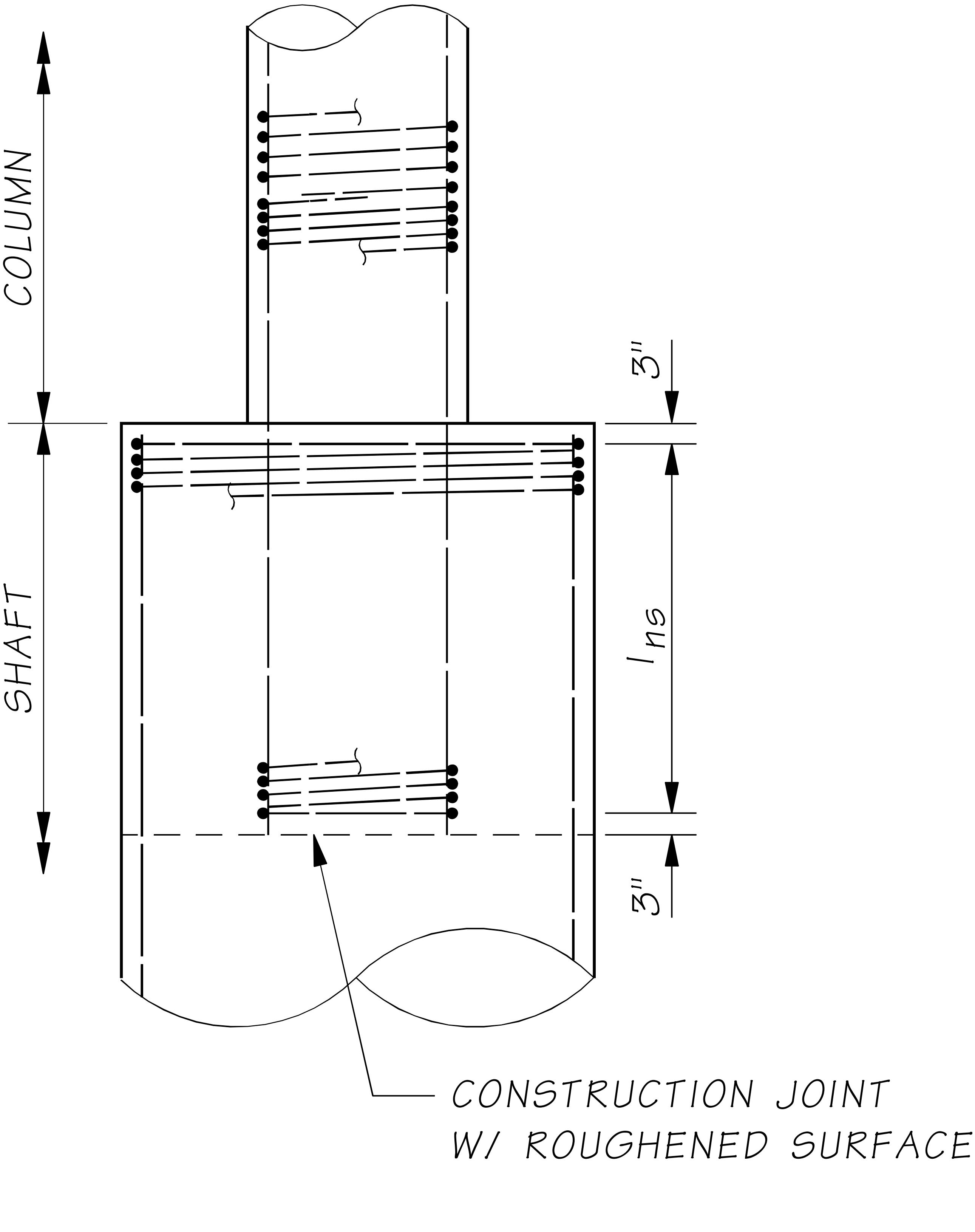
l_d = tension development length from AASHTO LRFD Section 5.11.2.1 for the column longitudinal reinforcement.

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 7.4.4-2 for an example of longitudinal development into shafts.



Longitudinal Development Into Shafts
Figure 7.4.4-2

7.4.5 Transverse Reinforcement

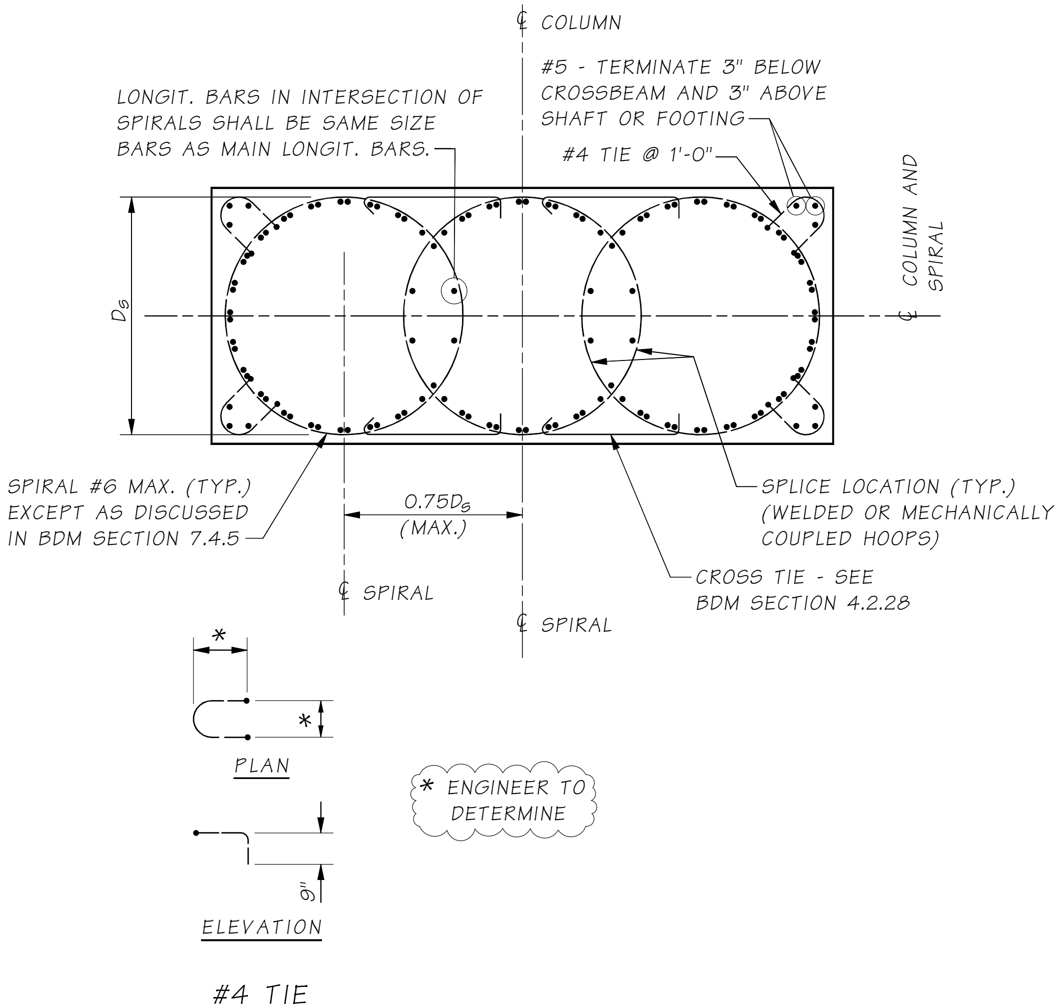
A. **General –** All transverse reinforcement in columns shall be deformed. Although allowed in the AASHTO LRFD Specification, plain bars or plain wire may not be used for transverse reinforcement.

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

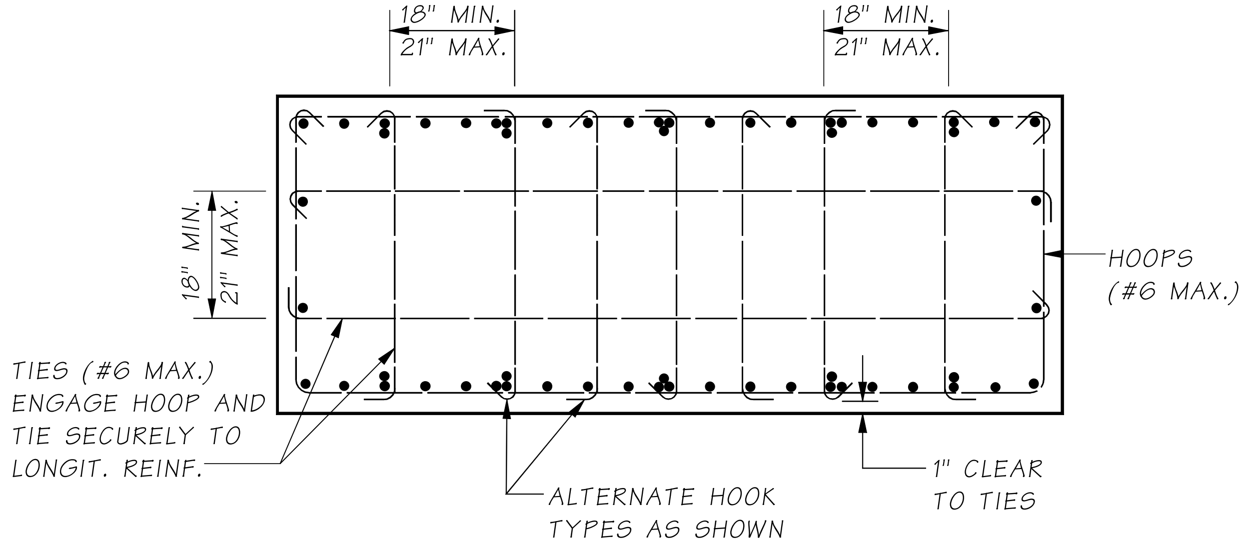
Columns in SDC B, C, and D shall use spiral or circular hoop transverse confinement reinforcement where possible, although rectangular hoops with ties may be used when large, odd shaped column sections are required.

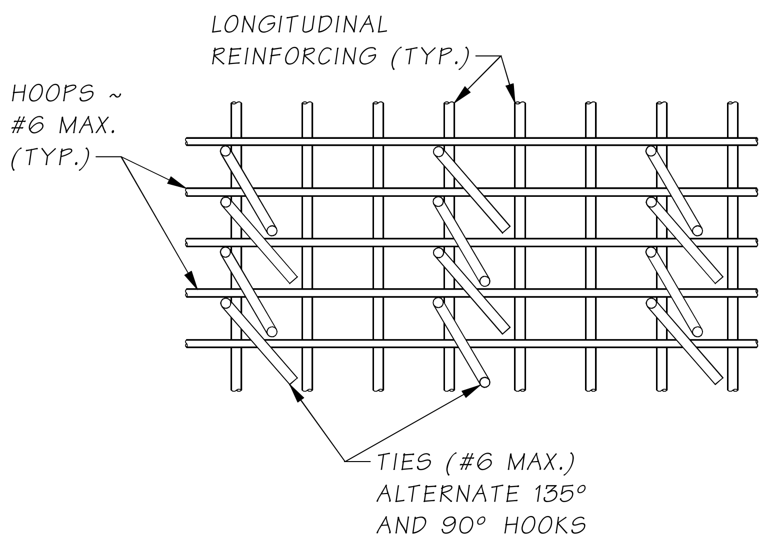
Spirals are the preferred confinement reinforcement and shall be used whenever a #6 spiral is sufficient to satisfy demands. When demands require reinforcement bars greater than #6, circular hoops of #7 through #9 may be used. Bundled spirals shall not be used for columns or shafts. Also, mixing of spirals 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.

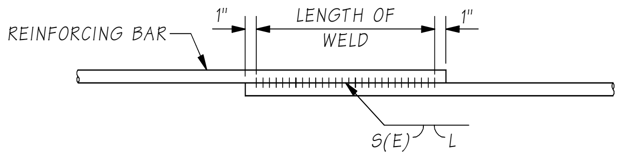
Although hooked lap splices are structurally acceptable, and permissible by AASHTO LRFD Specification for spirals or circular hoops, they shall not be allowed due to construction challenges. While placing concrete, tremies get caught in the protruding hooks, making accessibility to all areas and its 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.

Columns with circular hoop reinforcement shall have a minimum 2" concrete cover to the hoops to accommodate resistance butt weld “weld flash” that can extend up to ½" from the bar surface.

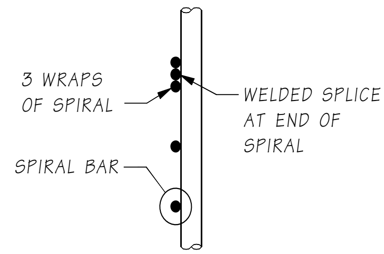
Field welded lap splices and termination welds of spirals of any size bar are not permitted in the plastic hinge region and should be clearly designated on the contract plans. If spirals are welded while in place around longitudinal steel reinforcement, there is a chance that an arc can occur between 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.



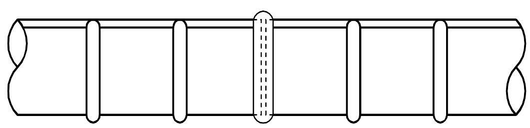
WELDED LAP SPLICE DETAIL

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

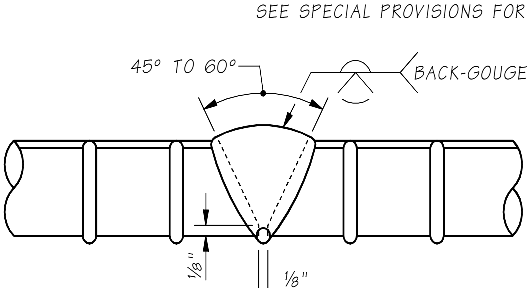


SPIRAL TERMINATION DETAIL



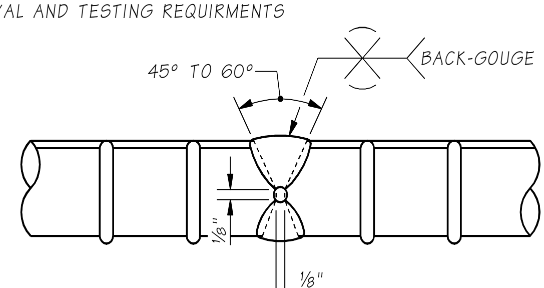
RESISTANCE BUTT JOINT DETAIL

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.

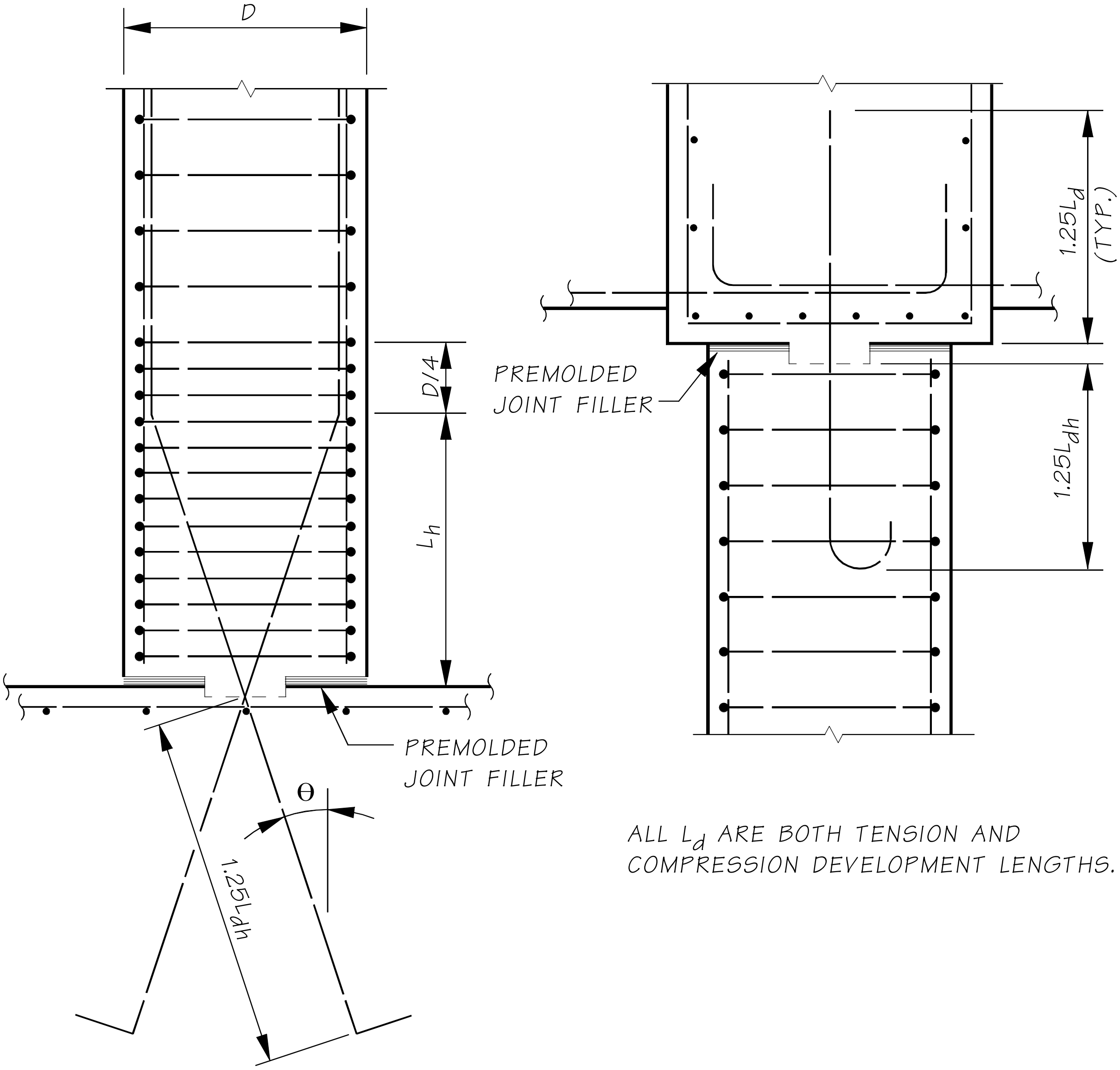
When the hinge reinforcement is bent, additional confinement reinforcing may be necessary to take the horizontal component from the bent hinge bars. The maximum spacing of confinement reinforcing for the hinge is the smaller of that required above and the following:

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

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**

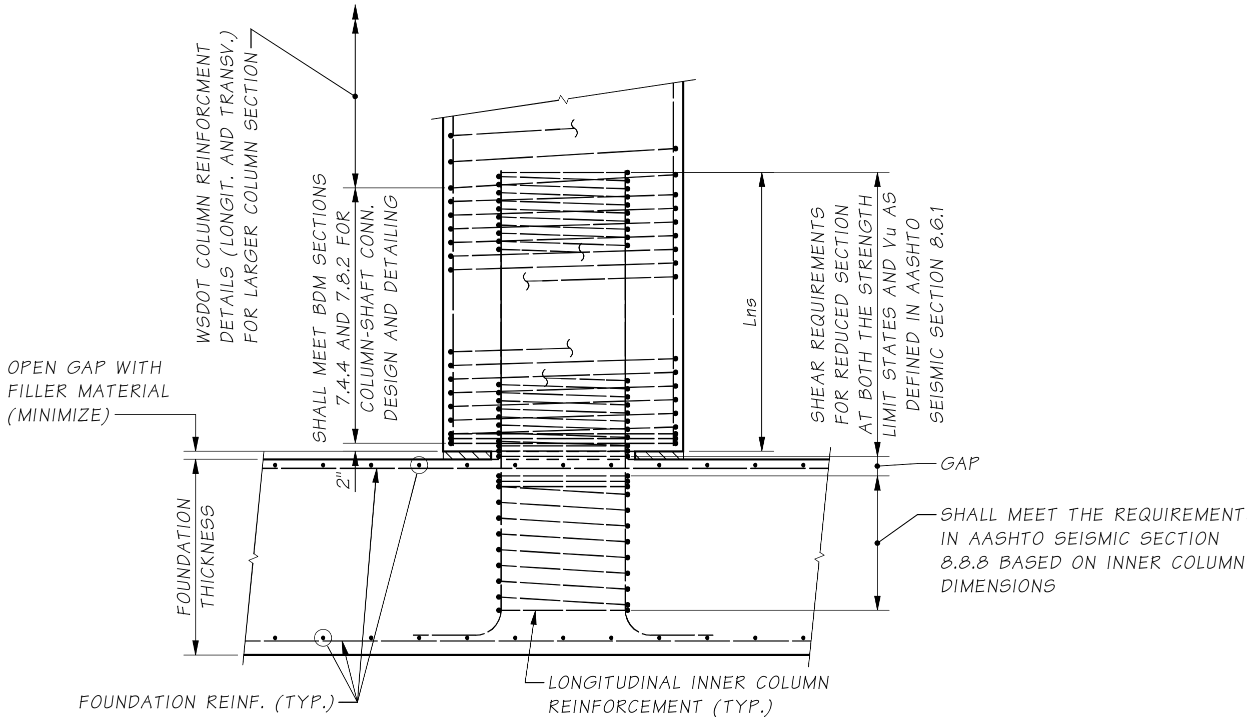
1. The WSDOT Bridge and Structures Office normal practices and procedures shall be met for the column design, with the following exceptions:

   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.



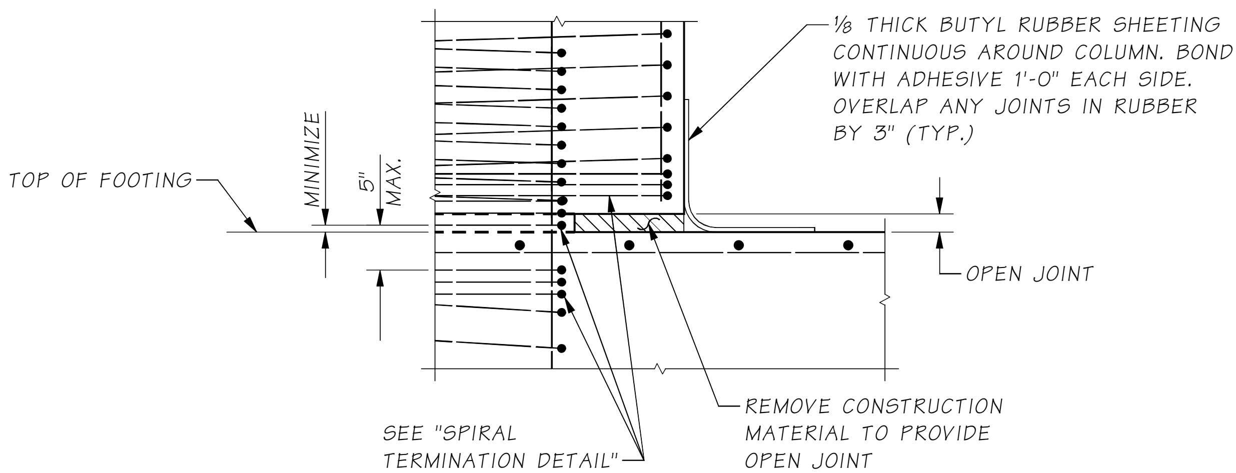
Pinned Column Base Reduced Column Fixity at Base
Figure 7.4.7-1

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.



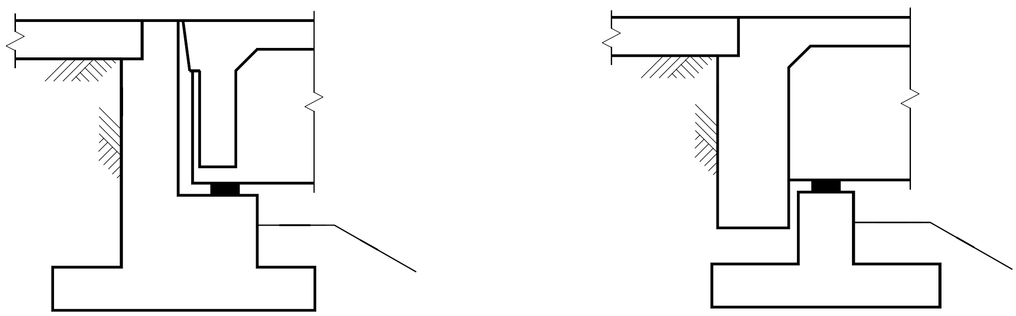
Open Gap Detail
Figure 7.4.7-2

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.

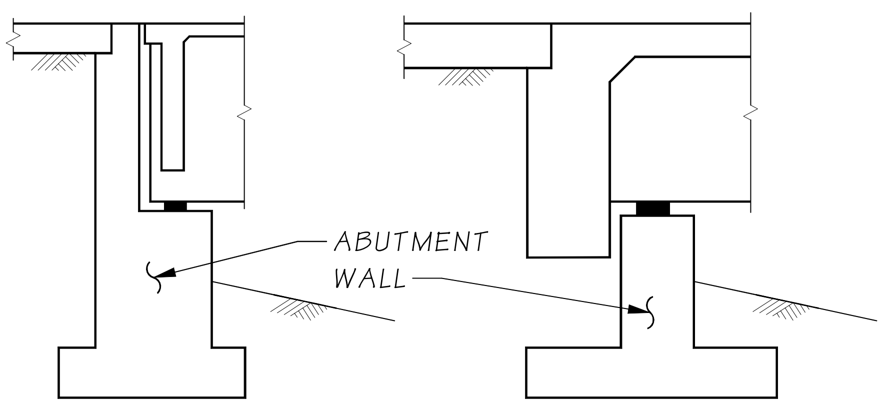
1. **Stub Abutments –** Stub abutments are short abutments where the distance from the girder seat to top of footing is less than approximately 4 feet, see Figure 7.5.1-1. The footing and wall can be considered as a continuous inverted T-beam. The analysis of this type abutment shall include investigation 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

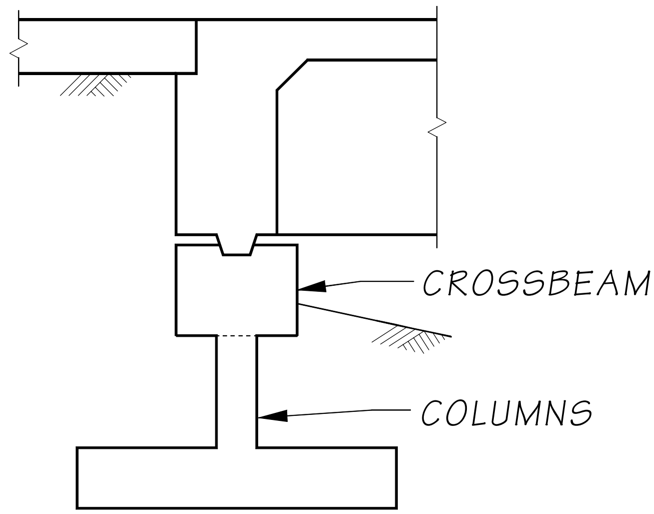
2. **Cantilever Abutments –** If the height of the wall from the bearing seat down to the bottom of the footing exceeds the clear distance between the girder bearings, the assumed 45° lines of influence from the girder reactions will overlap, and the dead load and live load from the superstructure can 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

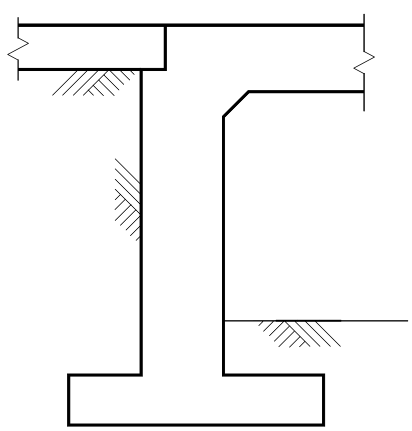
Figure 7.5.1-2

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).

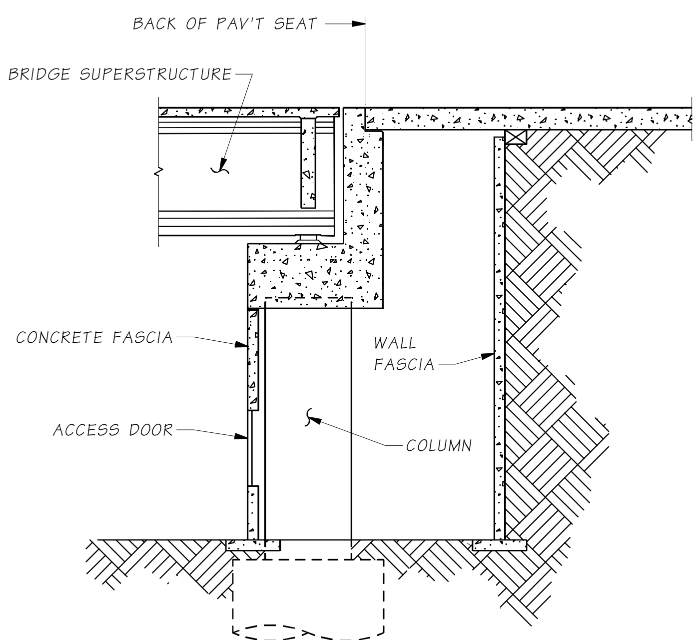


Rigid Frame Abutment
Figure 7.5.1-4

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″ above finished grade. Contact the State Bridge and Structures Architect for configuration and concrete surface treatments.



Bent-Type Abutment
Figure 7.5.1-5

6. **Abutments Supported by Mechanically Stabilized Earth Walls** – Bridge abutments may be supported on mechanically-stabilized earth (MSE) walls. Refer to Section 7.5.11 for specific requirements.

7.5.2 Embankment at Abutments

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.

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.

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.

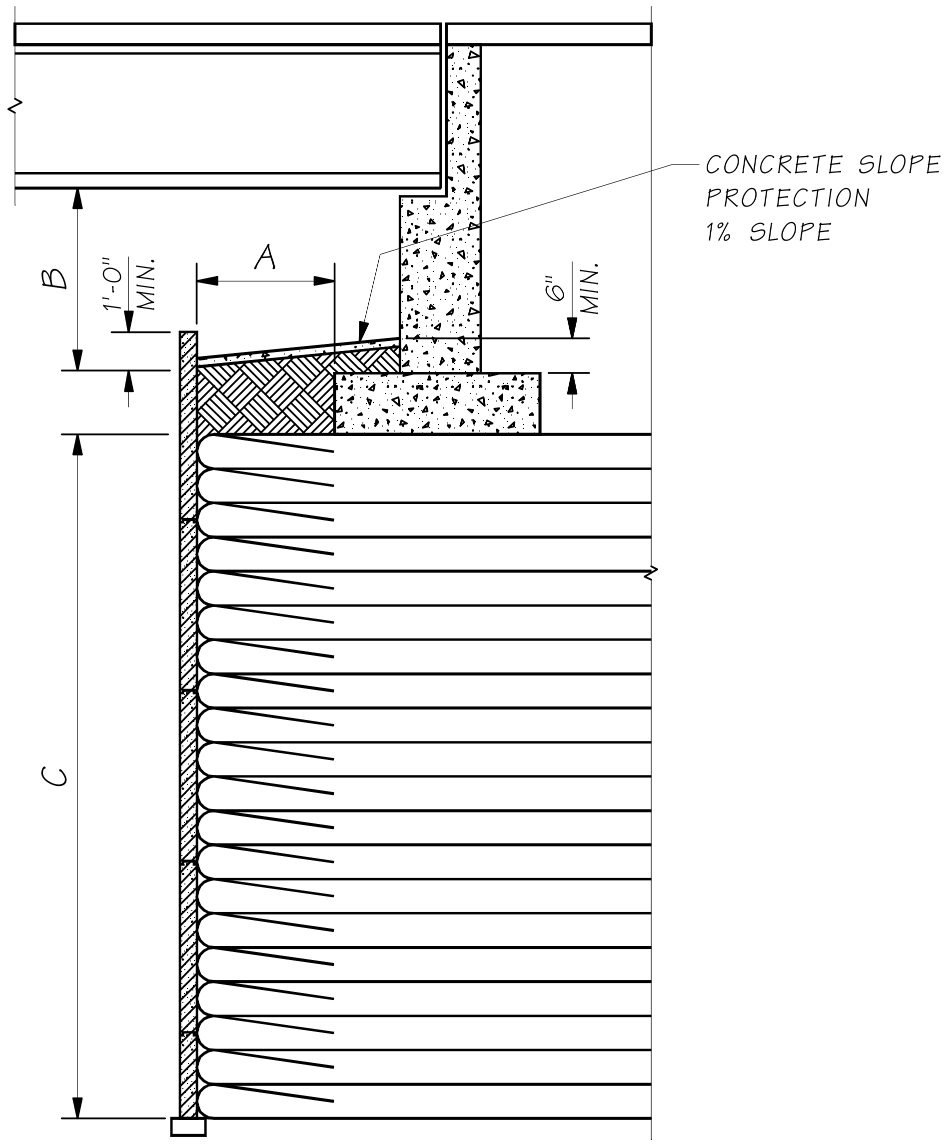
   For conventional footing supported abutments, the seismic horizontal acceleration coefficient, k_h, shall be taken as one half of the seismic horizontal acceleration coefficient assuming zero displacement, k_{h0}. Seismic active earth pressure, K_{AE}, shall be assumed to be a uniform pressure over the 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.



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

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.

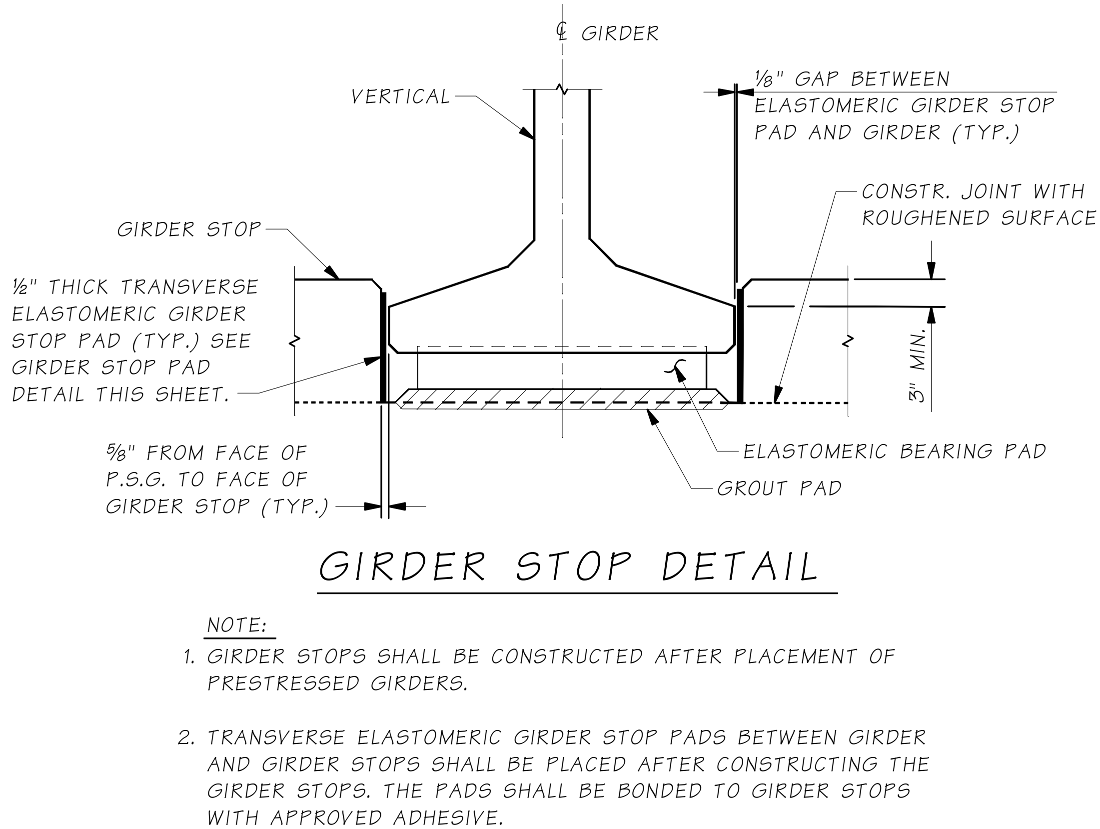
C. **Transverse Girder Stops** – Transverse girder stops are required for all abutments in order to transfer lateral loads from the superstructure to the abutment. Abutments shall normally be considered as part of the Earthquake Resisting System (ERS). Girder stops shall be full width between girder 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 within the face 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.

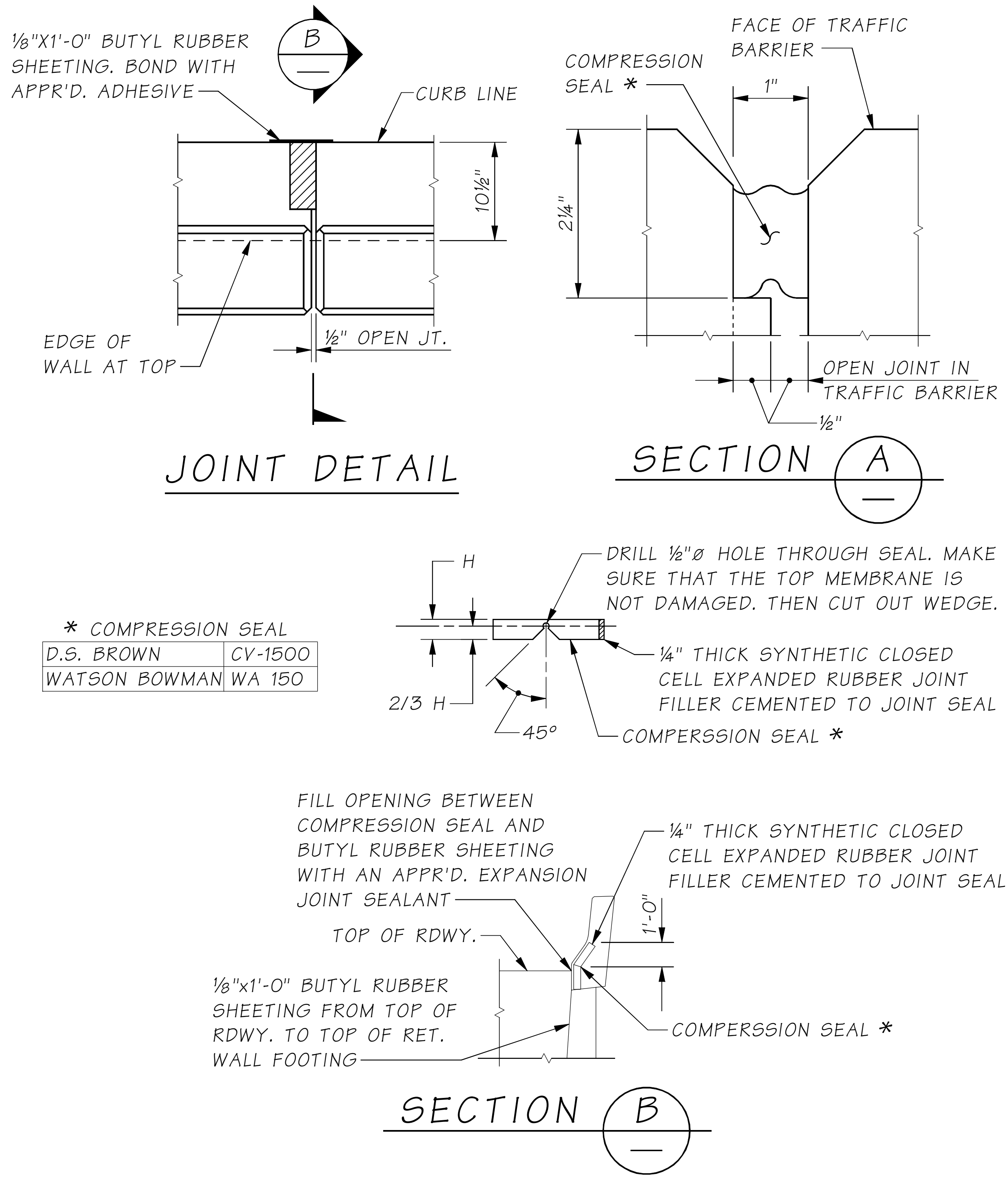
7.5.9 Abutment Wall Design

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.

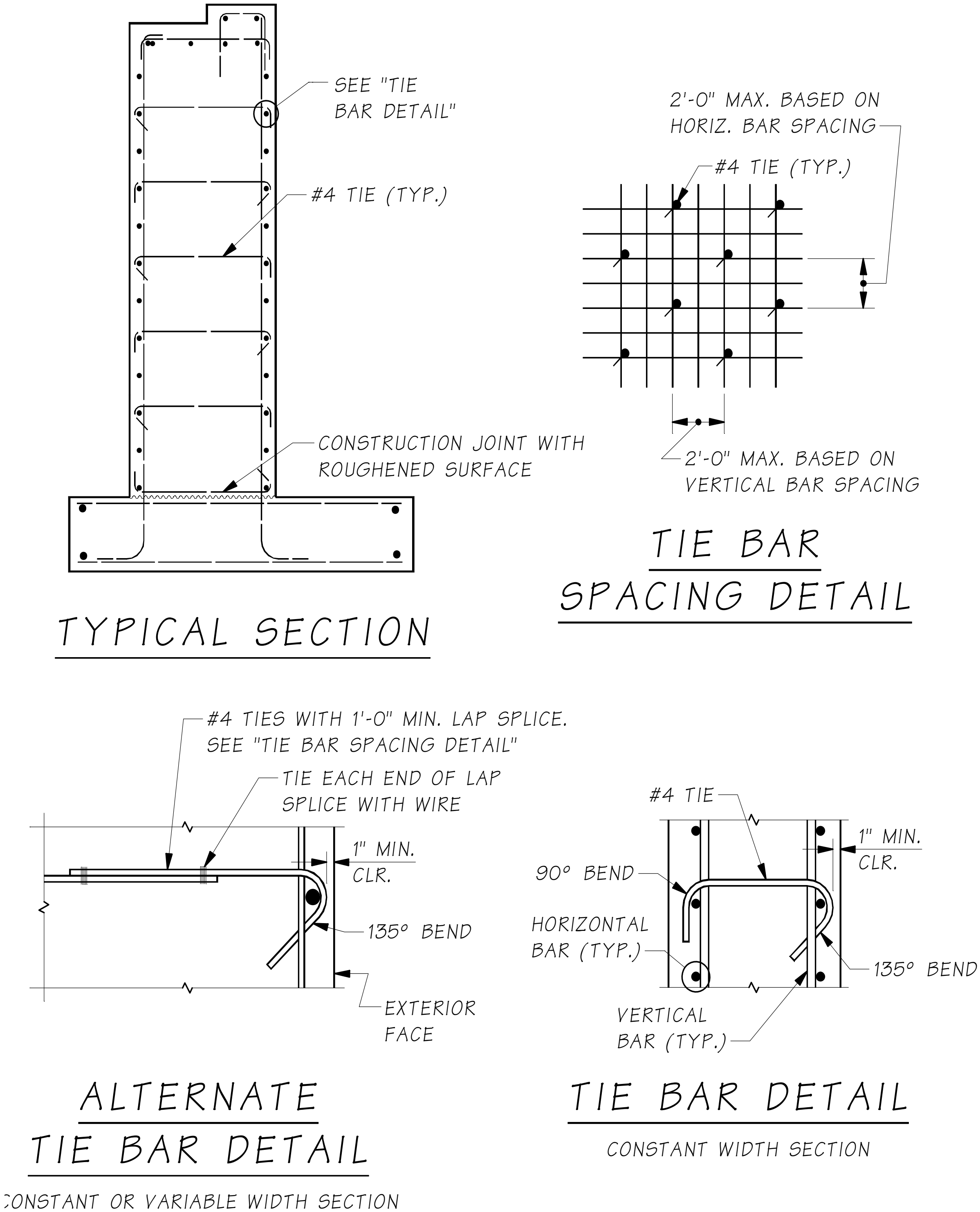
C. **Cross Ties** – The minimum cross tie reinforcement in abutment walls, except stub abutments, shall be #4 tie bars with 135° hooks, spaced at approximately 2′-0″ maximum center-to-center vertically and horizontally, see Figure 7.5.9-1.



* COMPRESSION SEAL	
D.S. BROWN	CV-1500
WATSON BOWMAN	WA 150

Open Joint Details Between Abutment and Retaining Walls

Figure 7.5.7-1



Cross Tie Details
Figure 7.5.9-1

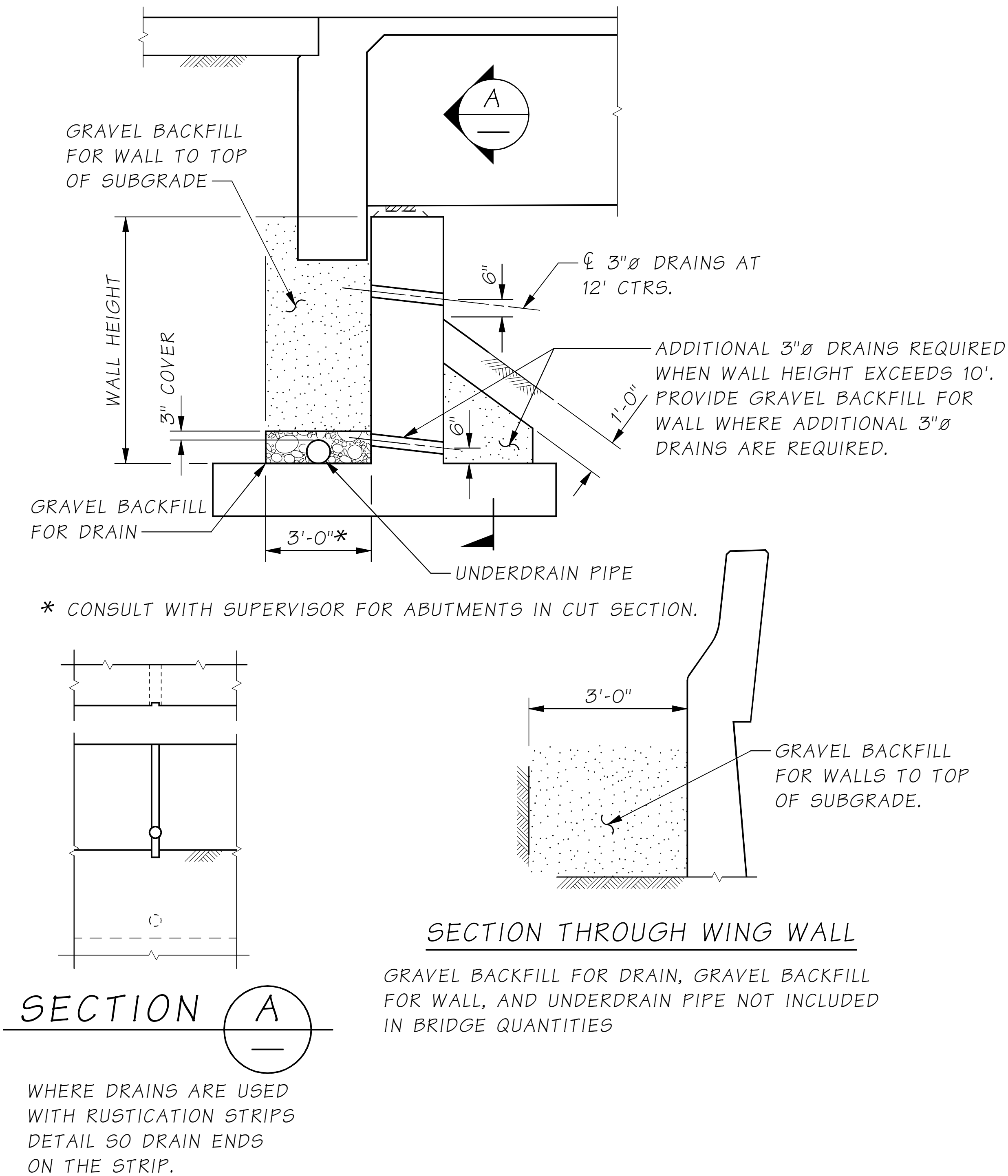
7.5.10 Drainage and Backfilling

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

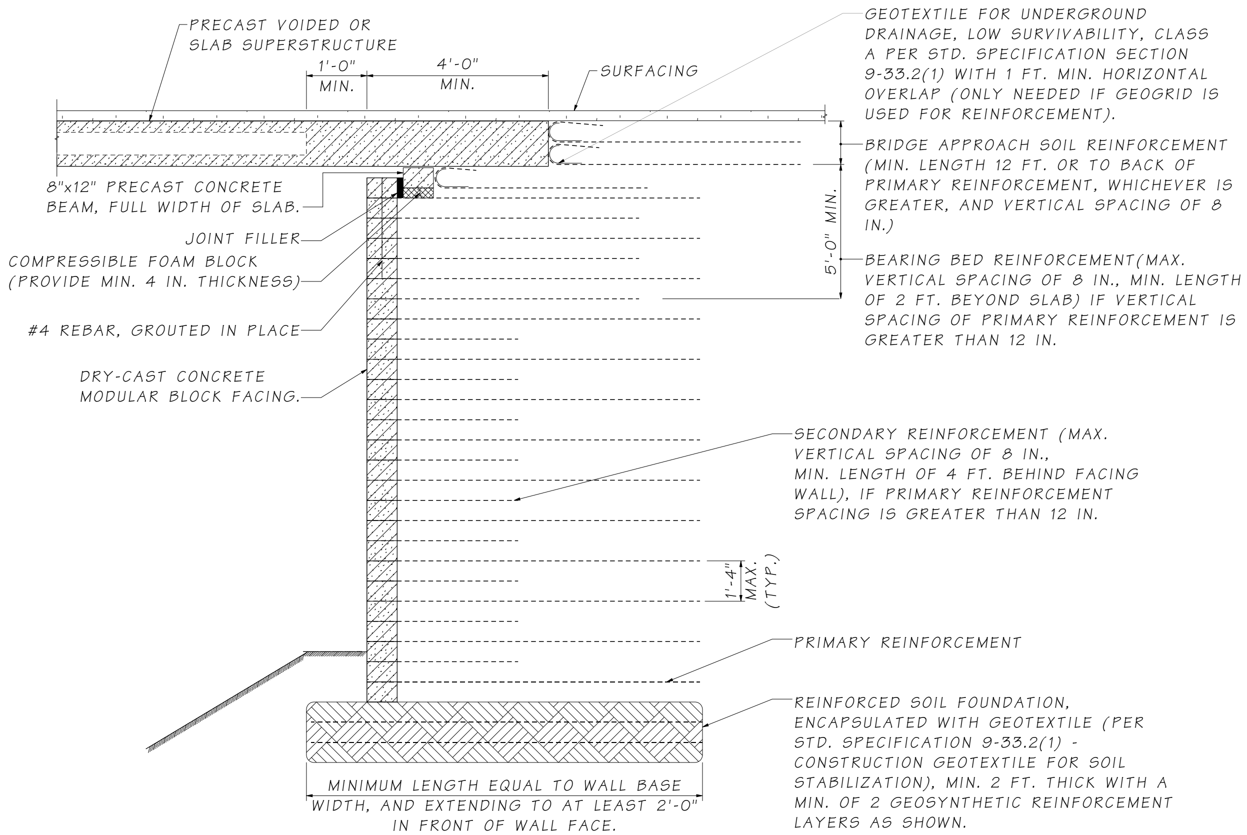
7.5.11 Abutments Supported By Mechanically-Stabilized Earth Walls

Bridge abutments may be supported on mechanically-stabilize earth (MSE) walls, including geosynthetic retaining walls (with and without structural facing), structural earth walls and reinforced soil. 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):

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

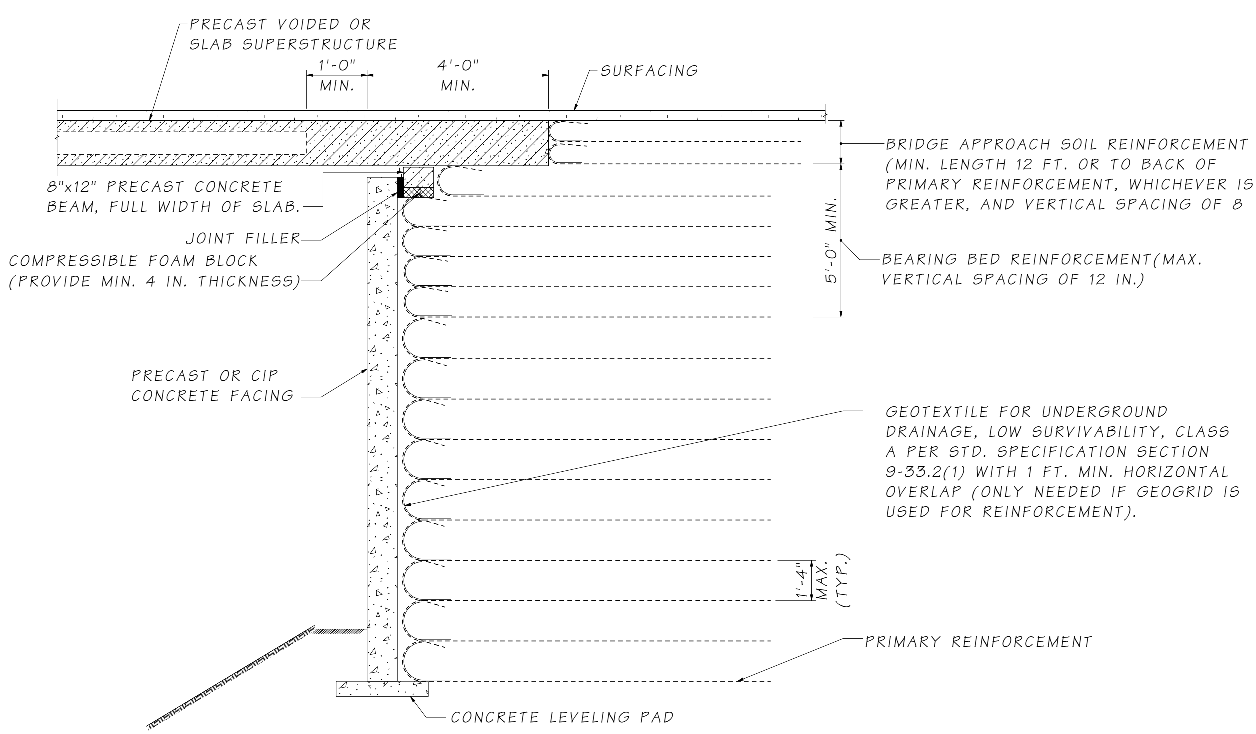
Bridges with MSE supported abutments shall be designed as one of two types described below, and shall satisfy the associated design requirements.

A. **Single-span bridges with precast slab superstructures supported directly on reinforced soil –** These bridges shall conform to the following requirements:
   1. Walls supporting abutments shall be special designed wall systems, and shall be one of two types:
      a. Geosynthetic walls with a stacked dry-cast modular concrete block facing. The top 3 rows of dry-cast modular concrete blocks shall be grouted with #4 rebar. See Figure 7.5.11-1
      b. Geosynthetic and structural earth walls with full-height concrete facing. See Figure 7.5.11-2
   2. The span length shall not exceed 60 feet.
   3. The superstructure shall include a 5” thick C.I.P. composite topping.
   4. The end of the precast superstructure shall be at least 4 ft. from the face of the MSE wall. Minimum seat width requirements shall be provided on the reinforced soil bearing area.
   5. A foam board detail shall be used to create a 1 ft. horizontal buffer between the bearing area and the wall facing.
   6. The vertical gap between the top of wall facing and the bottom of superstructure shall be 4” or 2% of the abutment height, whichever is greater.
   7. Prestressing strands in the zone bearing on reinforced soil shall have a minimum concrete cover of 2”. Transverse reinforcing steel within this zone shall have a minimum concrete cover of 1½”. All prestressing strand shall be removed to a 2” depth from the end of the slab. The voids shall be patched with epoxy grout.
   8. Where voided slab superstructures are used, the slab section shall be solid from the end of the slab to at least 1 ft. in front of the fascia.
   9. The abutment shall be designed for a bearing pressure at service loads not to exceed 2.0 TSF and a factored load at strength and extreme limit states not to exceed 3.5 TSF. The bearing pressure may be increased to 3.0 TSF at service loads and 4.5 TSF at strength and extreme limit states if a vertical settlement monitoring program is conducted in accordance with WSDOT GDM Section 15.5.3.5.
   10. Bridge approach slabs may be omitted.



Reinforced Soil Abutment with Dry-Cast Modular Block Facing

Figure 7.5.11-1



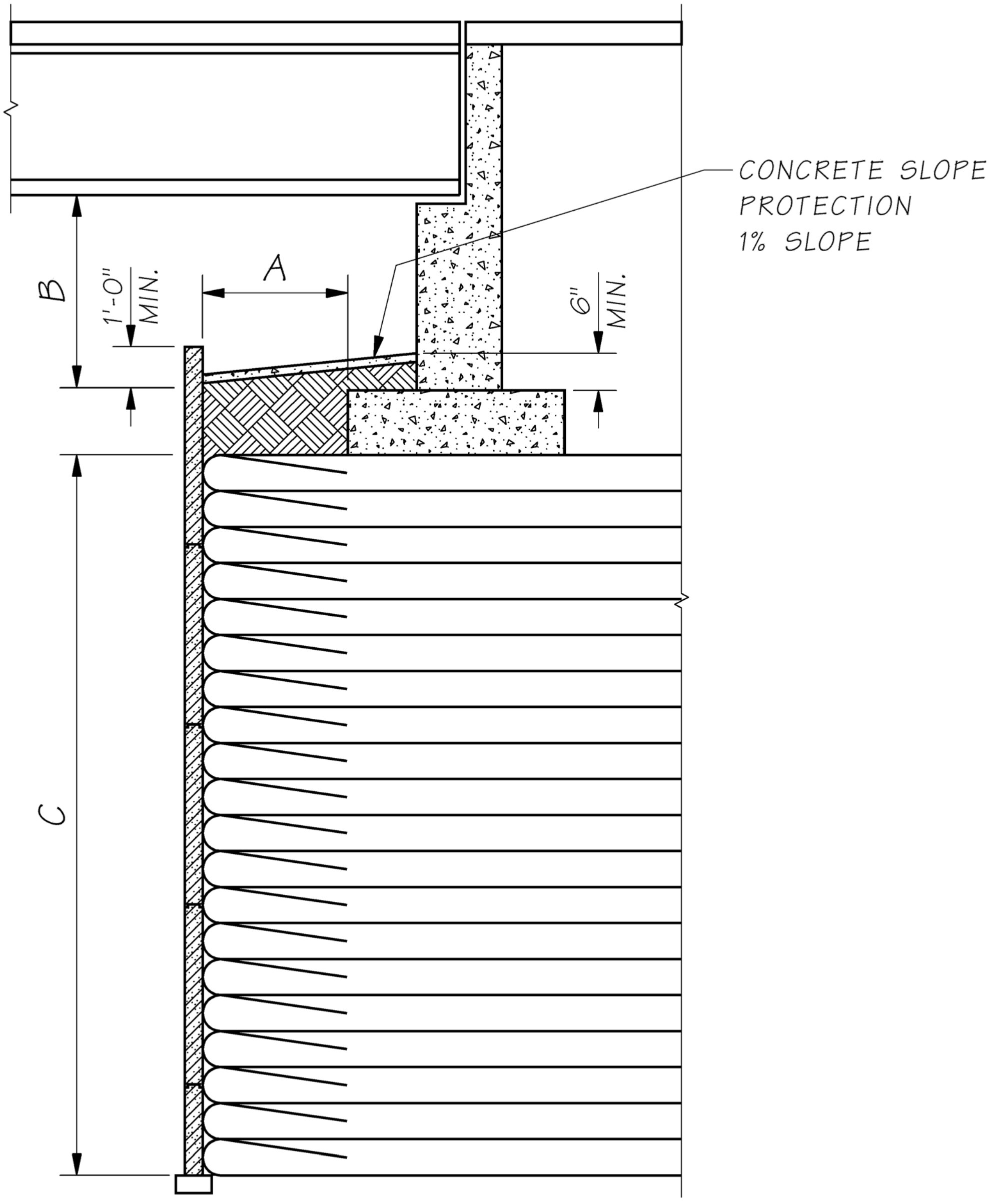
Reinforced Soil Abutment with Full-Height Concrete Facing

Figure 7.5.11-2

B. **Bridges with spread footing abutments supported by a geosynthetic wall or SE wall** – These bridges shall conform to the following requirements, see Figure 7.5.11-3:

1. Walls shall be 30 feet or less in total height, which includes the retained soil height up to the bottom of the embedded spread footing.
2. For SE walls, the front edge of the bridge footing shall be placed 4 ft. minimum from the back face of the fascia panel. For geosynthetic retaining walls with a wrapped face, the front edge of the bridge footing shall be placed 2 ft. minimum from the back face of the fascia panel.
3. The abutment footing shall be covered by at least 6 inch of soil for frost protection.
4. The superstructure of continuous span bridges shall be designed for differential settlement between piers.
5. Abutment spread footings shall be designed for bearing pressure at service loads not to exceed 2.0 TSF and factored load at strength and extreme limit states not to exceed 3.5 TSF. The bearing pressure may be increased to 3.0 TSF at service loads and 4.5 TSF at strength and extreme limit states if a vertical settlement monitoring program is conducted in accordance with WSDOT GDM section 15.5.3.5.
6. Walls supporting bridge abutments shall be special designed wall systems. Walls supporting permanent bridges shall have precast or C.I.P concrete fascia panels. Walls supporting temporary bridges may use dry-cast modular concrete block or welded wire facing.
7. Concrete slope protection shall be provided. Fall protection shall be provided in accordance with *Design Manual* Section 730.

Deviations from the design requirements require approval from the State Bridge Design Engineer and the State Geotechnical Engineer.



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

Spread Footing on SE Wall or Geosynthetic Wall
Figure 7.5.11-3

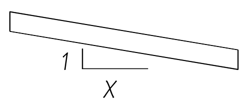
7.7 Footing Design

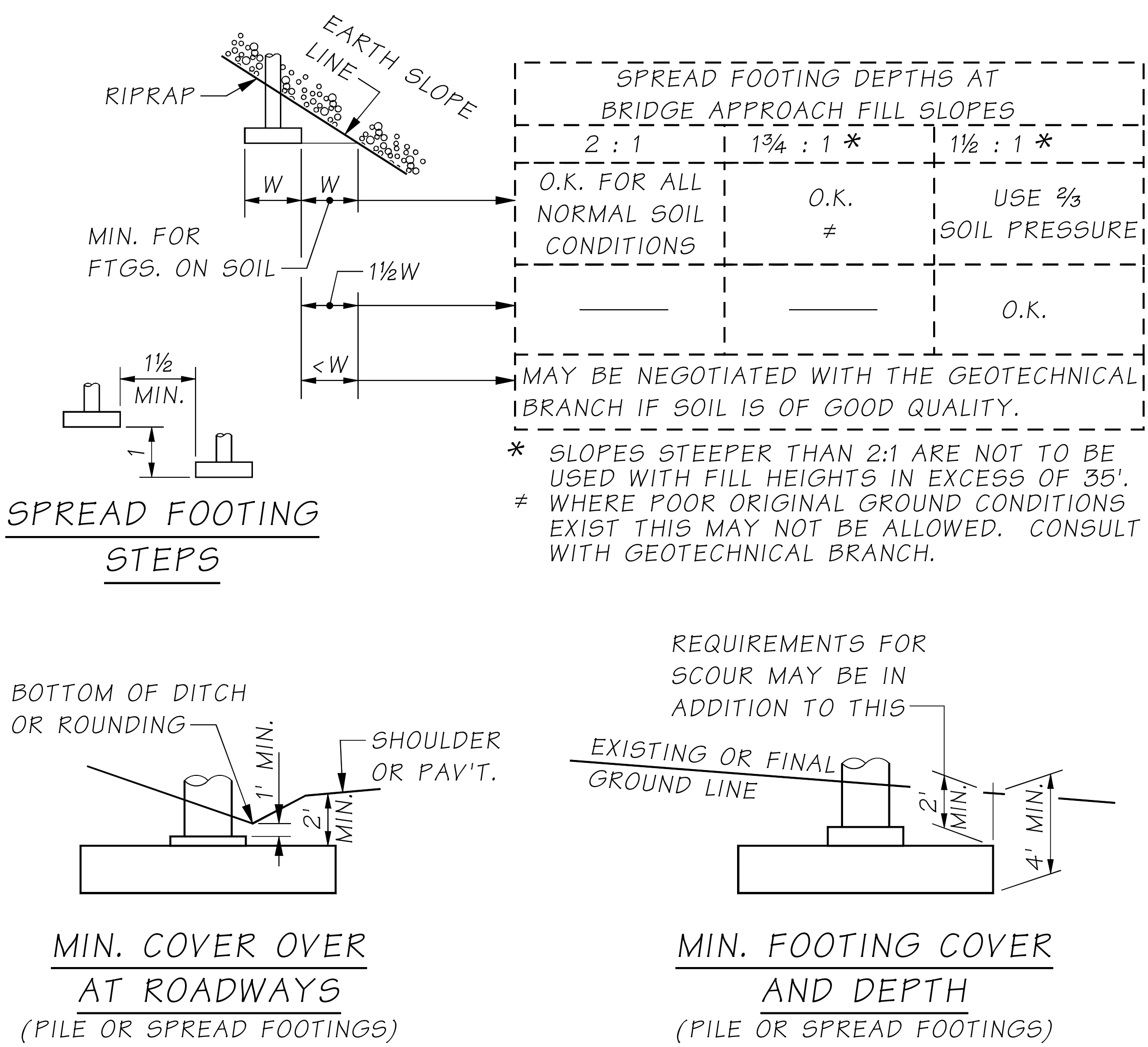
7.7.1 General Footing Criteria

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.

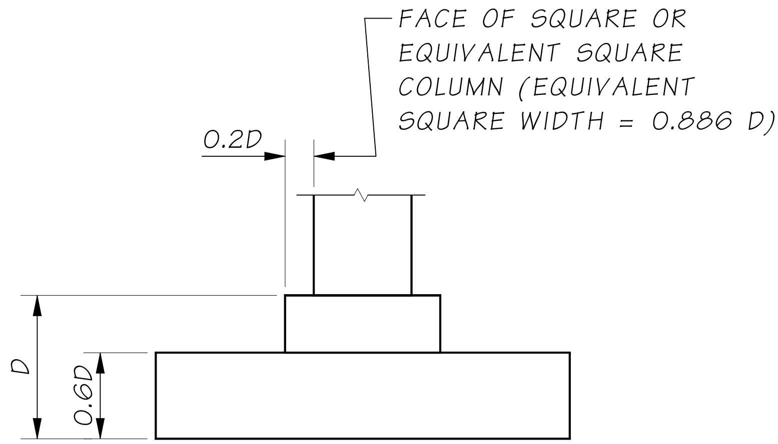
MAXIMUM WALL FOOTING SLOPE	
FOOTING TYPE	X : 1
PIER WALL	6 : 1
RETAINING WALL	6 : 1





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.

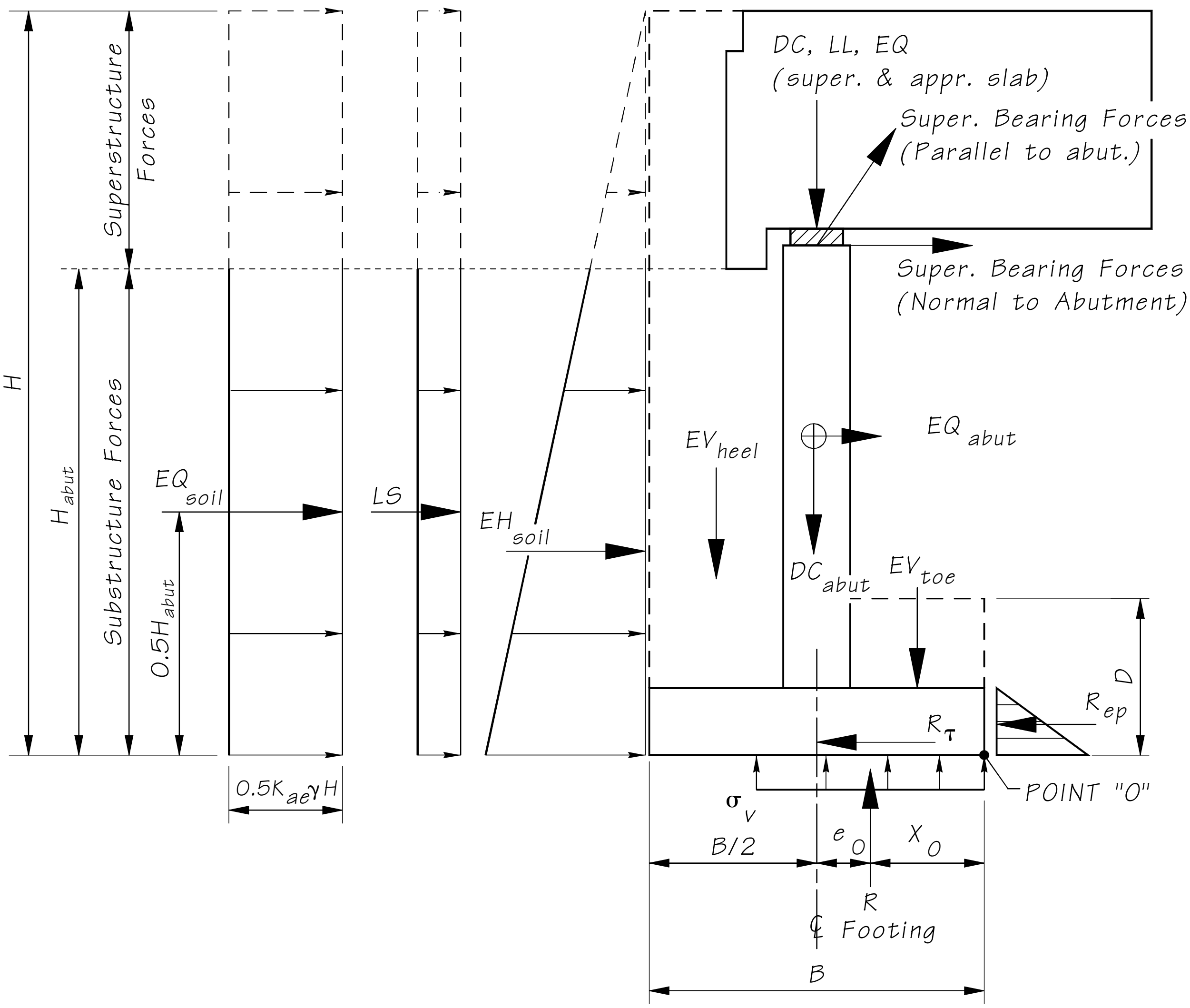
C. **Sliding Resistance - Strength and Extreme Event Limit States –** The geotechnical report will contain the following information to determine earth loads and the factored sliding resistance ($R_R = \phi R_n$)
   - Resistance factors for strength and extreme event limit states (ϕ_τ, ϕ_{ep})
   - If 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.

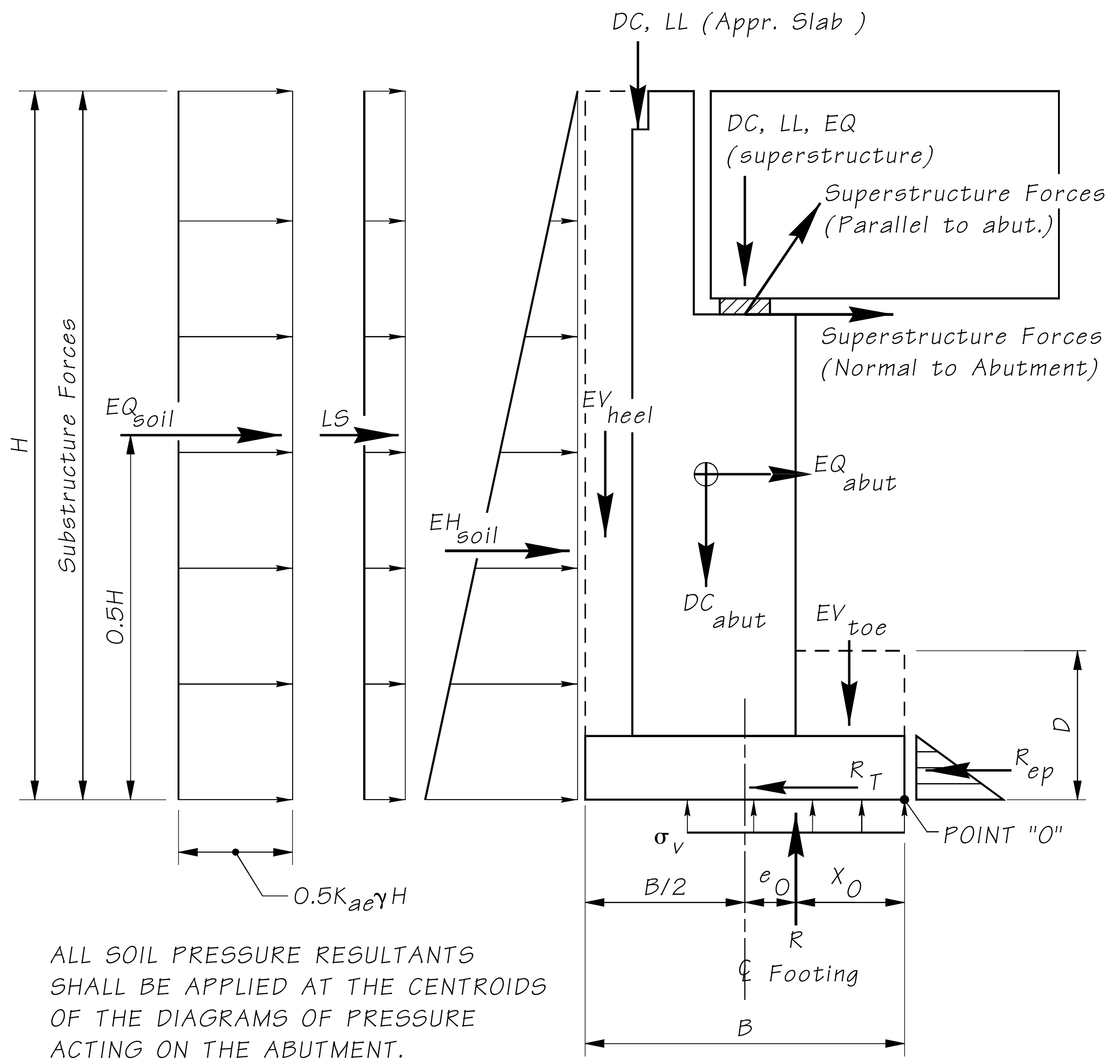
7.7.4 Spread Footing Design

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.

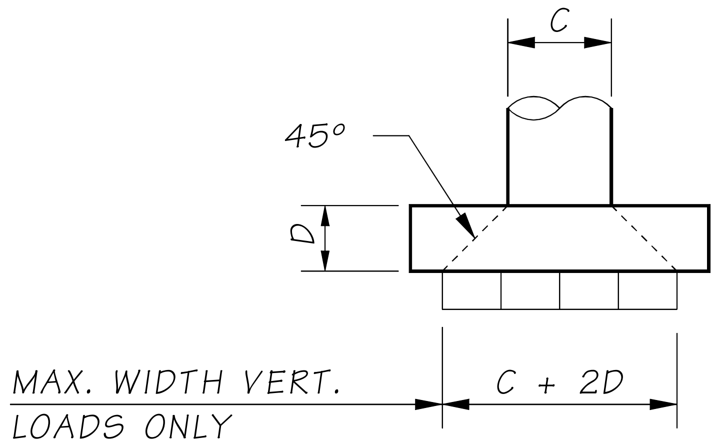
σ_{str} max = $R/B\ (1+ 6\ e/\mathrm{B})$

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
$tan\ \delta$	=	Coefficient of friction between the footing base and the soil
$tan\ \delta$	=	$tan\ \phi$ for cast-in-place concrete against soil
$tan\ \delta$	=	(0.8)$tan\ \phi$ 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.

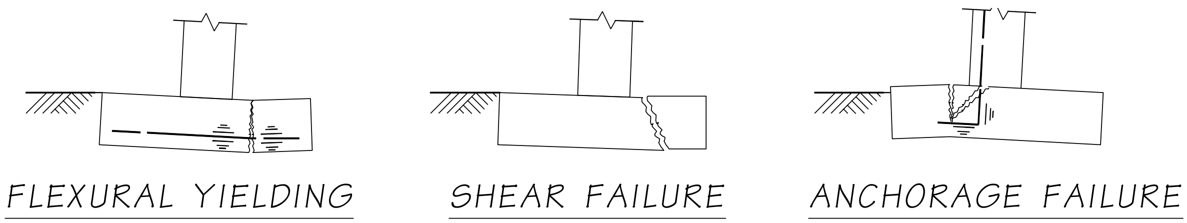


Figure 7.7.4-4

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.

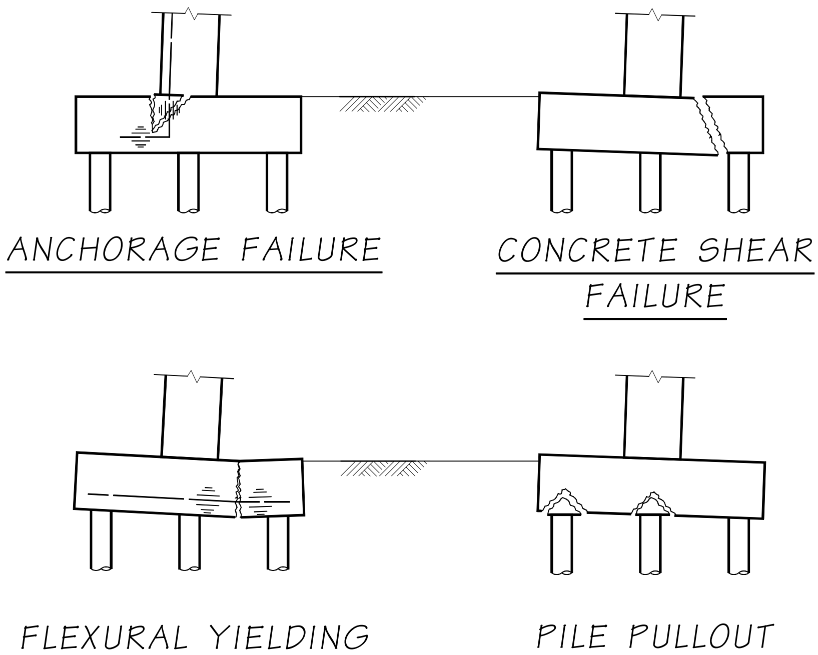
4. **Bottom Reinforcement** – Concrete design shall be in accordance with AASHTO LRFD Specifications. Reinforcement shall not be less than #6 bars at 12″ centers to account for uneven soil conditions and shrinkage stresses.

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 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.

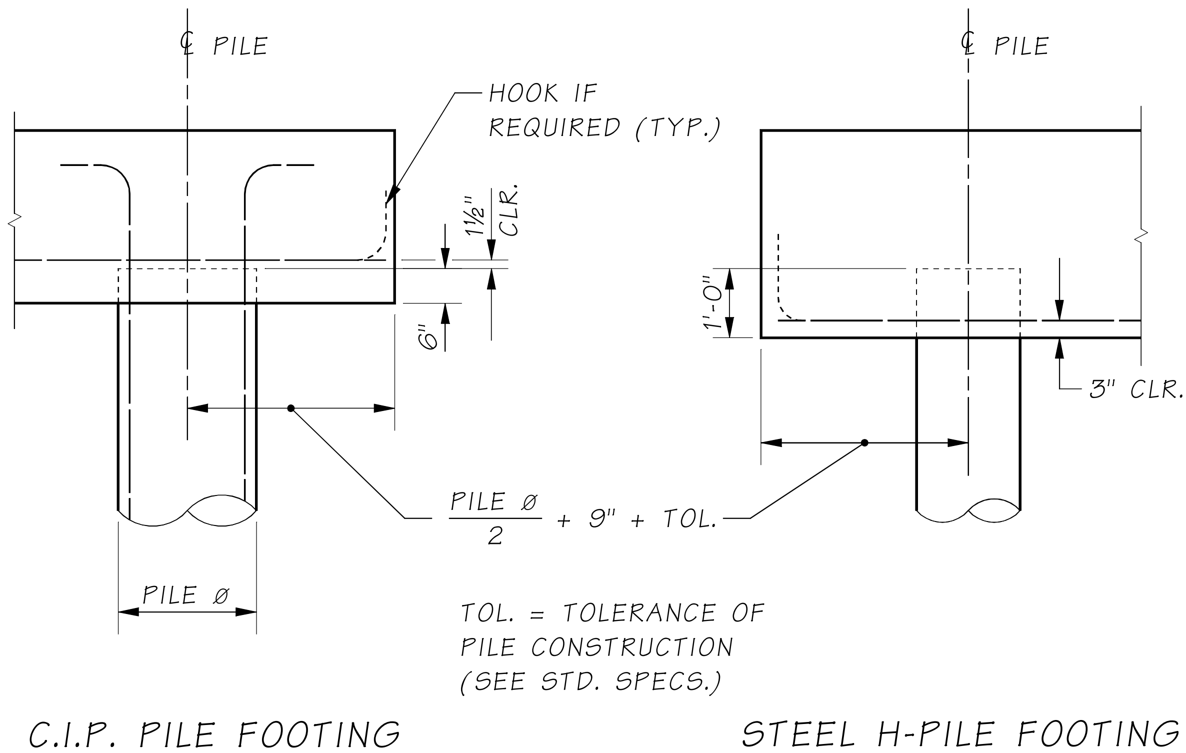
7.7.5 Pile-Supported Footing Design

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. The steel casing for cast-in-place concrete piles with reinforcing extending into footings shall be 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 casing 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 Shafts

7.8.1 Axial Resistance

The factored axial resistance of the 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.

$$R = \varphi_p R_p + \varphi_s R_s \qquad (7.8.1\text{-}1)$$

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. End bearing resistance, R_p, is typically provided by the Geotechnical Branch as a net value. Thus, the effective weight of the shaft can be reduced by the total weight of the excavated soil when examining compressive loads and resistances.

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 (100 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 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 shall be determined from the AASHTO LRFD Specifications and the WSDOT *Geotechnical Design Manual* M 46-03.

7.8.2 Structural Design and Detailing

Section 6-19 of the *Standard Specifications* should be reviewed as part of the design of shafts. The structural design of shafts is similar to column design. The following guidelines shall be followed:

A. For shaft foundation supporting columns in any SDC, Shafts shall be designed to resist the 1.25 times the plastic overstrength of the column above. Where elastic design methods are approved in SDC C or D, the shaft may be designed to resist 1.2 times the elastic seismic forces at the demand displacement.

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 shafts.

F. Cover requirements vary depending on the 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″

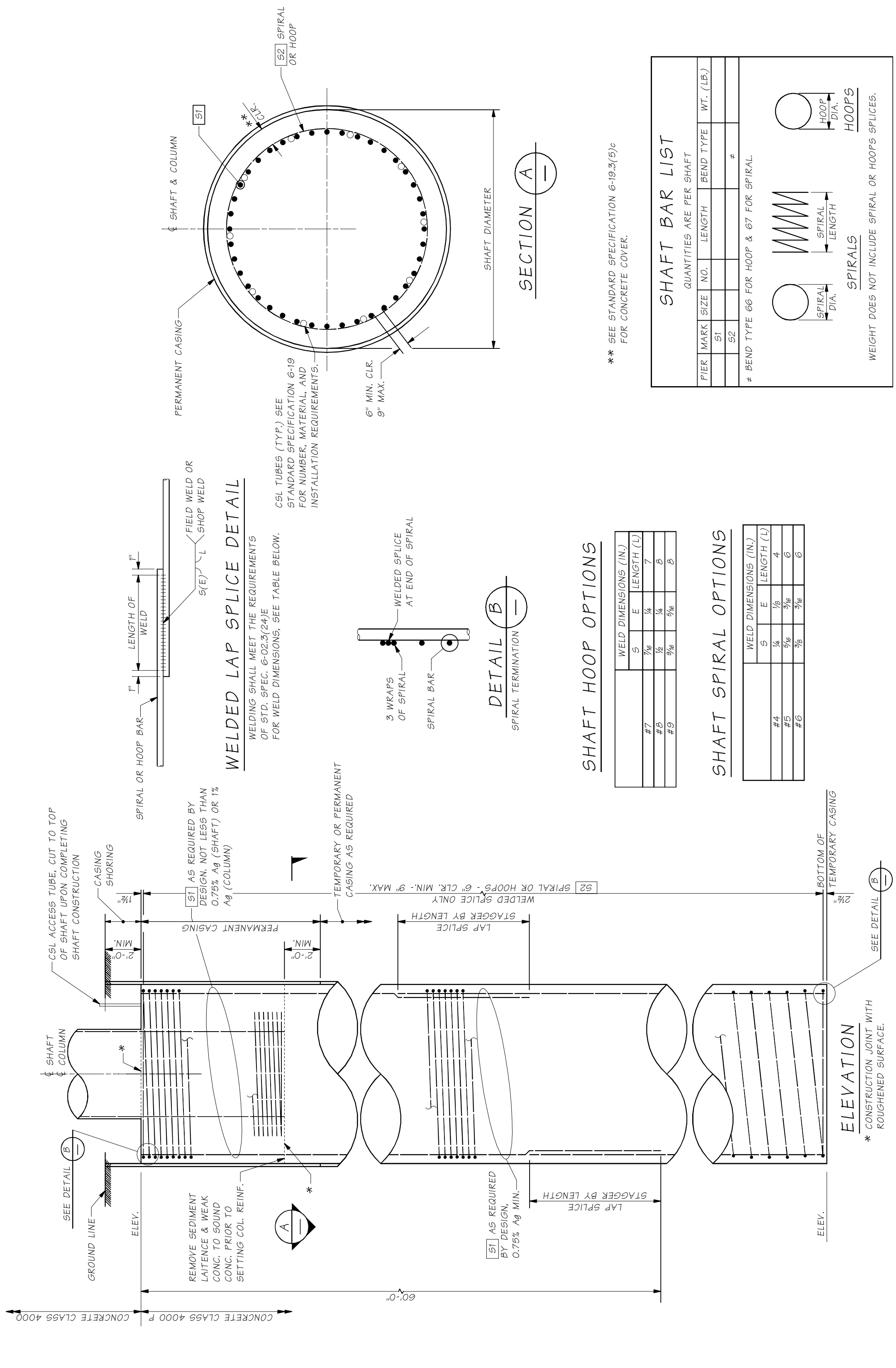
Section 6-19 of the *Standard Specifications* lists exceptions to these cover requirements when permanent slip casings are used in column splice zones.

G. In general, 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 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":

$$S_{max} = \frac{2\pi A_{sh} f_{ytr} l_s}{k A_l f_{ul}} \qquad (7.8.2\text{-}1)$$

Where:

S_{max} = Spacing of transverse shaft reinforcement
A_{sh} = Area of shaft spiral or transverse reinforcement bar
f_{ytr} = Yield strength of shaft transverse reinforcement
l_s = Standard splice length of the column reinforcement, per AASHTO LRFD.
A_l = Total 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. In the upper half of the splice zone, k = 1.0. In the lower half of the splice zone, this ratio could be determined from the column moment-curvature analysis using computer programs XTRACT or CSiBridge. To simplify this process, k = 0.5 could safely be used in most applications

The additional lateral reinforcement in the upper half of the oversized pile shafts is required to control cracking in this region. The volumetric ratio of transverse reinforcement throughout the splice zone shall not be less that that provided by a #6 spiral with a 6" pitch.

L. Longitudinal reinforcement shall be provided for the full length of 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 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 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 placed in low stress regions and staggered 2′-0″ minimum.

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.

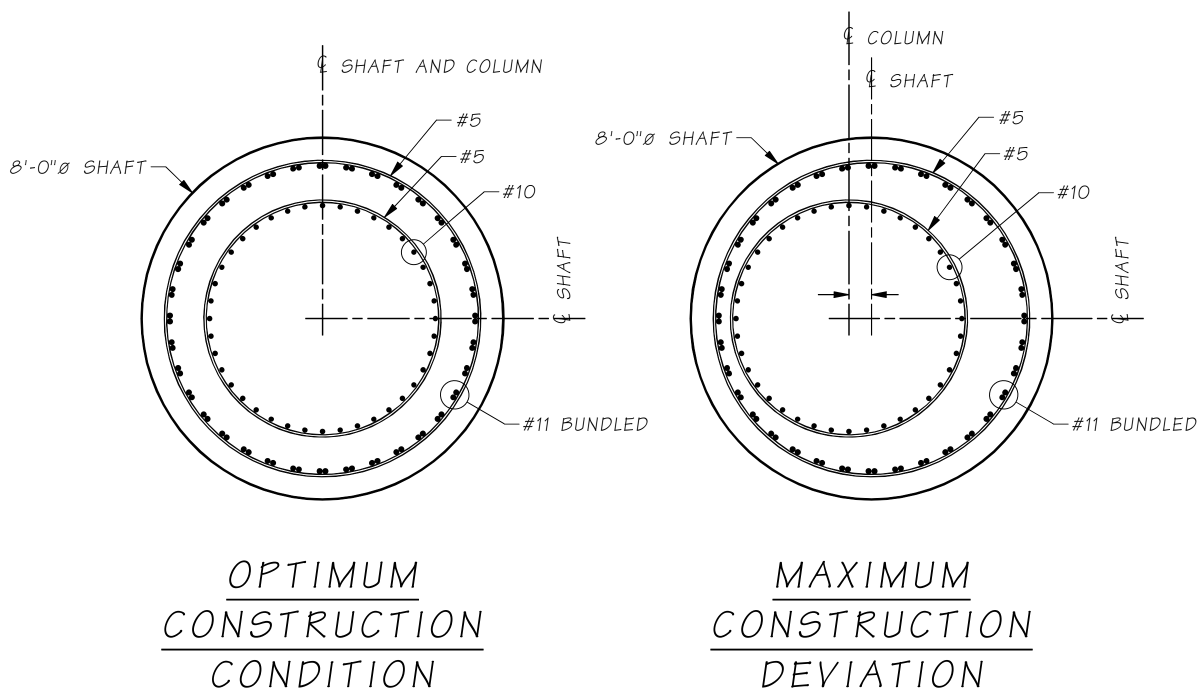


Figure 7.8.2-2

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.

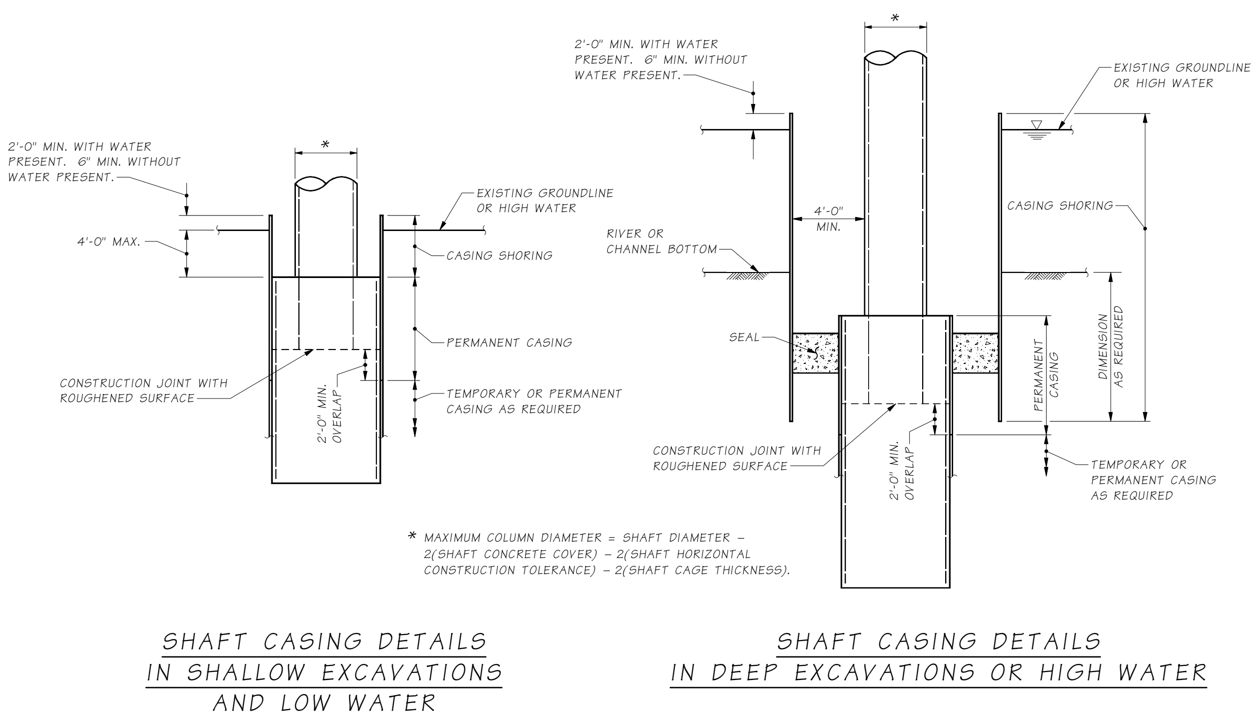
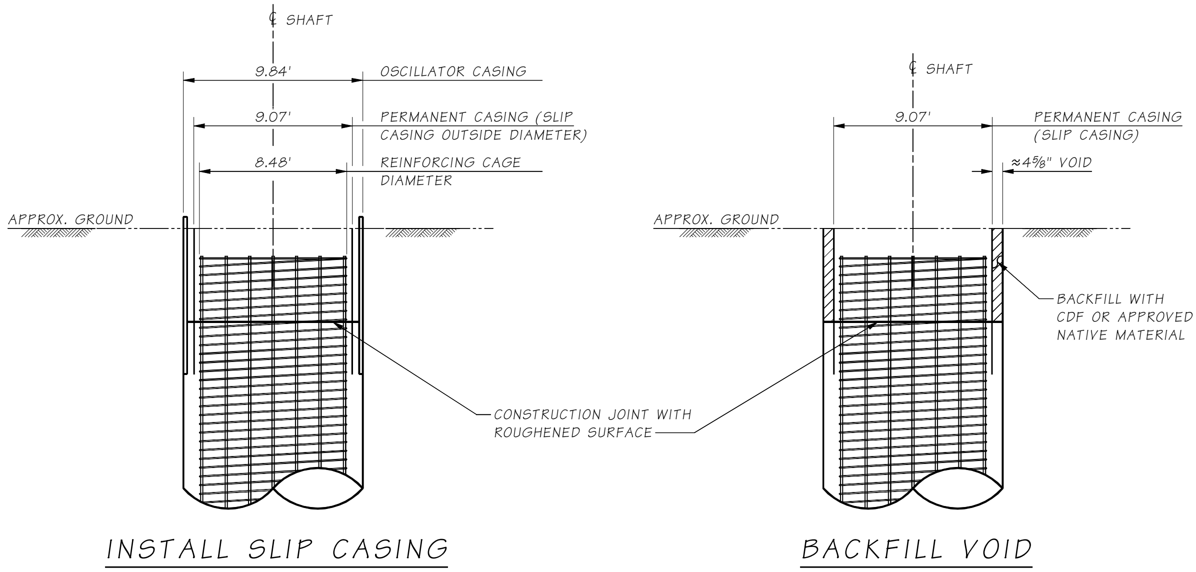


Figure 7.8.2-3

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.

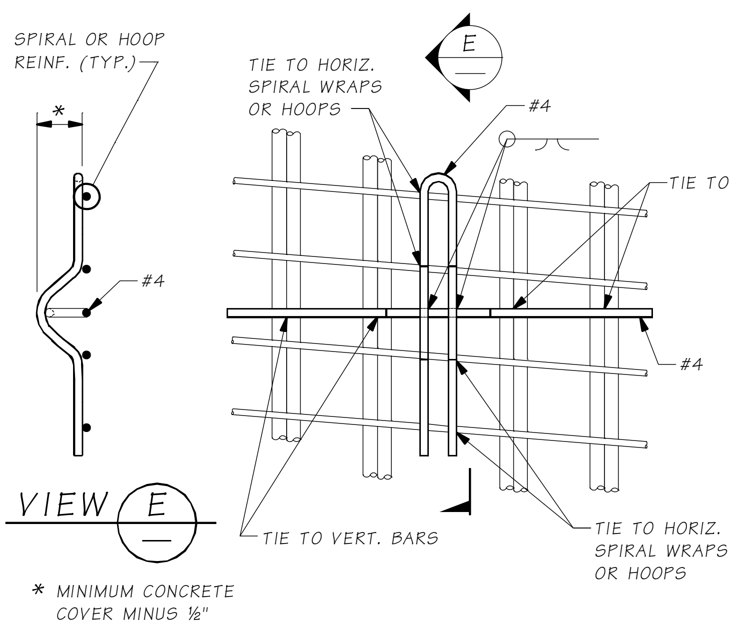
Y. Reinforcing bar centralizers shall be detailed in the plans as shown in Figure 7.8.2-5.

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 NOTES:

1. CENTRALIZERS SHALL BE EPOXY COATED OR PAINTED WITH INORGANIC ZINC AFTER FABRICATION.
2. EACH LEG SHALL BE TIED TO TWO VERTICAL BAR BUNDLES AND TWO SPIRAL WRAPS OR TWO HOOPS.
3. SEE STD. SPEC. 6-19.3(5)B FOR SPACING REQUIREMENTS.

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:

$$(P_{U\,pile}) = (P_{U\,pile\,group})/N + M_{U\,group}\,C/I_{group} + \gamma DD \qquad \text{(7.9.2-1)}$$

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 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.

7.10 Concrete-Filled Tubes

7.10.1 Scope

This section shall be taken to supersede AASHTO LRFD and AASHTO Seismic requirements for concrete-filled tubes (or pipes). The use of concrete-filled tubes (CFT) and reinforced concrete-filled tubes (RCFT) for bridge foundations requires approval from the WSDOT Bridge Design Engineer. CFT and RCFT shall not be used for bridge columns including extended-pile columns, and they shall not be utilized as the ductile elements of an earthquake resisting system.

CFT and RCFT have been shown to offer strength and stiffness beyond a conventional reinforced concrete (RC) member. And recent research has shown that CFT members can sustain large cyclic drifts with minimal damage.

The design methods herein regarding concrete-filled tubes are largely based on study, testing and recommendations compiled by the University of Washington (UW). RCFT offers limited resistance beyond that achieved with CFT (while potentially adding cost and complexity), so RCFT is recommended only as a transition between CFT and RC members.

The concrete for CFT members tested at the UW was a low-shrinkage, self-consolidating concrete. The nominal concrete strengths were 6 ksi and 10 ksi. This represents structural concrete with a minimum specified strength of 4 ksi, and an expected strength 25% to 50% larger.

Prior research has not evaluated the shear strength of RCFT. Though shear research is ongoing. The shear resistance of the steel will invariably be larger than the shear resistance of the concrete alone unless the *D/t* ratio of the tube is extremely large (approaching 200).

Prior CALTRANS and ARMY research programs studied two types of fully restrained connections for CFT pier to foundation connections. One of those two connections is readily usable as a CFT-to-cap connection. An annular ring is attached to the top of the CFT, and it is partially embedded into the pile cap. This anchored connection resists flexural loading from the pile through strutting action to the bottom of the pile cap (resulting from the portion of tube of the CFT that is in tension) and the top of the pile cap (resulting from the portion of tube of the CFT column that in compression). The tests show this connection is both simple to construct and fully effective in transferring flexure. The current ACI procedure (ACI 318-2011) was recommended by the UW as a conservative approach to design against punching shear in this type of connection.

Transition connections between RC shafts and CFT shafts have not been tested, but considerable analysis has been performed at the UW. Models have been developed to predict the strength of RCFT members, and this RCFT behavior may be used to provide increased strength over a significant length of the pile relative to conventional RC construction. Overstrength factors for capacity design of adjacent members and joint shear design at connections were not addressed in the research.

7.10.2 Design Requirements

A. **Materials –**

1. The concrete for CFT and RCFT shall be class 4000P. A reduced compressive design strength of $0.85f'_c$ shall be used for wet placed concrete. Low shrinkage concrete shall be required to ensure the concrete does not shrink relative to the steel tube.
2. Steel tubes shall conform to one of the following:
    - i. API 5L Grade X42 or X52 for longitudinal seam welded or helical (spiral) seam submerged-arc welded tube
    - ii. ASTM A 252 Grade 2 or 3 for longitudinal seam welded or helical (spiral) seam submerged-arc welded tube
    - iii. ASTM A 572 or ASTM A 588 for longitudinal seam welded tube

3. For capacity protected members at the extreme event limit state, expected material properties may be used to determine the expected nominal moment capacity. The expected yield strength, F_{ye}, for steel tubes shall be taken as $1.1F_y$.

B. **Limit States** – For strength limit states, the resistance factors for axial load effects on CFT shall be taken per AASHTO LRFD for tension- and compression- controlled reinforced concrete sections. The resistance factor for shear shall be taken as 0.85. For extreme event limit states, resistance factors shall be taken as 1.0.

C. **General Dimensions** – The minimum tube wall thickness shall not be taken less than 3/8 inch at the time of installation. To develop the full plastic capacity of CFT or RCFT members, it is necessary to ensure that local buckling does not occur prior to development of the strength of the tube. Therefore the following *D/t* limits are recommended:

1. For members subjected primarily to flexural loading:

$$\frac{D}{t} \leq 0.22\frac{E}{F_y} \tag{7.10.2-1}$$

2. For members subjected primarily to axial loading:

$$\frac{D}{t} \leq 0.15\frac{E}{F_y} \tag{7.10.2-2}$$

Where D is the outside diameter of the tube (in.), and t is the wall thickness of the tube (in.).

D. **Stiffness** – The effective stiffness, EI_{eff}, of circular CFT, as defined in Eq. 7.10.2-3, shall be used to evaluate deflections, deformations, buckling resistance, and moment magnification. The effective stiffness factor, *C*', is defined in Equation 7.10.2-4.

$$EI_{eff} = E_s I_S + C'E_c I_c \tag{7.10.2-3}$$

$$C' = 0.15 + \frac{P}{P_0} + \frac{A_s}{A_s + A_c} \leq 0.9 \tag{7.10.2-4}$$

P_0 is the nominal compressive resistance without moment, P is the factored axial load effect, and A_s is the combined area of the steel tube and steel reinforcing.

E. **Flexure and Axial Resistance** – The flexural strength of CFT and RCFT members may be determined using the plastic stress distribution method. The appropriate limit state stresses and geometry is shown in Figure 7.10.2-1.

Solutions for the interaction diagrams can be developed using parametric equations for $P(y)$ and $M(y)$ where y is the distance from the centroid to the neutral axis. A positive value of P is a net compressive force. M and y are positive with the sign convention shown in Figure 7.10.2-1. The parameter y varies between plus and minus r_i, where r_i is the radius of the concrete core.

Stress is assumed to be plastically developed over the following regions of the section:

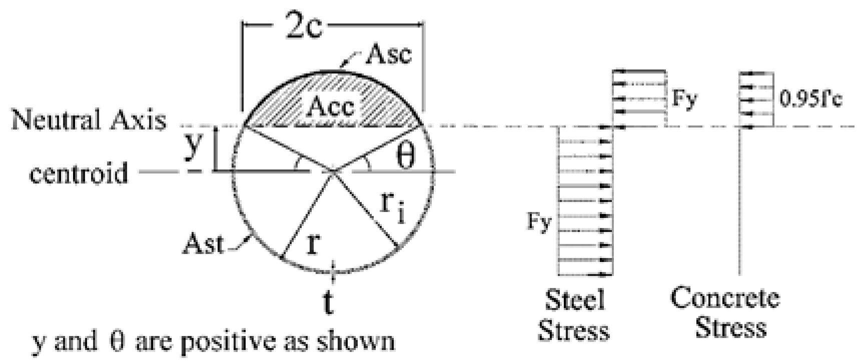
A_{cc} = area of concrete effective in compression

A_{sc} = area of the steel tube in compression

A_{st} = area of the steel tube in tension

A_{bc} = area of the internal steel reinforcing in compression

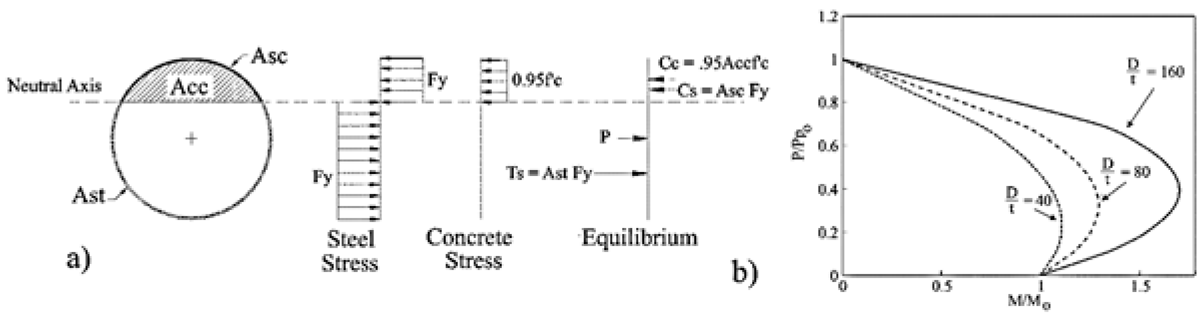
A_{bt} = area of the internal steel reinforcing in tension



Plastic Stress Distribution Method
Figure 7.10.2-1

Alternatively, a strain-compatibility analysis can be performed with appropriate plastic stress-strain relationships.

1. **CFT Interaction** – A parametric solution for the nominal interaction diagram can be developed using Figure 7.10.2-2 and Equations 7.10.2-5 through 7.10.2-9. Figure 7.10.2-2b also shows normalized interaction curves for various D/t ratios.



Plastic Stress Distribution for CFT
Figure 7.10.2-2

$$P_n(y) = \left(\left(\frac{\pi}{2} - \theta\right) r_i^{\,2} - yc\right) * 0.95f'_c - 4\theta t r_m F_y \quad (7.10.2\text{-}5)$$

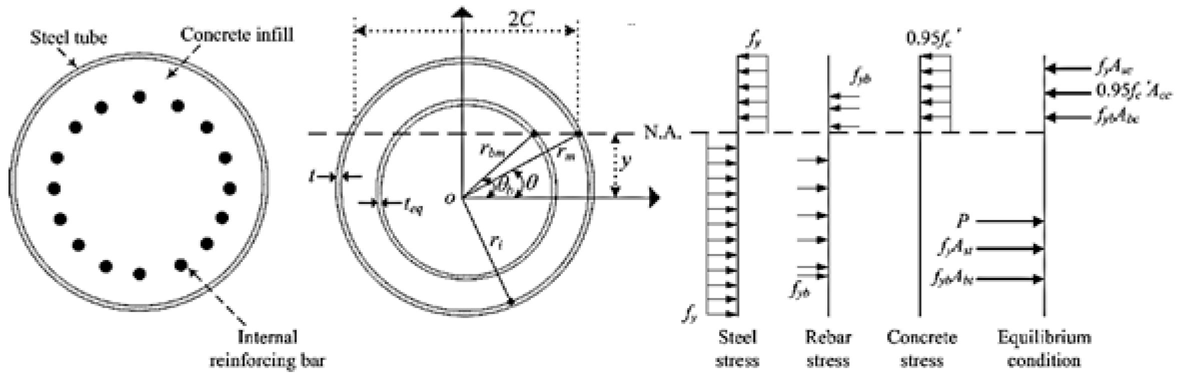
$$M_n(y) = \left(c(r_i^{\,2} - y^2) - \frac{c^3}{3}\right) * 0.95f'_c + 4ct\frac{r_m^{\,2}}{r_i} F_y \quad (7.10.2\text{-}6)$$

$$c = r_i cos\theta \quad (7.10.2\text{-}7)$$

$$\theta = sin^{-1}\left(\frac{y}{r_m}\right) \quad (7.10.2\text{-}8)$$

$$r_m = r - \frac{t}{2} \quad (7.10.2\text{-}9)$$

2. **RCFT Interaction** – A parametric solution for the nominal interaction diagram can be developed using Figure 7.10.2-3 and Equations 7.10.2-7 through 7.10.2-14. The internal steel reinforcing is idealized as a thin ring.



Plastic Stress Distribution for RCFT
Figure 7.10.2-3

$$P_n(y) = \left(\left(\frac{\pi}{2} - \theta\right) r_i^2 - yc\right) * 0.95f'_c - 4\theta t r_m F_y - t_b r_{bm}\left(4\theta_b F_{yb} + (\pi - 2\theta_b)0.95f'_c\right) \quad (7.10.2\text{-}10)$$

$$M_n(y) = \left(c(r_i^2 - y^2) - \frac{c^3}{3}\right) * 0.95f'_c + 4ct\frac{r_m^2}{r_i}F_y + 4t_b r_{bm} c_b (F_{yb} - 0.95f'_c) \quad (7.10.2\text{-}11)$$

$$c_b = r_b cos\theta_b \quad (7.10.2\text{-}12)$$

$$\theta_b = sin^{-1}\left(\frac{y}{r_{bm}}\right) \quad (7.10.2\text{-}13)$$

$$t_b = \frac{nA_b}{2\pi r_{bm}} \quad (7.10.2\text{-}14)$$

The associated variables are defined as:

r = radius to the outside of the steel tube (in)

r_i = radius to the inside of the steel tube (in)

r_m = radius to the center of the steel tube (in)

r_{bm} = radius to the center of the internal reinforcing bars (in)

t = wall thickness of the tube (in)

t_b = wall thickness of a notional steel ring equivalent to the internal reinforcement (in)

c = one half the chord length of the tube in compression (in)

c_b = one half the chord length of a notional steel ring equivalent to the internal reinforcement in compression (in)

θ = angle used to define c (rad.)

θ_b = angle used to define c_b (rad.)

A_b = area of a typical steel bar comprising the internal reinforcement (in^2)

n = number of internal steel reinforcing bars

The requirements of AASHTO Seismic 8.16.2 for piles with permanent steel casing shall be applied to RCFT. Accordingly, the extent of longitudinal reinforcement may be reduced to only the upper portion of the member as needed to provide the required resistance of the member.

For CFT and RCFT, the area of the steel casing shall be included in the determination of the longitudinal reinforcement ratio. For RCFT, the minimum required longitudinal reinforcement ratio may be reduced to 0.005.

A. **Stability Considerations for Unbraced of Partially-braced Members** – Piles and shafts are typically assumed to be continually braced by the surrounding soil. Therefore they are not normally subject to P-Δ effects or other secondary effects. However, it is recognized that special circumstances such as scour, soil liquefaction, or other conditions may leave piles and shafts subject to less than full bracing. In these circumstances, it may be necessary to consider stability effects.

B. **Shear Resistance** – The shear resistance of CFT and RCFT shall be taken as:

$$V_n = V_s + 0.5V_c \tag{7.10.2-15}$$

Where:

V_s = nominal shear resistance of the circular steel tube alone, excluding stability

= $0.58*F_y*(0.5*A_g)$

V_c = nominal shear resistance of the concrete alone

= $0.0316*2*\sqrt{(f_c')}*A_c$ if P_u is compressive.

A_g (in^2) is the area of the steel tube. The resistance factor for shear shall be taken as 0.85 at strength limit states and 1.0 at extreme event limit states.

C. **Corrosion** – The design wall thickness for tubes shall be reduced for corrosion over a 75-year minimum design life. Corrosion rates are specified below, except that the design thickness loss due to corrosion shall not be taken to be less than 1/16 inch.

Soil embedded zone (undisturbed soil):	0.001 inch per year
Soil embedded zone (fill or disturbed natural soils):	0.003 inch per year
Immersed Zone (fresh water):	0.002 inch per year
Immersed Zone (salt water):	0.004 inch per year
Scour Zone (salt water):	0.005 inch per year

The corrosion rates are taken from July 2008 CALTRANS memo to Designers 3-1 and FHWA NHI-05-042 Design and Construction of Driven Pile Foundations.

7.10.3 CFT-to-Cap Connections

CFT-to-cap connections shall be designed as fully-restrained connections capable of resisting all load effects. The preferred connection to a concrete cap includes an annular ring at the top of the embedded tube. The connection design involves:

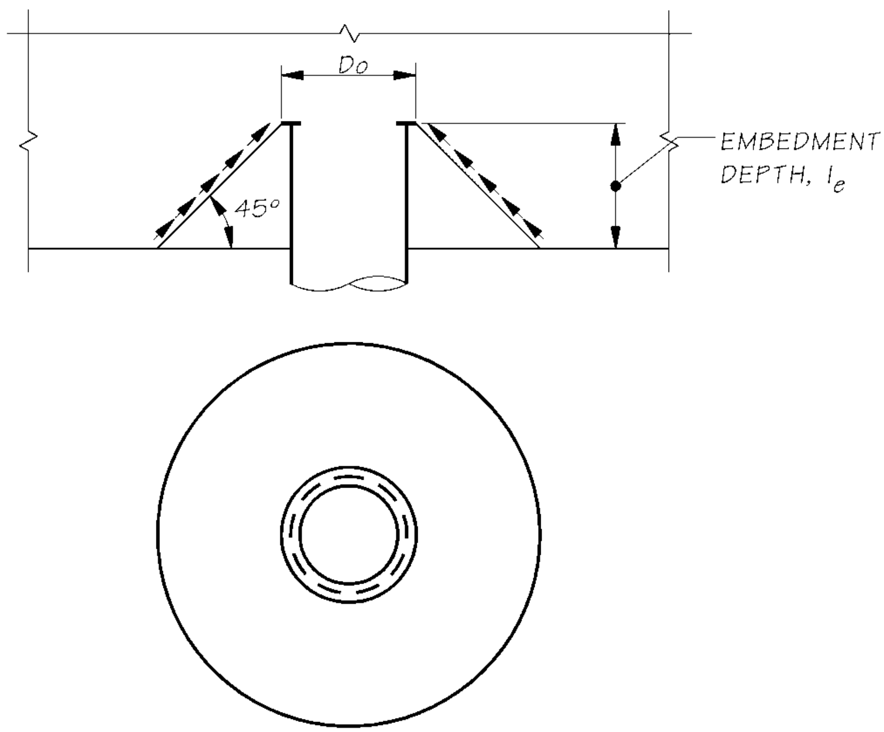
A. Design of the annular ring

B. Determination of the embedment depth

C. A punching shear evaluation in the cap

D. General design of the cap for flexure and shear

A. **Annular Ring** – An annular ring shall be welded to the end of the tube to provide anchorage and stress distribution, as shown in Figure 7.10.3-1. The ring shall be made of a steel of the same thickness and grade as the steel tube. The ring shall extend outside the tube a distance of 16*t* and shall extend inside the tube a distance of 8*t*, where t is the thickness of the tube.

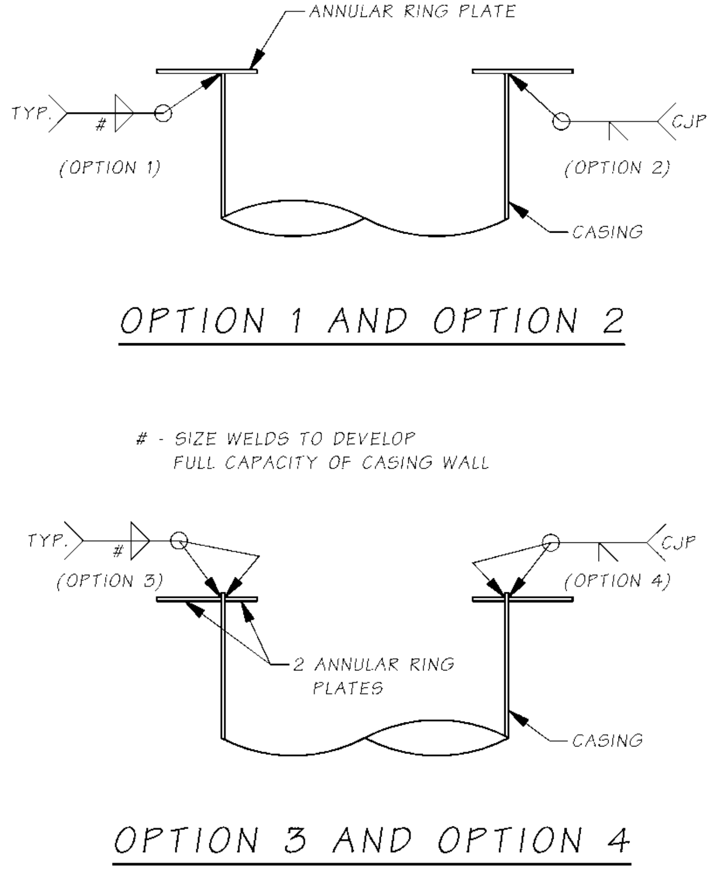


Cone Pullout Mechanism for Cap Connections
Figure 7.10.3-1

The ring shall be welded to the tube with complete joint penetration (CJP) welds or fillet welds on both the inside and outside of the tube. The fillet welds must be capable of developing the full tensile capacity of the tube. For this purpose, the minimum size, w, of the fillet welds shall be taken as:

$$w \geq \frac{1.47 F_u t}{F_{exx}} \quad (7.10.3\text{-}1)$$

Where F_u is the specified minimum tensile strength of the tube steel (ksi), and F_{exx} is the classification strength of the weld metal (ksi). Typical CFT weld details are shown in Figure 7.10.3-2.



Annular Ring Weld Detail
Figure 7.10.3-2

B. **Embedment –** The tube and the annular ring shall be embedded into the pile cap with adequate embedment depth to ensure ductile behavior of the connection. The minimum embedment length, l_e, shall be taken as:

$$l_e \geq \sqrt{\frac{{D_o}^2}{4} + \frac{DtF_u}{6\sqrt{f'_{cf}}}} - \frac{D_o}{2} \qquad (7.10.3\text{-}2)$$

$$h = \sqrt{\frac{D^2}{4} + \frac{250C_{max}}{\sqrt{f'_{cf}}}} - \frac{D}{2} \qquad (7.10.3\text{-}3)$$

$$C_{max} = C_c + C_s \qquad (7.10.3\text{-}4)$$

(7.10.3-2)

$$d_f \geq h + l_e \qquad (7.10.3\text{-}5)$$

$$d_e \geq \frac{D_o}{2} + 1.75l_e \qquad (7.10.\ 3\text{-}6)$$

$$s \leq \frac{l_e}{2.5} \qquad (7.10.3\text{-}7)$$

Where f'_{cf} (psi) is the specified 28-day compressive strength of the cap, D_o is the outside diameter of the annular ring as shown in Figure 7.10.3-1, and F_u is the minimum specified tensile strength of the tube (psi).

C. **Punching Shear** – The pile cap shall have adequate concrete depth, h, above the steel tube to preclude punching through the pile cap, taken as follows:

$$h = \sqrt{\frac{D^2}{4} + \frac{250C_{max}}{\sqrt{f'_{cf}}}} - \frac{D}{2} \quad (7.10.3\text{-}3)$$

Where the total compressive force of the couple, C_{max}, shall be taken as:

$$C_{max} = C_c + C_s \quad (7.10.3\text{-}4)$$

C_c and C_s are the compression forces in the concrete and the steel due to the combined bending and axial load as computed by the PSDM for the most extreme load effect.

D. **Pile Cap Reinforcement** – The pile cap should follow conventional design practice and must be adequate to sustain the foundation design loads. However, the concrete cap thickness shall be large enough to preclude punching shear and cone pullout of the CFT piles. The minimum concrete cap thickness, d_f, shall be taken as:

$$d_f \geq h + l_e \quad (7.10.3\text{-}5)$$

The edge distance from center-of-tube to the edge of the cap shall be large enough to accommodate concrete struts oriented 60 degrees from the vertical originating at the base of the ring. The minimum edge distance, d_e, shall be taken as:

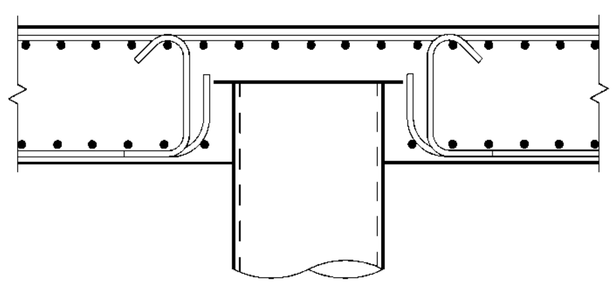
$$d_e \geq \frac{D_o}{2} + 1.75l_e \quad (7.10.\ 3\text{-}6)$$

$$s \leq \frac{l_e}{2.5} \quad (7.10.3\text{-}7)$$

(7.10.3-6)

CFT shall be adequately spaced to avoid intersecting concrete struts.

The cap shall be designed to resist all flexural load effects. The flexural reinforcement in both directions shall be spaced uniformly across the length and width of the cap, but the bottom mat of flexural reinforcement will be interrupted by the concrete tube. The interrupted bars shall be provided, but they shall not be relied on to contribute to the flexural resistance of the cap. Figure 7.10.3-3 shows the configuration of the longitudinal reinforcing where it conflicts with the steel tube. Standard 90° hooks shall be used.



Reinforcement Detail at Cap Connection
Figure 7.10.3-3

The cap shall be designed to resist all shear load effects. Note that the minimum required embedment results in an average shear stress in the critical area surrounding the tube of $6\sqrt{f'_c}$ (psi). Assuming the concrete is capable of resisting a shear stress of approximately $2\sqrt{f'_c}$, vertical reinforcement will be required to resist an average shear stress of approximately $4\sqrt{f'_c}$. Additional requirements for shear demand resulting from other load combinations must also be considered.

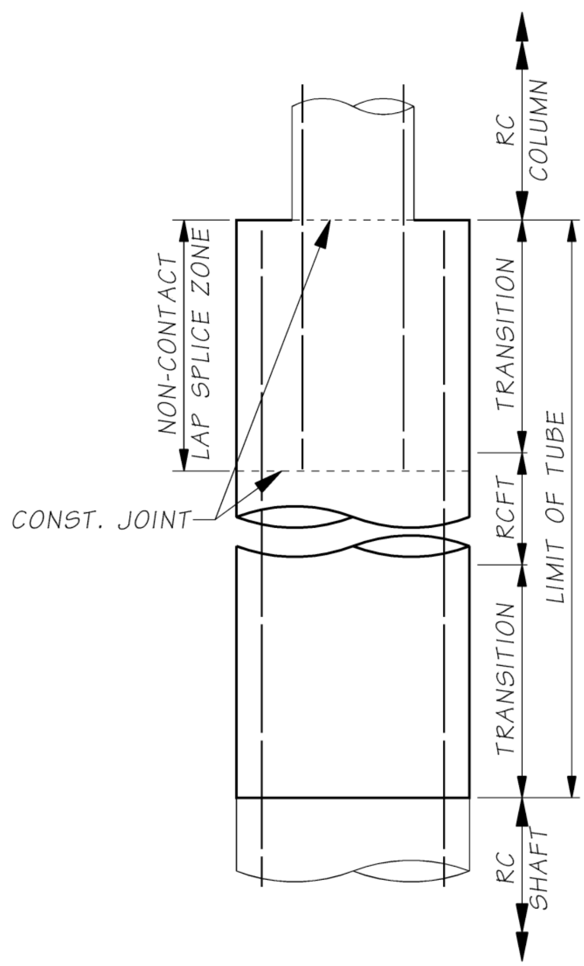
Additionally, vertical ties shall be provided within the anchorage regions such that at least two vertical ties intersect the pull-out cone depicted in Figure 7.10.3-1 on each side of the CFT subject to shear. Therefore vertical ties shall be placed in the region within $1.5l_e$ of the outside of the tube, and shall be placed at a maximum spacing s, taken as:

$$s \leq \frac{l_e}{2.5} \tag{7.10.3-7}$$

7.10.4 RCFT-to-Column Connections

Direct RCFT-to-column connections shall be designed as fully-restrained connections capable of resisting all load effects. The recommended RCFT shaft to reinforced concrete column connection is shown in Figure 7.10.4-1.

All column reinforcement shall be extended into the RCFT shaft for a length greater than or equal to the length required for noncontact lap splices between columns and shafts. The contribution of steel casing to the structural resistance of RCFT's varies from zero at the end of the tube to fully composite at the end of the transition zone. The transition zone length may be taken as 1.0D. The use of slip casing in determining the resistance for RFCT shafts is not permitted.



RCFT-to-Column Connection
Figure 7.10.4-1

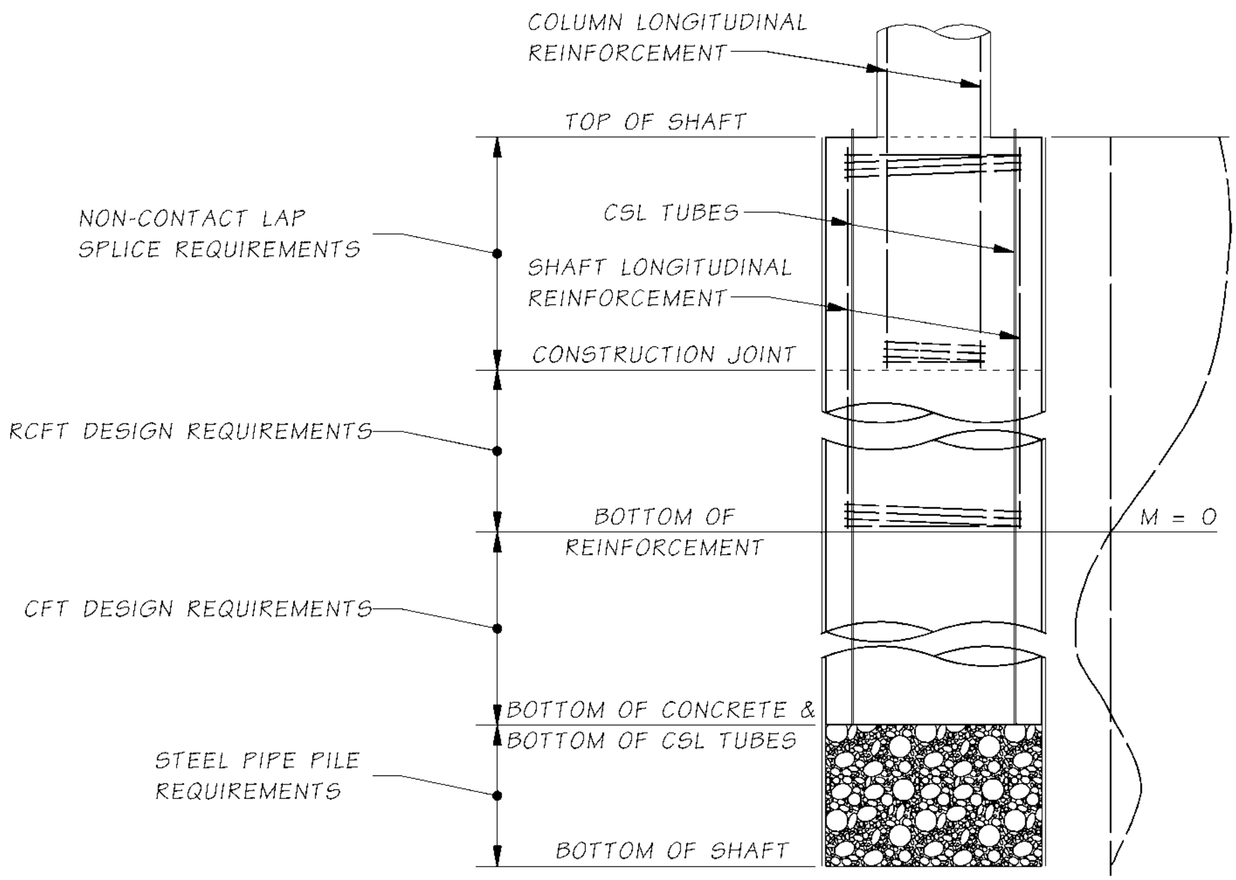
7.10.5 Partially-filled CFT

The use of partially-filled steel tubes for bridge foundations requires the approval of the WSDOT Bridge Design Engineer, and will only be used where conventional CFT members are grossly uneconomical or unconstructible.

Design zones of partially filled steel piles and shafts are shown in Figure 7.10.5-1. Longitudinal and transverse reinforcement shall extend to at least the first point of zero moment along the member under the peak loading condition.

Crosshole sonic log (CSL) testing shall be performed in accordance with Standard Specification Section 6-19.3(9). CSL tubes shall extend to the bottom of concrete.

Corrosion losses shall be considered on each exposed surface of the steel tube.



Partially-filled CFT
Figure 7.10.5-1

7.10.6 Construction Requirements

For CFT with tubes installed open-ended, the insides of the tube shall be cleaned with an appropriate tool to remove all adhering soil and other material.

Welding for ASTM A 252 pipe shall conform to AWS D1.1/D1.1M, latest edition, Structural Welding Code, except that all weld filler metal shall be low hydrogen material selected from Table 4.1 in AASHTO/AWS D1.5M/D1.5:2010 Bridge Welding Code. All seams and splices shall be complete penetration welds.

Welding and joint geometry for the seam shall be qualified in accordance with AWS D1.1/D1.1M, latest edition, Structural Welding Code. The Contractor may submit documentation of prior qualification to the Engineer to satisfy this requirement.

For the fabrication of helical (spiral) seam submerged-arc welded pipe piles, the maximum radial offset of strip/plate edges shall be 1/8 inch. The offset shall be transitioned with a taper weld and the slope shall not be less than a 1-to-2.5 taper. The weld reinforcement shall not be greater than 3/16 inches and misalignment of weld beads shall not exceed 1/8 inch.

If spirally welded pipe piles are allowed, skelp splices shall be located at least 1'-0" away from the annular ring.

Nondestructive evaluation (NDE) requirements for field welded splices shall be identified on the plans. The location of splices and NDE requirements shall be divided into 3 possible zones as determined by the Engineer:

1. No splices permitted – highly stressed areas
2. Splices permitted with 100% UT and visual inspection – moderately stressed areas
3. Splices permitted with 100% visual inspection – low stressed areas

7.10.7 Notation

A_b = area of a single bar for the internal reinforcement (in^2)

A_c = net cross-sectional area of the concrete (in^2)

A_g = cross-sectional area of the steel tube (in^2)

A_s = cross-sectional area of the steel tube and the longitudinal internal steel reinforcement (in^2)

c = one half the chord length of the tube in compression (in)

c_b = one half the chord length of a notional steel ring equivalent to the internal reinforcement in compression (in)

D = outside diameter of the tube (in.)

D_o = outside diameter of the annular ring (in.)

d_b = nominal diameter of a reinforcing bar (in)

d_e = minimum edge distance from center of CFT to edge of cap (in)

d_f = depth of cap (in)

E_c = elastic modulus of concrete (ksi)

EI_{eff} = effective composite flexural cross-sectional stiffness of CFT or RCFT (k-in^2)

E_s = elastic modulus of steel (ksi)

F_{exx} = classification strength of weld metal (ksi)

F_u = specified minimum tensile strength of steel (ksi)

F_y = specified minimum yield strength of steel (ksi)

F_{yb} = specified minimum yield strength of reinforcing bars used for internal reinforcement (ksi)

f_c' = minimum specified 28-day compressive strength of concrete (ksi)

f'_{cf} = minimum specified 28-day compressive strength of concrete in a cap or footing (psi)

h = cap depth above the CFT required to resist punching shear in a cap (in)

I_c = uncracked moment of inertial of the concrete about the centroidal axis (in^4)

I_s = moment of inertia of the steel tube and the longitudinal internal steel reinforcement about the centroidal axis (in^4)

l_e = Required embedment length for CFT embedded in a concrete cap (in)

$M(y)$ = nominal moment resistance as a function of the parameter y (kip-in)

M_o = plastic moment resistance of members without axial load (kip-in)

n = number of equally spaced longitudinal internal steel reinforcement

$P(y)$ = nominal compressive resistance as function of the parameter y (kips)

P_u = factored axial load acting on member (kip)

P_o = compressive resistance of a member without consideration of flexure (kips)

r = radius to the outside of the steel tube (in)

r_{bm} = radius to the center of the internal reinforcing bars (in)

r_i = radius to the inside of the steel tube (in)

r_m = radius to the center of the steel tube (in)

s = maximum spacing of shear reinforcing in pullout cone region (in)

t = wall thickness of the tube (in)

t_b = wall thickness of a notional steel ring equivalent to the internal reinforcement (in)

V_n = nominal shear resistance (kip)

w = fillet weld size (in)

y = parameter representing the distance from the centroid to the neutral axis of a CFT

θ_s = horizontal angle used to define c (rad.)

θ_b = horizontal angle used to define c_b (rad.)

7.99 References

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Chapter 8 Walls and Buried Structures

8.1 Retaining Walls

8.1.1 General

A retaining wall is a structure built to provide lateral support for a mass of earth or other material where a grade separation is required. Retaining walls depend either on their own weight, their own weight plus the additional weight of laterally supported material, or on a tieback system for their stability. Additional information is provided in Chapter 15 of the WSDOT *Geotechnical Design Manual* M 46-03.

Standard designs for reinforced concrete cantilevered retaining walls, noise barrier walls (precast concrete, cast-in-place concrete, or masonry), and geosynthetic walls are shown in the Standard Plans. The Region Design PE Offices are responsible for preparing the PS&E for retaining walls for which standard designs are available, in accordance with the WSDOT *Design Manual* M 22-01. However, the Bridge and Structures Office may prepare PS&E for such standard type retaining walls if such retaining walls are directly related to other bridge structures being designed by the Bridge and Structures Office.

Structural earth wall (SE) systems meeting established WSDOT design and performance criteria have been listed as "pre-approved" by the Bridge and Structures Office and the Materials Laboratory Geotechnical Branch. The PS&E for "pre-approved" structural earth wall systems shall be coordinated by the Region Design PE Office with the Bridge and Structures Office, and the Materials Laboratory Geotechnical Branch, in accordance with WSDOT *Design Manual* M 22-01.

The PS&E for minor non-structural retaining walls, such as rock walls, gravity block walls, and gabion walls, are prepared by the Region Design PE Offices in accordance with the WSDOT *Design Manual* M 22-01, and any other design input from the Region Materials Offic, Materials Laboratory Geotechnical Branch or Geotechnical Engineer.

All other retaining walls not covered by the Standard Plans such as soil nail walls, soldier pile walls, soldier pile tieback walls and all walls beyond the scope of the designs tabulated in the Standard Plans, are designed by the Bridge and Structures Office according to the design parameters provided by the Geotechnical Engineer.

The Hydraulics Branch of the Design Office should be consulted for walls that subject to floodwater or are located in a flood plain. The State Bridge and Structures Architect should review the architectural features and visual impact of the walls during the Preliminary Design stage. The designer is also directed to the retaining walls chapter in the WSDOT *Design Manual* M 22-01 and Chapter 15 of the WSDOT *Geotechnical Design Manual* M 46-03, which provide valuable information on the design of retaining walls.

8.1.2 Common Types of Walls

The majority of walls used by WSDOT are one of the following six types:

1. Proprietary Structural Earth (SE) Walls - *Standard Specification* Section 6-13.
2. Geosynthetic Walls (Temporary and Permanent) - *Standard Plan* D-3 and *Standard Specification* Section 6-14.
3. Standard Reinforced Concrete Cantilever Retaining Walls- *Standard Plans* D-10.10 through D-10.45 and *Standard Specification* Section 6-11.
4. Soldier Pile Walls and Soldier Pile Tieback Walls - *Standard Specification* Sections 6-16 and 6-17.
5. Soil Nail Walls - *Standard Specification* Section 6-15.
6. Noise Barrier Walls - *Standard Plan* D-2.04 through D-2.68 and *Standard Specification* Section 6-12.

Other wall systems, such as secant pile or cylinder pile walls, may be used based on the recommendation of the Geotechnical Engineer. These walls shall be designed in accordance with the current AASHTO LRFD.

A. **Pre-approved Proprietary Walls –** A wall specified to be supplied from a single source (patented, trademark, or copyright) is a proprietary wall. Walls are generally pre-approved for heights up to 33 ft. The Materials Laboratory Geotechnical Division will make the determination as to which pre-approved proprietary wall system is appropriate on a case-by-case basis. The following is a description of the most common types of proprietary walls:

   1. **Structural Earth Walls (SE) –** A structural earth wall is a flexible system consisting of concrete face panels or modular blocks that are held rigidly into place with reinforcing steel strips, steel mesh, welded wire, or geogrid extending into a select backfill mass. These walls will allow for some settlement and are best used for fill sections. The walls have two principal elements:

      - Backfill or wall mass: a granular soil with good internal friction (i.e. gravel borrow).
      - Facing: precast concrete panels, precast concrete blocks, or welded wire (with or without vegetation).

      Design heights in excess of 33 feet shall be approved by the Materials Laboratory Geotechnical Division. If approval is granted, the designer shall contact the individual structural earth wall manufacturers for design of these walls before the project is bid so details can be included in the Plans. See Appendix 8.1-A2 for details that need to be provided in the Plans for manufacturer designed walls.

      A list of current pre-approved proprietary wall systems is provided in Appendix 15-D of the WSDOT *Geotechnical Design Manual* M 46-03. For additional information see the retaining walls chapter in the WSDOT *Design Manual* M 22-01 and Chapter 15 of the WSDOT *Geotechnical Design Manual* M 46-03. For the SEW shop drawing review procedure see Chapter 15 of the WSDOT *Geotechnical Design Manual*.

   2. **Other Proprietary Walls –** Other proprietary wall systems such as crib walls, bin walls, or precast cantilever walls, can offer cost reductions, reduce construction time, and provide special aesthetic features under certain project specific conditions.

      A list of current pre-approved proprietary wall systems and their height limitations is provided in Appendix 15-D of the WSDOT *Geotechnical Design Manual* M 46-03. The Region shall refer to the retaining walls chapter in the WSDOT *Design Manual* M 22-01 for guidelines on the selection of wall types. The Materials Laboratory Geotechnical Division and the Bridge and Structures Office Preliminary Plans Unit must approve the concept prior to development of the PS&E.

B. **Geosynthetic Wrapped Face Walls –** Geosynthetic walls use geosynthetics for the soil reinforcement and part of the wall facing. Use of geosynthetic walls as permanent structures requires the placement of a cast-in-place, precast or shotcrete facing. Details for construction are shown in Standard Plan D-3, D-3.10 and D-3.11.

C. **Standard Reinforced Concrete Cantilever Walls –** Reinforced concrete cantilever walls consist of a base slab footing from which a vertical stem wall extends. These walls are suitable for heights up to 35 feet. Details for construction and the maximum bearing pressure in the soil are given in the Standard Plans D-10.10 to D-10.45.

   A major disadvantage of these walls is the low tolerance to post-construction settlement, which may require use of deep foundations (shafts or piling) to provide adequate support.

D. **Soldier Pile Walls and Soldier Pile Tieback Walls –** Soldier Pile Walls utilize wide flange steel members, such as W or HP shapes. The piles are usually spaced 6 to 10 feet apart. The main horizontal members are timber or precast concrete lagging designed to transfer the soil loads to the piles. For additional information see WSDOT *Geotechnical Design Manual* M 46-03 Chapter 15. See Appendix 8.1-A3 for typical soldier pile wall details.

E. **Soil Nail Walls –** The basic concept of soil nailing is to reinforce and strengthen the existing ground by installing steel bars called "nails" into a slope or excavation as construction proceeds from the "top down". Soil nailing is a technique used to stabilize moving earth, such as a landslide, or as temporary shoring. Soil anchors are used along with the strength of the soil to provide stability. The Geotechnical Engineer designs the soil nail system whereas the Bridge and Structures Office designs the wall fascia. Presently, the FHWA Publication FHWA-IF-03-017 "Geotechnical Engineering Circular No. 7 Soil Nail Walls" is being used for structural design of the fascia. See Appendix 8.1-A4 for typical soil nail wall details.

F. **Noise Barrier Walls –** Noise barrier walls are primarily used in urban or residential areas to mitigate noise or to hide views of the roadway. Common types, as shown in the Standard Plans, include cast-in-place concrete panels (with or without traffic barrier), precast concrete panels (with or without traffic barrier), and masonry blocks. The State Bridge and Structures Architect should be consulted for wall type selection. Design criteria for noise barrier walls are based on AASHTO's *Guide Specifications for Structural Design of Sound Barriers*. Details of these walls are available in the Standard Plans D-2.04 to D-2.68. The Noise Barriers chapter of the WSDOT *Design Manual* M 22-01 tabulates the design wind speeds and various exposure conditions used to determine the appropriate wall type.

8.1.3 Design

A. **General –** All designs shall follow procedures as outlined in AASHTO LRFD Chapter 11, the WSDOT *Geotechnical Design Manual* M 46-03, and this manual. See Appendix 8.1-A1 for a summary of design specification requirements for walls.

All construction shall follow procedures as outlined in the WSDOT *Standard Specifications for Road, Bridge, and Municipal Construction*, latest edition.

The Geotechnical Engineer will provide the earth pressure diagrams and other geotechnical design requirements for special walls to be designed by the Bridge and Structures Office. Pertinent soil data will also be provided for pre-approved proprietary structural earth walls (SEW), non-standard reinforced concrete retaining walls, and geosynthetic walls.

B. **Standard Reinforced Concrete Cantilever Retaining Walls –** The Standard Plan reinforced concrete retaining walls have been designed in accordance with the requirements of the AASHTO LRFD Bridge Design Specifications 4th Edition 2007 and interims through 2008.

1. Western Washington Walls (Types 1 through 4)

   a. The seismic design of these walls has been completed using and effective Peak Ground Acceleration of 0.51g. Extreme Event stability of the wall was based on 100% of the wall inertia force combined with 50% of the seismic earth pressure.

   b. Active Earth pressure distribution was linearly distributed per Section 7.7.4. The corresponding Ka values used for design were 0.24 for wall Types 1 and 2, and 0.36 for Types 3 and 4.

   c. Seismic Earth pressure distribution was uniformly distributed per WSDOT *Geotechnical Design Manual* M 46-03, Nov. 2008, Section 15.4.2.9, and was supplemented by AASHTO LRFD Bridge Design Specifications (Fig. 11.10.7.1-1). The corresponding Kae values used for design were 0.43 for Types 1 and 2, and 0.94 for Types 3 and 4.

   d. Passive Earth pressure distribution was linearly distributed. The corresponding Kp value used for design was 1.5 for all walls. For Types 1 and 2, passive earth pressure was taken over the depth of the footing. For Types 3 and 4, passive earth pressure was taken over the depth of the footing and the height of the shear key.

   e. The retained fill was assumed to have an angle of internal friction of 36 degrees and a unit weight of 130 pounds per cubic foot. The friction angle for sliding stability was assumed to be 32 degrees.

f. Load factors and load combinations used per AASHTO LRFD Bridge Design Specifications 3.4.1-1 and 2. Stability analysis performed per AASHTO LRFD Bridge Design Specifications Section 11.6.3 and C11.5.5-1& 2.

g. Wall Types 1 and 2 were designed for traffic barrier collision forces, as specified in AASHTO LRFD Bridge Design Specifications section A13.2 for TL-4. These walls have been designed with this force distributed over the distance between wall section expansion joints (48 feet).

2. Eastern Washington Walls (Types 5 through 8)

a. The seismic design of these walls has been completed using and effective Peak Ground Acceleration of 0.2g. Extreme Event stability of the wall was based on 100% of the wall inertia force combined with 50% of the seismic earth pressure.

b. Active Earth pressure distribution was linearly distributed per Section 7.7.4 of this manual. The corresponding Ka values used for design were 0.36 for wall Types 5 and 6, and 0.24 for Types 7 and 8.

c. Seismic Earth pressure distribution was uniformly distributed per WSDOT *Geotechnical Design Manual* M 46-03, Nov. 2008, Section 15.4.2.9, and was supplemented by AASHTO LRFD Bridge Design Specifications (Fig. 11.10.7.1-1). The corresponding Kae values used for design were 0.55 for Types 5 and 6, and 0.30 for Types 7 and 8.

d. Passive Earth pressure distribution was linearly distributed, and was taken over the depth of the footing and the height of the shear key. The corresponding Kp value used for design was 1.5 for all walls.

e. The retained fill was assumed to have an angle of internal friction of 36 degrees and a unit weight of 130 pounds per cubic foot. The friction angle for sliding stability was assumed to be 32 degrees.

f. Load factors and load combinations used per AASHTO LRFD Bridge Design Specifications 3.4.1-1& 2. Stability analysis performed per AASHTO LRFD Bridge Design Specifications Section 11.6.3 and C11.5.5-1 & 2.

g. Wall Types 7 and 8 were designed for traffic barrier collision forces, as specified in AASHTO LRFD Bridge Design Specifications section A13.2 for TL-4. These walls have been designed with this force distributed over the distance between wall section expansion joints (48 feet).

C. **Non-Standard Reinforced Concrete Retaining Walls –** For retaining walls where a traffic barrier is to be attached to the top of the wall, the AASHTO LRFD Extreme Event loading for vehicular collision must be analyzed. These loads are tabulated in LRFD Table A13.2-1. Although the current yield line analysis assumptions for this loading are not applicable to retaining walls, the transverse collision load (F_t) may be distributed over the longitudinal length (L_t) at the top of barrier. At this point, the load is distributed at a 45 degree angle into the wall. Future updates to the LRFD code will address this issue.

For sliding, the passive resistance in the front of the footing may be considered if the earth is more than 2 feet deep on the top of the footing and does not slope downward away from the wall. The design soil pressure at the toe of the footing shall not exceed the allowable soil bearing capacity supplied by the Geotechnical Engineer. For retaining walls supported by deep foundations (shafts or piles), refer to Sections 7.7.5, 7.8 and 7.9 of this manual.

D. **Soldier Pile and Soldier Pile Tieback Walls**

1. **Permanent Ground Anchors (Tiebacks) –** See AASHTO LRFD Section 11.9 “Anchored Walls”. The Geotechnical Engineer will determine whether anchors can feasibly be used at a particular site based on the ability to install the anchors and develop anchor capacity. The presence of utilities or other underground facilities, and the ability to attain underground easement rights may also determine whether anchors can be installed.

The anchor may consist of bars, wires, or strands. The choice of appropriate type is usually left to the Contractor but may be specified by the designer if special site conditions exist that preclude the use of certain anchor types. In general, strands and wires have advantages with respect to tensile strength, limited work areas, ease of transportation, and storage. However, bars are more easily protected against corrosion, and are easier to develop stress and transfer load.

The geotechnical report will provide a reliable estimate of the feasible factored design load of the anchor, recommended anchor installation angles (typically 10° to 45°), no-load zone dimensions, and any other special requirements for wall stability for each project.

Both the "tributary area method" and the "hinge method" as outlined in AASHTO LRFD Section C11.9.5.1 are considered acceptable design procedures to determine the horizontal anchor design force. The capacity of each anchor shall be verified by testing. Testing shall be done during the anchor installation (See *Standard Specification Section* 6-17.3(8) and WSDOT *Geotechnical Design Manual* M 46-03).

a. The horizontal anchor spacing typically follows the pile spacing of 6 to 10 feet. The vertical anchor spacing is typically 8 to 12 feet. A minimum spacing of 4 feet in both directions is not recommended because it can cause a loss of effectiveness due to disturbance of the anchors during installation.

b. For permanent ground anchors, the anchor DESIGN LOAD, T, shall be according to AASHTO LRFD. For temporary ground anchors, the anchor DESIGN LOAD, T, may ignore extreme event load cases.

c. The lock-off load is 60 percent of the controlling factored design load for temporary and permanent walls (see WSDOT *Geotechnical Design Manual* M 46-03 Chapter 15).

2. **Permanent Ground Anchor Corrosion Protection –** The Geotechnical Engineer will specify the appropriate protection system; the two primary types are:

   a. Simple Protection: The use of simple protection relies on Portland cement grout to protect the tendon, bar, or strand in the bond zone. The unbonded lengths are sheaths filled with anti-corrosion grease, heat shrink sleeves, and secondary grouting after stressing. Except for secondary grouting, the protection is usually in place prior to insertion of the anchor in the hole.

   b. Double Protection: a corrugated PVC, high-density polyethylene, or steel tube accomplishes complete encapsulation of the anchor tendon. The same provisions of protecting the unbonded length for simple protection are applied to those for double protection.

3. **Design of Soldier Pile –** The soldier piles shall be designed for shear, bending, and axial stresses according to the latest AASHTO LRFD and WSDOT *Geotechnical Design Manual* M 46-03 design criteria. The bending moment shall be based on the elastic section modulus "S" for the entire length of the pile for all Load combinations

   a. Lateral Loads

      (1) Lateral loads are assumed to act over one pile spacing above the base of excavation in front of the wall. These lateral loads result from horizontal earth pressure, live load surcharge, seismic earth pressure, or any other applicable load.

      (2) Lateral loads are assumed to act over the shaft diameter below the base of excavation in front of the wall. These lateral loads result from horizontal earth pressure, seismic earth pressure or any other applicable load.

      (3) Passive earth pressure usually acts over three times the shaft diameter or pile spacing, whichever is smaller.

b. Depth of Embedment

The depth of embedment of soldier piles shall be the maximum embedment as determined from the following;

(1) 10 feet

(2) As recommended by the Geotechnical Engineer of Record

(3) As required for skin friction resistance and end bearing resistance.

(4) As required to satisfy horizontal force equilibrium and moment equilibrium about the bottom of the soldier pile for cantilever soldier piles without permanent ground anchors.

(5) As required to satisfy moment equilibrium of lateral force about the bottom of the soldier pile for soldier piles with permanent ground anchors.

4. **Design of Lagging –** Lagging for soldier pile walls, with and without permanent ground anchors, may be comprised of timber, precast concrete, or steel. The expected service life of timber lagging is 20 years which is less than the 75 year service life of structures designed in accordance with AASHTO LRFD.

The Geotechnical Engineer will specify when lagging shall be designed for an additional 250 psf surcharge due to temporary construction load or traffic surcharge. The lateral pressure transferred from a moment slab shall be considered in the design of soldier pile walls and laggings.

Temporary Timber Lagging – Temporary lagging is based on a maximum 36 month service life before a permanent fascia is applied over the lagging. The wall Design Engineer shall review the Geotechnical Recommendations or consult with the Geotechnical Engineer regarding whether the lagging may be considered as temporary as defined in Section 6-16.3(6) of the *Standard Specifications*. Temporary timber lagging shall be designed by the contractor in accordance with Section 6-16.3(6)B of the *Standard Specifications.*

Permanent Lagging – Permanent lagging shall be designed for 100% of the lateral load that could occur during the life of the wall in accordance with AASHTO LRFD Sections 11.8.5.2 and 11.8.6 for simple spans without soil arching. A reduction factor to account for soil arching effects may be used if permitted by the Geotechnical Engineer.

Timber lagging shall be designed in accordance with AASHTO LRFD Section 8.6. The size effect factor (CF_b) should be considered 1.0, unless a specific size is shown in the wall plans. The wet service factor (CM_b) should be considered 0.85 for a saturated condition at some point during the life of the lagging. The load applied to lagging should be applied at the critical depth. The design should include the option for the contractor to step the size of lagging over the height of tall walls, defined as walls over 15 feet in exposed face height.

Timber lagging designed as a permanent structural element shall consist of treated Douglas Fir-Larch, grade No. 2 or better. Hem-fir wood species, due to the inadequate durability in wet condition, shall not be used for permanent timber lagging. Permanent lagging is intended to last the design life cycle (75 years) of the wall. Timber lagging does not have this life cycle capacity but can be used when both of the following are applicable:

(1) The wall will be replaced within a 20 year period or a permanent fascia will be added to contain the lateral loads within that time period.

And,

(2) The lagging is visible for inspections during this life cycle.

5. **Design of Fascia Panels –** Cast-in-place concrete fascia panels shall be designed as a permanent load carrying member in accordance with AASHTO LRFD Section 11.8.5.2. For walls without permanent ground anchors the minimum structural thickness of the fascia panels shall be 9 inches. For walls with permanent ground anchors the minimum structural thickness of the fascia panels shall be 14 inches. Architectural treatment of concrete fascia panels shall be indicated in the plans.

   Concrete strength shall not be less than 4,000 psi at 28 days. The wall is to extend 2 feet minimum below the finish ground line adjacent to the wall.

   When concrete fascia panels are placed on soldier piles, a generalized detail of lagging with strongback (see Appendix 8.1-A3-5) shall be shown in the plans. This information will assist the contractor in designing formwork that does not overstress the piles while concrete is being placed.

   Precast concrete fascia panels shall be designed to carry 100% of the load that could occur during the life of the wall. When timber lagging (including pressure treated lumber) is designed to be placed behind a precast element, conventional design practice is to assume that lagging will eventually fail and the load will be transferred to the precast panel. If another type of permanent lagging is used behind the precast fascia panel, then the design of the fascia panel will be controlled by internal and external forces other than lateral pressures from the soil (weight, temperature, Seismic, Wind, etc.). The connections for precast panels to soldier piles shall be designed for all applicable loads and the designer should consider rigidity, longevity (to resist cyclic loading, corrosion, etc.), and load transfer.

   See Section 5.1.1 of this manual for use of shotcrete in lieu of cast-in-place conventional concrete for soldier pile fascia panels.

8.1.4 Miscellaneous Items

A. **Drainage –** Drainage features shall be detailed in the Plans.

   Permanent drainage systems shall be provided to prevent hydrostatic pressures developing behind the wall. A cut that slopes toward the proposed wall will invariably encounter natural subsurface drainage. Vertical chimney drains or prefabricated drainage mats can be used for normal situations to collect and transport drainage to a weep hole or pipe located at the base of the wall. Installing horizontal drains to intercept the flow at a distance well behind the wall may control concentrated areas of subsurface drainage (see WSDOT *Geotechnical Design Manual* M 46-03 Chapter 15).

   All reinforced concrete retaining walls shall have 3-inch diameter weepholes located 6 inches above final ground line and spaced about 12 feet apart. In case the vertical distance between the top of the footing and final ground line is greater than 10 feet, additional weepholes shall be provided 6 inches above the top of the footing. No weepholes are necessary in cantilever wingwalls.

   Weepholes can get clogged up or freeze up, and the water pressure behind the wall may start to increase. In order to keep the water pressure from building, it is important to have well draining gravel backfill and underdrains. Appropriate details must be shown in the Plans.

   No underdrain pipe or gravel backfill for drains is necessary behind cantilever wingwalls. A 3 foot minimum thickness of gravel backfill shall be shown in the Plans behind the cantilever wingwalls. Backfill material shall be included with the civil quantities (not the bridge quantities). If it is necessary to excavate existing material for the backfill, then this excavation shall be a part of the bridge quantities for "Structure Excavation Class A Incl. Haul".

B. **Scour –** The foundation for all walls constructed along rivers and streams shall be evaluated during design by the Hydraulics Engineer for scour in accordance with AASHTO LRFD Sec. 2.6.4.4.2. The wall foundation shall be located at least 2 feet below the scour depth in accordance with the WSDOT *Geotechnical Design Manual* M 46-03 Section15.4.5.

C. **Joints –** For cantilevered and gravity walls constructed without a traffic barrier attached to the top, joint spacing should be a maximum of 24 feet on centers. For cantilevered and gravity walls constructed with a traffic barrier attached to the top, joint spacing should be a maximum of 48 feet on centers or that determined for adequate distribution of the traffic collision loading. For counterfort walls, joint spacing should be a maximum of 32 feet on centers. For soldier pile and soldier pile tieback walls with concrete fascia panels, joint spacing should be 24 to 32 feet on centers. For precast units, the length of the unit depends on the height and weight of each unit. Odd panels for all types of walls shall normally be made up at the ends of the walls. Every joint in the wall shall provide for expansion. For cast-in-place construction, a minimum of ½ inch premolded filler should be specified in the joints. A compressible back-up strip of closed-cell foam polyethylene or butyl rubber with a sealant on the front face is used for precast concrete walls.

No joints other than construction joints shall be used in footings except at bridge abutments and where substructure changes such as spread footing to pile footing occur. In these cases, the footing shall be interrupted by a ½ inch premolded expansion joint through both the footing and the wall. The maximum spacing of construction joints in the footing shall be 120 feet. The footing construction joints should have a 6-inch minimum offset from the expansion joints in the wall.

D. **Architectural Treatment –** The type of surface treatment for retaining walls is decided on a project specific basis. Consult the State Bridge and Structures Architect during preliminary plan preparation for approval of all retaining wall finishes, materials and configuration. The wall should blend in with its surroundings and complement other structures in the vicinity.

E. **Shaft Backfill for Soldier Pile Walls –** Specify controlled density fill (CDF, 145 pcf) for soldier pile shafts (full height) when shafts are anticipated to be excavated in the dry

When under water concrete placement is anticipated for the soldier pile shafts, specify pumpable lean concrete.

F. **Detailing of Standard Reinforced Concrete Retaining Walls**

1. In general, the “H” dimension shown in the retaining wall Plans should be in foot increments. Use the actual design “H” reduced to the next lower even foot for dimensions up to 3 inches higher than the even foot.

   Examples: Actual height = 15′-3”↑, show “H” = 15’ on design plans
   Actual height > 15′-3”↑, show “H” = 16’ on design plans

   For walls that are not of a uniform height, “H” should be shown for each segment of the wall between expansion joints or at some other convenient location. On walls with a steep slope or vertical curve, it may be desirable to show 2 or 3 different “H” dimensions within a particular segment. The horizontal distance should be shown between changes in the “H” dimensions.

   The value for “H” shall be shown in a block in the center of the panel or segment. See Example, Figure 9.4.4-1.

2. Follow the example format shown in Figure 8.1.4-1.
3. Calculate approximate quantities using the Standard Plans.
4. Wall dimensions shall be determined by the designer using the Standard Plans.
5. Do not show any details given in the Standard Plans.
6. Specify in the Plans all deviations from the Standard Plans.
7. Do not detail reinforcing steel, unless it deviates from the Standard Plans.
8. For pile footings, use the example format with revised footing sizes, detail any additional steel, and show pile locations. Similar plan details are required for footings supported by shafts.

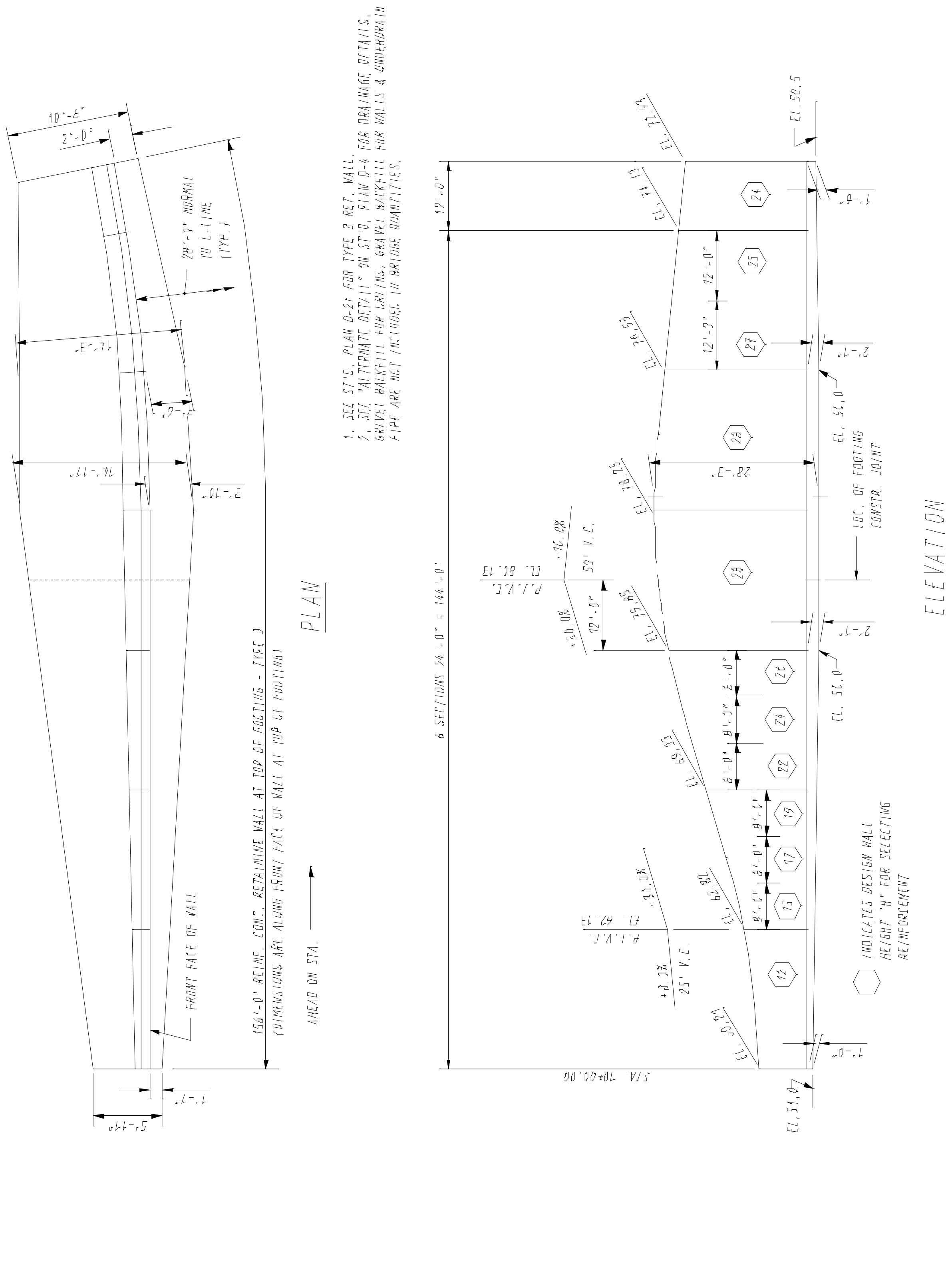

PLAN
ELEVATION
1. SEE ST'D. PLAN D-2f FOR TYPE 3 RET. WALL.
2. SEE "ALTERNATE DETAIL" ON ST'D. PLAN D-4 FOR DRAINAGE DETAILS, GRAVEL BACKFILL FOR DRAINS, GRAVEL BACKFILL FOR WALLS & UNDERDRAIN PIPE ARE NOT INCLUDED IN BRIDGE QUANTITIES.
156'-0" REINF. CONC. RETAINING WALL AT TOP OF FOOTING - TYPE 3
(DIMENSIONS ARE ALONG FRONT FACE OF WALL AT TOP OF FOOTING)
AHEAD ON STA.
FRONT FACE OF WALL
6 SECTIONS 24'-0" = 144'-0"
LOC. OF FOOTING CONSTR. JOINT
INDICATES DESIGN WALL HEIGHT "H" FOR SELECTING REINFORCEMENT

Figure 8.1.4-1

9.2 Bearings

9.2.1 General Considerations

Bridge bearings facilitate the transfer of vehicular and other environmentally imposed loads from the superstructure down to the substructure, and ultimately, to the ground. In fulfilling this function, bearings must accommodate anticipated movements (thermal expansion/contraction) while also restraining undesired movements (seismic displacements). Because the movements allowed by an adjacent expansion joint must be compatible with the movement restriction imposed by a bearing, bearings and expansion joints must be designed interdependently and in conjunction with the anticipated behavior of the overall structure.

Numerous types of bearings are used for bridges. These include steel reinforced elastomeric bearings, fabric pad sliding bearings, steel pin bearings, rocker bearings, roller bearings, steel pin bearings, pot bearings, spherical bearings, disk bearings, and seismic isolation bearings. Each of these bearings possess different characteristics in regard to vertical and horizontal load carrying capacity, vertical stiffness, horizontal stiffness, and rotational stiffness. A thorough understanding of these characteristics is essential for economical bearing selection and design. Spherical bearings, disk bearings, and pot bearings are sometimes collectively referred to as high load multi-rotational (HLMR) bearings.

9.2.2 Force Considerations

Bridge bearings must be explicitly designed to transfer all anticipated loads from the superstructure to the substructure. These forces may be directed vertically, longitudinally, or transversely with respect to the global orientation of the bridge. In accordance with LRFD provisions, most bearing design calculations are based upon service limit state stresses. Impact need not be applied to live load forces in the design of bearings.

Experience has empirically led to the following practical load capacity approximations for various bearing types:

Bearing Type	Approx. Load Capacity
Steel reinforced elastomeric (Method B)	Less than 800 kips
Fabric pad	Less than 600 kips
Steel pin	More than 600 kips
Spherical and disk	More than 800 kips
Seismic isolation	Less than 800 kips

9.2.3 Movement Considerations

Bridge bearings can be detailed to provide translational fixity, to permit free translation in any horizontal direction, or to permit guided translation. The movement restrictions thus imposed by a bearing must be compatible with the movements allowed by an adjacent expansion joint. Additionally, both bearings and expansion joints must be designed consistent with the anticipated load and deformation behavior of the overall structure. Design rotations shall be calculated as follows:

A. **Elastomeric and Fabric Pad Bearings –** The maximum service limit state rotation for bearings that do not have the potential to achieve hard contact between metal components shall be taken as the sum of unfactored dead and live load rotations plus an allowance for fabrication and construction uncertainties of 0.005 radians.

B. **HLMR Bearings –** Both service and strength limit state rotations are used in the design of HLMR bearings. These rotations must be shown on the plans to allow the manufacturer to properly design and detail a bearing.

   The service limit state rotation shown on the plans shall include an allowance for uncertainties of +/-0.005 radians.

The strength limit state rotation is used to assure that contact between hard metal or concrete surfaces is prevented under the full range of expected loading. In accordance with the AASHTO LRFD Bridge Design Specifications, the strength limit state rotation shown on the plans shall include allowances of:

1. For disc bearings, +/-0.005 radians for uncertainties
2. For other HLMR bearings, such as spherical, pot, and steel pin bearings, +/-0.005 radians for fabrication and installation tolerances and an additional +/-0.005 radians for uncertainties

9.2.4 Detailing Considerations

HLMR bearings shall be designed, detailed, fabricated, and installed to facilitate inspection, maintenance, and eventual replacement. Jacking points shall be identified in the contract drawings so that bearings can be reset, repaired, or replaced. In some instances, bearings may need to be reset in order to mitigate unintended displacements induced by construction sequences.

Prestressed concrete girder bridges having end Type A (semi - integral) need not be detailed to accommodate elastomeric bearing replacement at abutments. Prestressed concrete girder bridges having end Type B (L-type abutments) shall be designed and detailed to accommodate elastomeric bearing replacement at abutments. Specifically, girder stops and end diaphragms shall be detailed to accommodate the placement of hydraulic jacks. The standard end diaphragms for long-span girders may not have sufficient flexural and shear capacity to support jacking induced stresses. The designer shall check these and provide sufficient steel reinforcement to accommodate shear forces and bending moments induced by jacking. (Girder end Types A and B are depicted on Figures 5.6.2-4 and 5.6.2-5.) Incidentally, intermediate piers of prestressed concrete girder bridges having steel reinforced elastomeric bearings shall also be designed and detailed to facilitate bearing replacement.

9.2.5 Bearing Types

A. **Elastomeric Bearings –** Elastomeric bearings are perhaps the simplest and most economical of all bridge bearings. They are broadly classified into four types: plain elastomeric pads, fiberglass reinforced elastomeric pads, steel reinforced elastomeric pads, and cotton duck reinforced elastomeric pads. Of these four types, the latter two are used extensively for bridge construction. Incidentally, cotton duck reinforced elastomeric pads are generally referred to as fabric pad bearings. This subsection will address steel reinforced elastomeric bearings. A subsequent section will address fabric pad bearings.

   A steel reinforced elastomeric bearing consists of discrete steel shims vulcanized between adjacent discrete layers of elastomer. The vulcanization process occurs in an autoclave under conditions of high temperature and pressure. The constituent elastomer is either natural rubber or synthetic rubber (neoprene). Steel reinforced elastomeric bearings are commonly used with prestressed concrete girder bridges and may be used with other bridge types. Because of their relative simplicity and fabrication ease, steel reinforced elastomeric bearings offer significant economy relative to HLMR bearings.

   Steel reinforced elastomeric bearings rely upon the inherent shear flexibility of the elastomer layers to accommodate bridge movements in any horizontal direction. This shear flexibility also enhances their rotational flexibility. The steel shims limit the tendency for the elastomer layers to bulge laterally under compressive load.

   Steel reinforced elastomeric bearings can be designed by either the Method A or Method B procedure delineated in the LRFD provisions. Current WSDOT policy is to design all elastomeric bearings using the Method B provisions, which provides more relief in meeting rotational demands than Method A. The Method A design procedure is a carryover based upon more conservative interpretation of past theoretical analyses and empirical observations prior to research leading up to the publication of *NCHRP Report 596 Rotation Limits for Elastomeric Bearings.*

   Both Method A and Method B design procedures require determination of the optimal geometric parameters to achieve an appropriate balance of compressive, shear, and rotational stiffnesses and capacities. Fatigue susceptibility is controlled by limiting live load compressive stress. Delamination (of steel shim-elastomer interface) susceptibility is controlled by limiting total compressive stress. Assuring adequate shim thickness precludes yield and rupture of the steel shims. Excessive shear deformation is controlled and rotational flexibility is assured by providing adequate total elastomer height. Generally, total elastomer thickness shall be no less than twice the maximum anticipated lateral deformation. Overall bearing stability is controlled by limiting total bearing height relative to its plan dimensions. The most important design parameter for reinforced elastomeric bearings is the shape factor. The shape factor is defined as the plan area of the bearing divided by the area of the perimeter free to bulge (perimeter multiplied by thickness of one layer of elastomer).

Axial, rotational, and shear loading generate shear strain in the constituent elastomeric layers of a typical bearing. Computationally, Method B imposes a limit on the sum of these shear strains. It distinguishes between static and cyclic components of shear strain by applying an amplification factor of 1.75 to cyclic components to reflect cumulative degradation caused by repetitive loading.

In essence, elastomeric bearing design reduces to checking several mathematical equations while varying bearing plan dimensions, number of elastomeric layers and their corresponding thicknesses, and steel shim thicknesses. Because these calculations can become rather tedious, MS Excel spreadsheets have been developed and are available for designs using both Method A and Method B procedures. See the Bearing and Expansion Joint Specialist for these design tools.

LRFD design may result in thicker steel reinforced elastomeric bearings than previous designs, particularly for shorter span bridges. This is a consequence of the increased rotational flexibility required to accommodate the 0.005 radian allowance for uncertainties and partially to inherent conservatism built into the rotational capacity equations.

Although constituent elastomer has historically been specified by durometer hardness, shear modulus is the most important physical property of the elastomer for purposes of bearing design. Research has concluded that shear modulus may vary significantly among compounds of the same hardness. Accordingly, shear modulus shall be specified on the plans as 165 psi at 73ºF without reference to durometer hardness.

Elastomeric bearings shall conform to the requirements of AASHTO Specification M 251 *Plain and Laminated Elastomeric Bridge Bearings*. Shims shall be fabricated from ASTM A 1011 Grade 36 steel unless noted otherwise on the plans. Bearings shall be laminated in ½ inch thick elastomeric layers with a minimum total thickness of 1 inch. For overall bearing heights less than 5 inches, a minimum of ¼ inch of side clearance shall be provided over the steel shims. For overall heights greater than 5 inches, a minimum of ½ inch of side clearance shall be provided. Live load compressive deflection shall be limited to 1/16 inch. AASHTO Specification M 251 requires elastomeric bearings to be subjected to a series of tests, including a compression test at 150 percent of the total service load. For this reason, compressive dead load and live load shall be specified on the plans.

With respect to width, elastomeric bearings shall be designed and detailed as follows:

1. For prestressed concrete wide flange girders (WF42G, WF50G, WF58G, WF74G, and W95G), the edge of the bearing pad shall be set between 1 in. minimum and 9 in. maximum inside of the edge of the girder bottom flange.
2. For prestressed concrete I-girders, bulb-tee girders, and deck bulb-tee girders, the edge of the bearing pad shall be set 1 in. in side of the edge of the girder bottom flange.
3. For all prestressed concrete tub girders, the edge of the bearing shall be set 1in. inside of the edge of the bottom slab. Bearing pads for prestressed concrete tub girders shall be centered close to the centerline of each web.
4. For all prestressed concrete slabs, the edge of the bearing shall be set 1 in. inside of the edge of the slab. Two bearing pads and corresponding grout pads are required for each end of the prestressed concrete slabs. The need for steel shims shall be assessed during the bearing design.

As mentioned earlier, LRFD Article 14.4.2.1 requires that a 0.005 radian allowance for uncertainties be included in the design of steel reinforced elastomeric bearings. This allowance applies to both rotations θ_x and θ_y. The Article 14.4.2 Commentary somewhat ambiguously states "An owner may reduce the fabrication and setting tolerance allowances if justified by a suitable quality control plan; therefore, these tolerance limits are stated as recommendations rather than absolute limits." Consult with the Bearings and Expansion Joint Specialist in instances in which the 0.005 radian tolerance precludes convergence to a reasonable design solution.

In order to facilitate compressive load testing, future bearing replacement, and vertical geometry coordination, the following table shall be included in the Plans:

Bearing Design Table Service I Limit State	
Dead load reaction	--------- kips
Live load reaction (w/o impact)	--------- kips
Unloaded height	--------- inches
Loaded height (*DL*)	--------- inches
Shear modulus at 73º F	--------- psi

In the construction of precast prestressed concrete girder and steel girder bridges, elastomeric bearings are generally not offset to account for temperature during erection of the girders as are most other bearing systems. Girders may be set atop elastomeric bearings at temperatures other than the mean of the temperature range. This is statistically reconciled by assuming a maximum thermal movement in either direction of:

$$\Delta_{temp} = 0.75 \cdot \alpha \cdot L \cdot (T_{MaxDesign} - T_{MinDesign})$$

where $T_{MaxDesign}$ is the maximum anticipated bridge deck average temperature and $T_{MinDesign}$ is the minimum anticipated bridge deck average temperature during the life of the bridge.

For precast prestressed concrete girder bridges, the maximum thermal movement, Δ_{temp}, shall be added to shrinkage and long-term creep movements to determine total bearing height required. The shrinkage movement for this bridge type shall be half that calculated for a cast-in-place concrete bridge.

For cast-in-place concrete bridges, it is assumed that the temperature of concrete at placement is equal to the normal temperature, as defined by the *Standard Specifications*. Total shrinkage movement is added to the maximum thermal movement, Δ_{temp}, to determine required total height of the elastomeric bearing, as noted in Section 9.1.2-A.

B. **Fabric Pad Sliding Bearings –** Fabric pad sliding bearings incorporate fabric pads with a polytetrafluoroethylene (PTFE) - stainless steel sliding interface to permit large translational movements. Unlike a steel reinforced elastomeric bearing having substantial shear flexibility, the fabric pad alone cannot accommodate translational movements. Fabric pads can accommodate very small amounts of rotational movement; less than can be accommodated by more flexible steel reinforced elastomeric bearings. Practical size considerations limit the use of fabric pad bearings to total service load reactions under about 600 kips.

   PTFE, also referred to as Teflon, is available in several forms: unfilled sheet, dimpled lubricated, filled, and woven. Filled PTFE contains glass, carbon, or other chemically inert fibers that enhance its resistance to creep (cold flow) and wear. Interweaving high strength fibers through PTFE material creates woven PTFE. Dimpled PTFE contains dimples, which act as reservoirs for silicone grease lubricant.

   Friction coefficients for PTFE – stainless steel surfaces vary significantly as a function of PTFE type, contact pressure, and ambient temperature. The AASHTO LRFD provides friction coefficients as a function of these variables. Dimpled lubricated PTFE at high temperatures and high contact pressures typically yield the lowest friction coefficients. Filled PTFE at low temperatures and low contact pressures yield the highest friction coefficients.

In order to minimize frictional resistance, a Number 8 (Mirror) finish should be specified for all flat stainless steel surfaces in contact with PTFE. The low-friction characteristics of a PTFE – stainless steel interface are actually facilitated by fragmentary PTFE sliding against PTFE after the fragmentary PTFE particles are absorbed into the asperities of the stainless steel surface.

In fabric pad sliding bearings, the PTFE is generally recessed half its depth into a steel backing plate, which is generally bonded to the top of a fabric pad. The recess provides confinement that minimizes creep (cold flow). The stainless steel sheet is typically seal welded to a steel sole plate attached to the superstructure.

Silicone grease is not recommended for non-dimpled PTFE. Any grease will squeeze out under high pressure and attract potentially detrimental dust and other debris.

1. **Fabric Pad Design –** WSDOT's design criteria for fabric pad bearings are based upon manufacturers' recommendations, supported by years of satisfactory performance. These criteria differ from AASHTO LRFD provisions in that they recognize significantly more rotational flexibility in the fabric pad. Our maximum allowable service load average bearing pressure for fabric pad bearing design is 1,200 psi. WSDOT's maximum allowable service load edge bearing pressure for fabric pad bearing design is 2,000 psi. A 1,200 psi compressive stress corresponds to 10 percent strain in the fabric pad while a 2,000 psi compressive stress corresponds to 14 percent compressive strain. Based upon this information, the following design relationship can be established:

   $$\theta = \frac{2 \times (.14 - .10) \times T}{L}$$

   $$\theta = \frac{.08 \times T}{L}$$

   $$T = 12.5 \times \theta \times L$$

   Where Θ = rotation due to loading plus construction tolerances

   L = pad length (parallel to longitudinal axis of beam)

   T = fabric pad thickness required

   As an example:

   Given: $DL + LL$ = 240 kips

   Rotation = 0.015 radians

   Allowable bearing pad pressure = 1200 psi

   f'_c = 3000 psi

   Find: fabric pad plan area and thickness required

   Solution:

   Pad area required = 240,000/1200 = 200 in^2

   Try a 20″ wide × 10″ long fabric pad

   T = 12.5(.015)(10″) = 1.88″

   Solution: Use a 20″ × 10″ × 1⅞″ fabric pad.

2. **PTFE – Stainless Steel Sliding Surface Design –** PTFE shall be ⅛ in. thick and recessed 1/16 in. into a ½ in. thick steel plate that is bonded to the top of the fabric pad. With the PTFE confined in this recess, the LRFD code permits an average contact stress of 4,500 psi for all loads calculated at the service limit state and an average contact stress of 3,000 psi for permanent loads calculated at the service limit state. The LRFD code permits slightly higher edge contact stresses.

For example, suppose:

DL = 150 kips

LL = 90 kips

A_{PTFE} > (150 kips + 90 kips)/4.5 ksi = 53.3 in^2

A_{PTFE} > 150 kips/3 ksi = 50.0 in^2

Selected area of PTFE must exceed 53.3 in^2

Stainless steel sheet shall be finished to a No. 8 (Mirror) finish and seal welded to the sole plate.

C. **Pin Bearings –** Steel pin bearings are generally used to support heavy reactions with moderate to high levels of rotation about a single predetermined axis. This situation generally occurs with long straight steel plate girder superstructures.

D. **Rocker and Roller Type Bearings –** Steel rocker bearings have been used extensively in the past to allow both rotation and longitudinal movement while supporting large loads. Because of their seismic vulnerability and the more extensive use of steel reinforced elastomeric bearings, rocker bearings are no longer specified for new bridges.

Steel roller bearings have also been used extensively in the past. Roller bearings permit both rotation and longitudinal movement. Pintles are generally used to connect the roller bearing to the superstructure above and to the bearing plate below. Nested roller bearings have also been used in the past. Having been supplanted by more economical steel reinforced elastomeric bearings, roller bearings are infrequently used for new bridges today.

E. **Spherical Bearings –** A spherical bearing relies upon the low-friction characteristics of a curved PTFE - stainless steel interface to provide a high level of rotational flexibility in multiple directions. An additional flat PTFE - stainless steel surface can be incorporated into the bearing to additionally provide either guided or non-guided translational movement capability.

Woven PTFE is generally used on the curved surfaces of spherical bearings. Woven PTFE exhibits enhanced creep (cold flow) resistance and durability characteristics relative to unwoven PTFE. When spherical bearings are detailed to accommodate translational movement, woven PTFE is generally specified on the flat sliding surface also. The LRFD code permits an average contact stress of 4,500 psi for all loads calculated at the service limit state and an average contact stress of 3,000 psi for permanent loads calculated at the service limit state. The LRFD code permits slightly higher edge contact stresses.

Both stainless steel sheet and solid stainless steel have been used for the convex sliding surface of spherical bearings. According to one manufacturer, curved sheet is generally acceptable for contact surface radii greater than 14 in to 18. in For smaller radii, a solid stainless steel convex plate or a stainless steel inlay is used. The inlay is welded to the solid conventional steel. If the total height of the convex plate exceeds about 5 in, a stainless steel inlay will likely be more economical.

Most spherical bearings are fabricated with the concave surface oriented downward to minimize dirt infiltration between PTFE and the stainless steel surface. Structural analysis of the overall structure must recognize the center of rotation of the bearing not being coincident with the neutral axis of the girder above.

The contract drawings must show the diameter and height of the spherical bearing in addition to all dead, live, and seismic loadings. Total height depends upon the radius of the curved surface, diameter of the bearing, and total rotational capacity required. Consult the Bearing and Expansion Joint Specialist for design calculation examples. Additionally, sole plate connections, base plate, anchor bolts, and any appurtenances for horizontal force transfer must be detailed on the plans. The spherical bearing manufacturer is required to submit shop drawings and detailed structural design calculations of spherical bearing components for review by the Engineer.

F. **Disk Bearings –** A disk bearing is composed of an annular shaped urethane disk designed to provide moderate levels of rotational flexibility. A steel shear-resisting pin in the center provides resistance against lateral force. A flat PTFE - stainless steel surface can be incorporated into the bearing to also provide translational movement capability, either guided or non-guided.

G. **Seismic Isolation Bearings –** Seismic isolation bearings mitigate the potential for seismic damage by utilizing two related phenomena: dynamic isolation and energy dissipation. Dynamic isolation allows the superstructure to essentially float, to some degree, while substructure elements below move with the ground during an earthquake. The ability of some bearing materials and elements to deform in certain predictable ways allows them to dissipate earthquake energy that might otherwise damage critical structural elements.

Numerous seismic isolation bearings exist, each relying upon varying combinations of dynamic isolation and energy dissipation. These devices include lead core elastomeric bearings, high damping rubber, friction pendulum, hydraulic dampers, and various hybrid variations.

Effective seismic isolation bearing design requires a thorough understanding of the dynamic characteristics of the overall structure as well as the candidate isolation devices. Isolation devices are differentiated by maximum compressive load capacity, lateral stiffness, lateral displacement range, maximum lateral load capacity, energy dissipation per cycle, functionality in extreme environments, resistance to aging, fatigue and wear properties, and effects of size.

The Highway Innovative Technology Evaluation Center (HITEC) has developed guidelines for testing seismic isolation and energy dissipating devices. With the goal of disseminating objective information to design professionals, HITEC has tested and published technical reports on numerous proprietary devices. These tests include performance benchmarks, compressive load dependent characterization, frequency dependent characterization, fatigue and wear, environmental aging, dynamic performance at extreme temperatures, durability, and ultimate performance.

9.2.6 Miscellaneous Details

A. **Temporary Support before Grouting Masonry Plate –** The masonry plate of a HLMR bearing is generally supported on a grout pad that is installed after the bearing and superstructure girders above have been erected. This procedure allows the Contractor to level and slightly adjust the horizontal location of the bearing before immobilizing it by placing the grout pad. Several methods have been developed to temporarily support the masonry plate until the grout is placed. The two most commonly used methods will be discussed here.

1. **Shim Packs –** Multiple stacks of steel shim plates can be placed atop the concrete surface to temporarily support the weight of the girders on their bearings before grouting. engineering judgment must be used in selecting the number and plan size of the shims taking grout flowability and shim height adjustability into consideration.

2. **Two-step Grouting with Cast Sleeves –** A two-step grouting procedure with cast-in-place voided cores can be used for smaller HLMRs not generally subjected to uplift. Steel studs are welded to the underside of the masonry plate to coincide with the voided cores. With temporary shims installed between the top of the concrete surface and the underside of the masonry plate, the voided cores are fully grouted. Once the first stage grout has attained strength, the shims are removed, the masonry plate is dammed, and grout is placed between the top of the concrete surface and the underside of the masonry plate.

B. **Anchor Bolts –** Anchor bolts shall be designed to resist all horizontal shear forces and direct tension force due to uplift.

Anchor bolts shall be ASTM A 449 where strengths equal to ASTM A 325 are required and ASTM A 354, Grade BD, where strengths equal to ASTM A 490 are required. ASTM F 1554 bolts with supplemental Charpy test requirements shall be specified in applications in which the bolts are subject to seismic loading.

9.2.7 Contract Drawing Representation

High load multi-rotational bearings are generally depicted schematically in the contract drawings. Each bearing manufacturer has unique fabricating methods and procedures that allow it to fabricate a bearing most economically. Depicting the bearings schematically with loads and geometric requirements provides each manufacturer the flexibility to innovatively achieve optimal economy.

9.2.8 Shop Drawing Review

The manufacturer designs and develops shop drawings for high load multi-rotational bearings. The Engineer is responsible for checking and approving the calculations and shop drawings. The calculations shall verify the structural adequacy of all components of the bearing. Each bearing shall be detailed to permit the inspection and replacement of components.

9.2.9 Bearing Replacement Considerations

In some situations, existing bearings, or elements thereof, must be replaced consequent to excessive wear or seismic rehabilitation. Bearing replacement operations generally require lifting of superstructure elements using hydraulic jacks. The designer is responsible for calculating anticipated lifting loads and stipulating these loads on the contract drawings. Limitations on lift height shall also be specified. Consideration shall be given to lift height as it relates to adjacent expansion joints elements and adjoining sections of railing. Stresses induced as a consequence of differential lift height between multiple hydraulic jacks are generally addressed by stipulating restrictions in the plans or special provisions.

Past experience shows that actual lifting loads nearly always exceed calculated lifting loads. Many factors may contribute to this phenomenon, including friction in the hydraulic jack system and underestimation of superstructure dead loads. Unless the Bearing and Expansion Joint Specialist or the Bridge Design Engineer approves a variance, contract documents shall require that all hydraulic jacks be sized for 200 percent of the calculated lifting load. In all cases, the designer shall verify from manufacturer's literature that appropriate hydraulic jacks are available to operate within the space limitations imposed by a particular design situation.

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 *Standard Plans* C-70.10-00 and C-80.10-00 for details.

   If the difference in grade elevations is 4′-0″ or less, then the barrier shall be designed as a rigid system in accordance with AASHTO LRFD *Bridge Design Specifications* barrier loading with the following requirements:

   1. All applicable loads shall be applied in accordance to AASHTO LRFD Section 3. The traffic barrier shall satisfy AASHTO LRFD Section 13.
   2. For soil loads without vehicle impact loads, the barrier shall be designed as a retaining wall (barrier weight resists overturning and sliding). Passive soil resistance may be considered with concurrence by the geotechnical engineer.
   3. Vehicle impact loads shall be applied to the top of barrier on the side of the barrier retaining soil.
   4. For soil loads with vehicle impact loads, the AASHTO LRFD Extreme Event loading for vehicular collision shall also be analyzed. Equivalent Static Load (ESL) per NCHRP Report 663 may be applied as the transverse vehicle impact load for evaluating sliding, bearing, and overturning. Sliding resistance factor of 0.8 and overturning resistance factor of 0.5 shall be used when designing for the ESL (supersedes AASHTO LRFD 10.5.5.3.3). For TL-3 and TL-4 barrier systems, the ESL shall be 10 kips and for TL-5, the ESL shall be 23 kips.
   5. The length of the barrier required for stability shall be no more than 10 times the overall height limited to the length between barrier expansion joints. The barrier shall act as a rigid body behavior and be continuous throughout this length of barrier. Any coupling between adjacent barrier sections or friction that may exist between free edges of barrier and the surrounding soil shall be neglected.
   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 barrier 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.

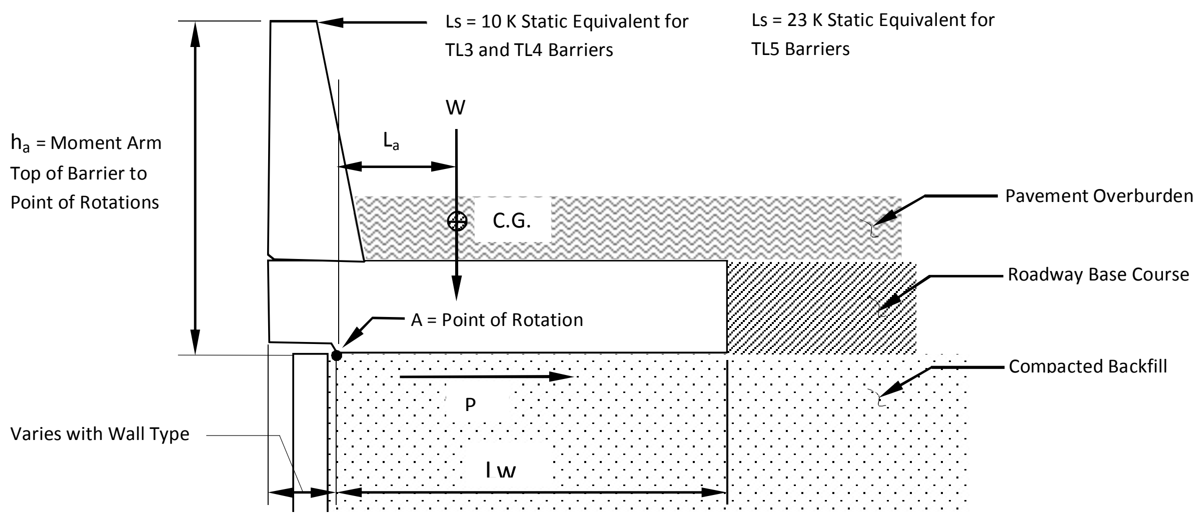
   Median traffic barriers with a grade difference greater than 4′-0″ shall be designed as standard plan retaining walls with a traffic barrier at the top and a barrier shape at the cut face.

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 resistance 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 Capacit**y – The structural capacity of the barrier and concrete moment slab shall be designed using impulse loads at appropriate Test Level (TL-3, TL-4, TL-5) applied to the top of the barrier in accordance with Sections 5 and 13 of *AASHTO LRFD Bridge Design Specifications*. Any section along the moment slab shall not fail in shear, bending, or torsion when the barrier is subjected to the design impact loads. 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.

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 continuous moment slab through rigid body behavior. Barrier shall also be continuous throughout this length of moment slab. 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 limited to the length between moment slab joints.

   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.

Chapter 10 Signs, Barriers, Approach Slabs, and Utilities

Contents

10.2 Bridge Traffic Barriers

10.2.1 General Guidelines

The design criteria for traffic barriers on structures shall be in accordance with Section 13 of the AASHTO LRFD *Bridge Design Specifications*. The following guidelines supplement the requirements in AASHTO LRFD.

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.

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.

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. If conditions require a higher test level, the test level shall be indicated in the general notes. 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 AASHTO LRFD Section 13 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

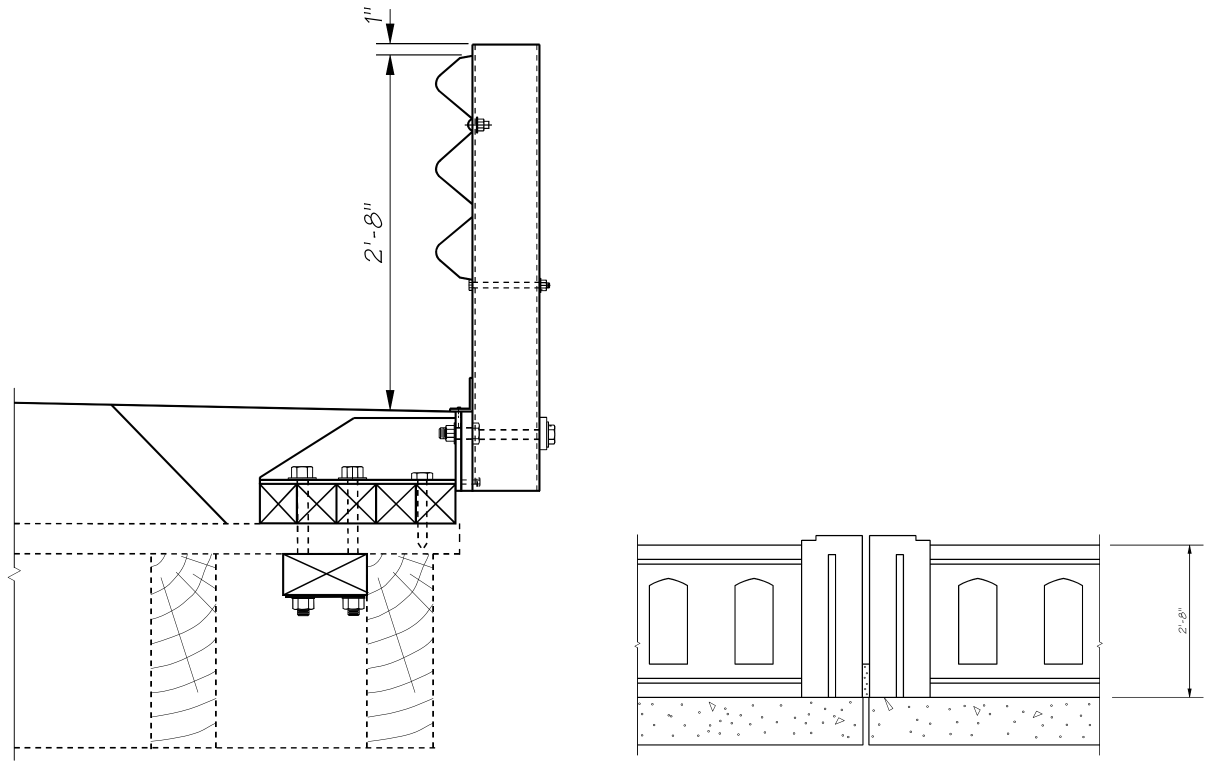
10.2.2 Bridge Railing Test Levels

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. Six different bridge railing test levels, TL-1 thru TL-6, and associated crash test/performance requirements are given in AASHTO LRFD Section 13 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/inside-txdot/division/bridge.html.

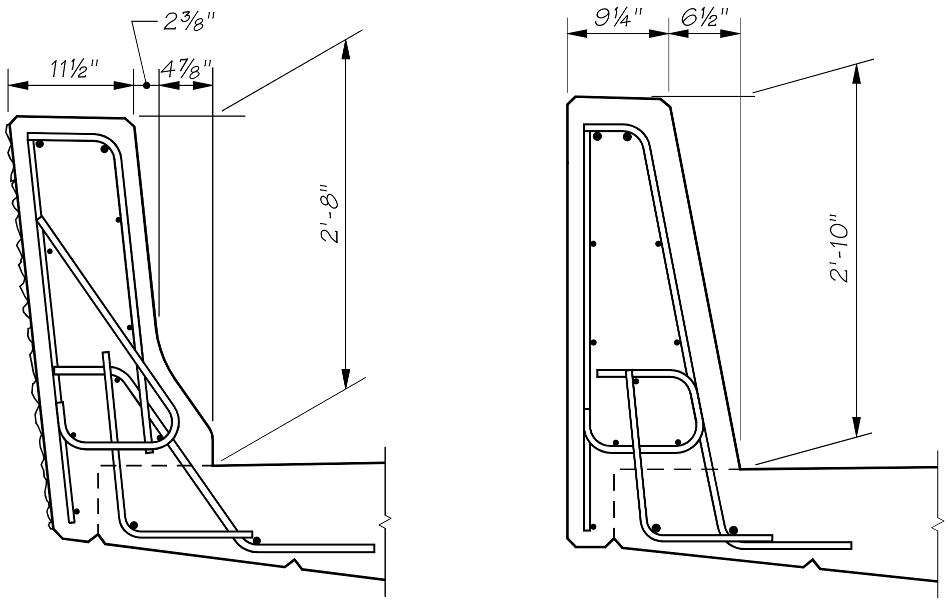


SL-1 Weak Post Texas T-411

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.

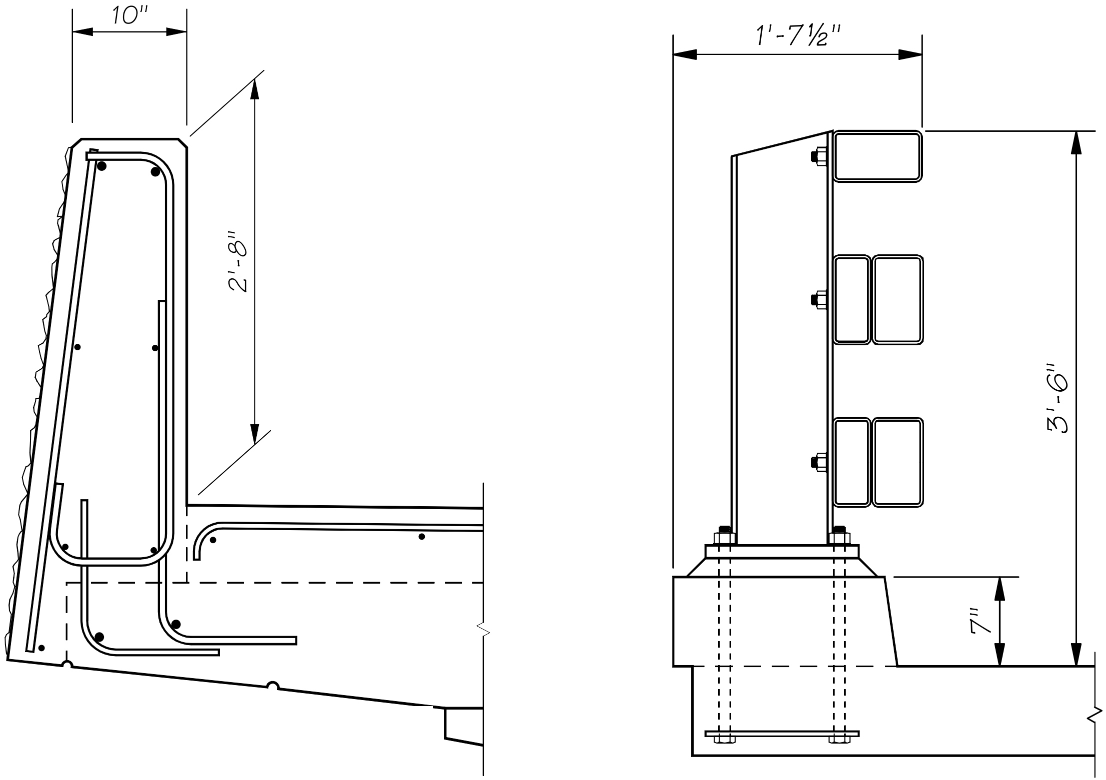


32″ F-Shape **34″ Single Slope**

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 www.oregon.gov/ODOT/HWY/ENGSERVICES/Pages/bridge_drawings.aspx.

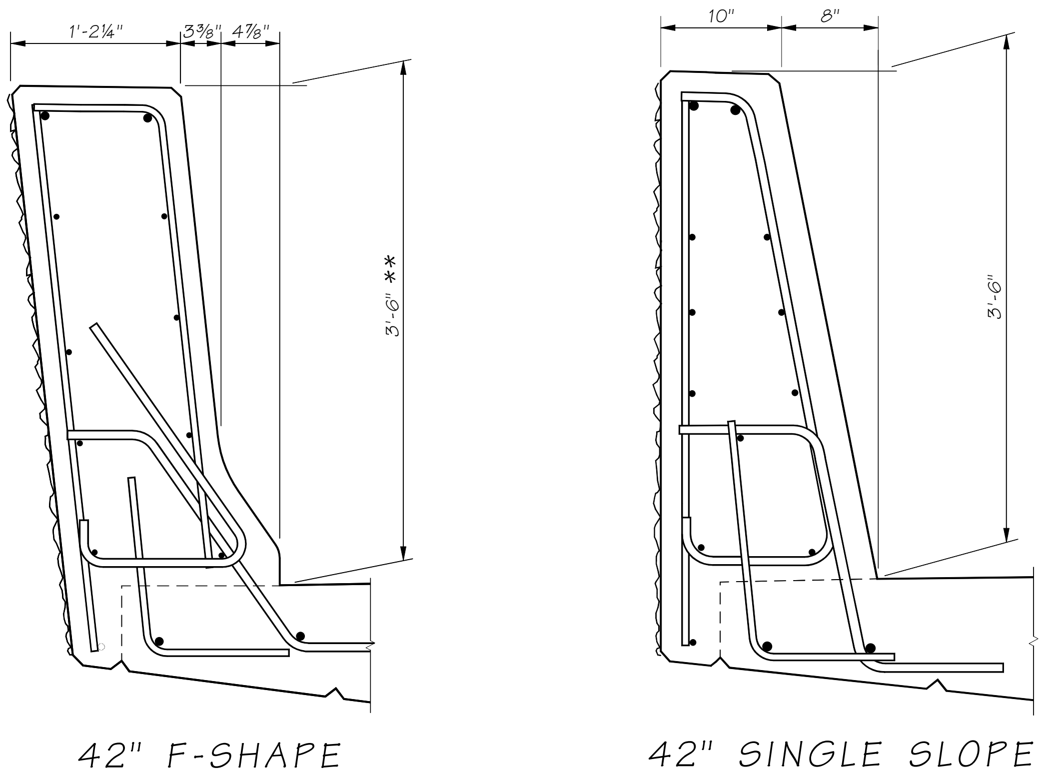


32″ Vertical **Oregon 3-Tube**

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 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 - $M_s = \frac{R_w \cdot H}{L_C + 2 \cdot H}$

and for an end barrier segment - $M_s = \frac{R_w \cdot H}{L_C + H}$

However, M_s need not be taken greater than M_c at the base. T shall be taken as:

for an interior barrier segment - $T = \frac{R_w}{L_C + 2 \cdot H}$

and for an end barrier segment - $T = \frac{R_w}{L_C + H}$

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 end section "Design F" detail or the w-beam end section "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)		Single Slope 34 in. (TL-4)		Type F 42 in. (TL-4)		Single Slope 42 in. (TL-4)		Type F 42 in. (TL-5)		Single Slope 42 in. (TL-5)	
		Interior	End*	Interior	End*	Interior	End*	Interior	End*	Interior	End*	Interior	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 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 *Standard Plans* C-70.10 and C-80.10 for details.

   If the difference in grade elevations is 4′-0″ or less, then the barrier shall be designed as a rigid system in accordance with AASHTO LRFD *Bridge Design Specifications* with the following requirements:

   1. All applicable loads shall be applied in accordance to AASHTO LRFD Section 3. The structural capacity of the differential grade barrier and supporting elements shall be designed for the required Test Level vehicle impact design forces in accordance with AASHTO LRFD Sections 5 and 13. Any section along the differential grade barrier and supporting elements shall not fail in shear, bending, or torsion when the barrier is subjected to the TL impact forces.
   2. For soil loads without vehicle impact loads, the barrier shall be designed as a retaining wall (barrier weight resists overturning and sliding). Passive soil resistance may be considered with concurrence by the geotechnical engineer.
   3. Vehicle impact loads shall be applied to the top of barrier on the side of the barrier retaining soil.
   4. For soil loads with vehicle impact loads, the AASHTO LRFD Extreme Event loading for vehicular collision shall also be analyzed. Equivalent Static Load (ESL) per NCHRP Report 663 may be applied as the transverse vehicle impact load for evaluating sliding, bearing, and overturning only. For TL-3 and TL-4 barrier systems, the ESL shall be 10 kips and for TL-5, the ESL shall be 23 kips.
   5. The length of the barrier required for stability shall be no more than 10 times the overall height limited to the length between barrier expansion joints (or one precast section). The barrier shall act as a rigid body behavior and shall be continuous throughout this length of barrier. Any coupling between adjacent barrier sections or friction that may exist between free edges of barrier and the surrounding soil shall be neglected.
   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 barrier 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.
   9. Barrier bottom shall be embedded a minimum 6" below roadway. Roadway subgrade and ballast shall be extended below whole width of differential grade barrier.

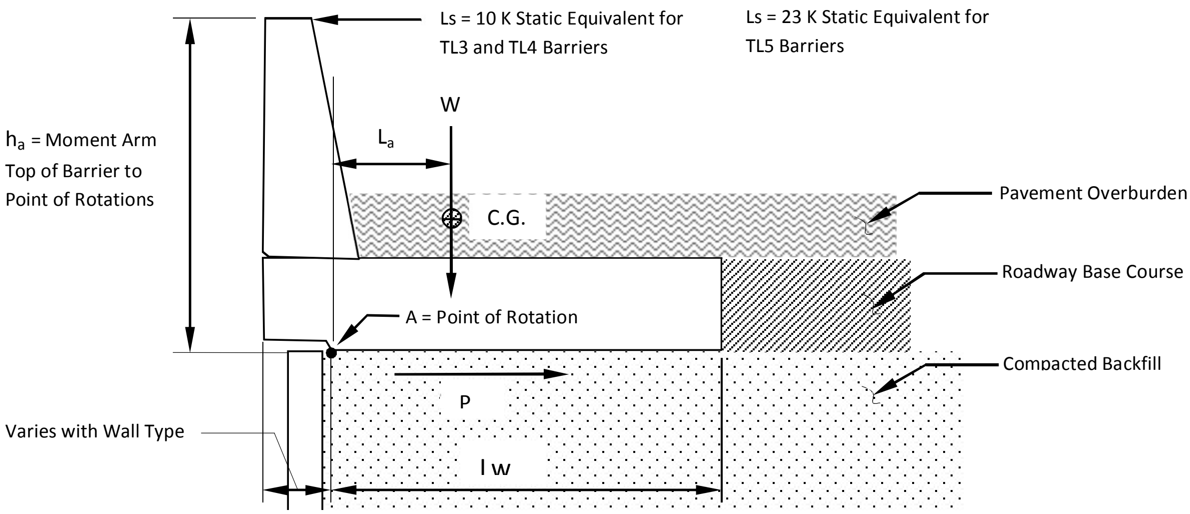
   Median traffic barriers with a grade difference greater than 4′-0″ shall be designed as standard plan retaining walls with a traffic barrier at the top and a barrier shape at the cut face.

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 resistance 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 loads at appropriate Test Level (TL-3, TL-4, TL-5) applied to the top of the barrier in accordance with Sections 5 and 13 of *AASHTO LRFD Bridge Design Specifications*. Any section along the moment slab shall not fail in shear, bending, or torsion when the barrier is subjected to the design impact loads. 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.

   The moment slab shall be designed as a deck supporting barrier in accordance to AASHTO LRFD A13.4.2 as modified by BDM 10.2.4.A. The moment slab reinforcement shall be designed to resist combined forces from the moment M_S (kip-ft/ft) and the tensile force T (kip/ft). M_S and T are determined from the lesser of the ultimate transverse resistance of barrier R_W (kip) and 120% of transverse vehicle impact force F_T (kip). M_S is not to be exceeded by the ultimate strength of barrier atits base M_C (kip-ft/ft).

2. **Global Stability** – Bearing stress, 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 continuous moment slab through rigid body behavior. Barrier shall also be continuous or have shear connections between barrier sections if precast throughout this length of moment slab. 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 limited to the length between moment slab joints.

   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
   φr = 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 shall 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 *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 *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 *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 (*Standard Plan* K-80.30) shall be anchored to the bridge deck as shown in *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, 17^{th} 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 (TL-2) 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 inAppendix 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.

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			
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 MONOTUBE SIGN STRUCTURES

GENERAL NOTES

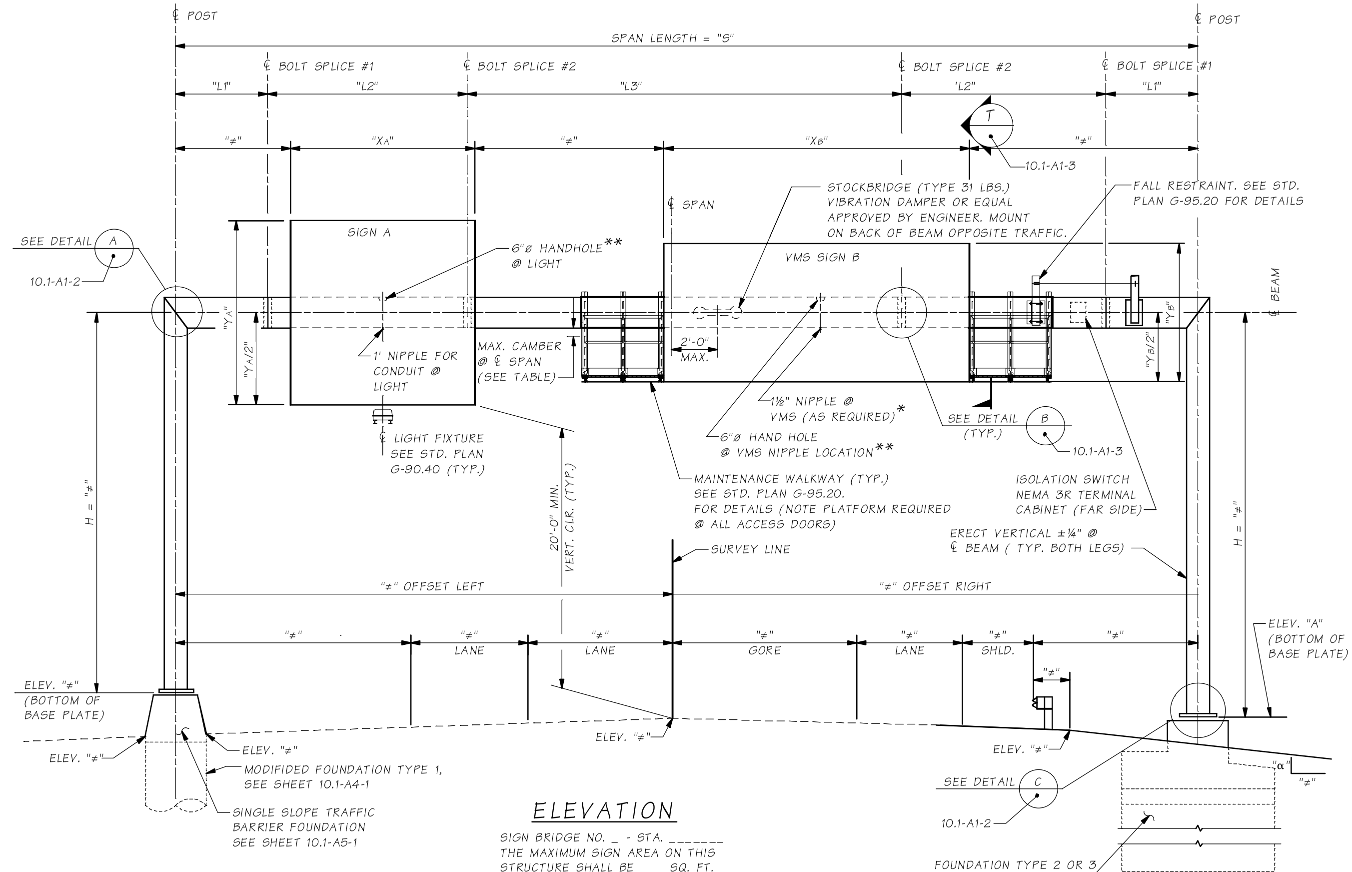
BRIDGE SHEET NO.

SHEET OF SHEETS

SR FILE NO. Job SHEET 1

10.1-A0-1

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.

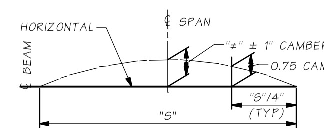
existing roadway lines dashed and new roadway is solid.

"≠" 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.	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

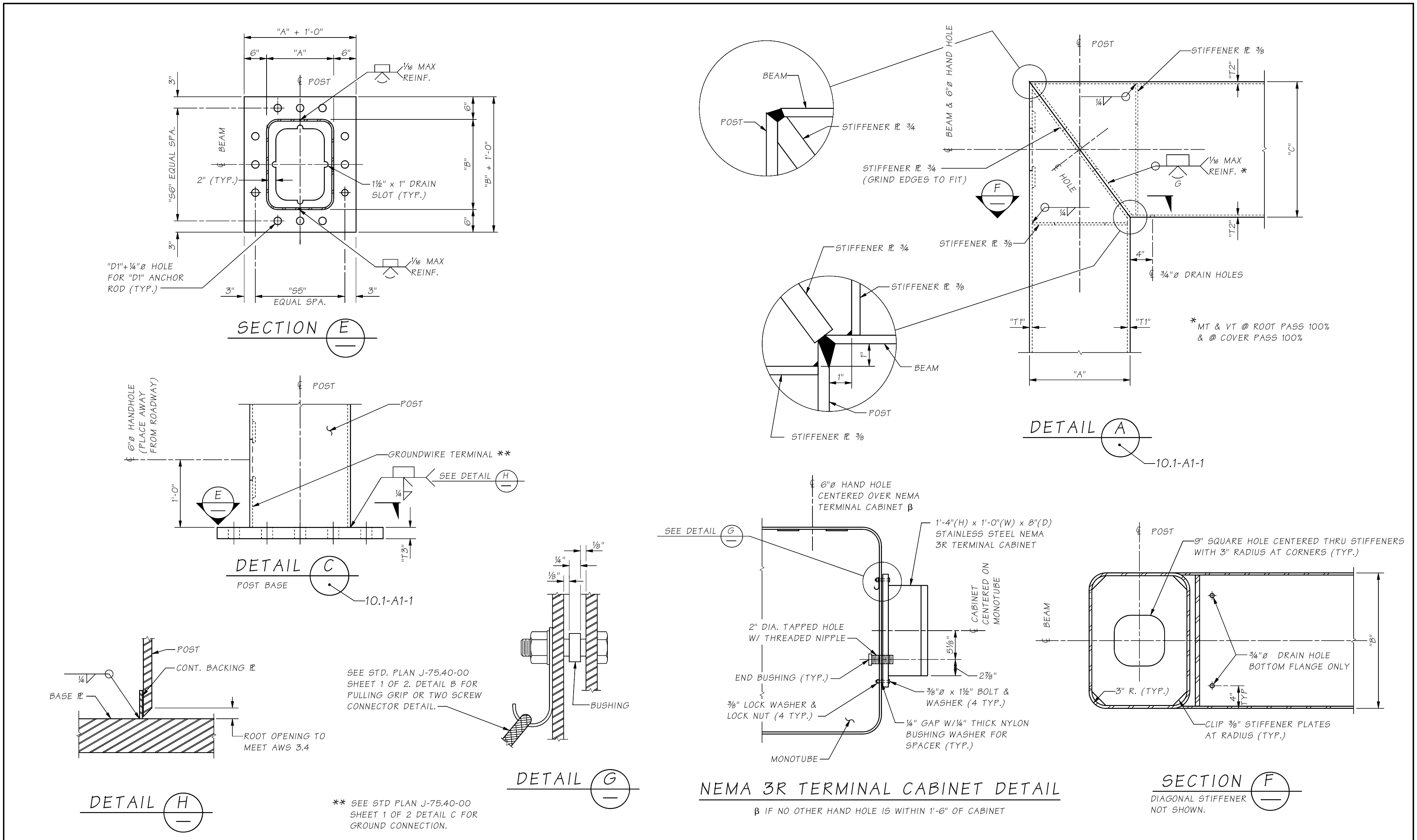
STANDARD MONOTUBE SIGN STRUCTURES

MONOTUBE SIGN BRIDGE LAYOUT

BRIDGE SHEET NO. / SHEET / OF / SHEETS

SHEET
JOB NO.
SR

Wed Jul 18 10:12:45 2012



Bridge Design Engr.	M:\STANDARDS\Sign Bridges\MT_SIGN_BRIDGE_STRUCT_DETAILS_1.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

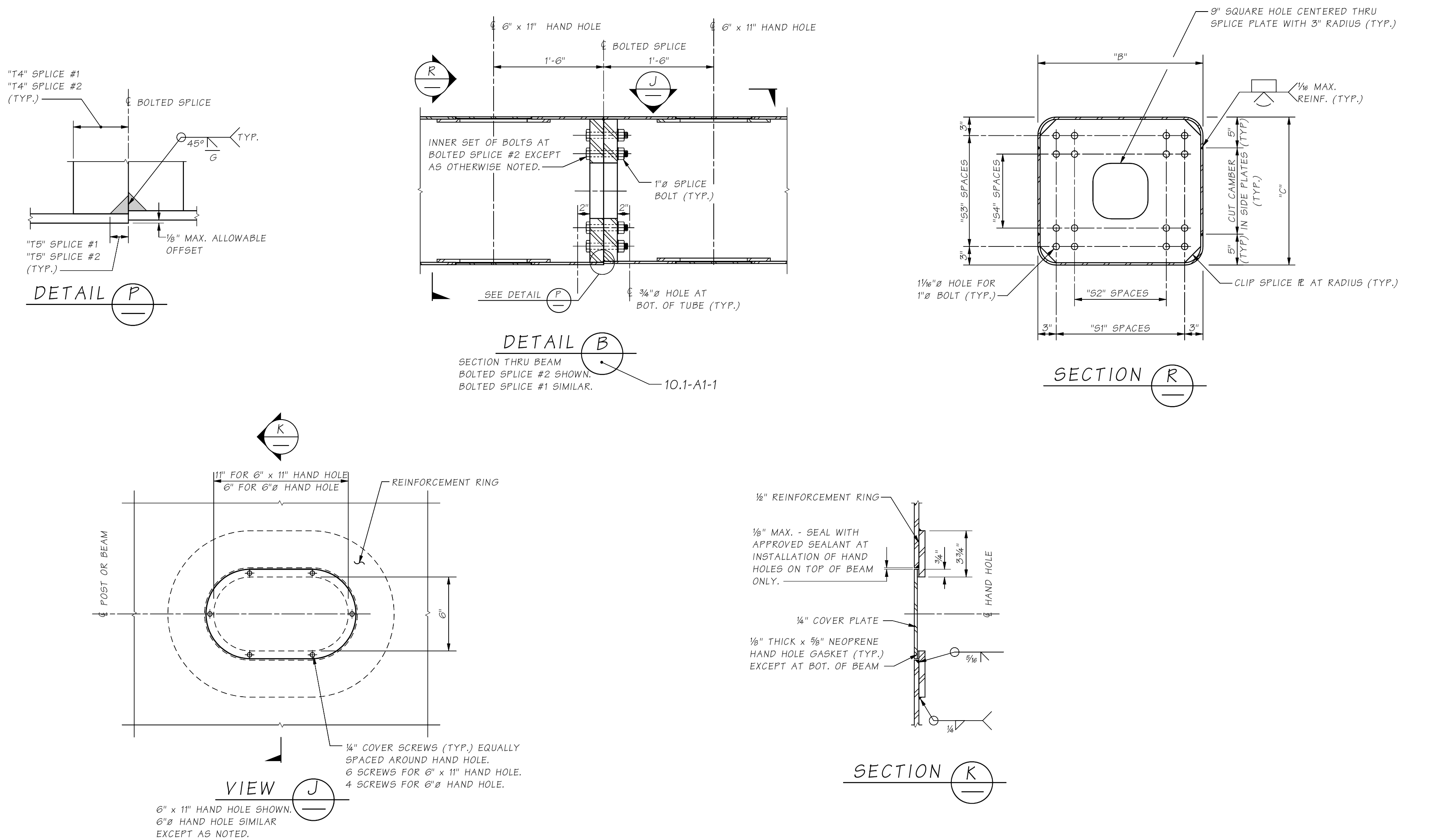
STANDARD SIGN BRIDGES

MONOTUBE SIGN BRIDGE STRUCTURAL DETAILS 1

BRIDGE SHEET NO. SHEET OF SHEETS

Fri Sep 03 09:42:26 2010

JOB NO. SR SHEET



Bridge Design Engr.	M:\STANDARDS\Sign Bridges\MT_SIGN_BRIDGE_STRUCT_DETAILS_2.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				

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Washington State Department of Transportation

STANDARD SIGN BRIDGES

MONOTUBE SIGN BRIDGE STRUCTURAL DETAILS 2

BRIDGE SHEET NO.
SHEET OF SHEETS

Fri Sep 03 09:42:28 2010

notes to the designer

delete all lights and electrical details, callouts & notes if no electrical items required on the sign structure.

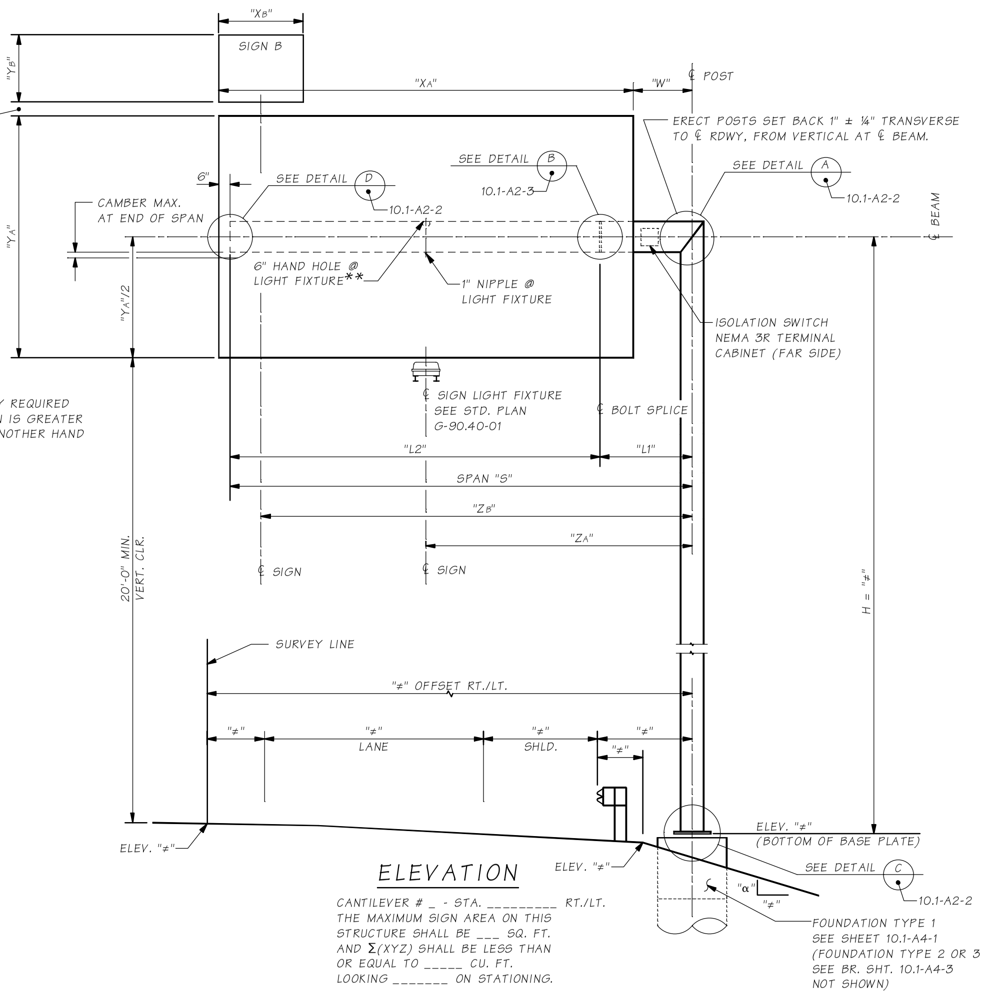
existing roadway lines dashed and new roadway is solid.

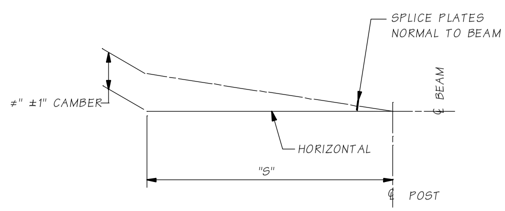
always show sign in front of structure.

remove notes that are not applicable.

"≠" is shown where dimensions are required.

** 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_CANTILEVER_LO.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

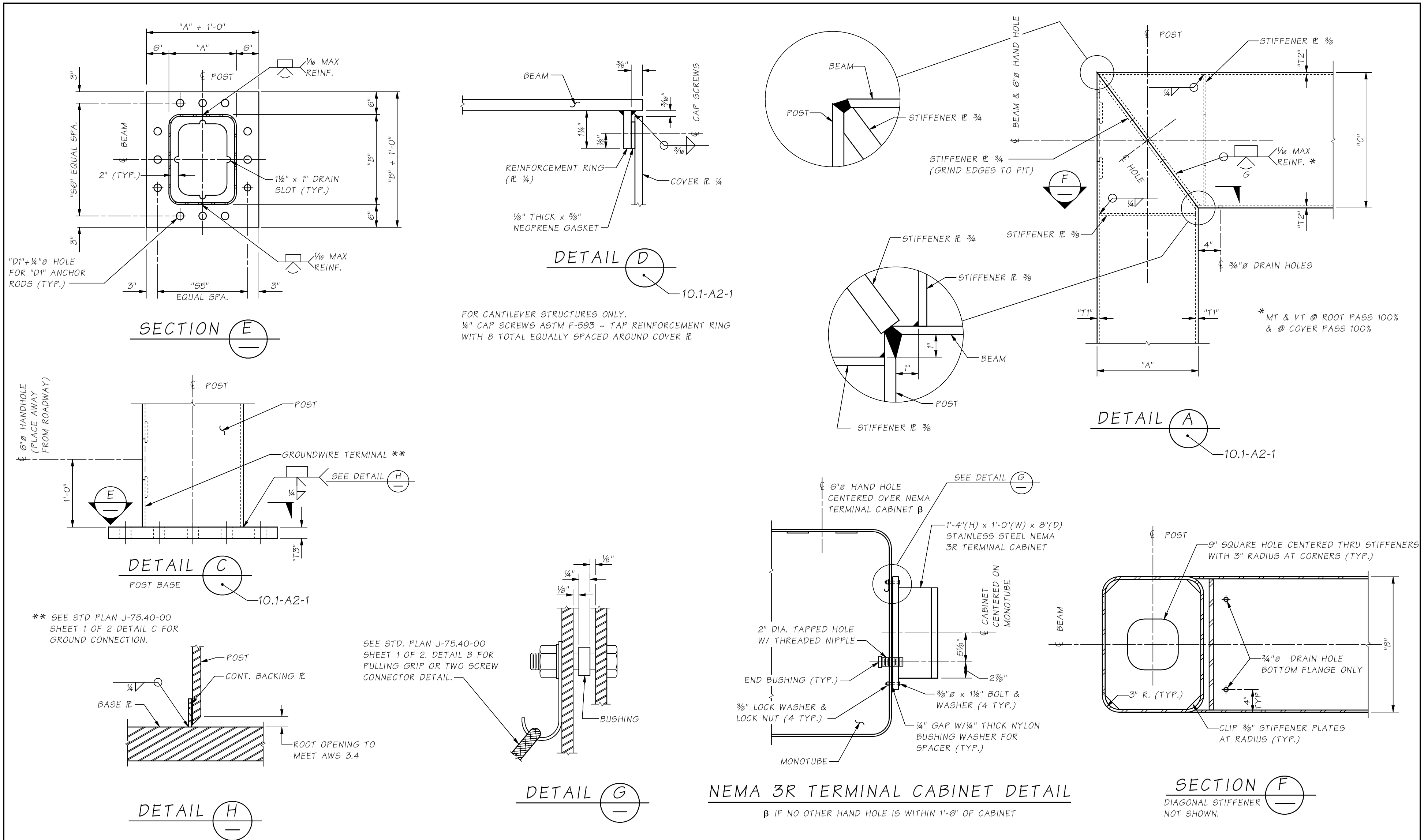
STANDARD MONOTUBE SIGN STRUCTURES

MONOTUBE CANTILEVER LAYOUT

BRIDGE SHEET NO.

SHEET OF SHEETS

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		JOB NUMBER				
Detailed By						
Bridge Projects Engr.						
Prelim. Plan By						
Architect/Specialist	DATE	REVISION	BY	APP'D		

BRIDGE AND STRUCTURES OFFICE

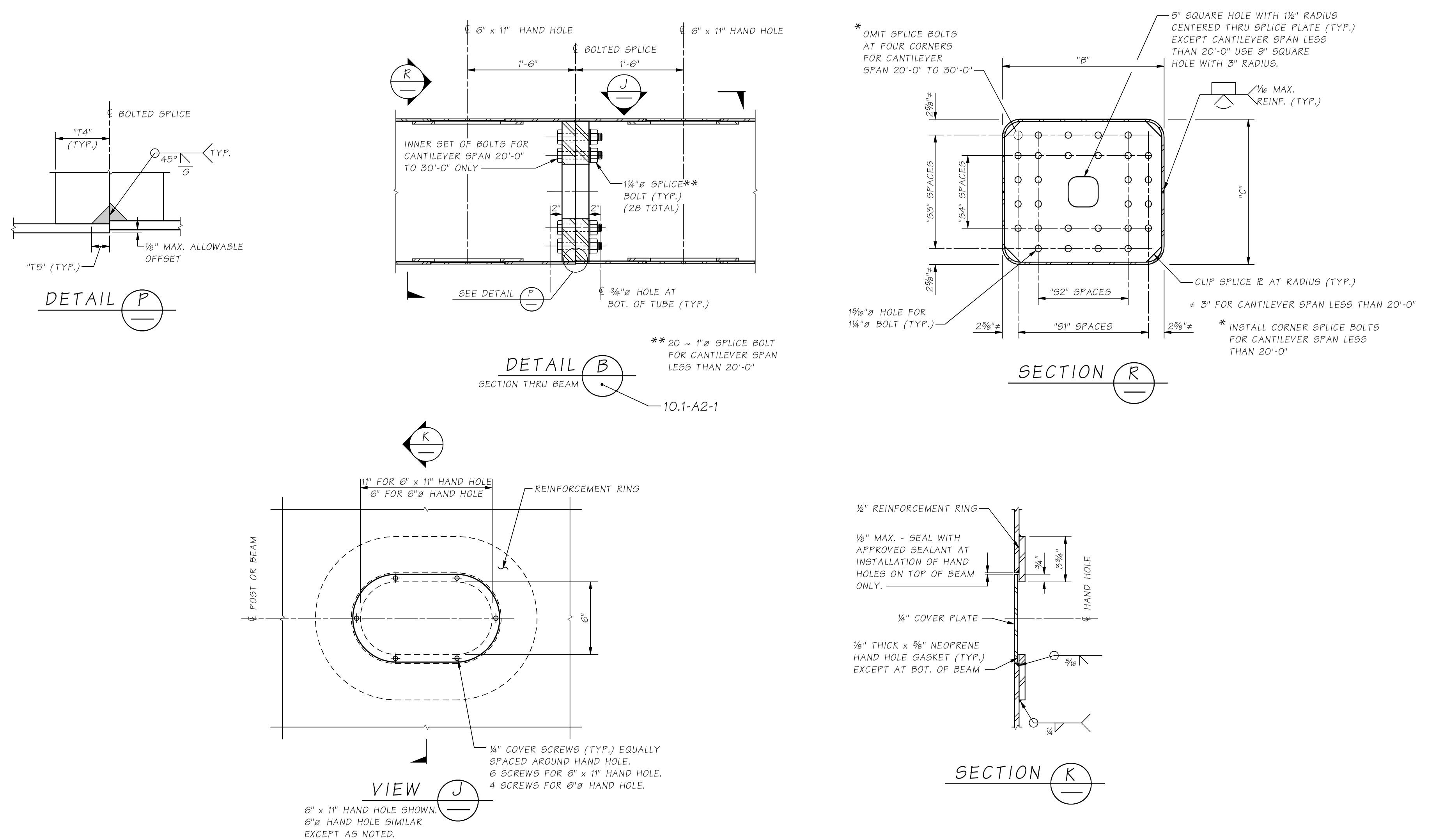
Washington State Department of Transportation

STANDARD SIGN BRIDGES

MONOTUBE CANTILEVER STRUCTURAL DETAILS 1

BRIDGE SHEET NO.
SHEET OF SHEETS

Fri Sep 03 09:41:56 2010



Bridge Design Engr.	M:\STANDARDS\Sign Bridges\MT_CANT_STRUCT_DETAILS_2.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 2

BRIDGE SHEET NO.

SHEET OF SHEETS

Fri Sep 03 09:41:59 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.

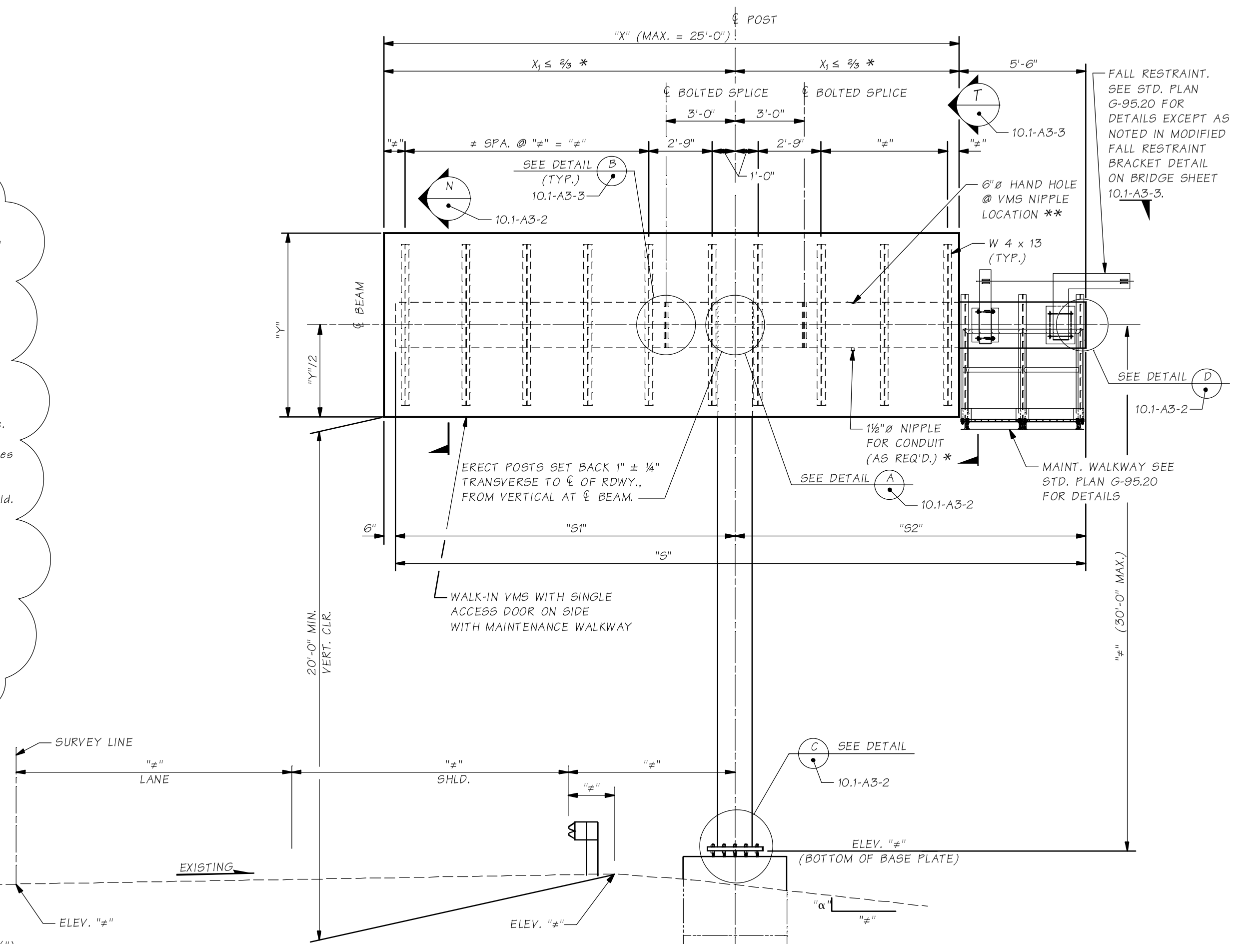
remove notes that are not applicable.

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.



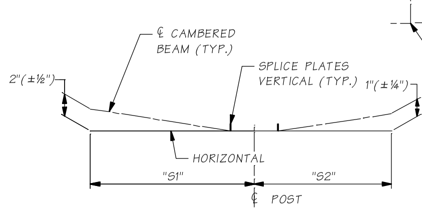
ELEVATION

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.



BEAM CAMBER DIAGRAM

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

Wed Jul 18 10:12:41 2012

REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
10	WASH.			
JOB NUMBER				

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

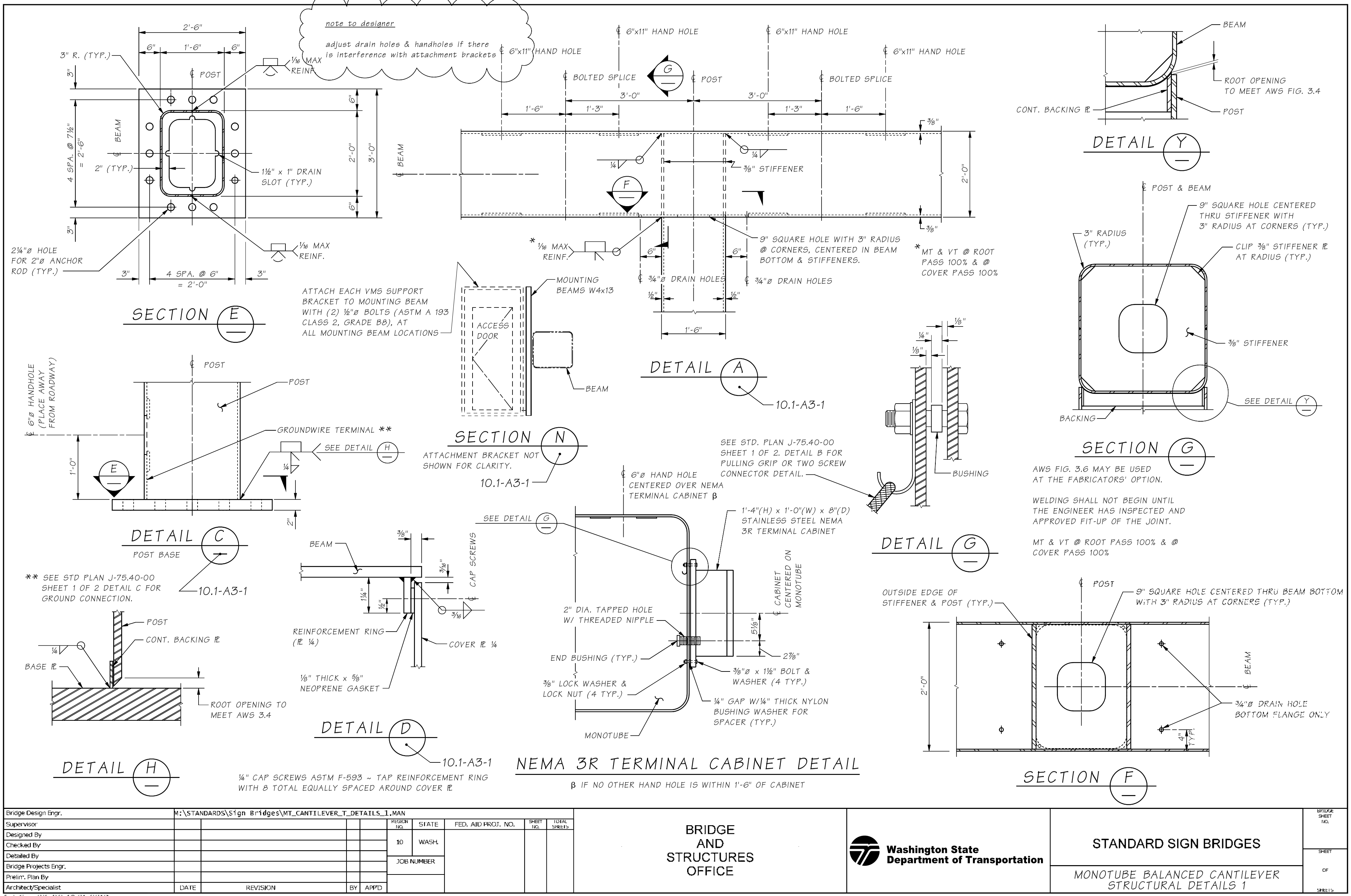
STANDARD MONOTUBE SIGN STRUCTURES

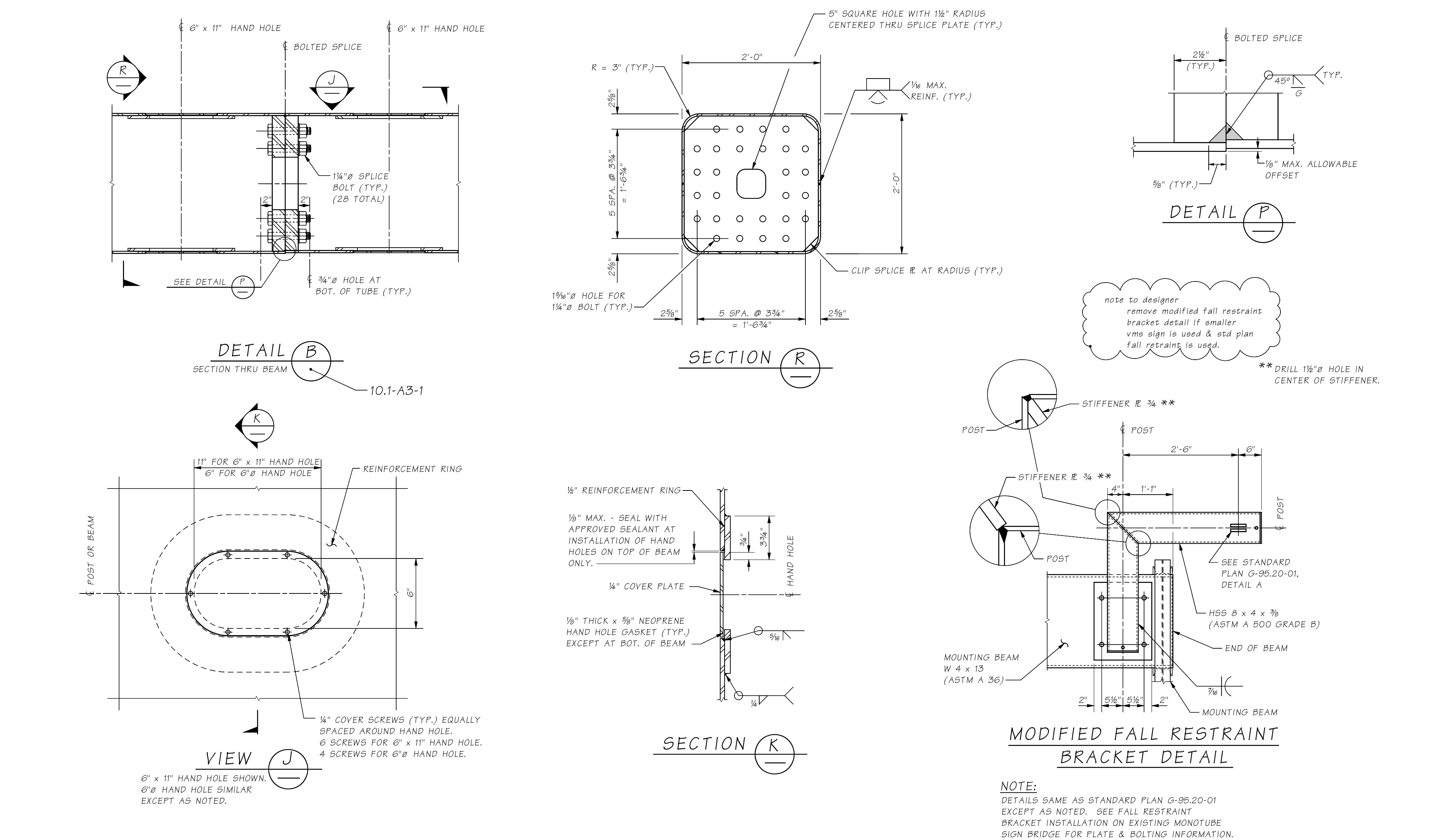
MONOTUBE BALANCED CANTILEVER LAYOUT

BRIDGE SHEET NO.

SHEET OF SHEETS

SR
FILE NO.
SHEET


note to designer
adjust drain holes & handholes if there is interference with attachment brackets
SECTION E
2'-6"
6"
1'-6"
3" R. (TYP.)
℄ POST
℄ BEAM
4 SPA. @ 7½" = 2'-6"
2" (TYP.)
1½" x 1" DRAIN SLOT (TYP.)
1/16 MAX REINF.
2¼"ø HOLE FOR 2"ø ANCHOR ROD (TYP.)
4 SPA. @ 6" = 2'-0"
2'-0"
3'-0"
3"
℄ 6"x11" HAND HOLE
℄ BOLTED SPLICE
℄ POST
3'-0"
1'-6"
1'-3"
3/8"
3/8" STIFFENER
9" SQUARE HOLE WITH 3" RADIUS @ CORNERS, CENTERED IN BEAM BOTTOM & STIFFENERS.
*MT & VT @ ROOT PASS 100% & @ COVER PASS 100%
℄ ¾"ø DRAIN HOLES
½"
1'-6"
DETAIL A
10.1-A3-1
ATTACH EACH VMS SUPPORT BRACKET TO MOUNTING BEAM WITH (2) ½"ø BOLTS (ASTM A 193 CLASS 2, GRADE B8), AT ALL MOUNTING BEAM LOCATIONS
MOUNTING BEAMS W4x13
ACCESS DOOR
BEAM
SECTION N
ATTACHMENT BRACKET NOT SHOWN FOR CLARITY.
10.1-A3-1
℄ 6"ø HANDHOLE (PLACE AWAY FROM ROADWAY)
POST
GROUNDWIRE TERMINAL **
SEE DETAIL H
1'-0"
2"
DETAIL C
POST BASE
10.1-A3-1
** SEE STD PLAN J-75.40-00 SHEET 1 OF 2 DETAIL C FOR GROUND CONNECTION.
POST
CONT. BACKING ℄
BASE ℄
ROOT OPENING TO MEET AWS 3.4
DETAIL H
BEAM
3/8"
CAP SCREWS
REINFORCEMENT RING (℄ ¼)
COVER ℄ ¼
⅛" THICK x ⅝" NEOPRENE GASKET
DETAIL D
10.1-A3-1
¼" CAP SCREWS ASTM F-593 ~ TAP REINFORCEMENT RING WITH 8 TOTAL EQUALLY SPACED AROUND COVER ℄
SEE STD. PLAN J-75.40-00 SHEET 1 OF 2. DETAIL B FOR PULLING GRIP OR TWO SCREW CONNECTOR DETAIL.
℄ 6"ø HAND HOLE CENTERED OVER NEMA TERMINAL CABINET β
SEE DETAIL G
1'-4"(H) x 1'-0"(W) x 8"(D) STAINLESS STEEL NEMA 3R TERMINAL CABINET
℄ CABINET CENTERED ON MONOTUBE
2" DIA. TAPPED HOLE W/ THREADED NIPPLE
END BUSHING (TYP.)
⅜" LOCK WASHER & LOCK NUT (4 TYP.)
MONOTUBE
5⅛"
2⅞"
⅜"ø x 1½" BOLT & WASHER (4 TYP.)
¼" GAP W/¼" THICK NYLON BUSHING WASHER FOR SPACER (TYP.)
NEMA 3R TERMINAL CABINET DETAIL
β IF NO OTHER HAND HOLE IS WITHIN 1'-6" OF CABINET
BUSHING
DETAIL G
BEAM
ROOT OPENING TO MEET AWS FIG. 3.4
CONT. BACKING ℄
POST
DETAIL Y
℄ POST & BEAM
9" SQUARE HOLE CENTERED THRU STIFFENER WITH 3" RADIUS AT CORNERS (TYP.)
3" RADIUS (TYP.)
CLIP ⅜" STIFFENER ℄ AT RADIUS (TYP.)
⅜" STIFFENER
SEE DETAIL Y
BACKING
SECTION G
AWS FIG. 3.6 MAY BE USED AT THE FABRICATORS' OPTION.
WELDING SHALL NOT BEGIN UNTIL THE ENGINEER HAS INSPECTED AND APPROVED FIT-UP OF THE JOINT.
MT & VT @ ROOT PASS 100% & @ COVER PASS 100%
℄ POST
OUTSIDE EDGE OF STIFFENER & POST (TYP.)
9" SQUARE HOLE CENTERED THRU BEAM BOTTOM WITH 3" RADIUS AT CORNERS (TYP.)
¾"ø DRAIN HOLE BOTTOM FLANGE ONLY
4" TYP.
SECTION F
Bridge Design Engr.
Supervisor
Designed By
Checked By
Detailed By
Bridge Projects Engr.
Prelim. Plan By
Architect/Specialist
M:\STANDARDS\Sign Bridges\MT_CANTILEVER_T_DETAILS_1.MAN
REGION NO.
STATE
FED. AID PROJ. NO.
SHEET NO.
TOTAL SHEETS
10
WASH.
JOB NUMBER
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



Bridge Design Engr.	M:\STANDARDS\Sign Bridges\MT_CANTILEVER_T_DETAILS_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				

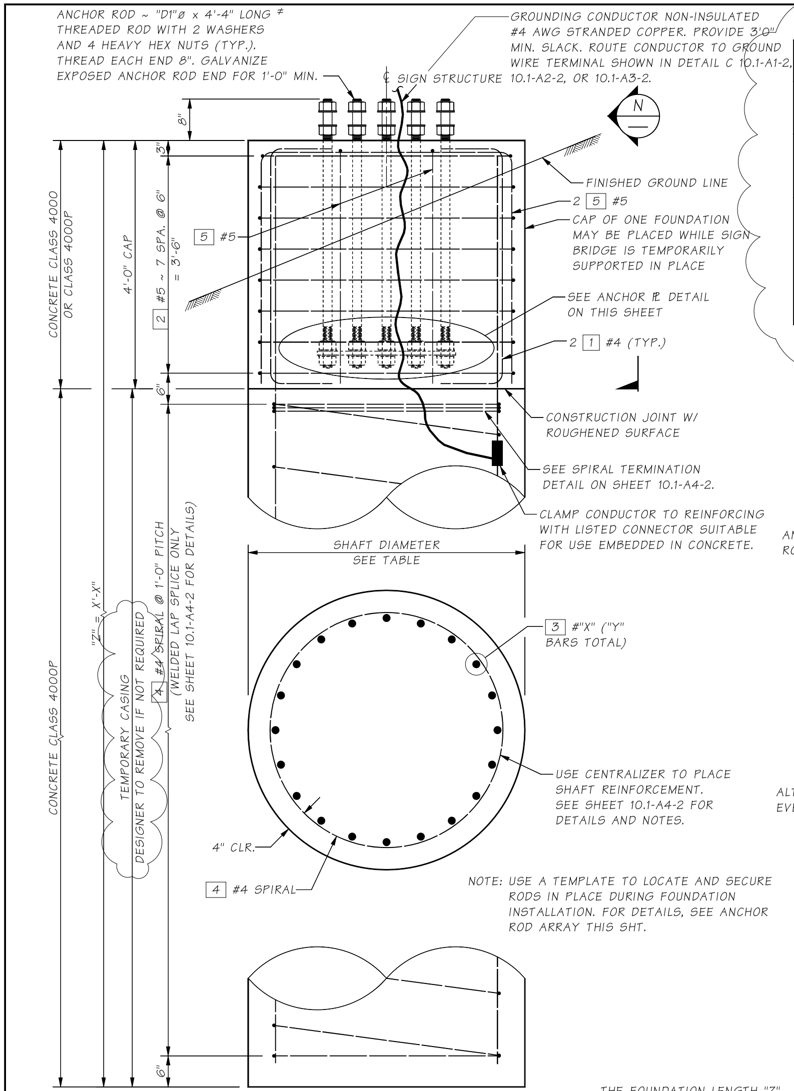
BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

STANDARD SIGN BRIDGES

MONOTUBE BALANCED CANTILEVER STRUCTURAL DETAILS 2

BRIDGE SHEET NO.
SHEET OF SHEETS



ELEVATION

NOTE: USE A TEMPLATE TO LOCATE AND SECURE RODS IN PLACE DURING FOUNDATION INSTALLATION. FOR DETAILS, SEE ANCHOR ROD ARRAY THIS SHT.

THE FOUNDATION LENGTH "Z" IS BASED ON AN ALLOWABLE LATERAL BEARING PRESSURE OF _______.

TYPE 1 FOUNDATION TABLE

	SPAN LENGTH ("S")	SHAFT DIAMETER	DEPTH ("Z")	FOUNDATION REINFORCEMENT SIZE ("X")	FOUNDATION REINFORCEMENT NUMBER ("Y")
CANTILEVER SIGN	LESS THAN 20'-0"	4'-6"	12'-6"	9	12
	20'-0" TO 30'-0"	4'-6"	15'-0"	9	16
SIGN BRIDGE	LESS THAN 60'-0"	4'-6"	15'-6"	9	18
	60'-0" TO 90'-0"	4'-6"	19'-0"	9	20
	90'-0" TO 120'-0"	5'-0"	21'-9"	11	15 BUNDLED PAIRS
	120'-0" TO 150'-0"	5'-0"	23'-6"	11	17 BUNDLED PAIRS

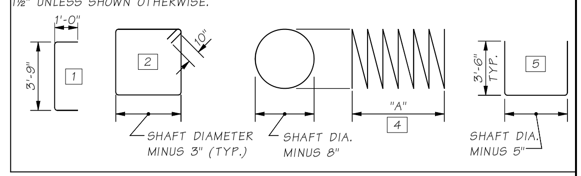
SHAFT DEPTH "Z" IS BASED ON ALLOWABLE LATERAL BEARING PRESSURE OF 2500 PSF OR MORE.

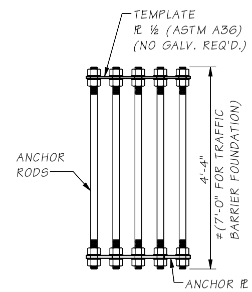
BENDING DIAGRAM

MARK	LOCATION	SIZE	NO. REQ'D.	LENGTH	
1	CAP VERTICAL	4	4	5'-6½"	
2	CAP HOOPS	5	8	"A"	
3	SHAFT VERTICAL	"X"	"Y"	"Z"-6"	STR.
4	SHAFT SPIRAL	4	1	"A"	
5	CAP TOP	5	4	"A"	

NOTE: CONCRETE COVER SHALL BE 1½" UNLESS SHOWN OTHERWISE.

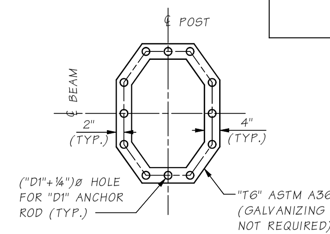
"A" ~ DETERMINE FROM PLANS





ANCHOR ROD ARRAY

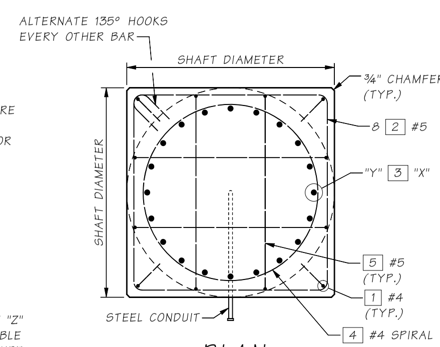
SHIP THE ANCHOR ROD ARRAY ASSEMBLY AS A COMPLETE UNIT.



ANCHOR ℄ DETAIL

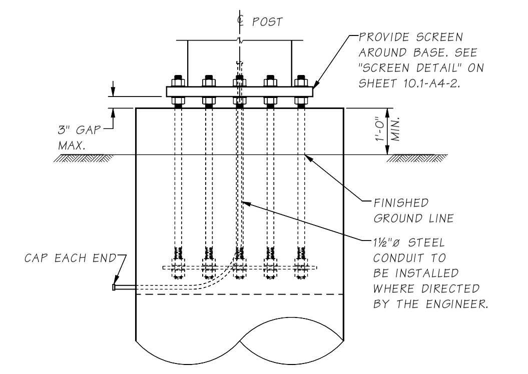
SEE SIGN STRUCTURE BASE PLATE FOR ANCHOR ROD PATTERN

note to designer
remove foundation table for contract plans.



PLAN

ANCHOR RODS NOT SHOWN.



VIEW N

Bridge Design Engr.	M:\STANDARDS\Sign Bridges\MT_SIGN_BRIDGE_FND_TYPE_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

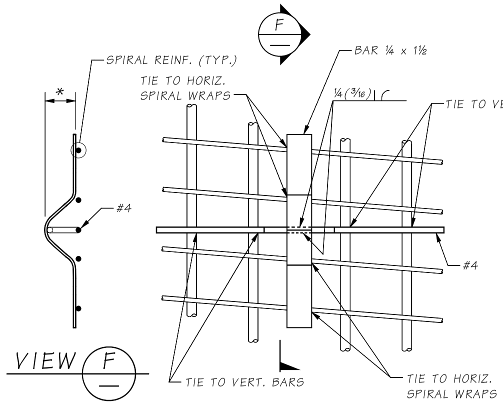
Washington State Department of Transportation

STANDARD SIGN BRIDGES

MONOTUBE SIGN STRUCTURES FOUNDATION TYPE 1 - SHEET 1 OF 2

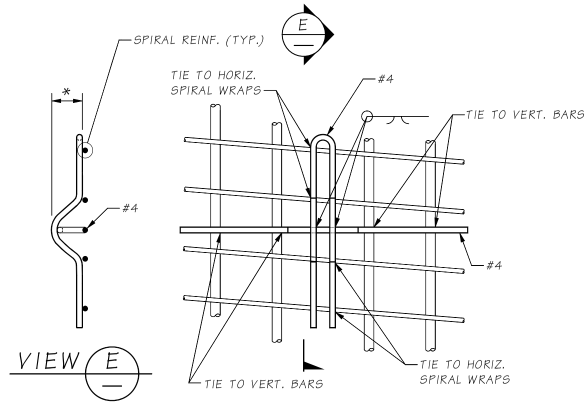
BRIDGE SHEET NO.
SHEET OF SHEETS

Fri Sep 03 09:42:13 2010



CENTRALIZER DETAIL

EPOXY COAT CENTRALIZER OR
PAINT WITH INORGANIC ZINC
AFTER FABRICATION (OPTION 2)



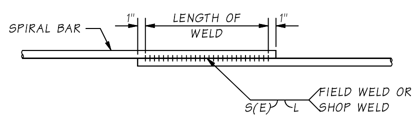
CENTRALIZER DETAIL

EPOXY COAT CENTRALIZER OR
PAINT WITH INORGANIC ZINC
AFTER FABRICATION (OPTION 1)

CENTRALIZER DETAIL NOTES:
EACH LEG SHALL BE TIED TO ONE VERTICAL BARS & TWO SPIRALS.

SEE SPECIAL PROVISIONS FOR SPACING REQUIREMENTS.

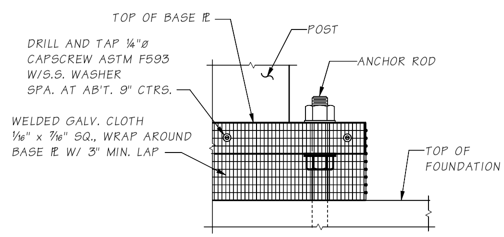
* CONCRETE COVER - MINUS ½"



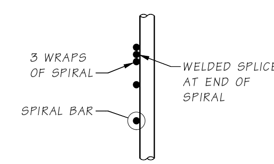
WELDED LAP SPLICE DETAIL

WELDING SHALL MEET THE REQUIREMENTS
OF STD. SPEC. 6-02.3(24)E
FOR WELD DIMENSIONS, SEE TABLE BELOW.

DEFORMED BAR	WELD DIMENSIONS (IN.)		
	S	E	LENGTH (L)
#4	¼	⅛	4



SCREEN DETAIL



SPIRAL TERMINATION DETAIL

Bridge Design Engr.	M:\STANDARDS\Sign Bridges\MT_SIGN_BRIDGE_FND_TYPE_1_2.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

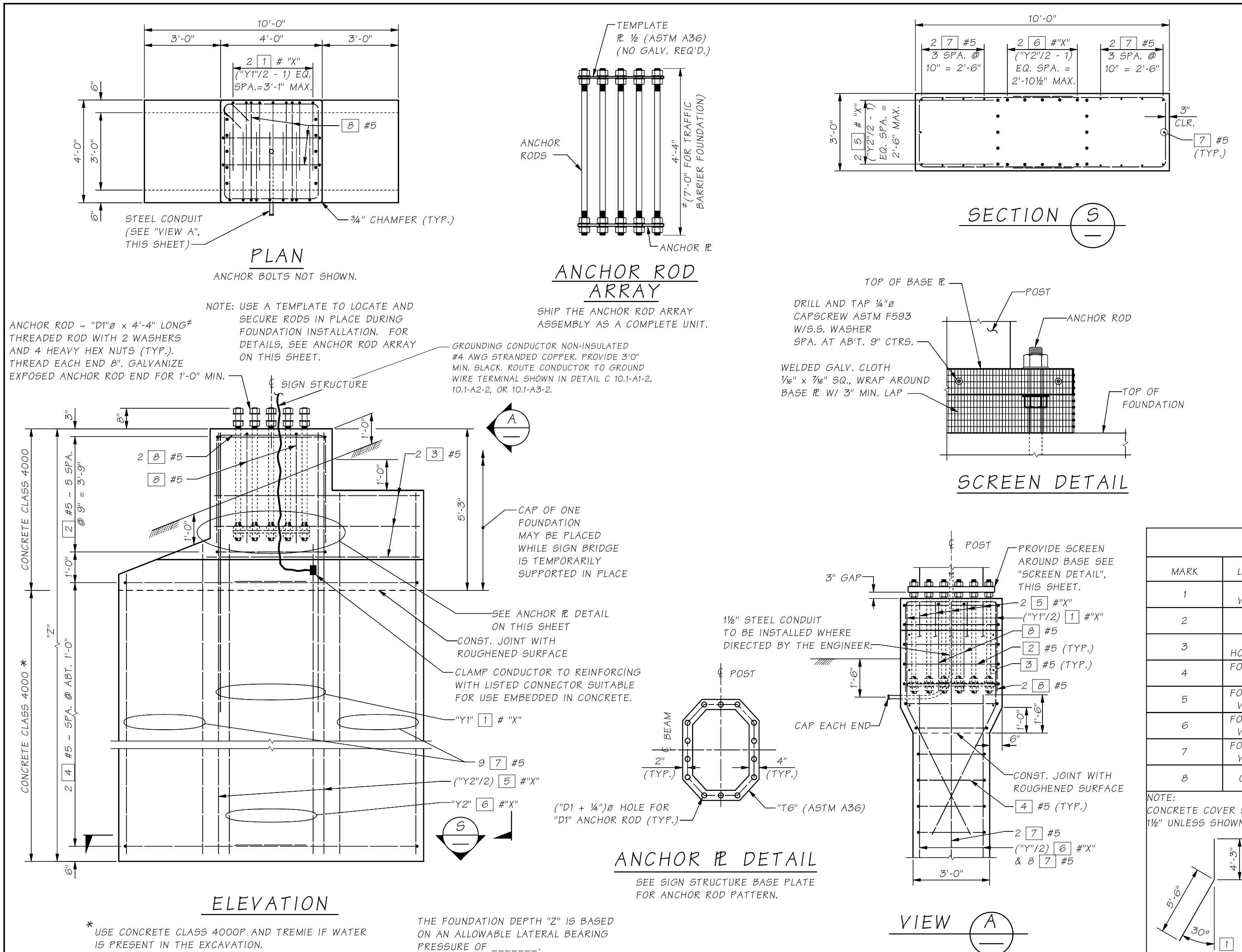
STANDARD SIGN BRIDGES

MONOTUBE SIGN STRUCTURES
FOUNDATION TYPE 1 - SHEET 2 OF 2

BRIDGE SHEET NO.
SHEET OF SHEETS

Fri Sep 03 09:42:17 2010

SR JOB NO. SHEET



TYPE 2 FOUNDATION TABLE**

STRUCTURE TYPE	SPAN LENGTH ("S")	DEPTH ("Z")	REINFORCEMENT SIZE ("X")	SPACING ("Y1")	SPACING ("Y2")
CANTILEVER SIGN	LESS THAN 20'-0"	7'-3"	9	8	4
	20'-0" TO 30'-0"	9'-6"	9	12	8
SIGN BRIDGE	LESS THAN 60'-0"	9'-0"	9	12	8
	60'-0" TO 90'-0"	11'-0"	9	16	12
	90'-0" TO 120'-0"	12'-9"	11	14	10
	120'-0" TO 150'-0"	13'-6"	11	16	14

* SHAFT DEPTH "Z" IS BASED ON ALLOWABLE LATERAL BEARING PRESSURE OF 2500 PSF OR MORE.

TYPE 3 FOUNDATION TABLE**

STRUCTURE TYPE	SPAN LENGTH ("S")	** DEPTH ("Z")	REINFORCEMENT SIZE ("X")	SPACING ("Y1")	SPACING ("Y2")
CANTILEVER SIGN	LESS THAN 20'-0"	11'-0"	9	8	4
	20'-0" TO 30'-0"	13'-9"	9	12	8
SIGN BRIDGE	LESS THAN 60'-0"	12'-9"	9	12	8
	60'-0" TO 90'-0"	15'-6"	9	16	12
	90'-0" TO 120'-0"	17'-9"	11	14	10
	120'-0" TO 150'-0"	19'-6"	11	16	14

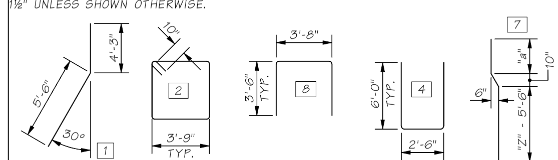
** SHAFT DEPTH "Z" IS BASED ON ALLOWABLE LATERAL BEARING PRESSURE BETWEEN 1500 PSF AND 2499 PSF.

BENDING DIAGRAM

MARK	LOCATION	SIZE	NO. REQ'D.	LENGTH	
1	CAP VERTICAL	"X"	"Y1"	9'-8"	
2	CAP HOOPS	5	6	16'-0"	
3	CAP HORIZONTAL	5	6	"A"	
4	FOUNDATION HOOPS	5	16	15'-5"	STR.
5	FOUNDATION VERTICAL	"X"	"Y2"	"Z" - 6"	
6	FOUNDATION VERTICAL	"X"	"Y2"	"Z" = 4'-0"	
7	FOUNDATION VERTICAL	5	18	"A"	STR.
8	CAP TOP	5	4	10'-4"	STR.

NOTE:
CONCRETE COVER SHALL BE 1½" UNLESS SHOWN OTHERWISE.

"A" ~ DETERMINE FROM PLANS



Bridge Design Engr.	M:\STANDARDS\Sign Bridges\MT_SIGN_BRIDGE_FND_TYPE_2AND3.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

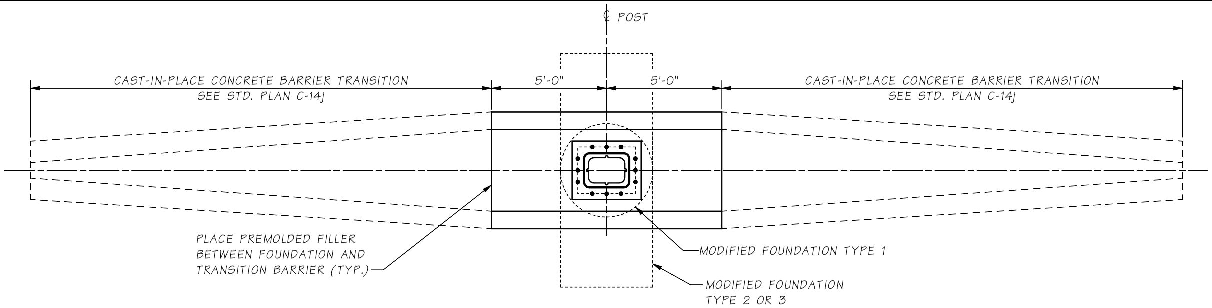
STANDARD SIGN BRIDGES

MONOTUBE SIGN STRUCTURES FOUNDATION TYPES 2 AND 3

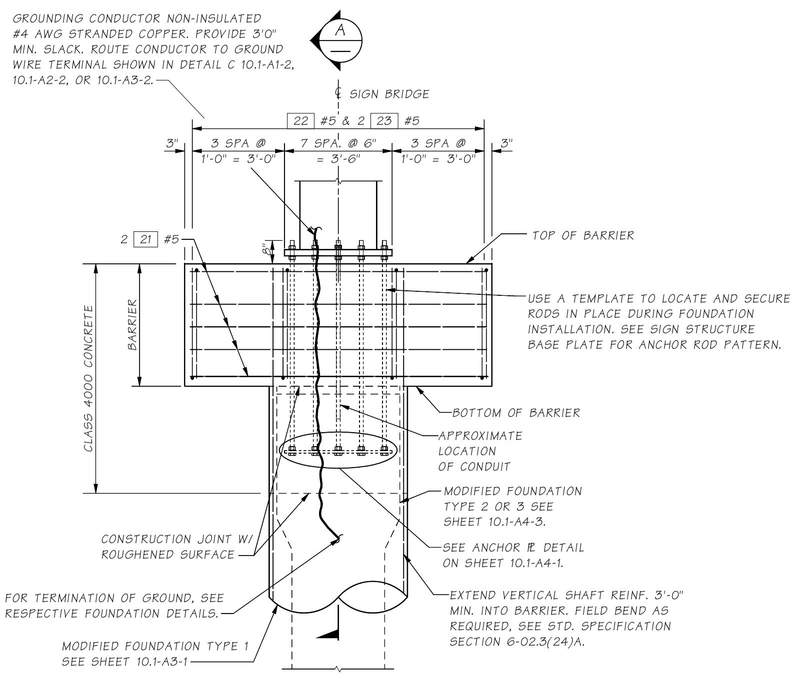
BRIDGE SHEET NO. | SHEET OF SHEETS

Fri Sep 03 09:42:18 2010

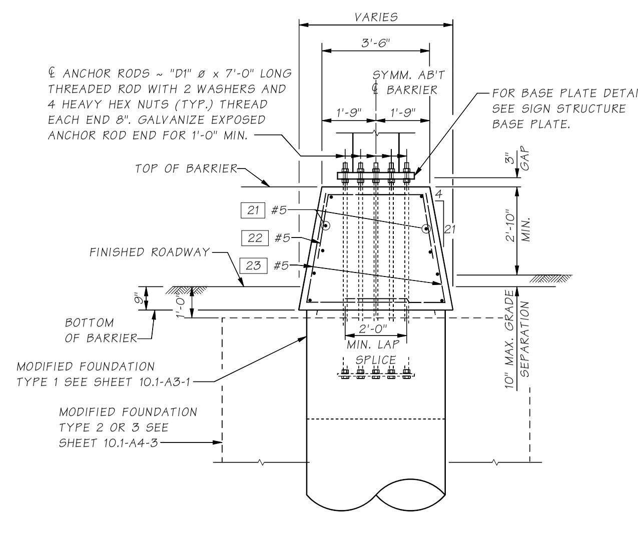
JOB NO. SR SHEET



PLAN
BARRIER TRANSITION
ANCHOR RODS NOT SHOWN.



ELEVATION

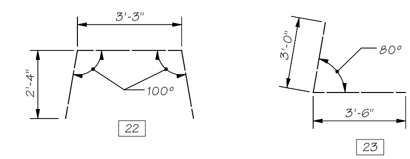


SECTION A

BENDING DIAGRAM

MARK	LOCATION	SIZE	NO. REQ'D	LENGTH	
21	MEDIAN BARRIER HORIZONTAL	5	10	9'-9"	STR.
22	MEDIAN BARRIER VERTICAL	5	14	7'-8"	
23	MEDIAN BARRIER VERTICAL	5	28	6'-4"	

NOTE:
CONCRETE COVER SHALL BE 1½" UNLESS SHOWN OTHERWISE.



Bridge Design Engr.	M:\STANDARDS\Sign Bridges\MT_DBL_FACED_SINGLE_SLOPE_FND.MAN		
Supervisor	REGION NO. 10	STATE WASH.	FED. AID PROJ. NO.
Designed By	JOB NUMBER		
Checked By			
Detailed By			
Bridge Projects Engr.			
Prelim. Plan By			
Architect/Specialist	DATE	REVISION	BY APP'D

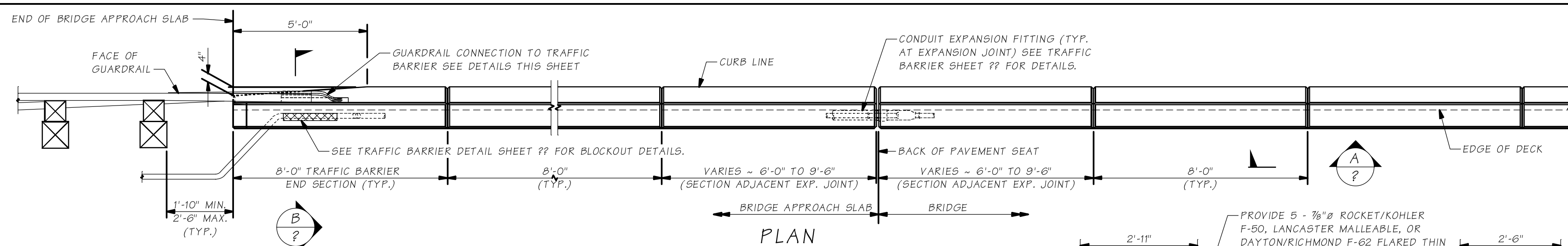
BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

STANDARD SIGN BRIDGES

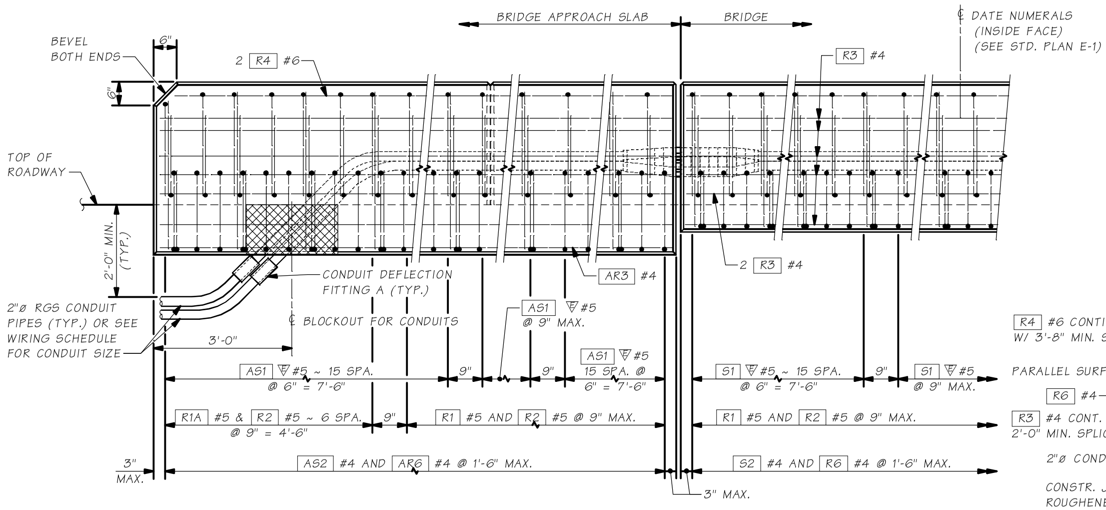
MONOTUBE SIGN STRUCTURE
SINGLE SLOPE TRAFFIC BARRIER FOUNDATION

Fri Sep 03 09:42:09 2010



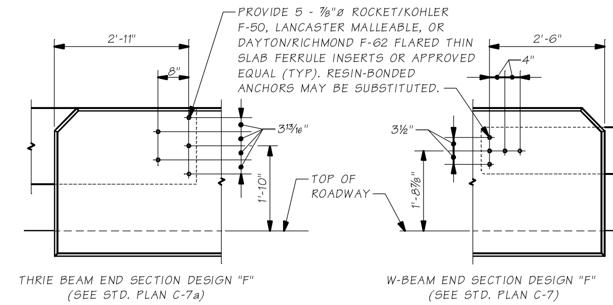
PLAN
TRAFFIC BARRIER

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.



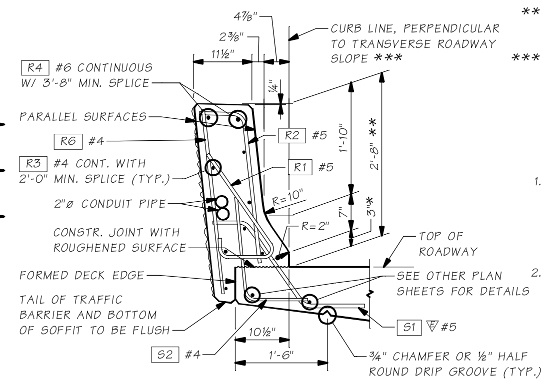
OUTSIDE ELEVATION
END OF TRAFFIC BARRIER

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\SHAPE F BARRIER SHT 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

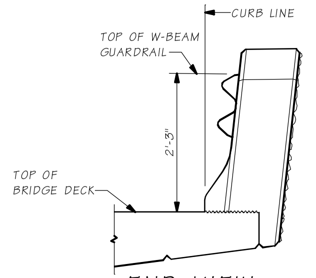
TRAFFIC BARRIER - SHAPE F
DETAILS 1 OF 3

BRIDGE SHEET NO. | SHEET OF SHEETS

Thu Dec 19 14:35:31 2013

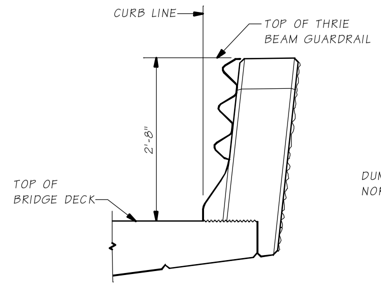
JOB NO. SR SHEET 1

Last revised on : 12/13/2012



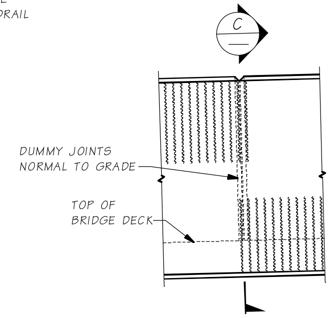
END VIEW

W-BEAM SHOWN WITH "D" CONNECTION OR "F" CONNECTION (SEE STD. PLAN C-5).

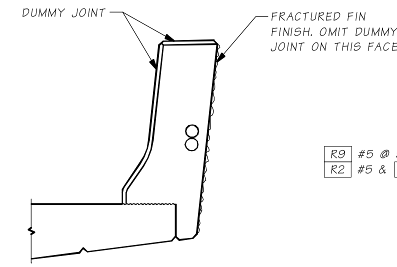


END VIEW

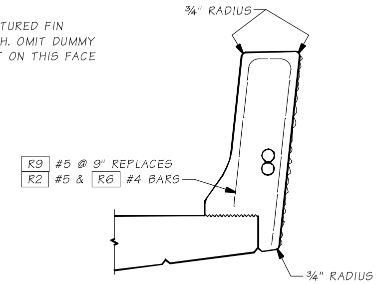
THRIE BEAM SHOWN WITH "D" CONNECTION OR "F" CONNECTION (SEE STD. PLAN C-5).



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.

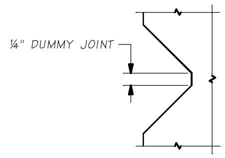
JUNCTION BOX LOCATIONS

STATION	OFFSET	"TS" OR "LT"

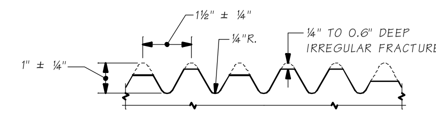
TS = TRAFFIC SYSTEM
LT = LIGHTING SYSTEM

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.



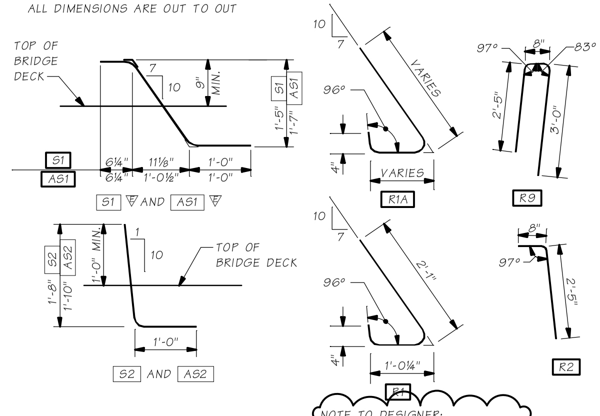
DUMMY JOINT DETAIL



FRACTURED FIN FINISH

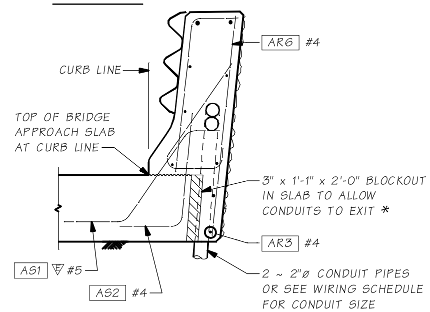
BENDING DIAGRAM

ALL DIMENSIONS ARE OUT TO OUT



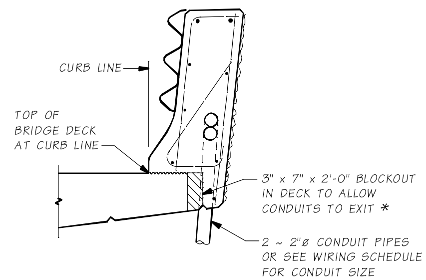
FOR W1 & W2 BARS SEE WINGWALL OR RETAINING WALL PLANS.

NOTE TO DESIGNER:
S1 AND S2 LENGTH BASED ON STANDARD DECK THICKNESS.



SECTION B ? APPROACH SLAB

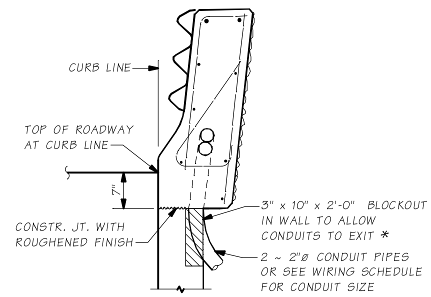
FOR DETAILS NOT SHOWN SEE "OUTSIDE ELEVATION" AND "TYPICAL SECTION - TRAFFIC BARRIER"



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.
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\SHAPE F BARRIER SHT 2.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

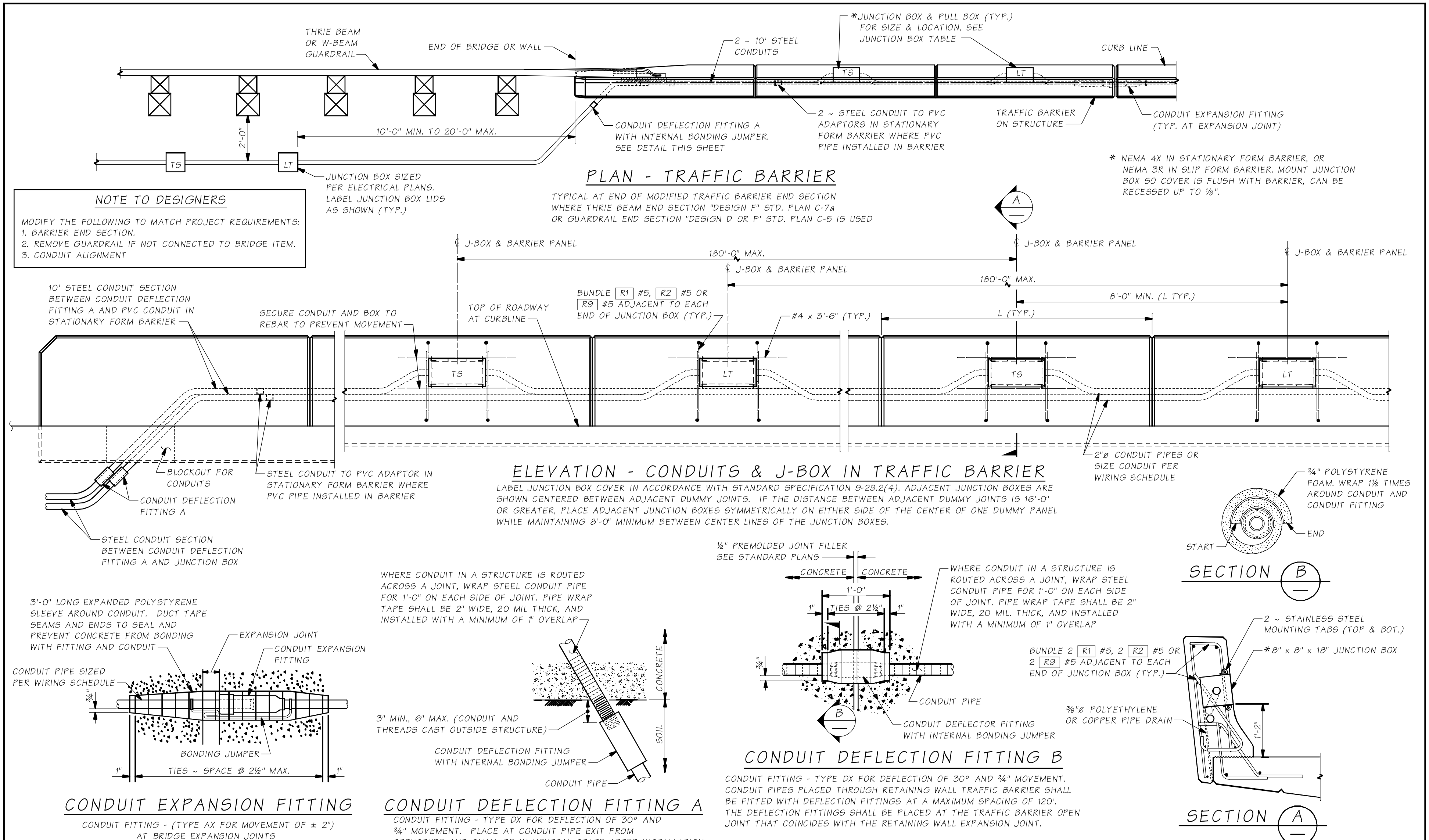
TRAFFIC BARRIER - SHAPE F DETAILS 2 OF 3

BRIDGE SHEET NO.
SHEET OF SHEETS

Thu Dec 19 14:35:31 2013

SHEET
JOB NO.
SR

Last revised on : 11/14/2012



Bridge Design Engr.	M:\STANDARDS\Traffic Barriers\Shape F\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

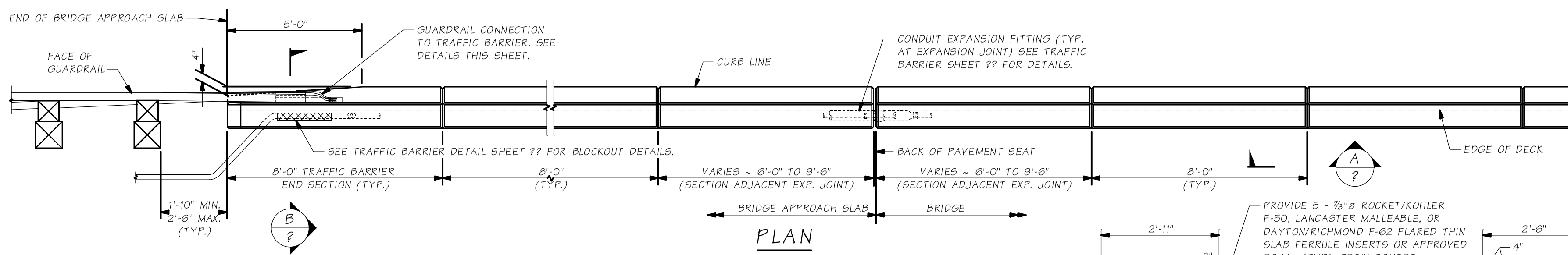
STANDARD TRAFFIC BARRIERS

TRAFFIC BARRIER - SHAPE F DETAIL 3 OF 3

BRIDGE SHEET NO. SHEET OF SHEETS

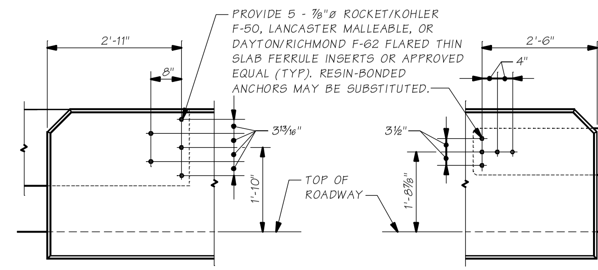
Thu Dec 19 14:35:31 2013

SR JOB NO. SHEET



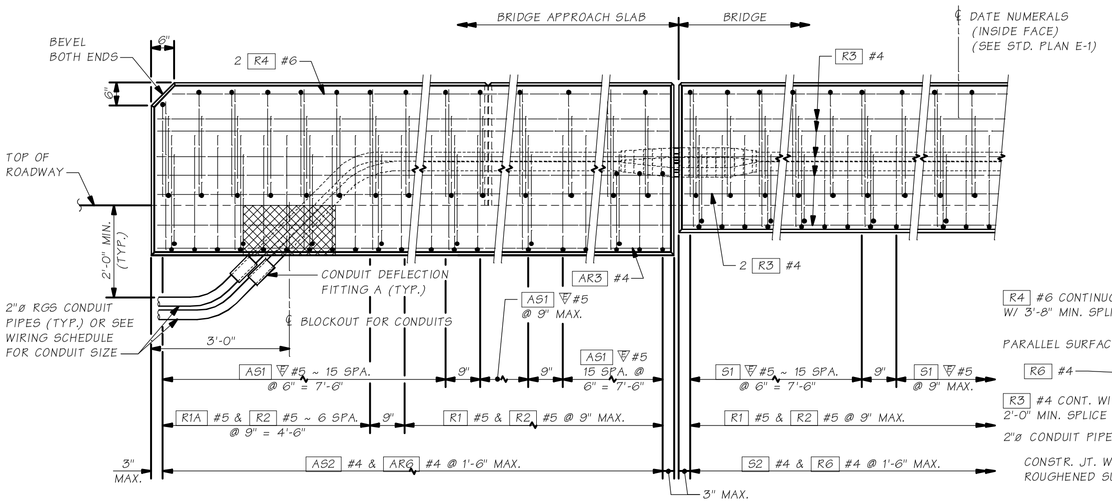
PLAN
TRAFFIC BARRIER

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.



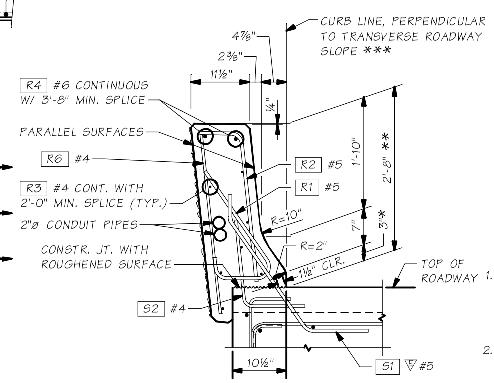
OUTSIDE ELEVATION
TRAFFIC BARRIER - GUARDRAIL CONNECTION

(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"

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						
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 TRAFFIC BARRIER

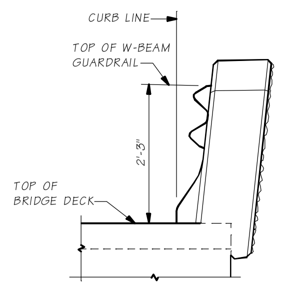
TRAFFIC BARRIER - SHAPE F
FLAT SLAB - DETAILS 1 OF 3

BRIDGE SHEET NO.
SHEET
OF
SHEETS

Thu Dec 19 14:35:32 2013

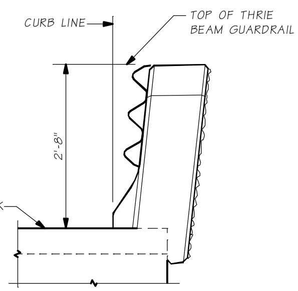
SR
FILE NO.
SHEET

Last revised on : 12/13/2012



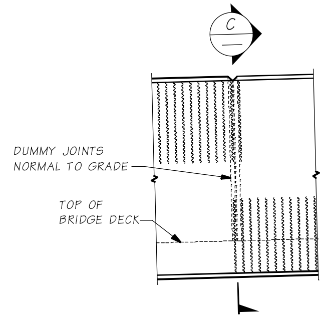
END VIEW

W-BEAM SHOWN WITH "D" CONNECTION OR "F" CONNECTION (SEE STD. PLAN C-5).

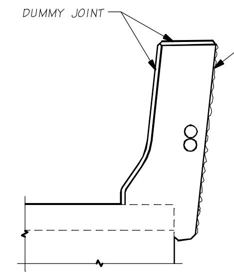


END VIEW

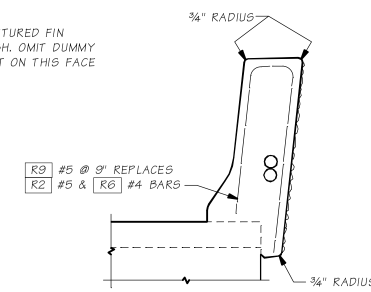
THRIE BEAM SHOWN WITH "D" CONNECTION OR "F" CONNECTION (SEE STD. PLAN C-5).



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.

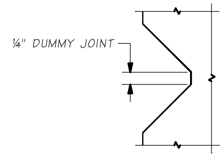
JUNCTION BOX LOCATIONS

STATION	OFFSET	"TS" OR "LT"

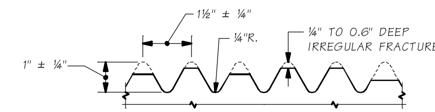
TS = TRAFFIC SYSTEM
LT = LIGHTING SYSTEM

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



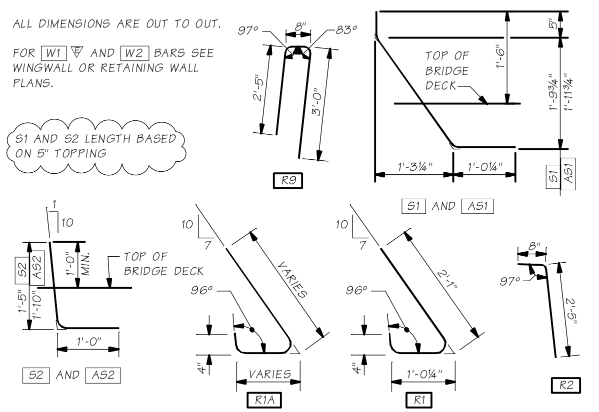
FRACTURED FIN FINISH

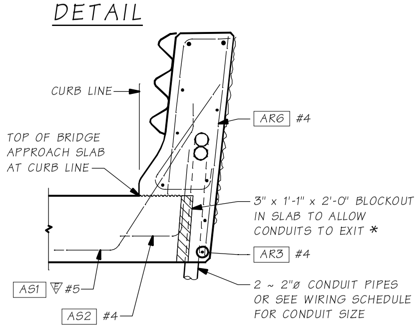
BENDING DIAGRAM

ALL DIMENSIONS ARE OUT TO OUT.

FOR W1 ∇ AND W2 BARS SEE WINGWALL OR RETAINING WALL PLANS.

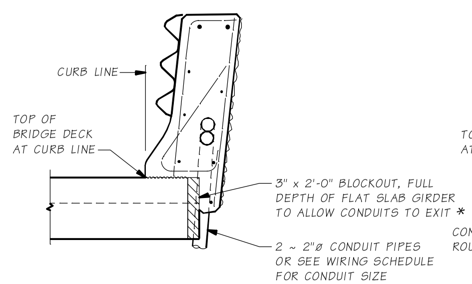
S1 AND S2 LENGTH BASED ON 5" TOPPING





SECTION B/? APPROACH SLAB

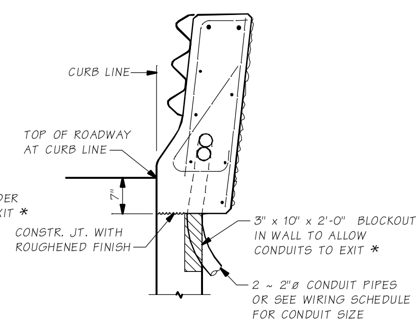
FOR DETAILS NOT SHOWN SEE "OUTSIDE ELEVATION" AND "TYPICAL SECTION - TRAFFIC BARRIER"



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				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

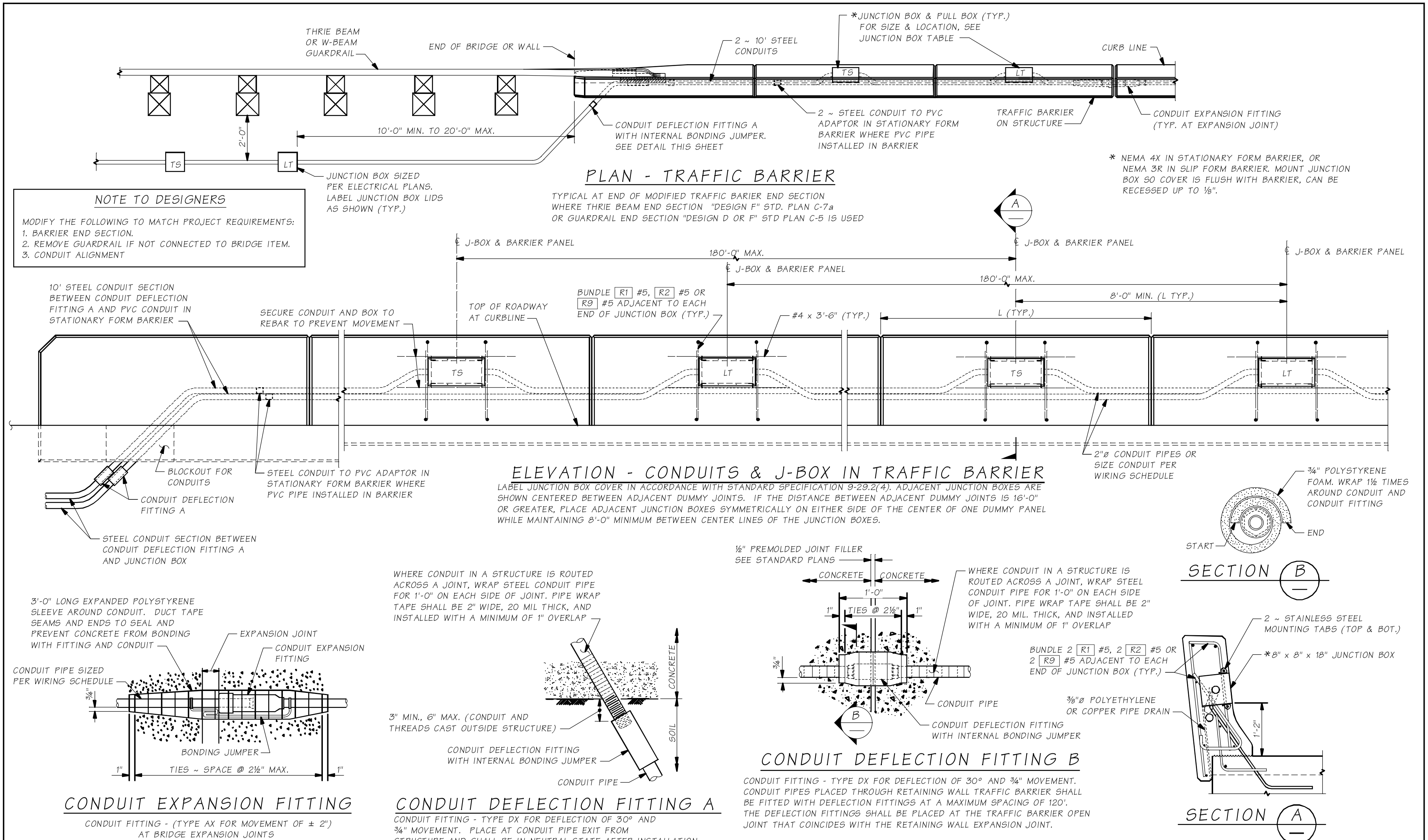
TRAFFIC BARRIER - SHAPE F
FLAT SLAB - DETAILS 2 OF 3

BRIDGE SHEET NO.
SHEET OF SHEETS

Thu Dec 19 14:35:32 2013

SR
FILE NO.
SHEET 0

Last revised on : 11/14/2012



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

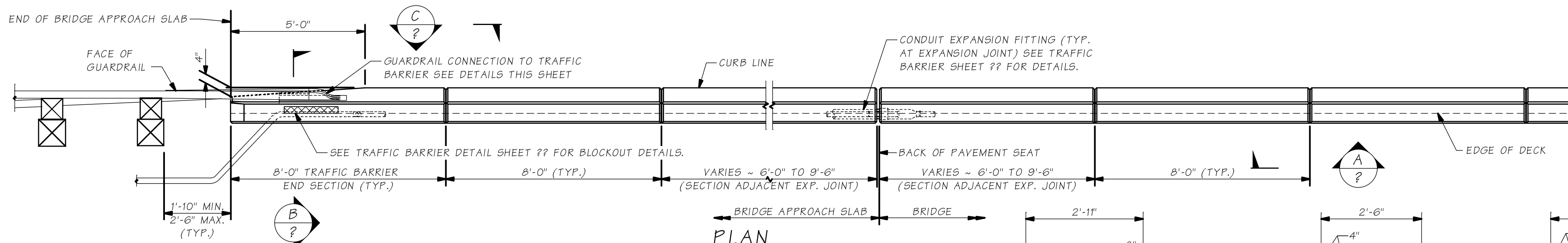
STANDARD TRAFFIC BARRIERS

TRAFFIC BARRIER - FLAT SLAB
FLAT SLAB - DETAILS 3 OF 3

BRIDGE SHEET NO. | SHEET OF SHEETS

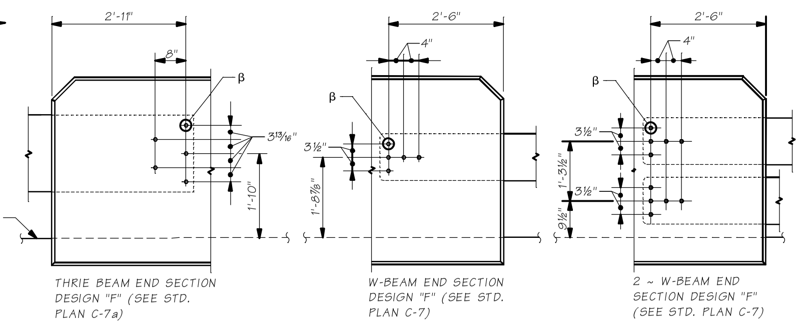
Thu Dec 19 14:35:32 2013

SR | FILE NO. | SHEET 0



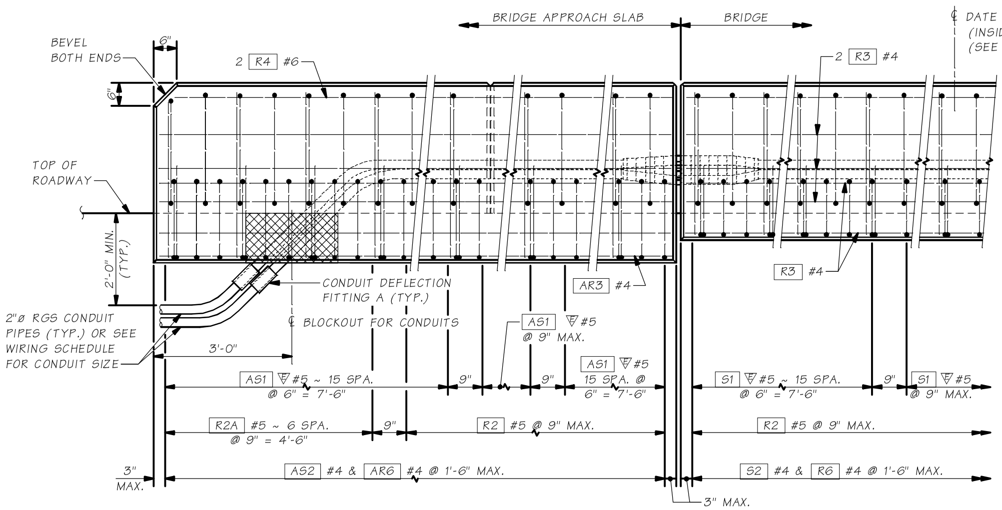
PLAN
TRAFFIC BARRIER

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.



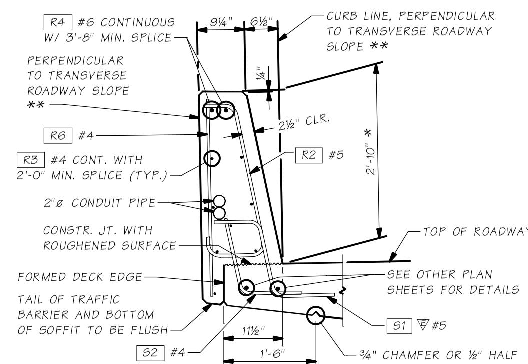
OUTSIDE ELEVATION
TRAFFIC BARRIER - GUARDRAIL CONNECTION

(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.

* 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.
Supervisor
Designed By
Checked By
Detailed By
Bridge Projects Engr.
Prelim. Plan By
Architect/Specialist

M:\STANDARDS\Traffic Barriers\Single Slope\SSTB SHEET 1.MAN

REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
10	WASH.			

JOB NUMBER

DATE | REVISION | BY | APP'D

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

STANDARD TRAFFIC BARRIERS

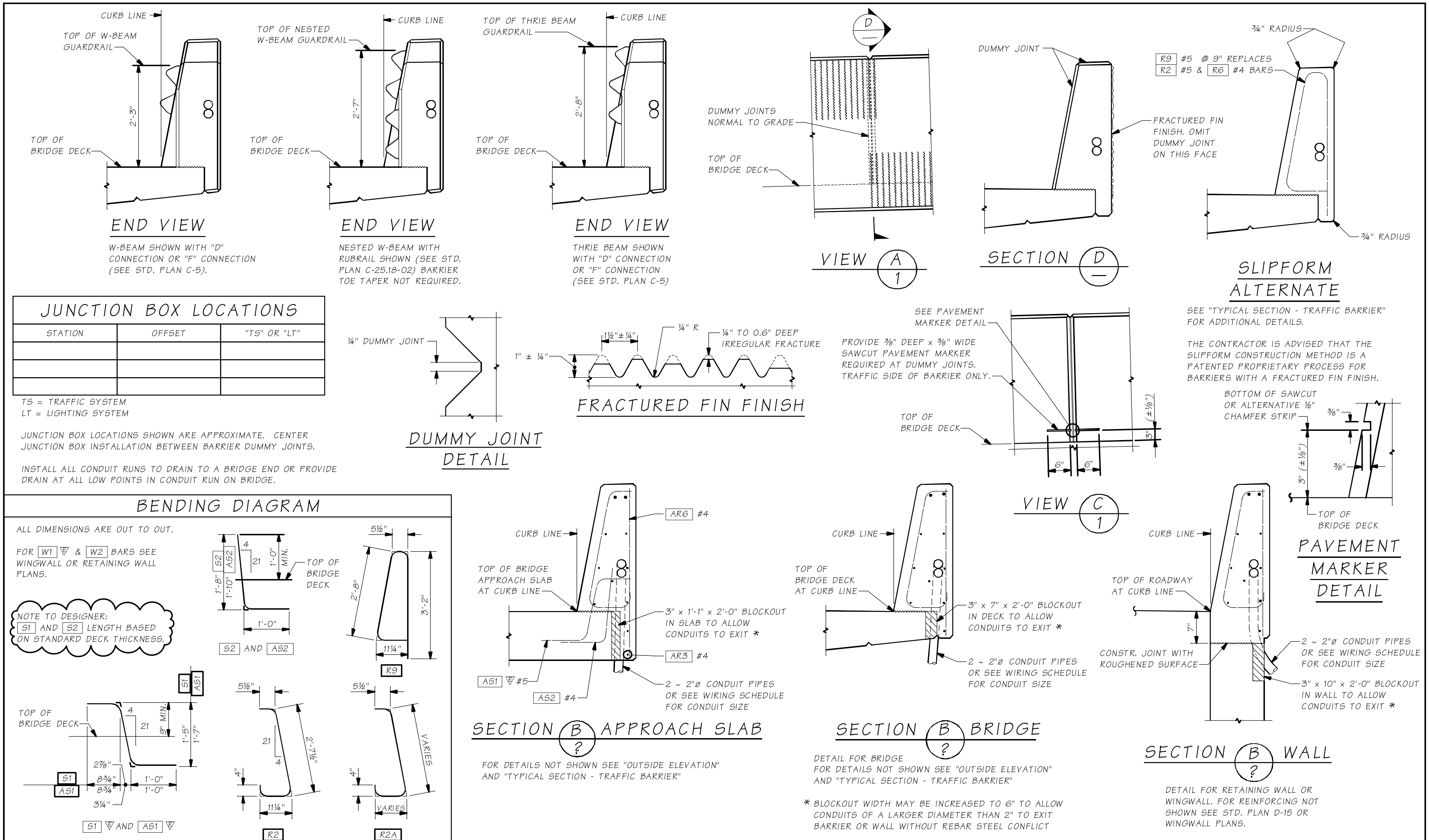
TRAFFIC BARRIER - SINGLE SLOPE DETAILS 1 OF 3

BRIDGE SHEET NO. SHEET OF SHEETS

Thu Dec 19 14:35:33 2013

JOB NO. SR SHEET 1

Last revised on : 11/30/2012



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\Single Slope\SSTB SHEET 2.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

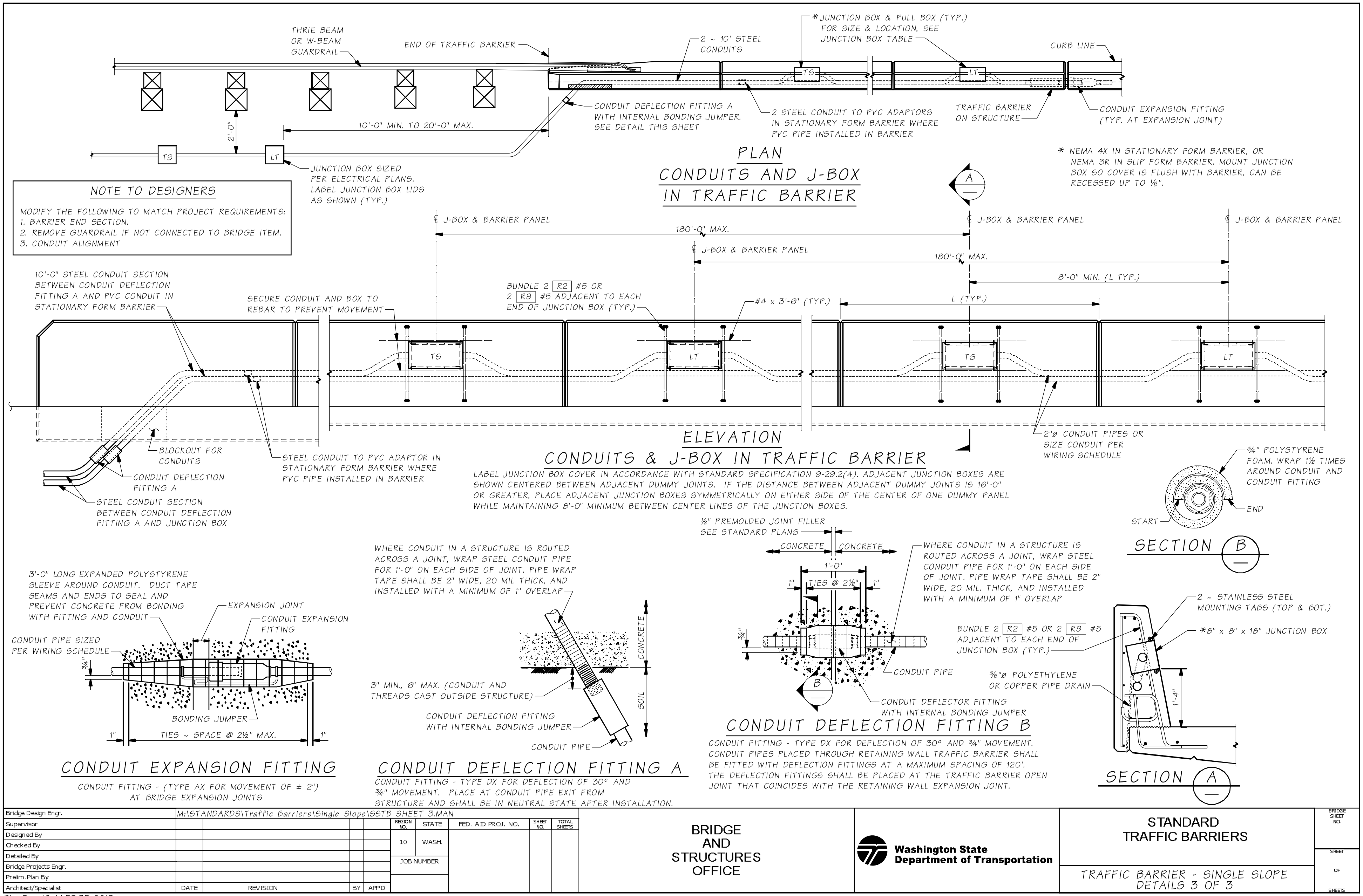
TRAFFIC BARRIER - SINGLE SLOPE DETAILS 2 OF 3

BRIDGE SHEET NO.
SHEET OF SHEETS

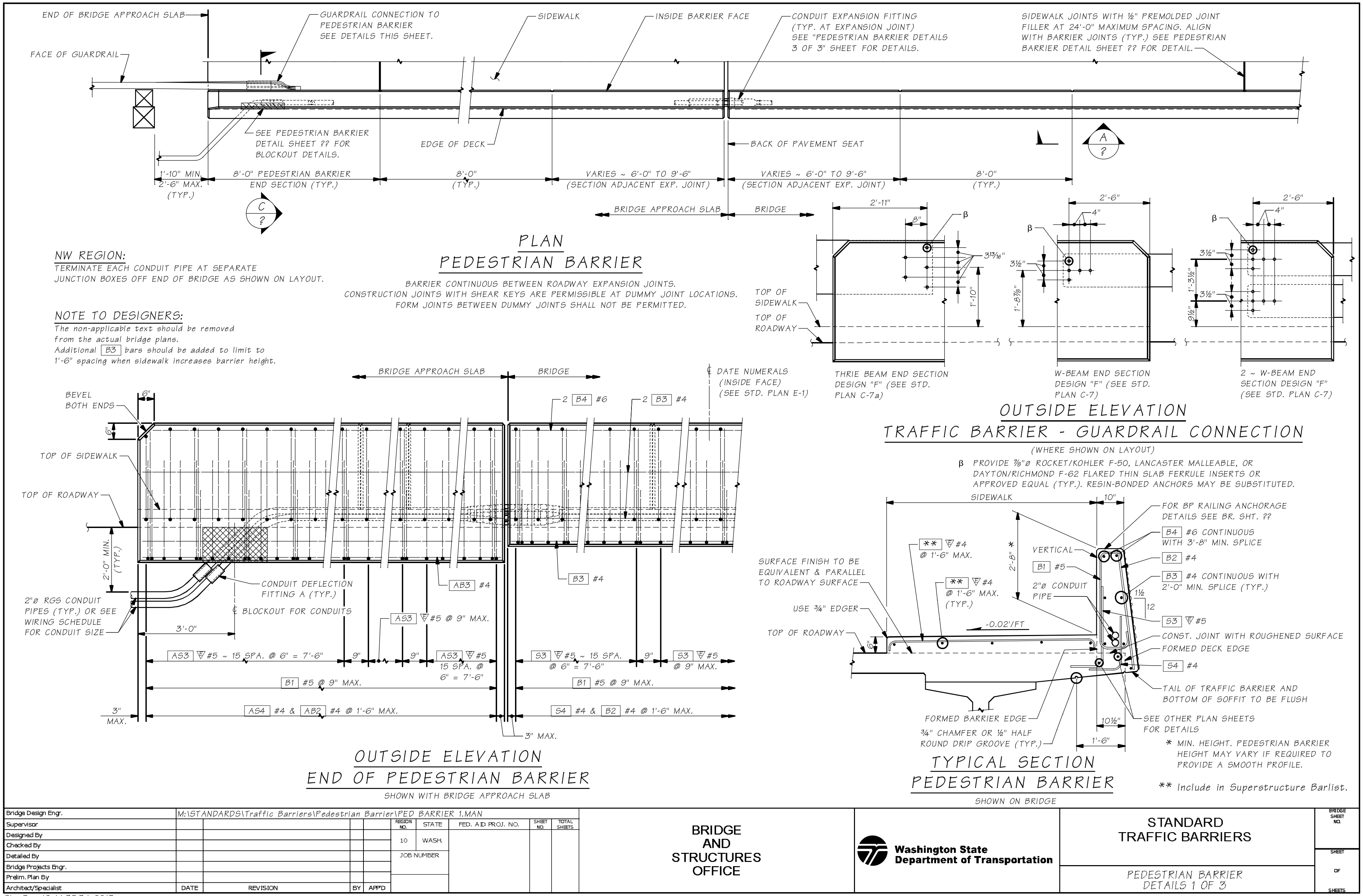
Thu Dec 19 14:35:33 2013

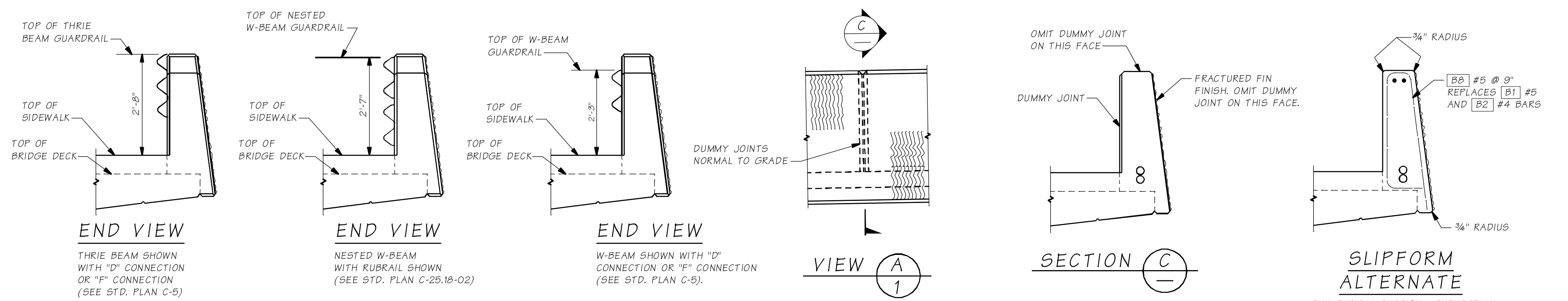
SHEET
JOB NO.
SR

Last revised on : 11/14/2012



PLAN
CONDUITS AND J-BOX IN TRAFFIC BARRIER
THRIE BEAM OR W-BEAM GUARDRAIL
END OF TRAFFIC BARRIER
2 ~ 10' STEEL CONDUITS
*JUNCTION BOX & PULL BOX (TYP.) FOR SIZE & LOCATION, SEE JUNCTION BOX TABLE
CURB LINE
CONDUIT DEFLECTION FITTING A WITH INTERNAL BONDING JUMPER. SEE DETAIL THIS SHEET
2 STEEL CONDUIT TO PVC ADAPTORS IN STATIONARY FORM BARRIER WHERE PVC PIPE INSTALLED IN BARRIER
TRAFFIC BARRIER ON STRUCTURE
CONDUIT EXPANSION FITTING (TYP. AT EXPANSION JOINT)
10'-0" MIN. TO 20'-0" MAX.
2'-0"
TS
LT
JUNCTION BOX SIZED PER ELECTRICAL PLANS. LABEL JUNCTION BOX LIDS AS SHOWN (TYP.)
* NEMA 4X IN STATIONARY FORM BARRIER, OR NEMA 3R IN SLIP FORM BARRIER. MOUNT JUNCTION BOX SO COVER IS FLUSH WITH BARRIER, CAN BE RECESSED UP TO ⅛".
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.)
10'-0" STEEL CONDUIT SECTION BETWEEN CONDUIT DEFLECTION FITTING A AND PVC CONDUIT IN STATIONARY FORM BARRIER
SECURE CONDUIT AND BOX TO REBAR TO PREVENT MOVEMENT
BUNDLE 2 R2 #5 OR 2 R9 #5 ADJACENT TO EACH END OF JUNCTION BOX (TYP.)
#4 x 3'-6" (TYP.)
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
2"ø CONDUIT PIPES OR SIZE CONDUIT PER WIRING SCHEDULE
ELEVATION
CONDUITS & J-BOX IN TRAFFIC BARRIER
LABEL JUNCTION BOX COVER IN ACCORDANCE WITH STANDARD SPECIFICATION 9-29.2(4). 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.
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 AND SHALL BE IN NEUTRAL STATE AFTER INSTALLATION.
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 DEFLECTOR 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 TRAFFIC BARRIER SHALL BE FITTED WITH DEFLECTION FITTINGS AT A MAXIMUM SPACING OF 120'. THE DEFLECTION FITTINGS SHALL BE PLACED AT THE TRAFFIC BARRIER OPEN JOINT THAT COINCIDES WITH THE RETAINING WALL EXPANSION JOINT.
2 ~ STAINLESS STEEL MOUNTING TABS (TOP & BOT.)
*8" x 8" x 18" JUNCTION BOX
BUNDLE 2 R2 #5 OR 2 R9 #5 ADJACENT TO EACH END OF JUNCTION BOX (TYP.)
⅜"ø POLYETHYLENE OR COPPER PIPE DRAIN
1'-4"
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\Single Slope\SSTB SHEET 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
TRAFFIC BARRIER - SINGLE SLOPE DETAILS 3 OF 3
BRIDGE SHEET NO.
SHEET
OF
SHEETS
Thu Dec 19 14:35:33 2013


PLAN
PEDESTRIAN 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.
END OF BRIDGE APPROACH SLAB
FACE OF GUARDRAIL
GUARDRAIL CONNECTION TO PEDESTRIAN BARRIER SEE DETAILS THIS SHEET.
SIDEWALK
INSIDE BARRIER FACE
CONDUIT EXPANSION FITTING (TYP. AT EXPANSION JOINT) SEE "PEDESTRIAN BARRIER DETAILS 3 OF 3" SHEET FOR DETAILS.
SIDEWALK JOINTS WITH ½" PREMOLDED JOINT FILLER AT 24'-0" MAXIMUM SPACING. ALIGN WITH BARRIER JOINTS (TYP.) SEE PEDESTRIAN BARRIER DETAIL SHEET ?? FOR DETAIL.
SEE PEDESTRIAN BARRIER DETAIL SHEET ?? FOR BLOCKOUT DETAILS.
EDGE OF DECK
BACK OF PAVEMENT SEAT
1'-10" MIN. 2'-6" MAX. (TYP.)
8'-0" PEDESTRIAN BARRIER END SECTION (TYP.)
8'-0" (TYP.)
VARIES ~ 6'-0" TO 9'-6" (SECTION ADJACENT EXP. JOINT)
BRIDGE APPROACH SLAB
BRIDGE
NW REGION:
TERMINATE EACH CONDUIT PIPE AT SEPARATE JUNCTION BOXES OFF END OF BRIDGE AS SHOWN ON LAYOUT.
NOTE TO DESIGNERS:
The non-applicable text should be removed from the actual bridge plans.
Additional B3 bars should be added to limit to 1'-6" spacing when sidewalk increases barrier height.
OUTSIDE ELEVATION
END OF PEDESTRIAN BARRIER
SHOWN WITH BRIDGE APPROACH SLAB
BEVEL BOTH ENDS
TOP OF SIDEWALK
TOP OF ROADWAY
2"ø RGS CONDUIT PIPES (TYP.) OR SEE WIRING SCHEDULE FOR CONDUIT SIZE
CONDUIT DEFLECTION FITTING A (TYP.)
℄ BLOCKOUT FOR CONDUITS
℄ DATE NUMERALS (INSIDE FACE) (SEE STD. PLAN E-1)
2 B4 #6
2 B3 #4
AB3 #4
B3 #4
AS3 #5 @ 9" MAX.
AS3 #5 ~ 15 SPA. @ 6" = 7'-6"
AS3 #5 15 SPA. @ 6" = 7'-6"
S3 #5 ~ 15 SPA. @ 6" = 7'-6"
S3 #5 @ 9" MAX.
B1 #5 @ 9" MAX.
AS4 #4 & AB2 #4 @ 1'-6" MAX.
S4 #4 & B2 #4 @ 1'-6" MAX.
3" MAX.
OUTSIDE ELEVATION
TRAFFIC BARRIER - GUARDRAIL CONNECTION
(WHERE SHOWN ON LAYOUT)
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.
THRIE BEAM END SECTION DESIGN "F" (SEE STD. PLAN C-7a)
W-BEAM END SECTION DESIGN "F" (SEE STD. PLAN C-7)
2 ~ W-BEAM END SECTION DESIGN "F" (SEE STD. PLAN C-7)
TOP OF SIDEWALK
TOP OF ROADWAY
TYPICAL SECTION
PEDESTRIAN BARRIER
SHOWN ON BRIDGE
SURFACE FINISH TO BE EQUIVALENT & PARALLEL TO ROADWAY SURFACE
USE ¾" EDGER
TOP OF ROADWAY
** #4 @ 1'-6" MAX.
** #4 @ 1'-6" MAX. (TYP.)
-0.02'/FT
VERTICAL
B1 #5
2"ø CONDUIT PIPE
FOR BP RAILING ANCHORAGE DETAILS SEE BR. SHT. ??
B4 #6 CONTINUOUS WITH 3'-8" MIN. SPLICE
B2 #4
B3 #4 CONTINUOUS WITH 2'-0" MIN. SPLICE (TYP.)
S3 #5
CONST. JOINT WITH ROUGHENED SURFACE
FORMED DECK EDGE
S4 #4
TAIL OF TRAFFIC BARRIER AND BOTTOM OF SOFFIT TO BE FLUSH
SEE OTHER PLAN SHEETS FOR DETAILS
FORMED BARRIER EDGE
¾" CHAMFER OR ½" HALF ROUND DRIP GROOVE (TYP.)
* MIN. HEIGHT. PEDESTRIAN BARRIER HEIGHT MAY VARY IF REQUIRED TO PROVIDE A SMOOTH PROFILE.
** Include in Superstructure Barlist.
M:\STANDARDS\Traffic Barriers\Pedestrian Barrier\PED BARRIER 1.MAN
BRIDGE AND STRUCTURES OFFICE
Washington State Department of Transportation
STANDARD TRAFFIC BARRIERS
PEDESTRIAN BARRIER DETAILS 1 OF 3
Thu Dec 19 14:35:34 2013



SEE TYPICAL SECTION - PEDESTRIAN 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.

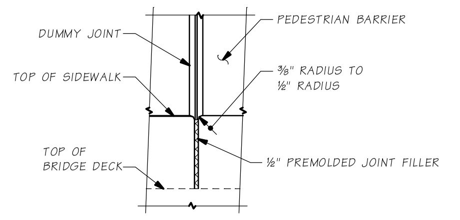
JUNCTION BOX LOCATIONS

STATION	OFFSET	"TS" OR "LT"

TS = TRAFFIC SYSTEM
LT = LIGHTING SYSTEM

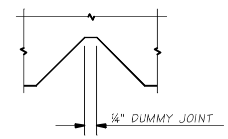
JUNCTION BOX LOCATIONS SHOWN ARE APPROXIMATE. CENTER JUNCTION BOX INSTALLATION BETWEEN BARRIER DUMMY JOINTS.

INSTALL ALL CONDUIT RUNS TO DRAIN TO A BRIDGE END OR PROVIDE DRAIN AT ALL LOW POINTS IN CONDUIT RUN ON BRIDGE.

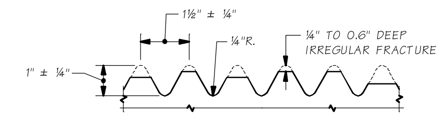


SIDEWALK JOINT DETAIL

SIDEWALK REINFORCING STEEL NOT SHOWN FOR CLARITY.



DUMMY JOINT DETAIL

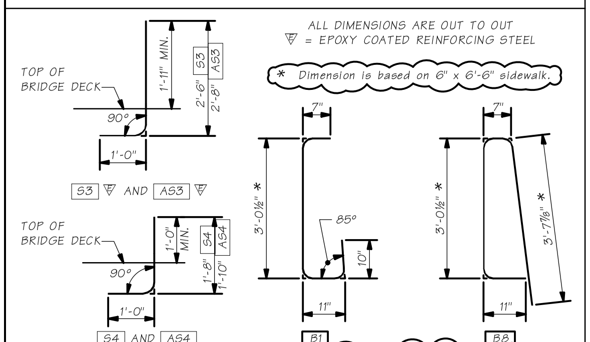


FRACTURED FIN FINISH

BENDING DIAGRAM

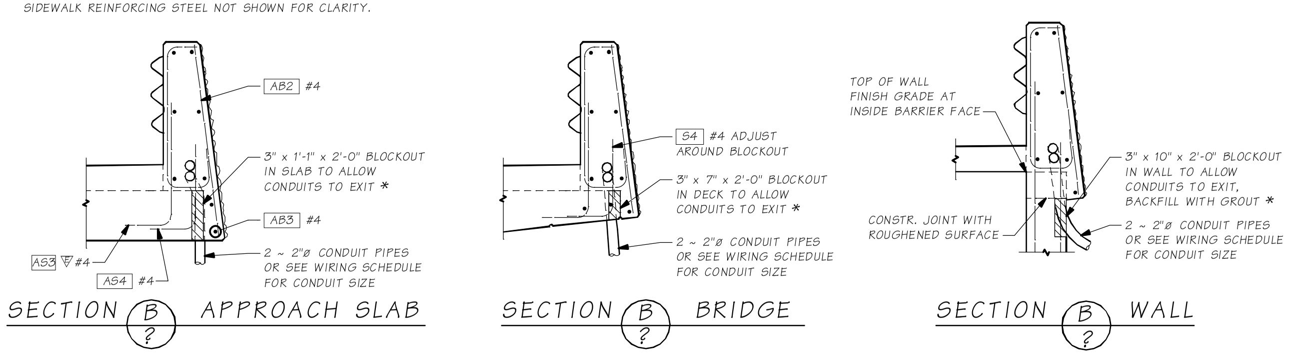
ALL DIMENSIONS ARE OUT TO OUT
∇ = EPOXY COATED REINFORCING STEEL

* Dimension is based on 6" x 6'-6" sidewalk.



FOR W1 & W2 BARS SEE WINGWALL OR RETAINING WALL PLANS.

NOTE TO DESIGNER:
S3 AND S4 LENGTH BASED ON STANDARD DECK THICKNESS.



FOR DETAILS NOT SHOWN SEE "OUTSIDE ELEVATION" AND "TYPICAL SECTION ~ PEDESTRIAN BARRIER"

FOR DETAILS NOT SHOWN SEE "OUTSIDE ELEVATION" AND "TYPICAL SECTION ~ PEDESTRIAN 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 RETAINING WALL OR WINGWALL. FOR REINFORCING NOT SHOWN SEE STD. PLAN D-15 OR 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\Pedestrian Barrier\PED BARRIER 2.MAN

REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
10	WASH.			

JOB NUMBER

BRIDGE AND STRUCTURES OFFICE

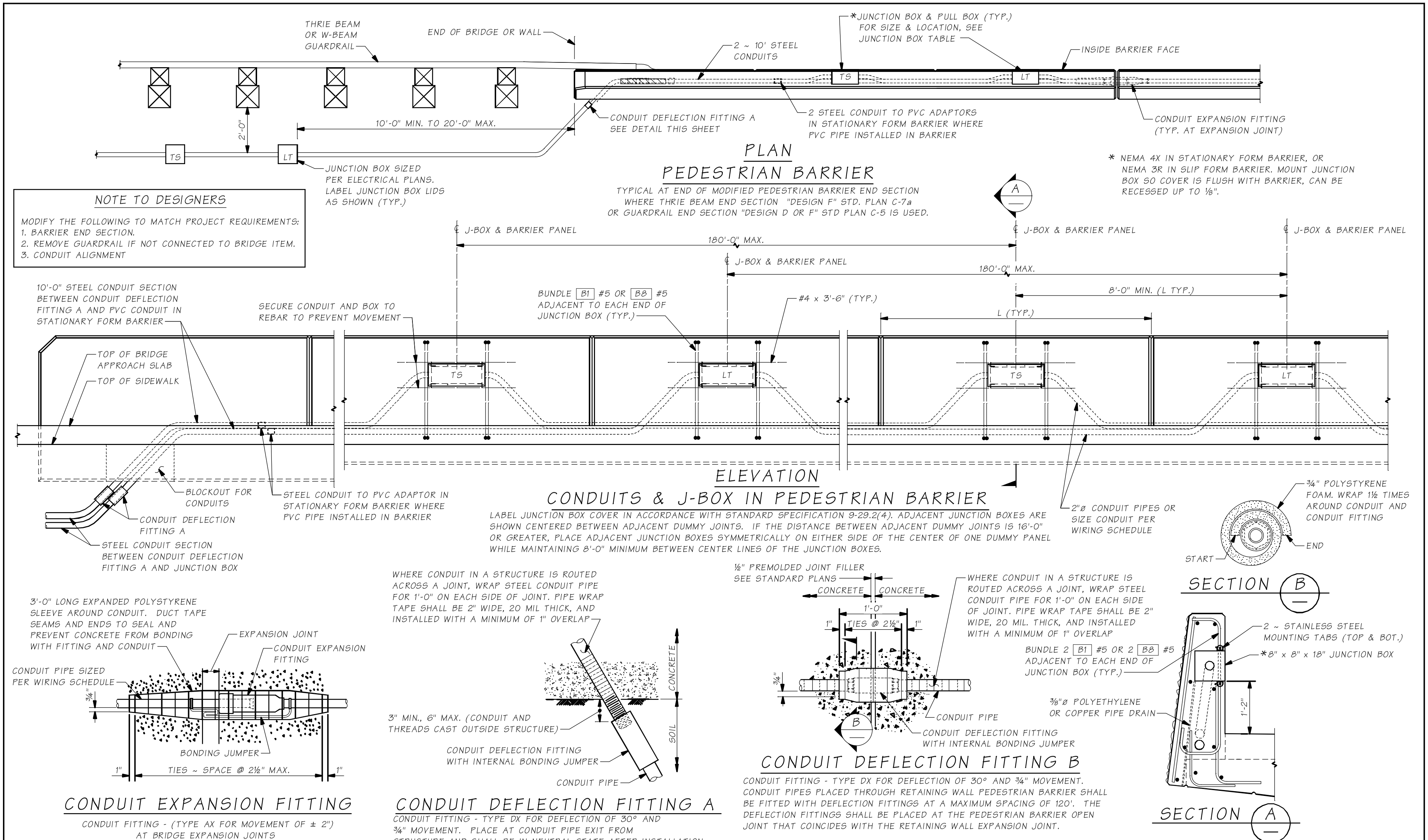
Washington State Department of Transportation

STANDARD TRAFFIC BARRIERS

PEDESTRIAN BARRIER DETAILS 2 OF 3

BRIDGE SHEET NO.
SHEET OF SHEETS

Thu Dec 19 14:35:34 2013



Bridge Design Engr.	M:\STANDARDS\Traffic Barriers\Pedestrian Barrier\PED BARRIER 3.MAN						
Supervisor		REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS	
Designed By		10	WASH.				
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Bridge Projects Engr.							
Prelim. Plan By							
Architect/Specialist	DATE	REVISION	BY	APP'D			

BRIDGE AND STRUCTURES OFFICE

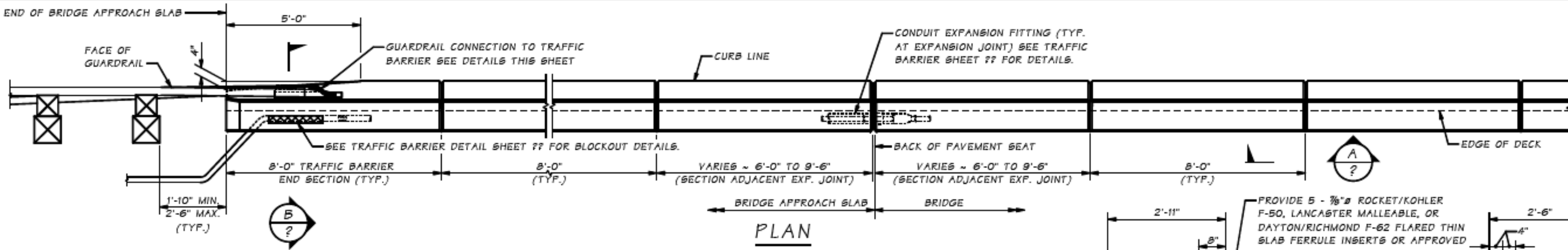
Washington State Department of Transportation

STANDARD TRAFFIC BARRIERS

PEDESTRIAN BARRIER DETAILS 3 OF 3

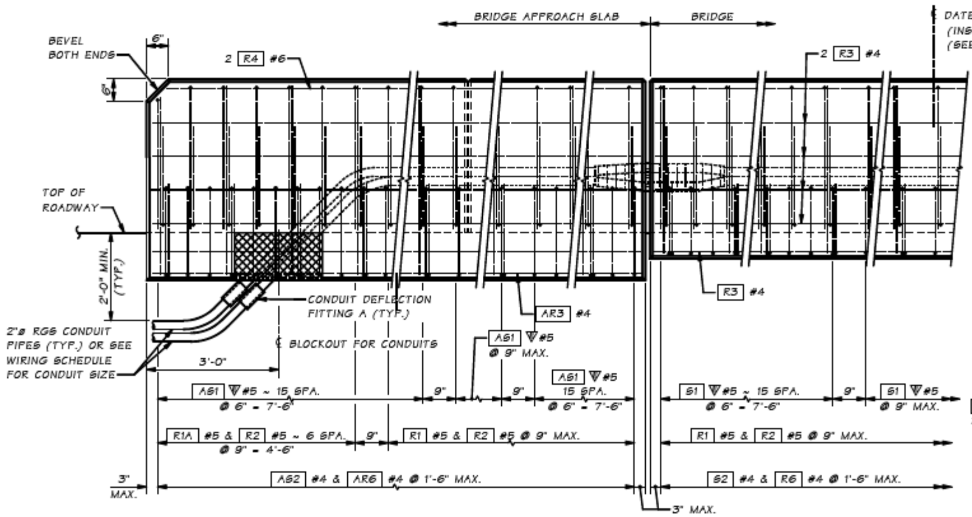
BRIDGE SHEET NO. | SHEET OF SHEETS

Thu Dec 19 14:35:35 2013



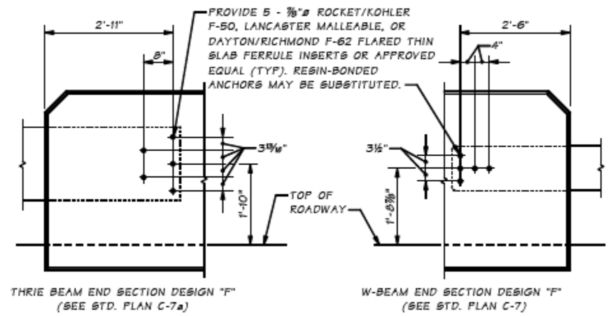
PLAN
TRAFFIC BARRIER

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.



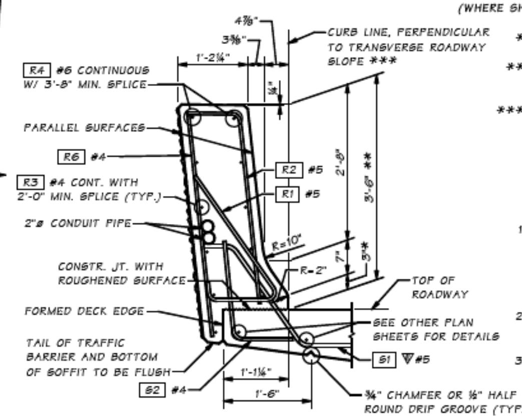
OUTSIDE ELEVATION
END OF TRAFFIC BARRIER

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.
3. General Notes or traffic barrier details shall include the test level (TL-4) of the barrier.

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. 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 - SHAPE F 42" DETAILS 1 OF 3

BRIDGE SHEET NO. | SHEET | OF | SHEETS

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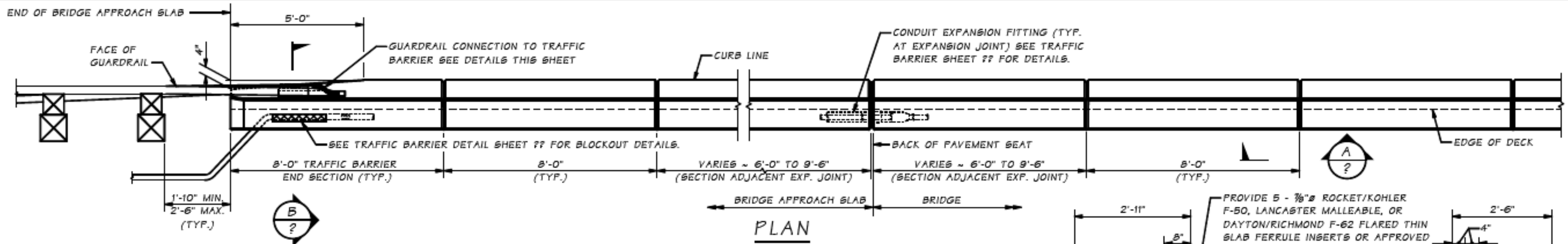
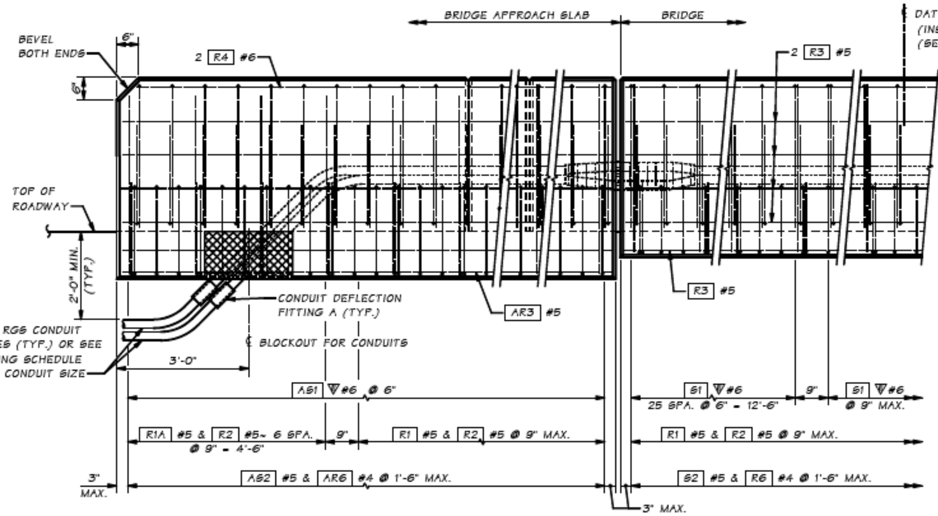


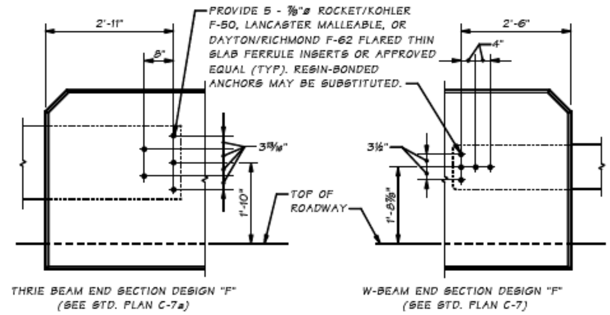
PLAN
TRAFFIC BARRIER

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.



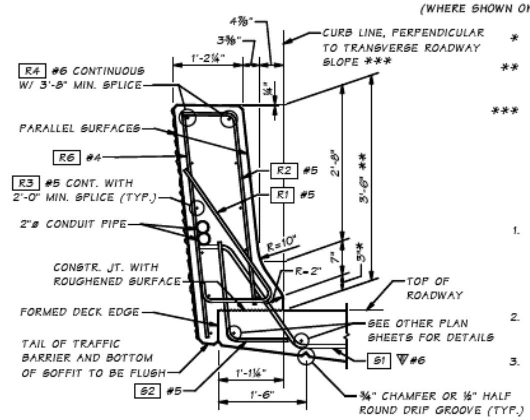
OUTSIDE ELEVATION
END OF TRAFFIC BARRIER

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.
3. General Notes or traffic barrier details shall include the test level (TL-5) of the barrier.

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. 10	STATE WASH.	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
Designed By		JOB NUMBER				
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Architect/Specialist	DATE	REVISION	BY	APP'D		

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BRIDGE AND STRUCTURES OFFICE

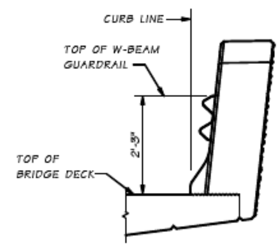
Washington State Department of Transportation

STANDARD TRAFFIC BARRIERS

TRAFFIC BARRIER - SHAPE F 42" DETAILS 1 OF 3

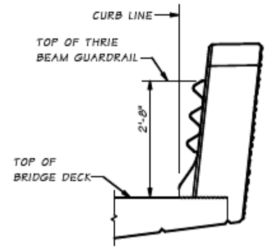
BRIDGE SHEET NO. | SHEET | OF | SHEETS

SR | JOB NO. | SHEET 1



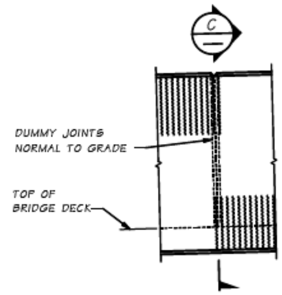
END VIEW

W-BEAM SHOWN WITH "D" CONNECTION OR "F" CONNECTION (SEE STD. PLAN C-5).

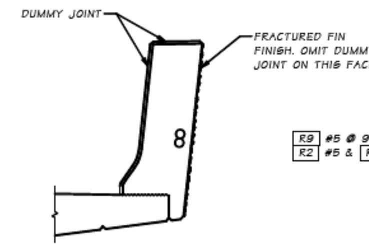


END VIEW

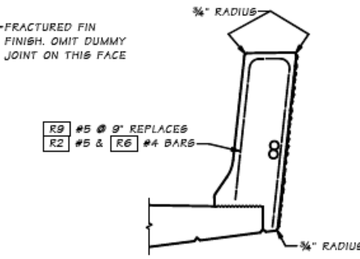
THRIE BEAM SHOWN WITH "D" CONNECTION OR "F" CONNECTION (SEE STD. PLAN C-5).



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.

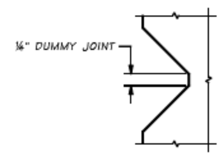
JUNCTION BOX LOCATIONS

STATION	OFFSET	"TS" OR "LT"

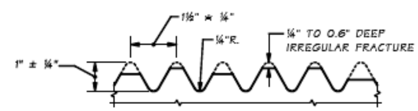
TS = TRAFFIC SYSTEM
LT = LIGHTING SYSTEM

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.



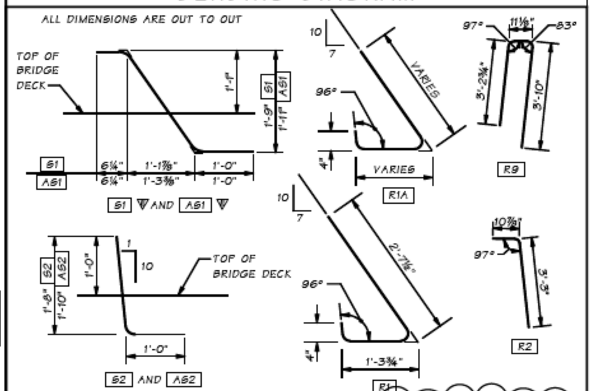
DUMMY JOINT DETAIL



FRACTURED FIN FINISH

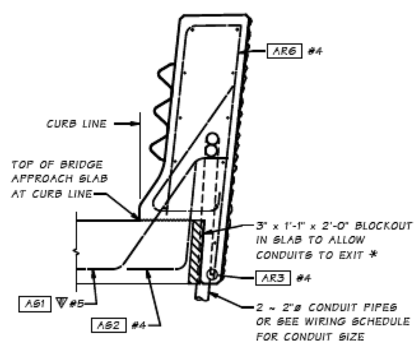
BENDING DIAGRAM

ALL DIMENSIONS ARE OUT TO OUT



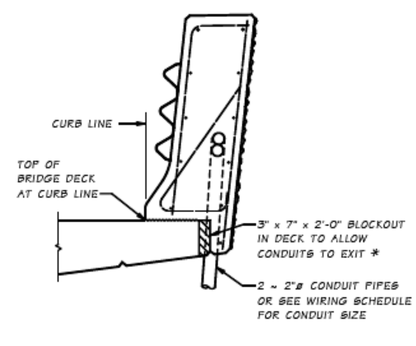
FOR W1 ▽ & W2 BARS SEE WINGWALL OR RETAINING WALL PLANS.

NOTE TO DESIGNER:
S1 AND S2 LENGTH BASED ON STANDARD DECK THICKNESS.



SECTION (B/?) APPROACH SLAB

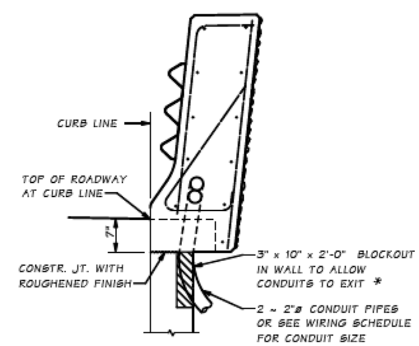
FOR DETAILS NOT SHOWN SEE "OUTSIDE ELEVATION" AND "TYPICAL SECTION - TRAFFIC BARRIER"



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 WINGWALL. FOR REINFORCING NOT SHOWN SEE WINGWALL PLANS.

Bridge Design Engr.
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Bridge Projects Engr.
Prelim. Plan By
Architect/Specialist
M:\STANDARDS\Traffic Barriers\Shape F\42IN SHAPE F BARRIER SHT 2 - TL 4.MAN
DATE | REVISION | BY | APP'D
REGION NO. 10 | STATE WASH. | FED. AID PROJ. NO. | SHEET NO. | TOTAL SHEETS
JOB NUMBER

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

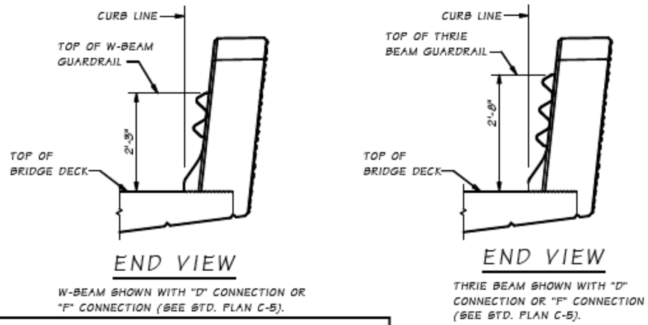
STANDARD TRAFFIC BARRIER

TRAFFIC BARRIER - SHAPE F 42"
DETAIL 2 OF 3

BRIDGE S. SET NO.
SHEET
OF
SHEETS

JOB NO.
SR
SHEET

Wed Feb 19 13:29:04 2014



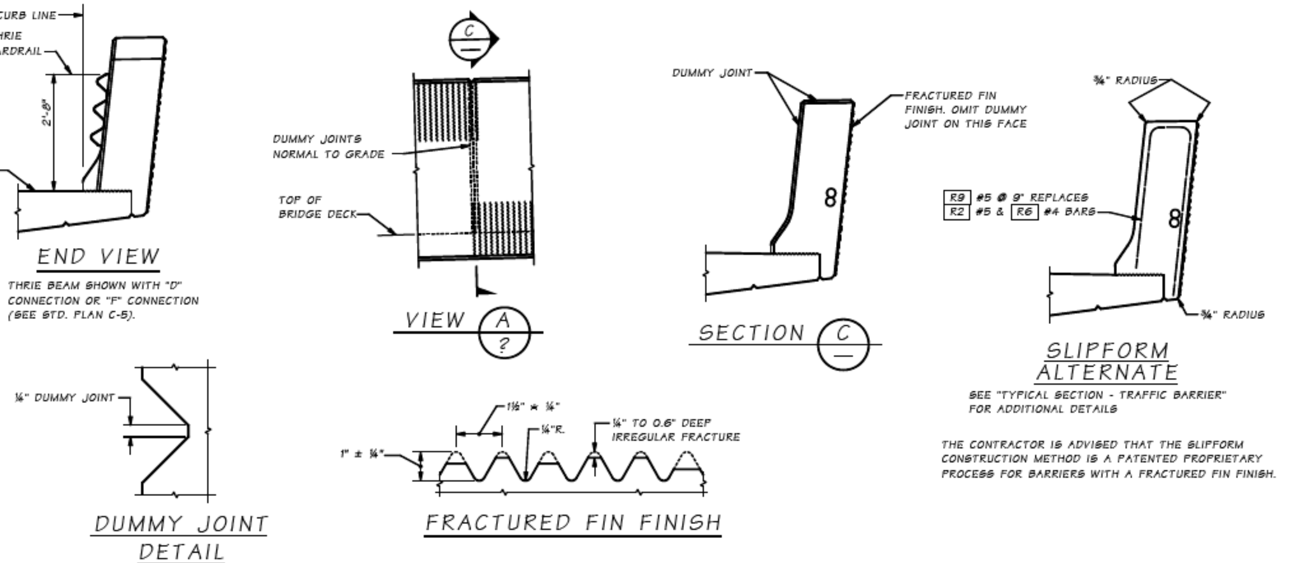
JUNCTION BOX LOCATIONS

STATION	OFFSET	"TS" OR "LT"

TS = TRAFFIC SYSTEM
LT = LIGHTING SYSTEM

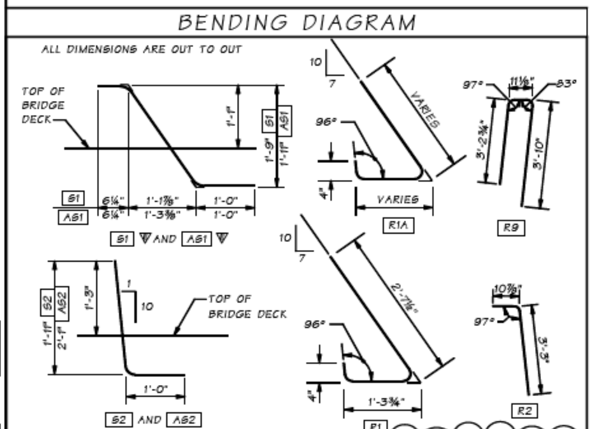
JUNCTION BOX LOCATIONS SHOWN ARE APPROXIMATE. CENTER JUNCTION BOX INSTALLATION BETWEEN BARRIER DUMMY JOINTS.

INSTALL ALL CONDUIT RUNS TO DRAIN TO A BRIDGE END OR PROVIDE DRAIN AT ALL LOW POINTS IN CONDUIT RUN ON BRIDGE.



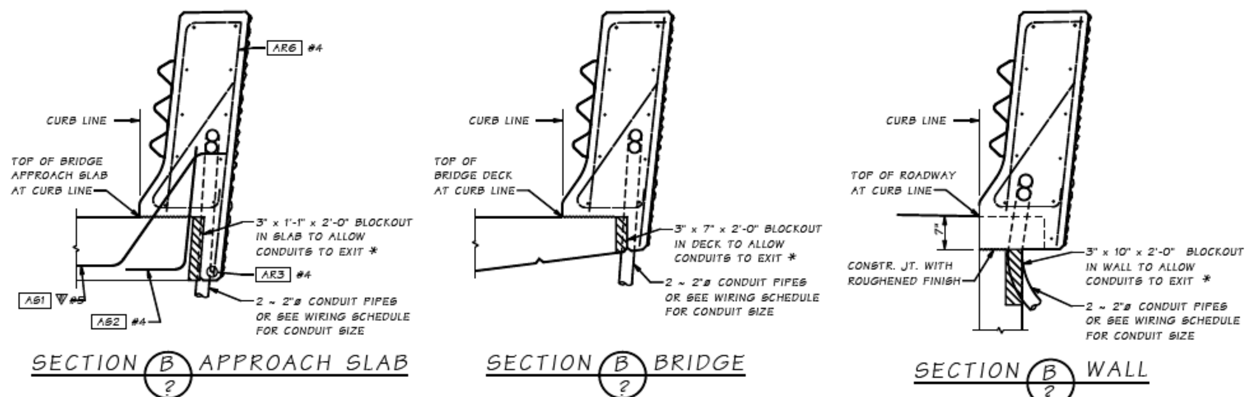
SEE "TYPICAL SECTION - TRAFFIC BARRIER" FOR ADDITIONAL DETAILS

THE CONTRACTOR IS ADVISED THAT THE SLIPFORM CONSTRUCTION METHOD IS A PATENTED PROPRIETARY PROCESS FOR BARRIERS WITH A FRACTURED FIN FINISH.



FOR W1 & W2 BARS SEE WINGWALL OR RETAINING WALL PLANS.

NOTE TO DESIGNER:
S1 AND S2 LENGTH BASED ON STANDARD DECK THICKNESS.



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.

M:\STANDARDS\Traffic Barriers\Shape F\ShapeF_Barrier_SHT 2 - TL 5.MAN

Bridge Design Engr.					
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				

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Washington State Department of Transportation

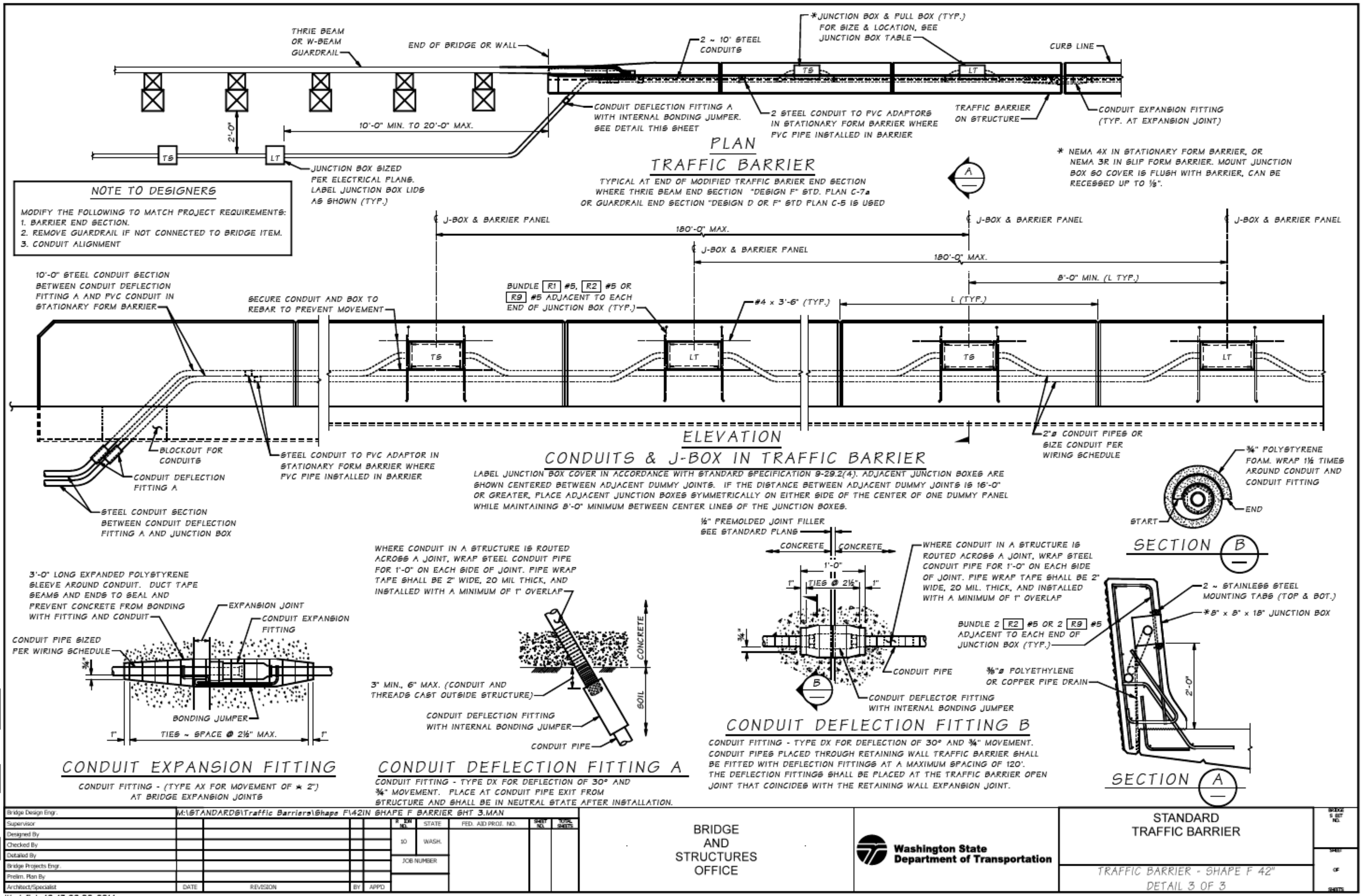
STANDARD TRAFFIC BARRIER

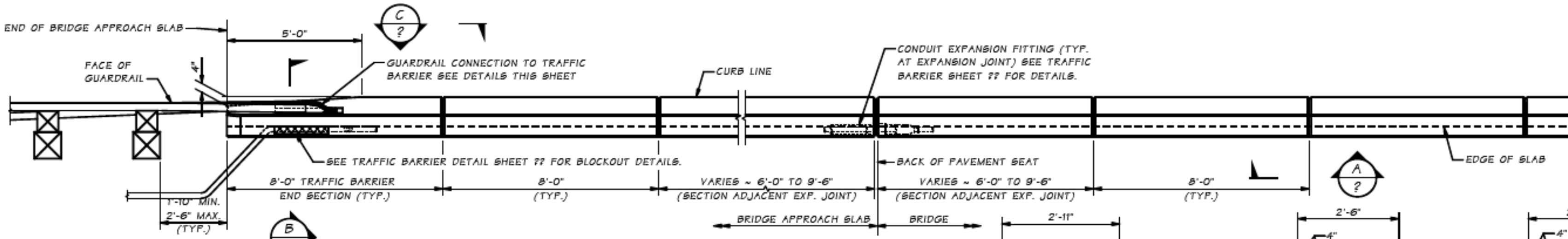
TRAFFIC BARRIER - SHAPE F 42"
DETAIL 2 OF 3

BRIDGE S. SET NO.
SHEET OF SHEETS

Wed Feb 19 13:29:04 2014

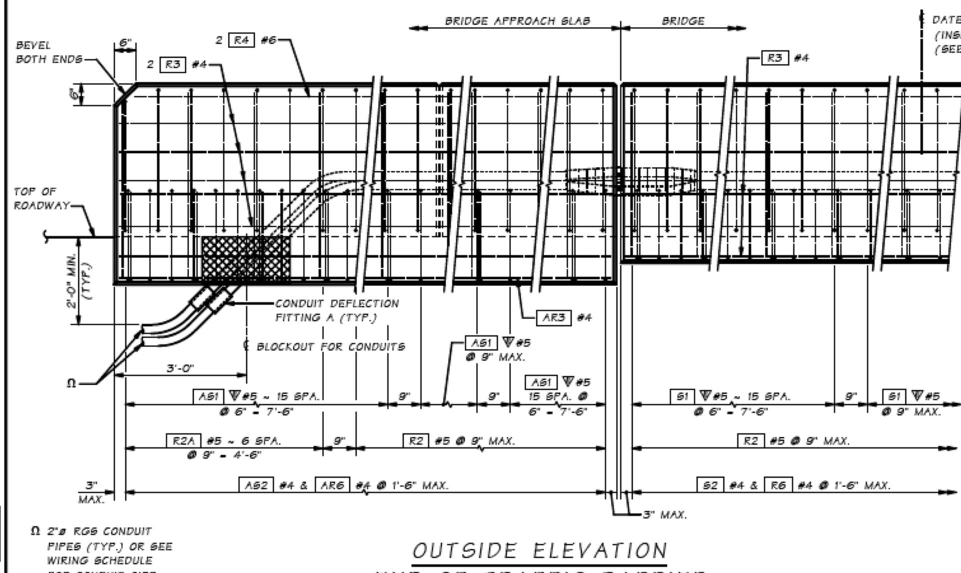
SR. JOB NO. SHEET


PLAN
TRAFFIC BARRIER
TYPICAL AT END OF MODIFIED TRAFFIC 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
THRIE BEAM OR W-BEAM GUARDRAIL
END OF BRIDGE OR WALL
2 ~ 10' STEEL CONDUITS
*JUNCTION BOX & PULL BOX (TYP.) FOR SIZE & LOCATION, SEE JUNCTION BOX TABLE
CURB LINE
CONDUIT DEFLECTION FITTING A WITH INTERNAL BONDING JUMPER. SEE DETAIL THIS SHEET
2 STEEL CONDUIT TO PVC ADAPTORS IN STATIONARY FORM BARRIER WHERE PVC PIPE INSTALLED IN BARRIER
TRAFFIC BARRIER ON STRUCTURE
CONDUIT EXPANSION FITTING (TYP. AT EXPANSION JOINT)
10'-0" MIN. TO 20'-0" MAX.
2'-0"
JUNCTION BOX SIZED PER ELECTRICAL PLANS. LABEL JUNCTION BOX LIDS AS SHOWN (TYP.)
* NEMA 4X IN STATIONARY FORM BARRIER, OR NEMA 3R IN SLIP FORM BARRIER. MOUNT JUNCTION BOX SO COVER IS FLUSH WITH BARRIER, CAN BE RECESSED UP TO ⅛".
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.)
10'-0" STEEL CONDUIT SECTION BETWEEN CONDUIT DEFLECTION FITTING A AND PVC CONDUIT IN STATIONARY FORM BARRIER
SECURE CONDUIT AND BOX TO REBAR TO PREVENT MOVEMENT
BUNDLE R1 #5, R2 #5 OR R9 #5 ADJACENT TO EACH END OF JUNCTION BOX (TYP.)
#4 x 3'-6" (TYP.)
TS
LT
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
2"ø CONDUIT PIPES OR SIZE CONDUIT PER WIRING SCHEDULE
ELEVATION
CONDUITS & J-BOX IN TRAFFIC BARRIER
LABEL JUNCTION BOX COVER IN ACCORDANCE WITH STANDARD SPECIFICATION 9-29.2(4). 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.
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
BONDING JUMPER
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 AND SHALL BE IN NEUTRAL STATE AFTER INSTALLATION.
½" PREMOLDED JOINT FILLER SEE STANDARD PLANS
CONCRETE
1'-0"
TIES @ 2½"
CONDUIT PIPE
CONDUIT DEFLECTOR 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 TRAFFIC BARRIER SHALL BE FITTED WITH DEFLECTION FITTINGS AT A MAXIMUM SPACING OF 120'. THE DEFLECTION FITTINGS SHALL BE PLACED AT THE TRAFFIC BARRIER OPEN JOINT THAT COINCIDES WITH THE RETAINING WALL EXPANSION JOINT.
2 ~ STAINLESS STEEL MOUNTING TABS (TOP & BOT.)
*8" x 8" x 18" JUNCTION BOX
BUNDLE 2 R2 #5 OR 2 R9 #5 ADJACENT TO EACH END OF JUNCTION BOX (TYP.)
⅜"ø POLYETHYLENE OR COPPER PIPE DRAIN
2'-0"
SECTION A
M:\STANDARDS\Traffic Barriers\Shape F\42IN SHAPE F BARRIER SHT 3.MAN
Bridge Design Engr.
Supervisor
Designed By
Checked By
Detailed By
Bridge Projects Engr.
Prelim. Plan By
Architect/Specialist
DATE
REVISION
BY
APP'D
STATE
FED. AID PROJ. NO.
10
WASH.
JOB NUMBER
BRIDGE AND STRUCTURES OFFICE
Washington State Department of Transportation
STANDARD TRAFFIC BARRIER
TRAFFIC BARRIER - SHAPE F 42"
DETAIL 3 OF 3
Wed Feb 19 13:29:02 2014



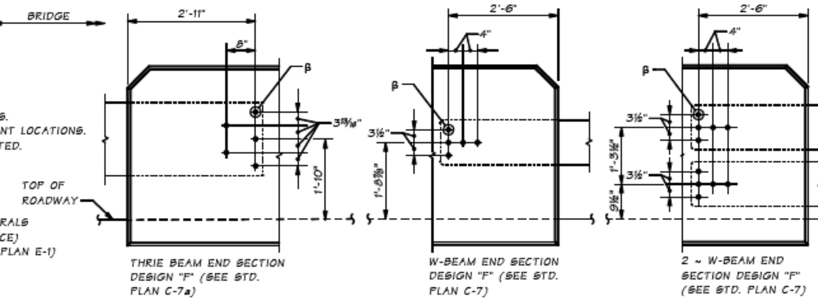
PLAN
TRAFFIC BARRIER

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.



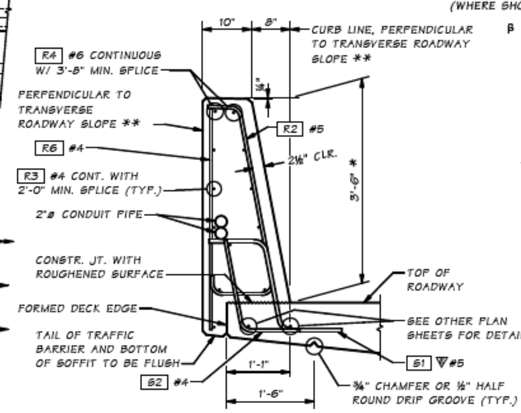
OUTSIDE ELEVATION
END OF TRAFFIC BARRIER

SHOWN WITH BRIDGE APPROACH SLAB



OUTSIDE ELEVATION
TRAFFIC BARRIER - GUARDRAIL CONNECTION

(WHERE SHOWN ON LAYOUT)



TYPICAL SECTION
TRAFFIC BARRIER

SHOWN ON BRIDGE

β PROVIDE ⅝"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.

* 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.
3. General Notes or traffic barrier details shall include the test level (TL-4) of the barrier.

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	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					

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BRIDGE AND STRUCTURES OFFICE

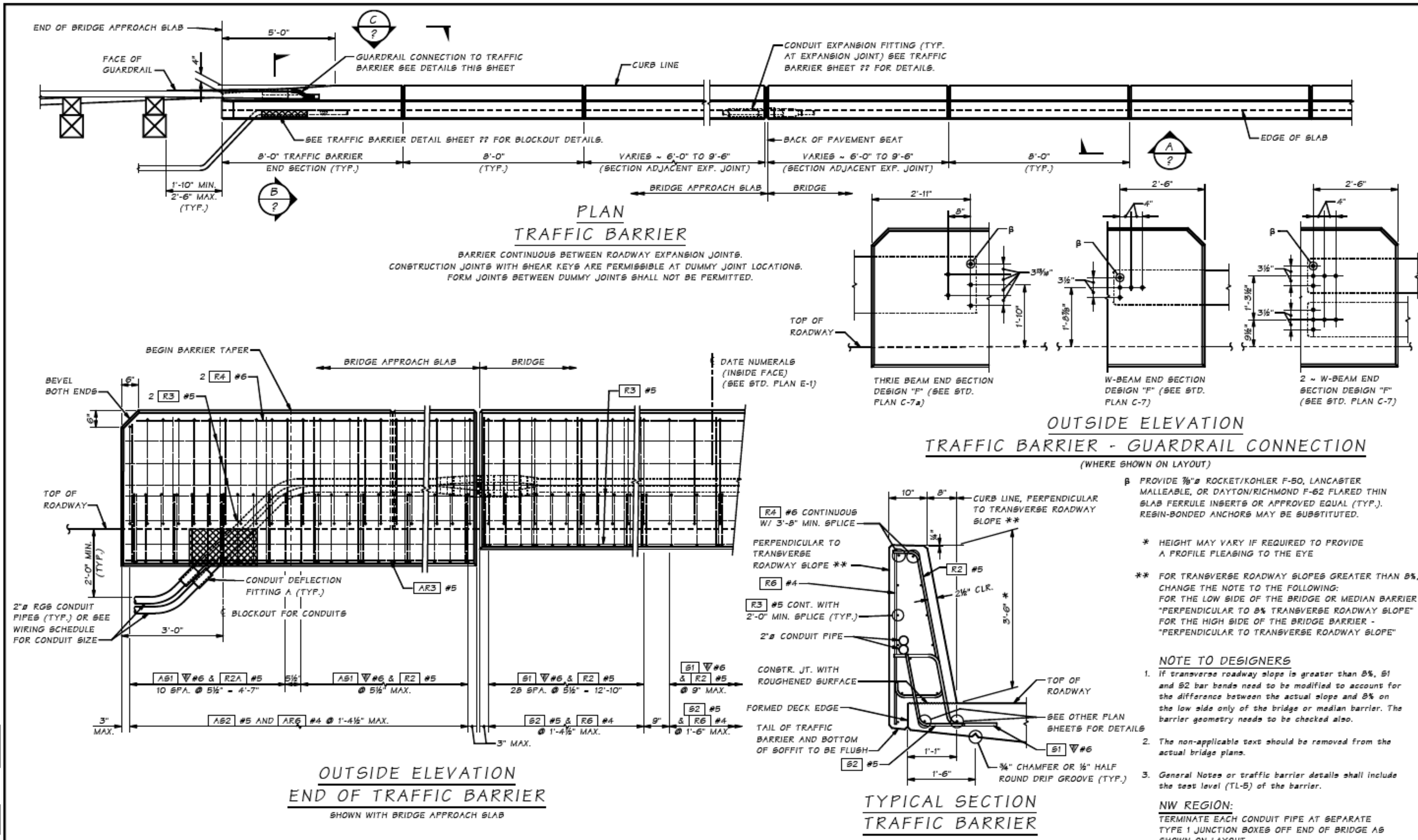
Washington State Department of Transportation

STANDARD TRAFFIC BARRIERS

TRAFFIC BARRIER - SINGLE SLOPE 42" DETAILS 1 OF 3

BRIDGE SHEET NO. / SHEET OF SHEETS

JOB NO.
SR
SHEET



β PROVIDE ⅝"⌀ 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.

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** 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.
3. General Notes or traffic barrier details shall include the test level (TL-5) of the barrier.

NW REGION:
TERMINATE EACH CONDUIT PIPE AT SEPARATE TYPE 1 JUNCTION BOXES OFF END OF BRIDGE AS SHOWN ON LAYOUT.

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REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
10	WASH.			

JOB NUMBER

DATE | REVISION | BY | APP'D

BRIDGE AND STRUCTURES OFFICE

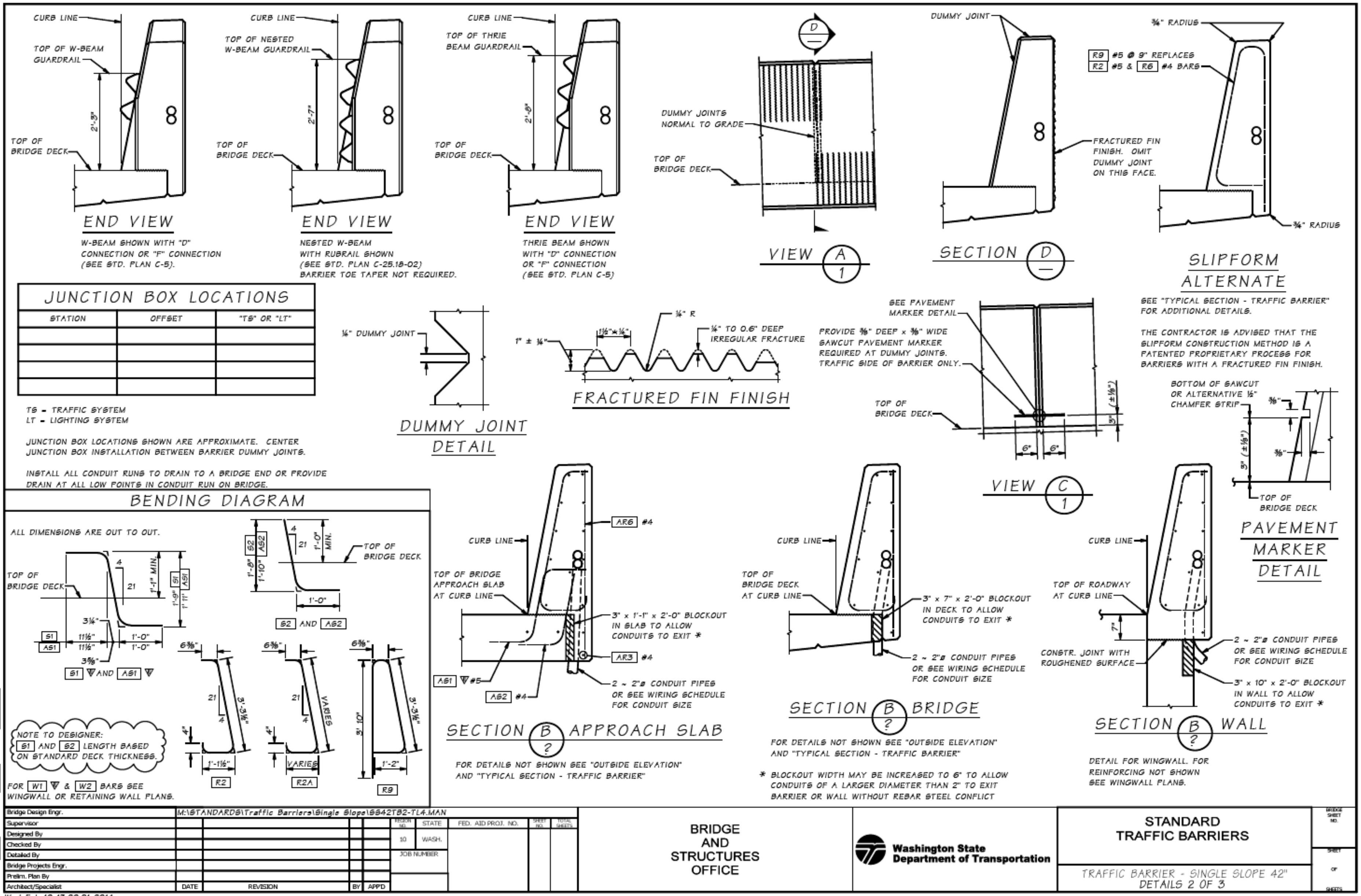
Washington State Department of Transportation

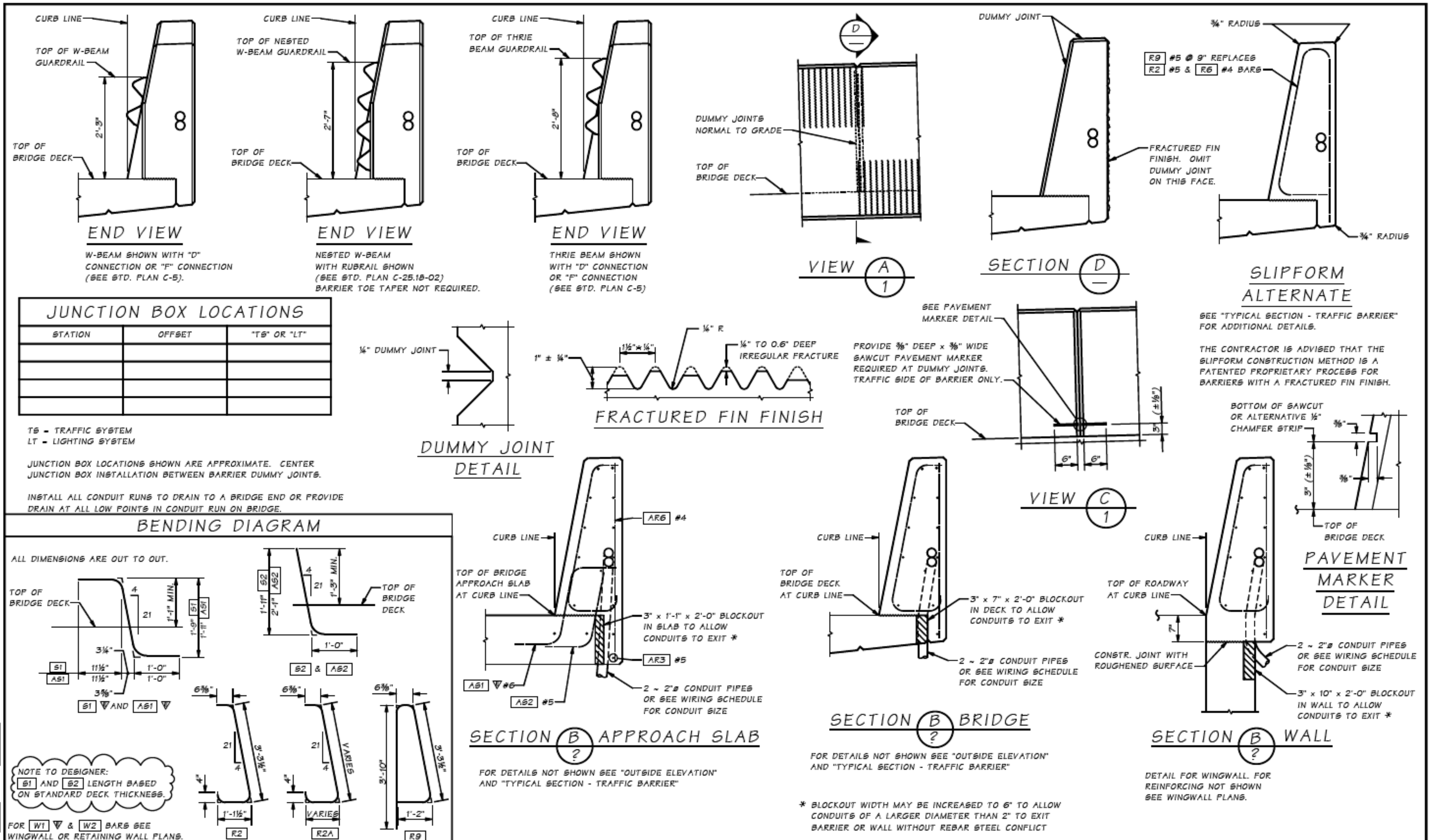
STANDARD TRAFFIC BARRIERS

TRAFFIC BARRIER - SINGLE SLOPE 42" DETAILS 1 OF 3

BRIDGE SHEET NO. / SHEET / OF / SHEETS

Wed Feb 19 13:29:02 2014


END VIEW
W-BEAM SHOWN WITH "D" CONNECTION OR "F" CONNECTION (SEE STD. PLAN C-5).
END VIEW
NESTED W-BEAM WITH RUBRAIL SHOWN (SEE STD. PLAN C-25.18-02) BARRIER TOE TAPER NOT REQUIRED.
END VIEW
THRIE BEAM SHOWN WITH "D" CONNECTION OR "F" CONNECTION (SEE STD. PLAN C-5)
CURB LINE
TOP OF W-BEAM GUARDRAIL
TOP OF NESTED W-BEAM GUARDRAIL
TOP OF THRIE BEAM GUARDRAIL
TOP OF BRIDGE DECK
2'-3"
2'-7"
2'-8"
VIEW A 1
DUMMY JOINTS NORMAL TO GRADE
SECTION D
DUMMY JOINT
SLIPFORM ALTERNATE
¾" RADIUS
R9 #5 @ 9" REPLACES R2 #5 & R6 #4 BARS
FRACTURED FIN FINISH. OMIT DUMMY JOINT ON THIS FACE.
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
¼" DUMMY JOINT
FRACTURED FIN FINISH
¼" R
¼" TO 0.6" DEEP IRREGULAR FRACTURE
1" ± ¼"
1½" ± ¼"
SEE PAVEMENT MARKER DETAIL
PROVIDE ⅜" DEEP x ⅜" WIDE SAWCUT PAVEMENT MARKER REQUIRED AT DUMMY JOINTS. TRAFFIC SIDE OF BARRIER ONLY.
VIEW C 1
6" 6"
PAVEMENT MARKER DETAIL
BOTTOM OF SAWCUT OR ALTERNATIVE ½" CHAMFER STRIP
⅜"
3" (±⅛")
BENDING DIAGRAM
ALL DIMENSIONS ARE OUT TO OUT.
S1 AND AS1
S2 AND AS2
R2
R2A
R9
3'-3½"
VARIES
3'-10"
1'-1½"
1'-2"
NOTE TO DESIGNER: S1 AND S2 LENGTH BASED ON STANDARD DECK THICKNESS.
FOR W1 & W2 BARS SEE WINGWALL OR RETAINING WALL PLANS.
SECTION B ? APPROACH SLAB
TOP OF BRIDGE APPROACH SLAB AT CURB LINE
AR6 #4
AR3 #4
AS1 #5
AS2 #4
3" x 1'-1" x 2'-0" BLOCKOUT IN SLAB TO ALLOW CONDUITS TO EXIT *
2 ~ 2"ø CONDUIT PIPES OR SEE WIRING SCHEDULE FOR CONDUIT SIZE
FOR DETAILS NOT SHOWN SEE "OUTSIDE ELEVATION" AND "TYPICAL SECTION - TRAFFIC BARRIER"
SECTION B ? BRIDGE
TOP OF BRIDGE DECK AT CURB LINE
3" x 7" x 2'-0" BLOCKOUT IN DECK TO ALLOW CONDUITS TO EXIT *
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
TOP OF ROADWAY AT CURB LINE
CONSTR. JOINT WITH ROUGHENED SURFACE
3" x 10" x 2'-0" BLOCKOUT IN WALL TO ALLOW CONDUITS TO EXIT *
DETAIL FOR WINGWALL. FOR REINFORCING NOT SHOWN SEE WINGWALL PLANS.
M:\STANDARDS\Traffic Barriers\Single Slope\SS42TB2-TL4.MAN
10 WASH.
BRIDGE AND STRUCTURES OFFICE
Washington State Department of Transportation
STANDARD TRAFFIC BARRIERS
TRAFFIC BARRIER - SINGLE SLOPE 42" DETAILS 2 OF 3
Wed Feb 19 13:29:01 2014



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.

M:\STANDARDS\Traffic Barriers\Single Slope\SS42TB2-TL5.MAN

Bridge Design Engr.			
Supervisor			
Designed By			
Checked By			
Detailed By			
Bridge Projects Engr.			
Prelim. Plan By			
Architect/Specialist	DATE	REVISION	BY APP'D

REGION NO. 10 | STATE WASH. | FED. AID PROJ. NO. | SHEET NO. | TOTAL SHEETS | JOB NUMBER

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

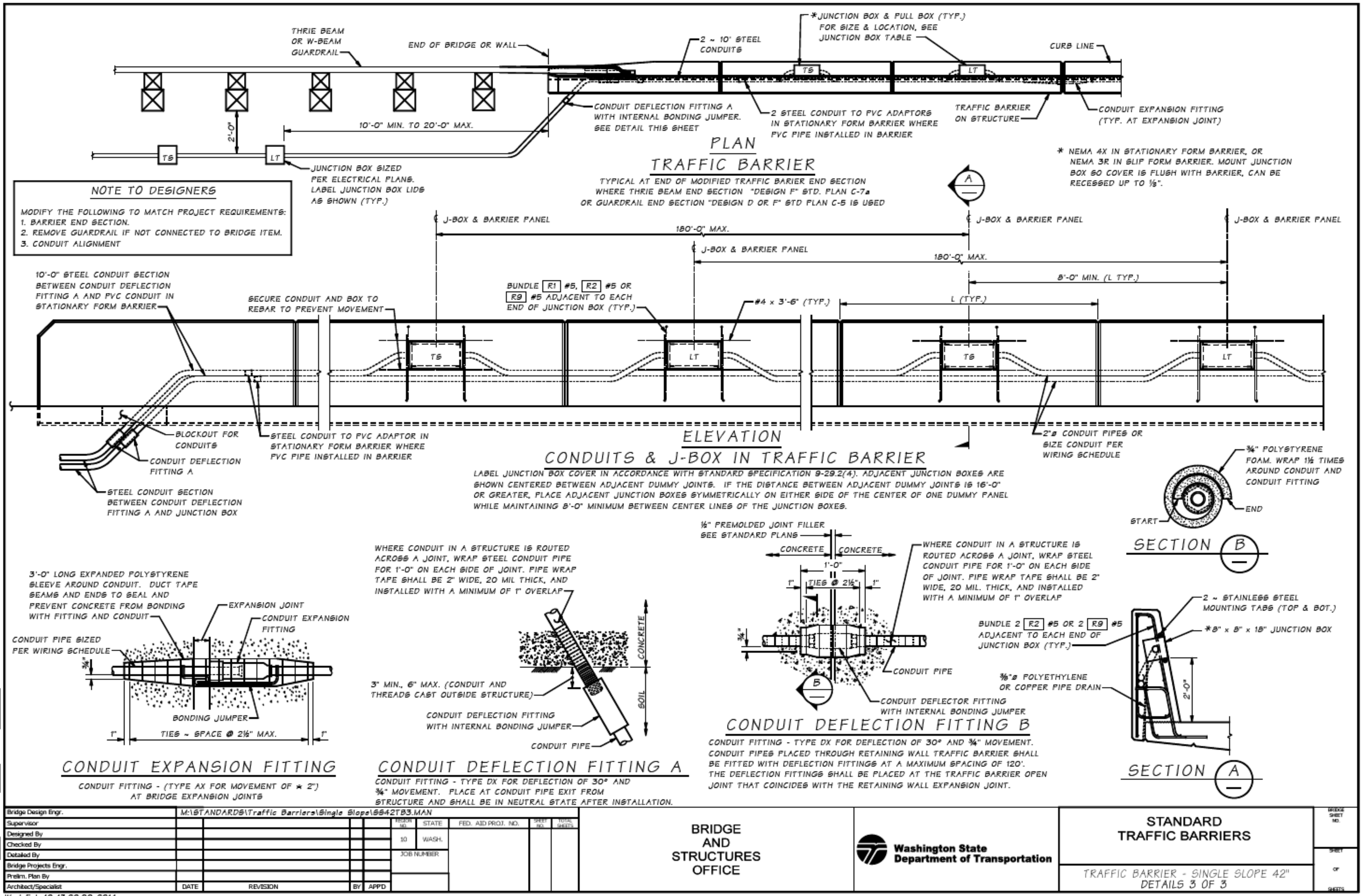
STANDARD TRAFFIC BARRIERS

TRAFFIC BARRIER - SINGLE SLOPE 42" DETAILS 2 OF 3

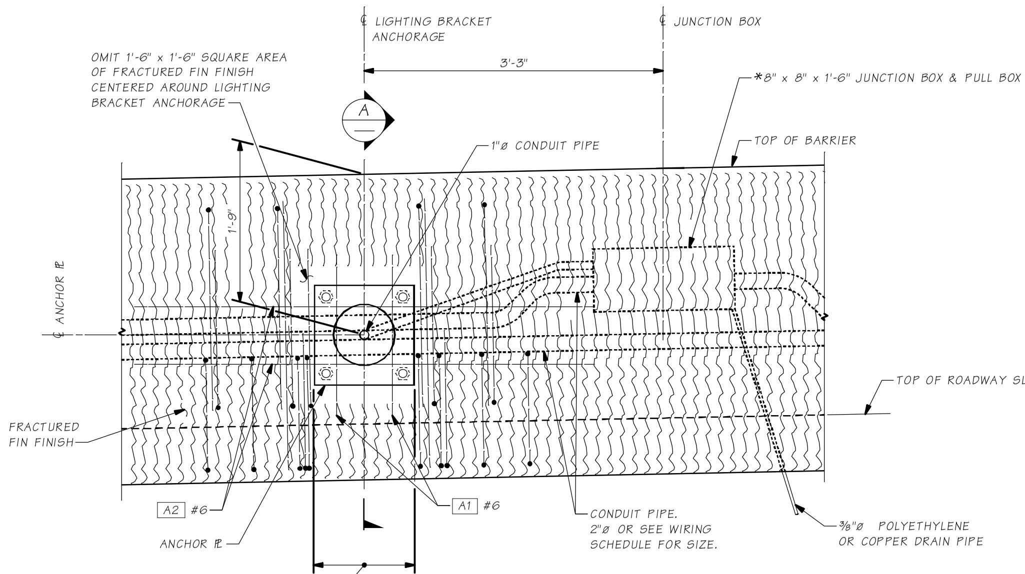
BRIDGE SHEET NO. | SHEET OF SHEETS

Wed Feb 19 13:29:01 2014

JOB NO. SR SHEET

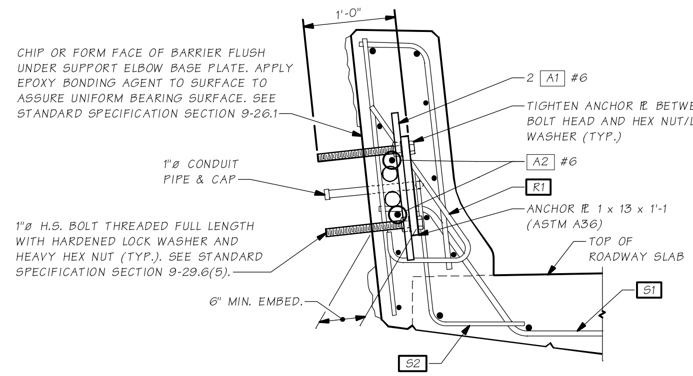

THRIE BEAM OR W-BEAM GUARDRAIL
END OF BRIDGE OR WALL
2 ~ 10' STEEL CONDUITS
*JUNCTION BOX & PULL BOX (TYP.) FOR SIZE & LOCATION, SEE JUNCTION BOX TABLE
CURB LINE
TS
LT
2'-0"
10'-0" MIN. TO 20'-0" MAX.
CONDUIT DEFLECTION FITTING A WITH INTERNAL BONDING JUMPER. SEE DETAIL THIS SHEET
2 STEEL CONDUIT TO PVC ADAPTORS IN STATIONARY FORM BARRIER WHERE PVC PIPE INSTALLED IN BARRIER
TRAFFIC BARRIER ON STRUCTURE
CONDUIT EXPANSION FITTING (TYP. AT EXPANSION JOINT)
JUNCTION BOX SIZED PER ELECTRICAL PLANS. LABEL JUNCTION BOX LIDS AS SHOWN (TYP.)
PLAN
TRAFFIC BARRIER
TYPICAL AT END OF MODIFIED TRAFFIC BARIER 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
* NEMA 4X IN STATIONARY FORM BARRIER, OR NEMA 3R IN SLIP FORM BARRIER. MOUNT JUNCTION BOX SO COVER IS FLUSH WITH BARRIER, CAN BE RECESSED UP TO ⅛".
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.)
10'-0" STEEL CONDUIT SECTION BETWEEN CONDUIT DEFLECTION FITTING A AND PVC CONDUIT IN STATIONARY FORM BARRIER
SECURE CONDUIT AND BOX TO REBAR TO PREVENT MOVEMENT
BUNDLE R1 #5, R2 #5 OR R9 #5 ADJACENT TO EACH END OF JUNCTION BOX (TYP.)
#4 x 3'-6" (TYP.)
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
2"ø CONDUIT PIPES OR SIZE CONDUIT PER WIRING SCHEDULE
ELEVATION
CONDUITS & J-BOX IN TRAFFIC BARRIER
LABEL JUNCTION BOX COVER IN ACCORDANCE WITH STANDARD SPECIFICATION 9-29.2(4). 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.
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 AND SHALL BE IN NEUTRAL STATE AFTER INSTALLATION.
½" PREMOLDED JOINT FILLER SEE STANDARD PLANS
CONCRETE
1'-0"
TIES @ 2½"
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 DEFLECTOR 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 TRAFFIC BARRIER SHALL BE FITTED WITH DEFLECTION FITTINGS AT A MAXIMUM SPACING OF 120'. THE DEFLECTION FITTINGS SHALL BE PLACED AT THE TRAFFIC BARRIER OPEN JOINT THAT COINCIDES WITH THE RETAINING WALL EXPANSION JOINT.
2 ~ STAINLESS STEEL MOUNTING TABS (TOP & BOT.)
BUNDLE 2 R2 #5 OR 2 R9 #5 ADJACENT TO EACH END OF JUNCTION BOX (TYP.)
*8" x 8" x 18" JUNCTION BOX
⅝"ø POLYETHYLENE OR COPPER PIPE DRAIN
2'-0"
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\Single Slope\SS42TB3.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
TRAFFIC BARRIER - SINGLE SLOPE 42" DETAILS 3 OF 3
BRIDGE SHEET NO.
SHEET
OF
SHEETS
Wed Feb 19 13:29:00 2014

ANCHORAGE BARLIST			
MARK #	SIZE	LENGTH	BEND TYPE
A1	6	1'-9"	STRAIGHT
A2	6	5'-0"	STRAIGHT



ELEVATION

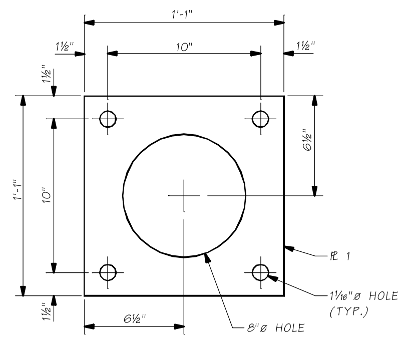
* NEMA 4X IN STATIONARY FORM BARRIER, OR NEMA 3R IN SLIP FORM BARRIER. MOUNT JUNCTION BOX SO COVER IS FLUSH WITH BARRIER, CAN BE RECESSED UP TO ⅛".



SECTION A

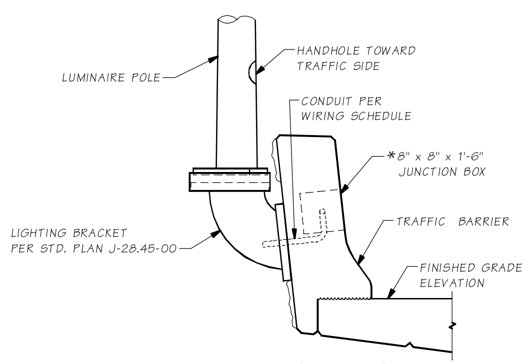
SEE TRAFFIC BARRIER SHEETS FOR INFORMATION NOT SHOWN.

NOTE:
INSTALL ALL CONDUIT RUNS SO THEY DRAIN TO A BRIDGE END OR PROVIDE DRAIN AT ALL LOW POINTS WITHIN CONDUIT RUN. SEE "TRAFFIC BARRIER SHEETS" FOR BARRIER DIMENSIONS.



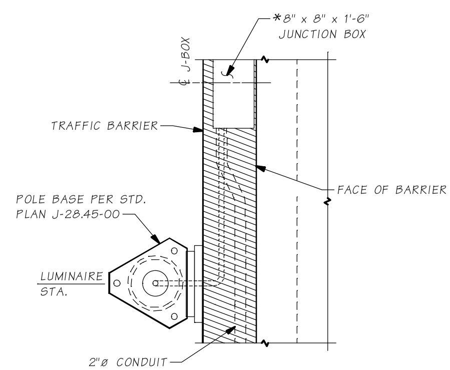
ANCHOR PLATE

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.

Bridge Design Engr.	M:\STANDARDS\Traffic Barriers\Luminare\LUMANCHORAGE-TBF.man							
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Bridge Projects Engr.								
Prelim. Plan By								
Architect/Specialist	DATE	REVISION	BY	APP'D				

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

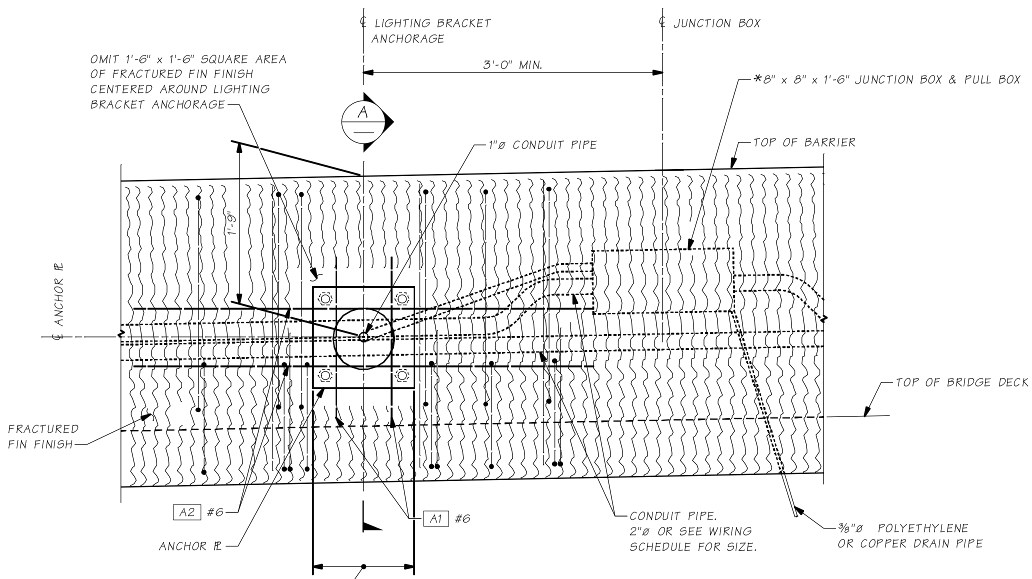
STANDARD TRAFFIC BARRIERS

TRAFFIC BARRIER - SHAPE F
LUMINAIRE ANCHORAGE DEATILS

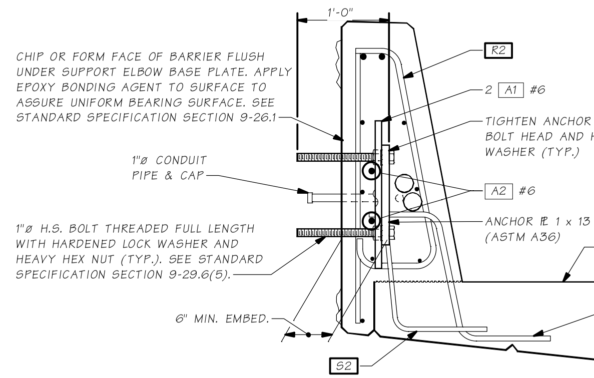
BRIDGE SHEET NO.
SHEET
OF
SHEETS

Thu Dec 19 14:35:35 2013

ANCHORAGE BARLIST			
MARK #	SIZE	LENGTH	BEND TYPE
A1	6	1'-9"	STRAIGHT
A2	6	5'-0"	STRAIGHT



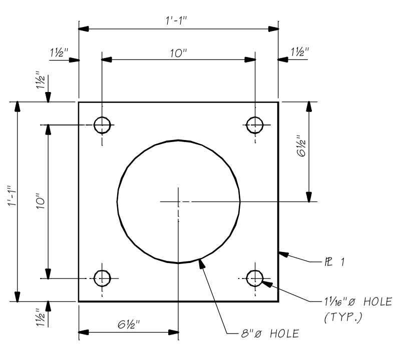
ELEVATION



SECTION A

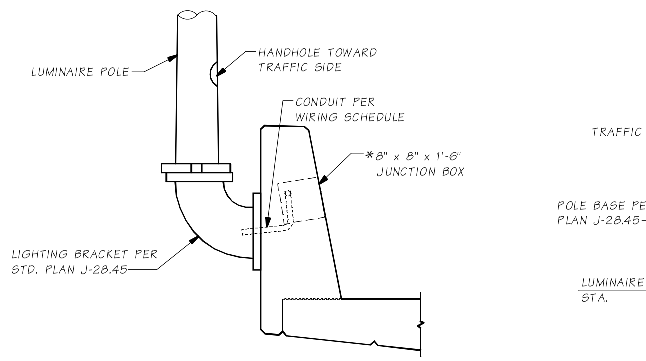
SEE TRAFFIC BARRIER SHEETS FOR INFORMATION NOT SHOWN.

* NEMA 4X IN STATIONARY FORM BARRIER, OR NEMA 3R IN SLIP FORM BARRIER. MOUNT JUNCTION BOX SO COVER IS FLUSH WITH BARRIER, CAN BE RECESSED UP TO ⅛".



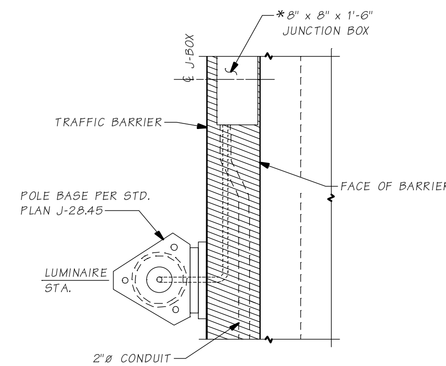
ANCHOR PLATE

GALVANIZE PER AASHTO M 111



ELEVATION PROFILE
LUMINAIRE SUPPORT

SHOWN FOR BRIDGE, WALLS ARE SIMILAR



PLAN

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.

LUMINAIRE POLE LOCATIONS ON BRIDGE & WALLS	
STATION	OFFSET
XXX 0+000.000	0.000 RT.
XXX 0+000.000	0.000 RT.

LAST REVISED: 7/16/2013

Bridge Design Engr.
Supervisor
Designed By
Checked By
Detailed By
Bridge Projects Engr.
Prelim. Plan By
Architect/Specialist

M:\STANDARDS\Traffic Barriers\Luminare\LUMANCHORAGE-SSTB.MAN

DATE	REVISION	BY	APP'D

REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
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JOB NUMBER

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

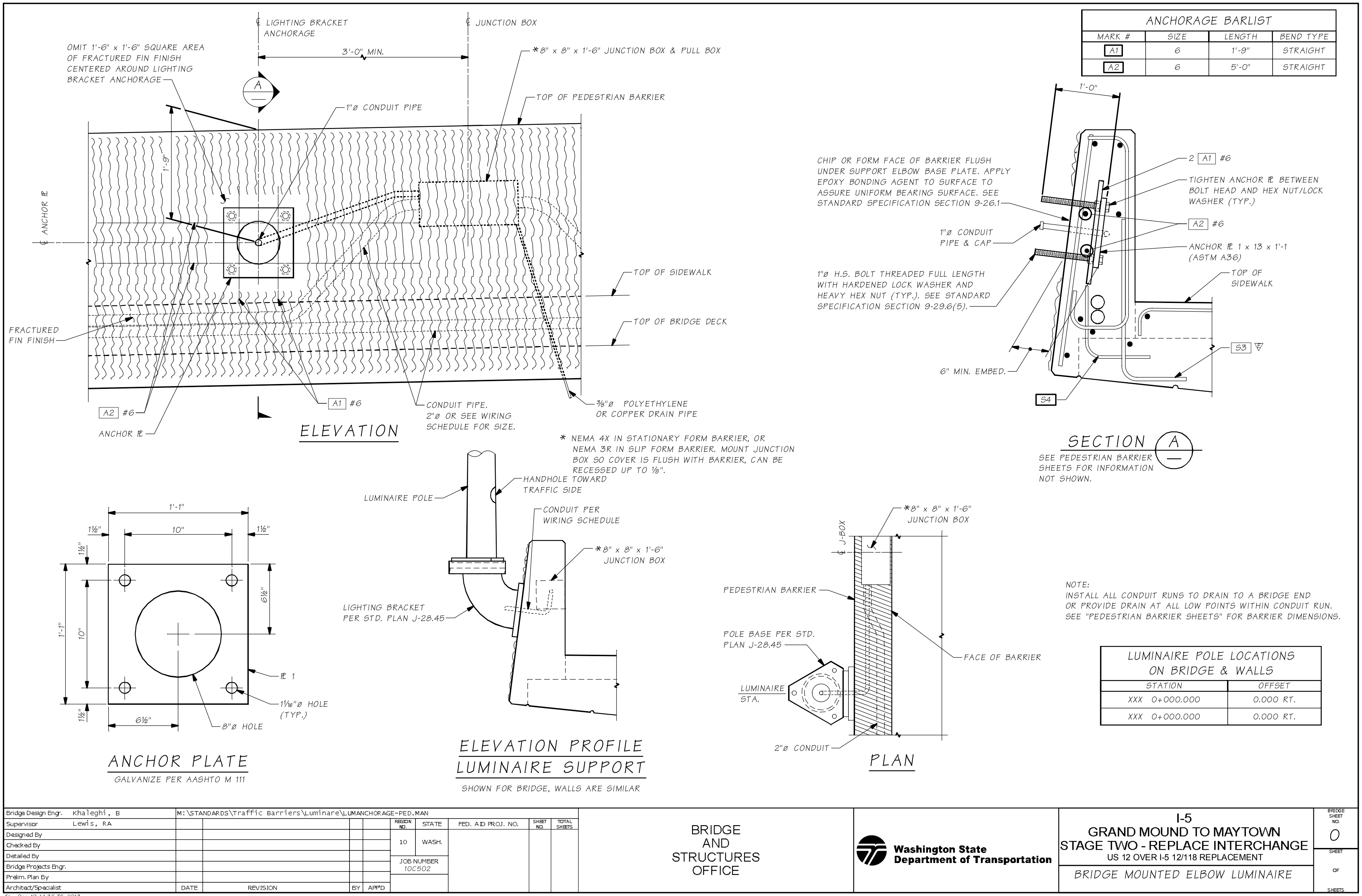
STANDARD TRAFFIC BARRIERS

TRAFFIC BARRIER - SINGLE SLOPE
LUMINAIRE ANCHORAGE DEATILS

BRIDGE SHEET NO.
SHEET OF SHEETS

Thu Dec 19 14:35:35 2013

JOB NO. SR SHEET



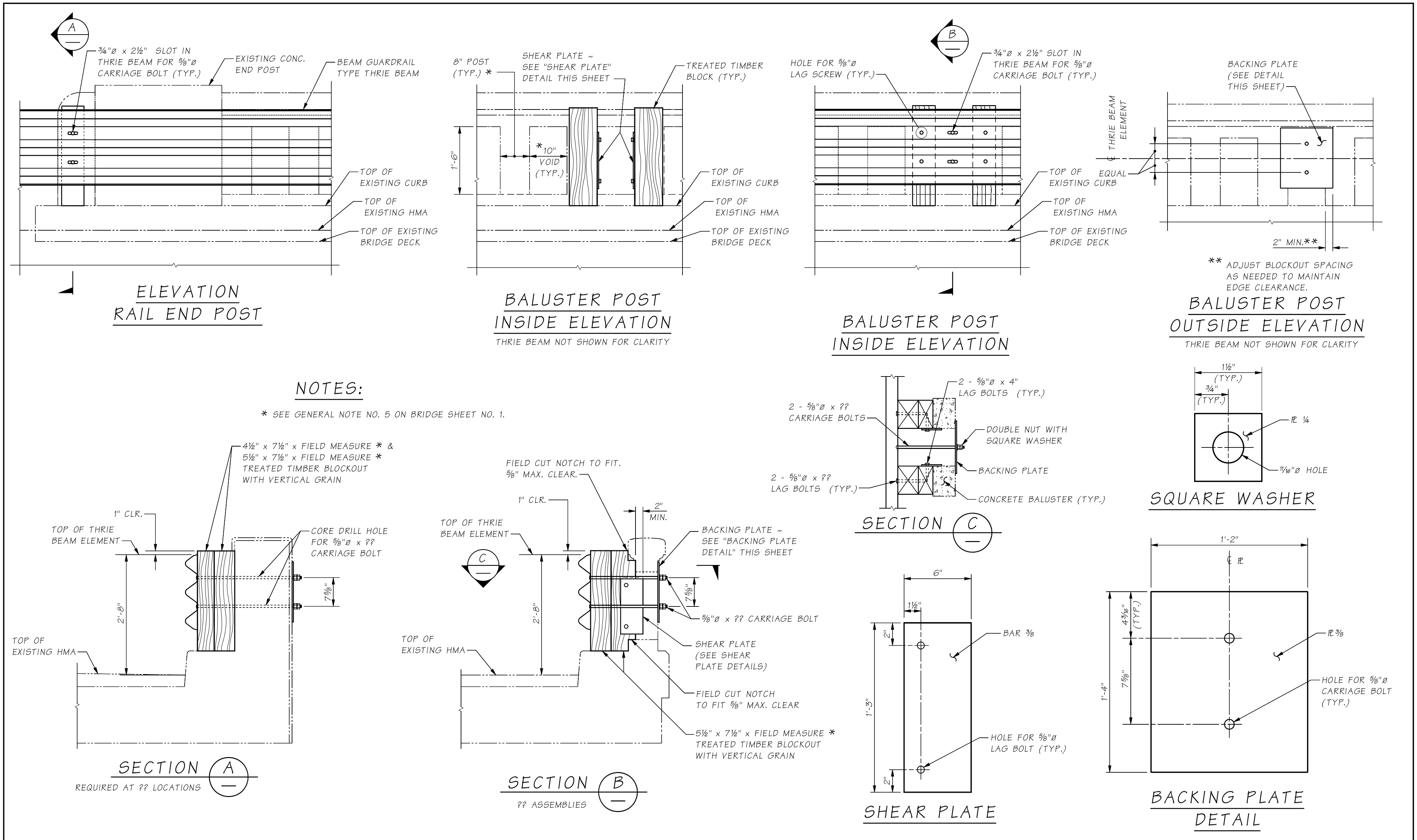
ANCHORAGE BARLIST

MARK #	SIZE	LENGTH	BEND TYPE
A1	6	1'-9"	STRAIGHT
A2	6	5'-0"	STRAIGHT

NOTE:
INSTALL ALL CONDUIT RUNS TO DRAIN TO A BRIDGE END OR PROVIDE DRAIN AT ALL LOW POINTS WITHIN CONDUIT RUN. SEE "PEDESTRIAN BARRIER SHEETS" FOR BARRIER DIMENSIONS.

LUMINAIRE POLE LOCATIONS ON BRIDGE & WALLS

STATION	OFFSET
XXX 0+000.000	0.000 RT.
XXX 0+000.000	0.000 RT.



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REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
10	WASH.			

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BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

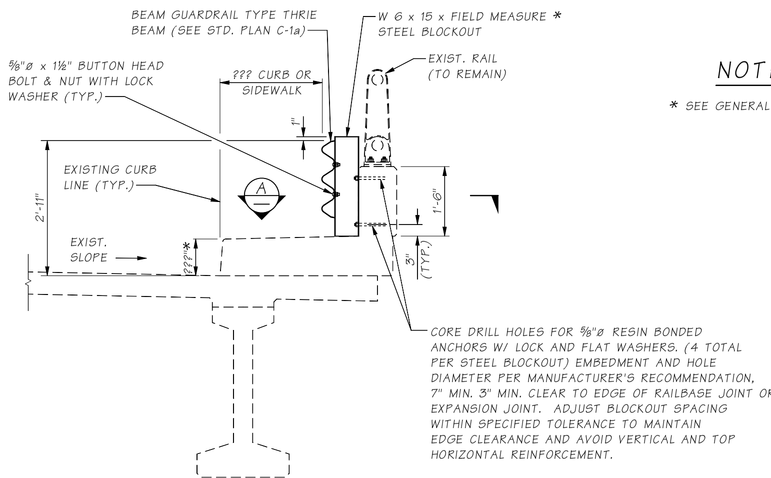
STANDARD RAIL RETROFITS

Thrie Beam Retrofit Concrete Baluster

BRIDGE SHEET NO. 0 — SHEET OF SHEETS

Thu Sep 02 14:04:37 2010

SR ___ JOB NO. ___ SHEET 0

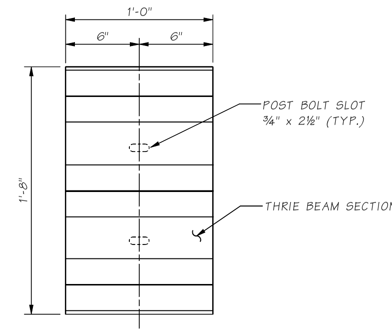


TYPICAL SECTION
AT STEEL BLOCKOUT

(??? ASSEMBLIES REQUIRED)

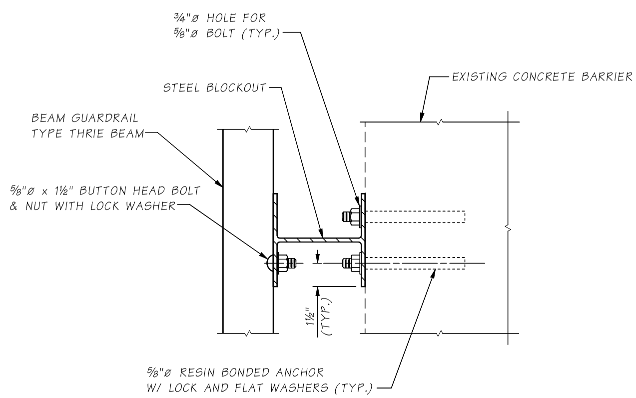
NOTES:

* SEE GENERAL NOTE NO. 5 ON BRIDGE SHEET NO. 1.

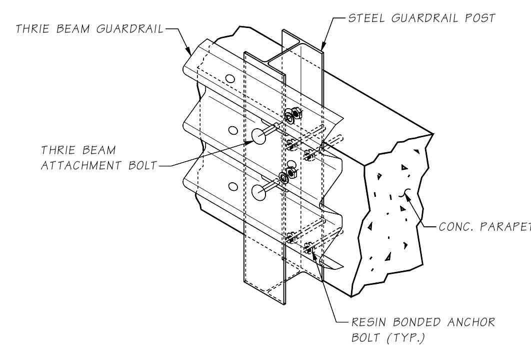


BACKUP PLATE

BACKUP PLATE REQUIRED AT POST WHERE NO THRIE BEAM GUARDRAIL SPLICE OCCURS.



SECTION A



ISOMETRIC VIEW

Bridge Design Engr.	M:\STANDARDS\Traffic Barriers\Rail Retrofit\PARAPET STANDARD.MAN							
Supervisor				REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
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Architect/Specialist	DATE	REVISION	BY	APP'D				

Thu Sep 02 14:04:40 2010

BRIDGE
AND
STRUCTURES
OFFICE

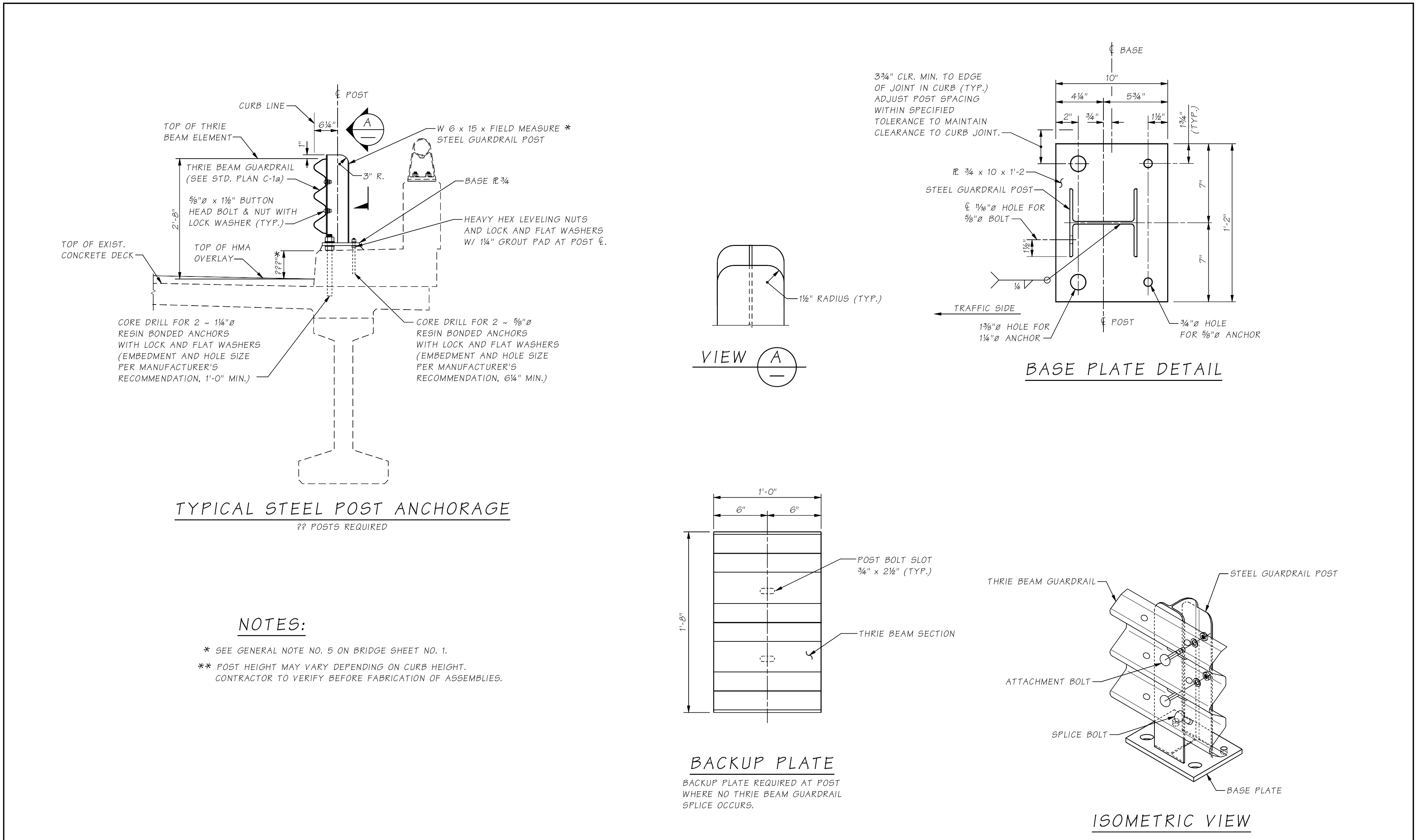
Washington State
Department of Transportation

STANDARD
RAIL RETROFITS

THRIE BEAM RETROFIT CONCRETE RAILBASE

BRIDGE SHEET NO. 0

SHEET OF SHEETS



NOTES:

* SEE GENERAL NOTE NO. 5 ON BRIDGE SHEET NO. 1.

** POST HEIGHT MAY VARY DEPENDING ON CURB HEIGHT. CONTRACTOR TO VERIFY BEFORE FABRICATION OF ASSEMBLIES.

Bridge Design Engr.	M:\STANDARDS\Traffic Barriers\Rail Retrofit\STEEL POST STANDARD.MAN				
Supervisor		REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO. / TOTAL SHEETS
Designed By		10	WASH.		
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Architect/Specialist	DATE / REVISION / BY / APP'D				

Thu Sep 02 14:04:42 2010

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

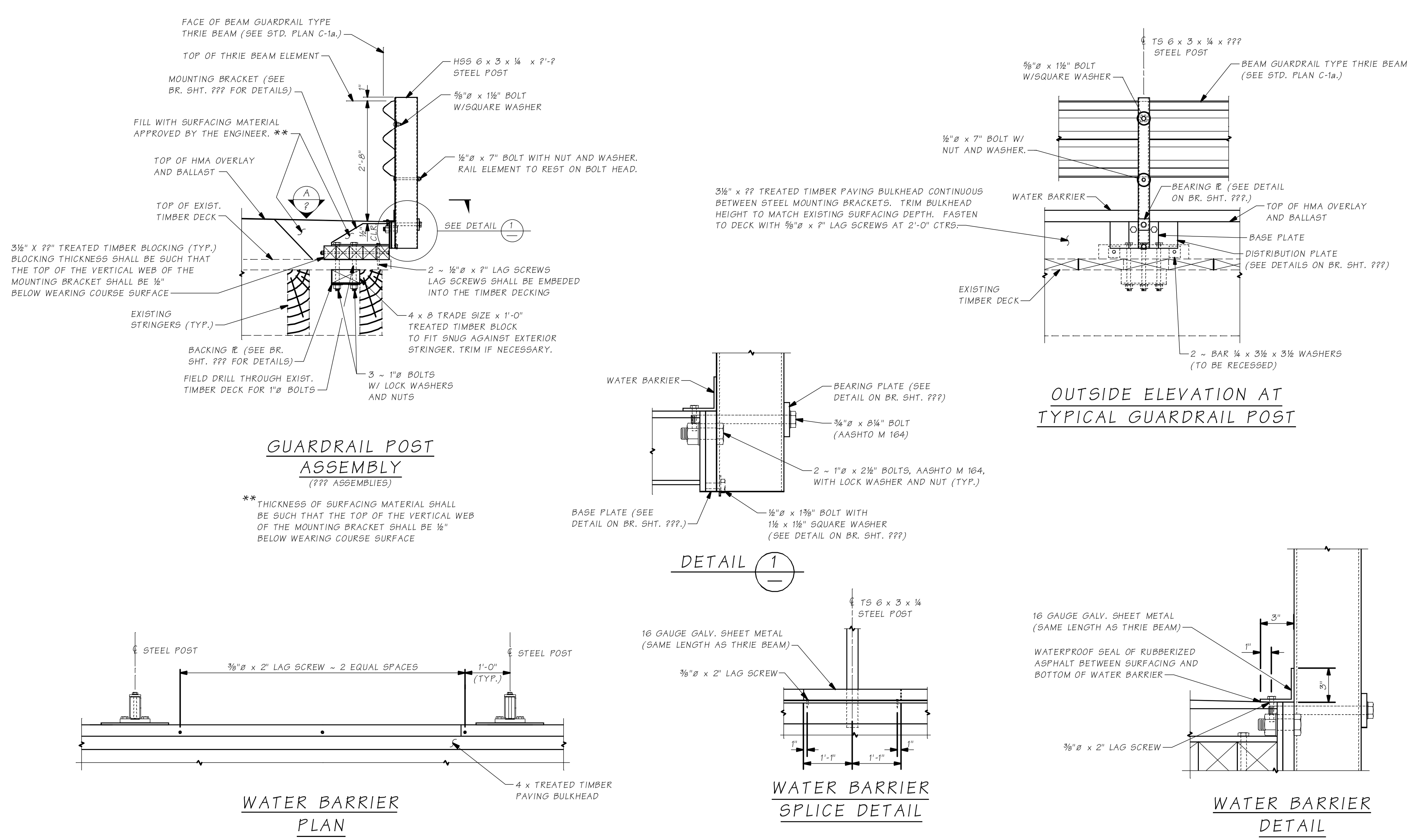
STANDARD RAIL RETROFITS

THRIE BEAM RETROFIT CONCRETE CURB

BRIDGE SHEET NO. 0

SHEET OF SHEETS

SR / JOB NO. / SHEET 0



Bridge Design Engr.	M:\STANDARDS\Traffic Barriers\Rail Retrofit\SL1 - 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					

Thu Dec 19 14:35:35 2013

BRIDGE AND STRUCTURES OFFICE

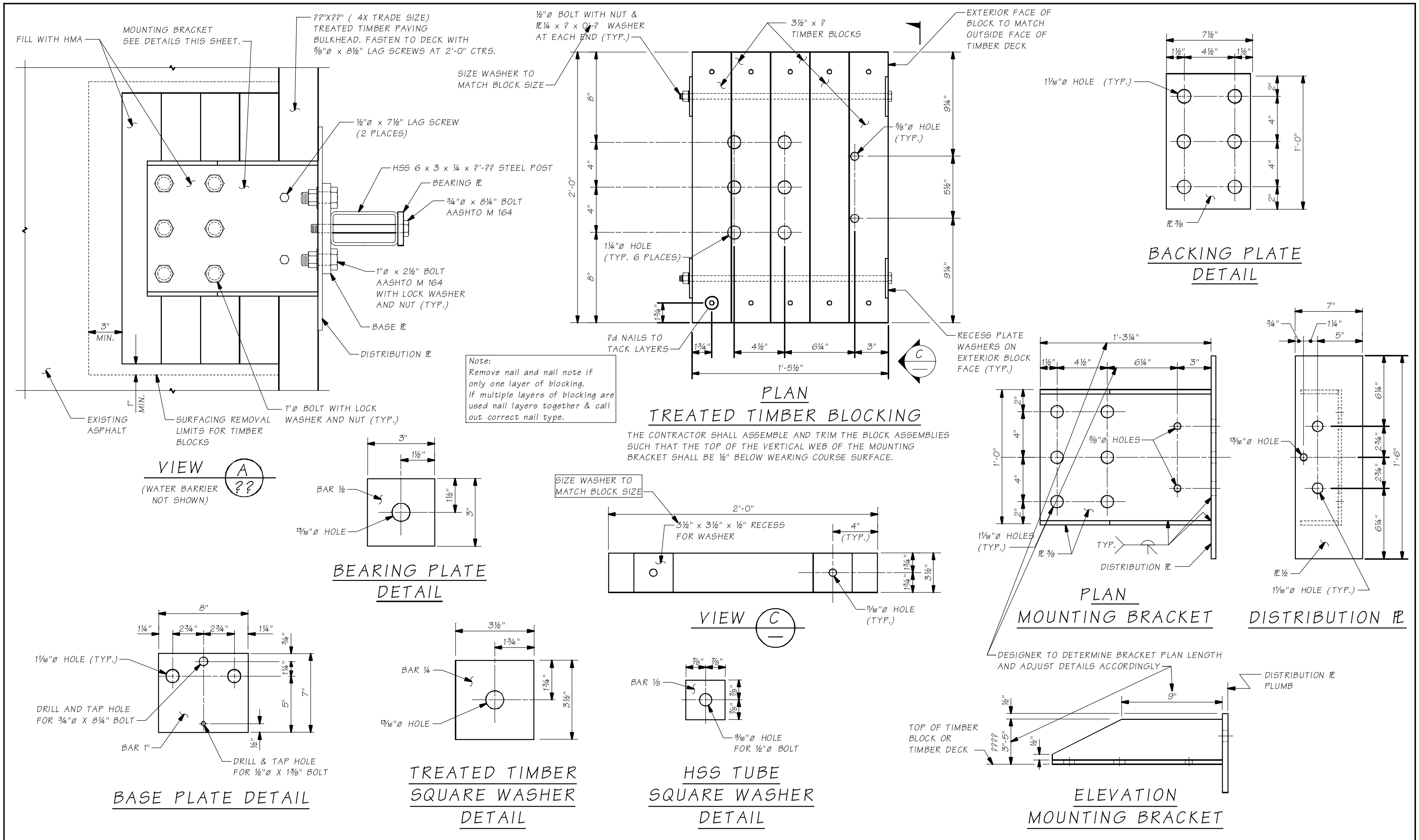
Washington State Department of Transportation

STANDARD RAIL RETROFITS

WP THRIE BEAM RETROFIT
SL1 - DETAILS 1 OF 2

BRIDGE SHEET NO. | SHEET OF SHEETS

SR | JOB NO. | SHEET 0



Bridge Design Engr.	M:\STANDARDS\Traffic Barriers\Rail Retrofit\SL1 - 2.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			

Thu Dec 19 14:35:35 2013

BRIDGE AND STRUCTURES OFFICE

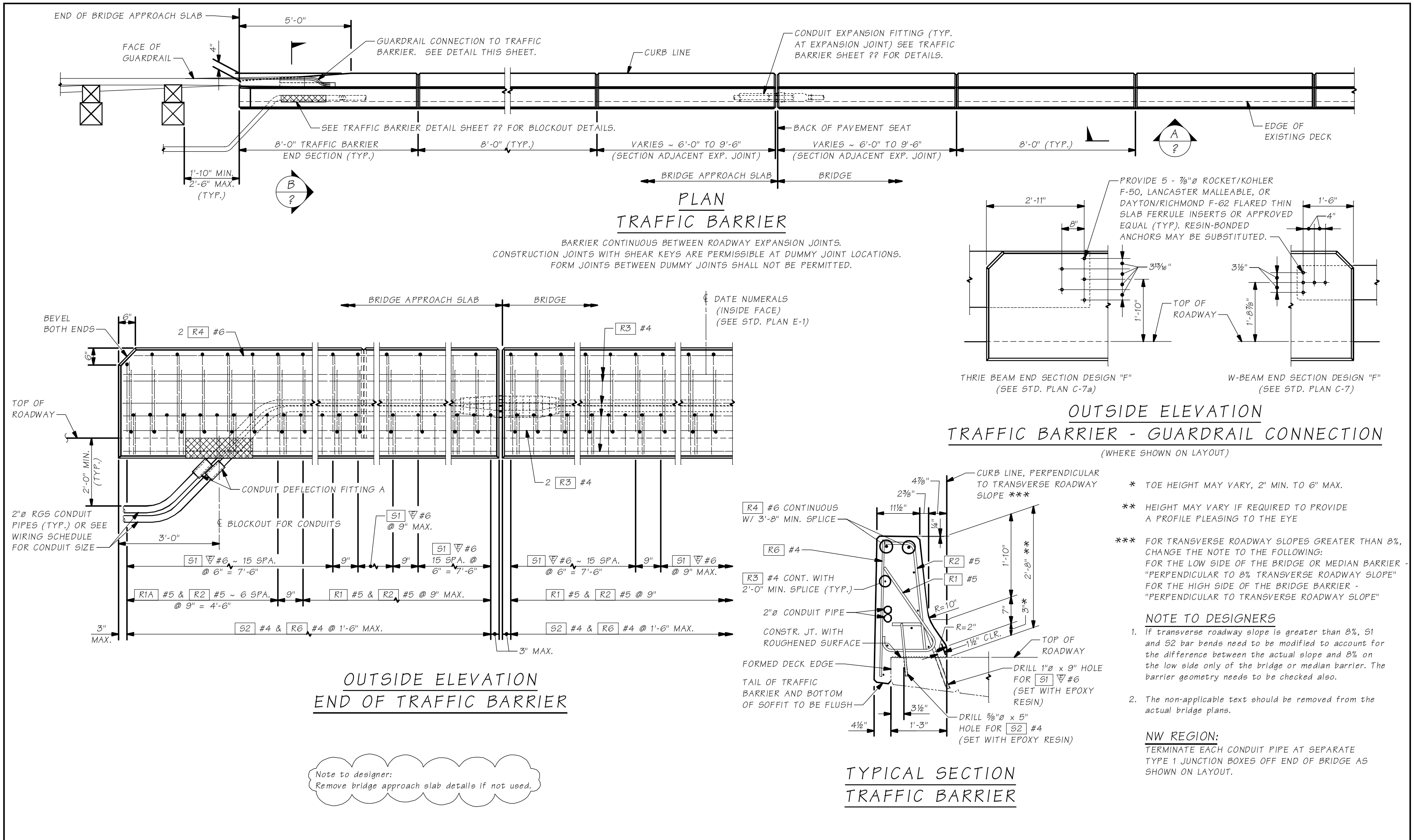
Washington State Department of Transportation

STANDARD RAIL RETROFITS

WP THRIE BEAM RETROFIT
SL1 - DETAILS 2 OF 2

BRIDGE SHEET NO. 0 SHEET OF SHEETS

SR JOB NO. SHEET 0



* 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\Rehab\TB1-R.MAN							
Supervisor				REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
Designed By				10	WASH.			
Checked By				JOB NUMBER				
Detailed By								
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BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

STANDARD TRAFFIC BARRIERS

TRAFFIC BARRIER - SHAPE F REHABILITATION - DETAILS 1 OF 3

BRIDGE SHEET NO.

SHEET OF SHEETS

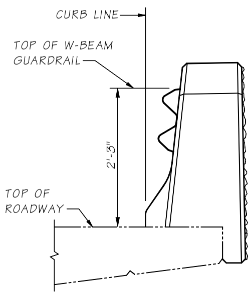
Thu Dec 19 14:35:35 2013

JOB NO.

SR

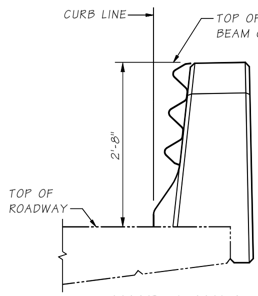
SHEET

Last revised on : 11/30/2012



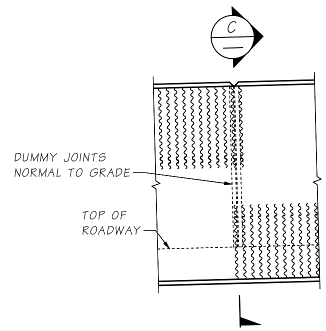
END VIEW

W-BEAM SHOWN WITH "D" CONNECTION

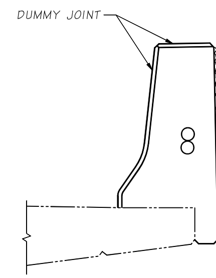


END VIEW

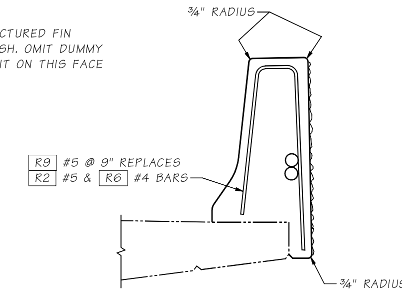
THRIE BEAM SHOWN WITH "D" CONNECTION



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.

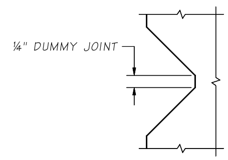
JUNCTION BOX LOCATIONS

STATION	OFFSET	"TS" OR "LT"

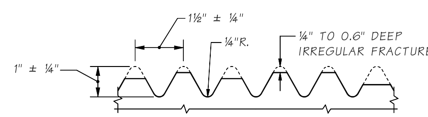
TS = TRAFFIC SYSTEM
LT = LIGHTING SYSTEM

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



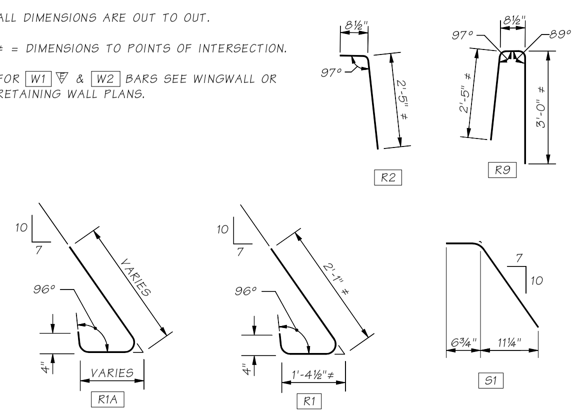
FRACTURED FIN FINISH

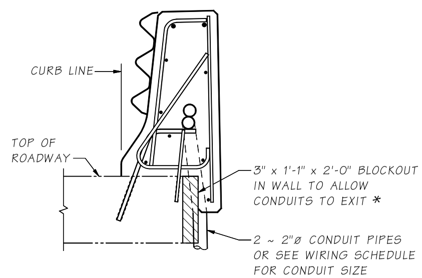
BENDING DIAGRAM

ALL DIMENSIONS ARE OUT TO OUT.

≠ = DIMENSIONS TO POINTS OF INTERSECTION.

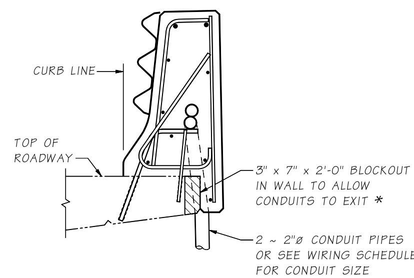
FOR W1 ∇ & W2 BARS SEE WINGWALL OR RETAINING WALL PLANS.





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.

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Architect/Specialist	DATE	REVISION	BY	APP'D	

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10	WASH.			
JOB NUMBER				

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

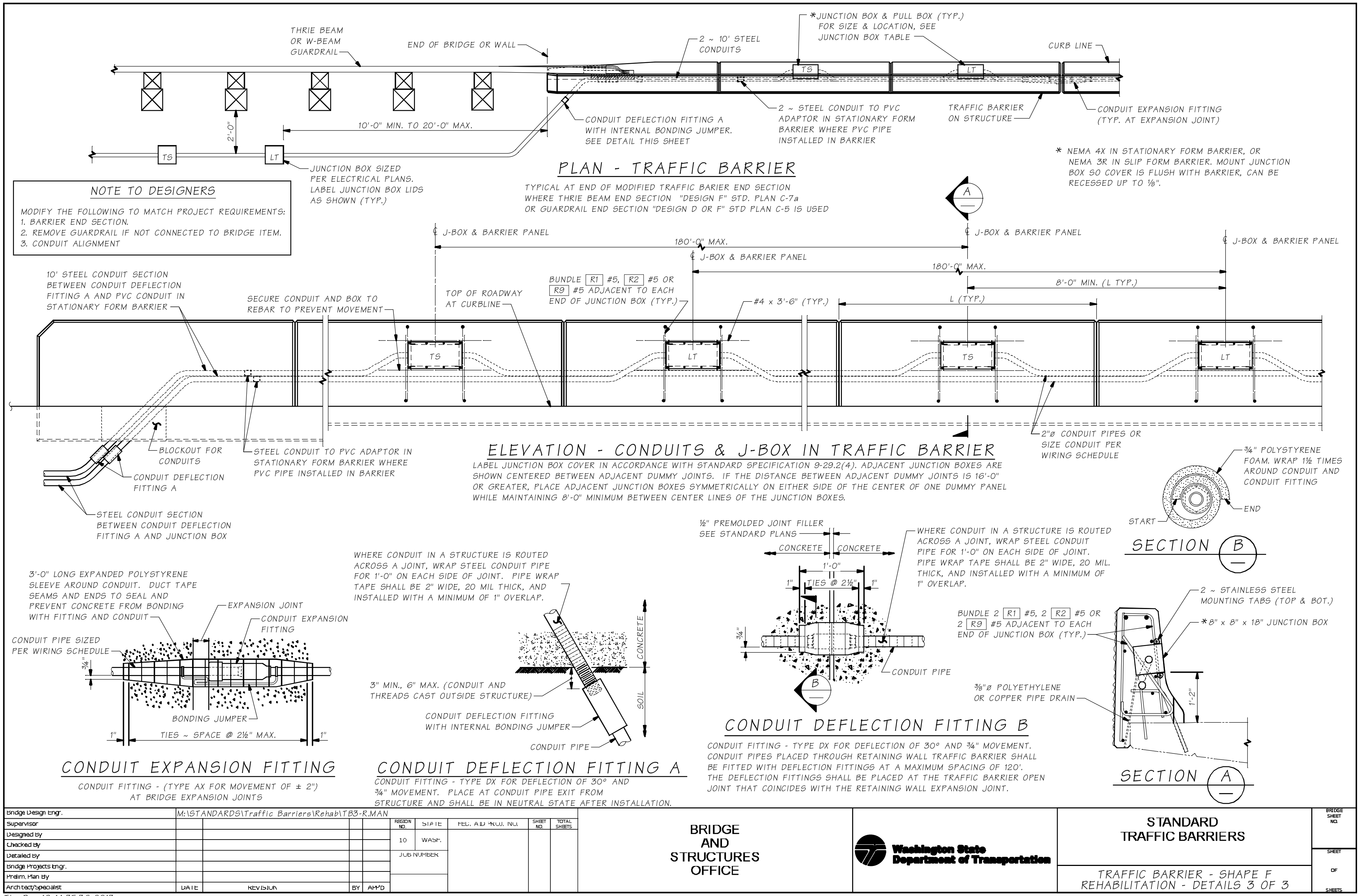
STANDARD TRAFFIC BARRIERS

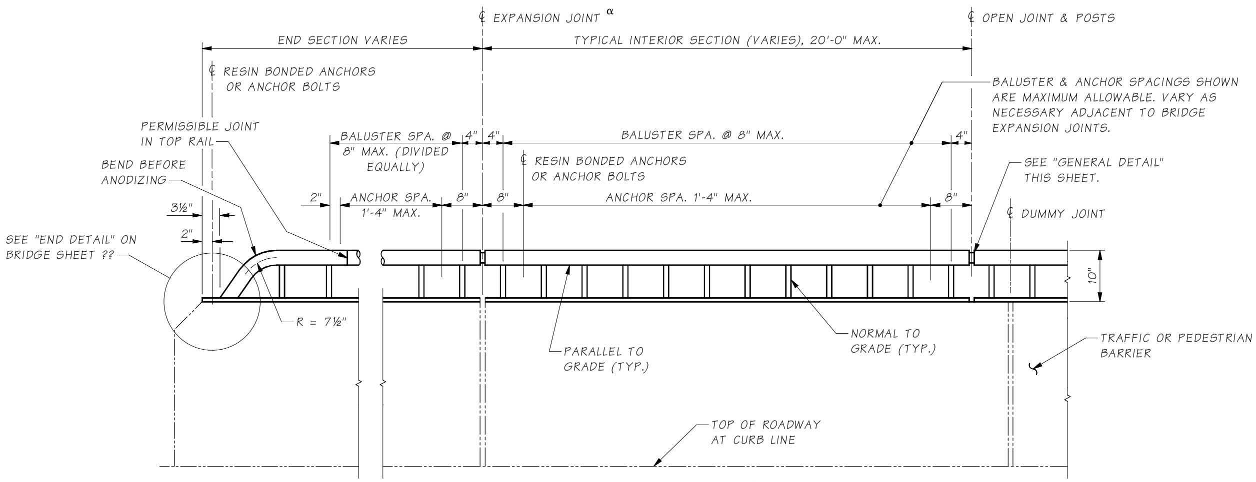
TRAFFIC BARRIER - SHAPE F
REHABILITATION - DETAILS 2 OF 3

BRIDGE SHEET NO.
SHEET
OF
SHEETS

Thu Sep 02 14:04:54 2010

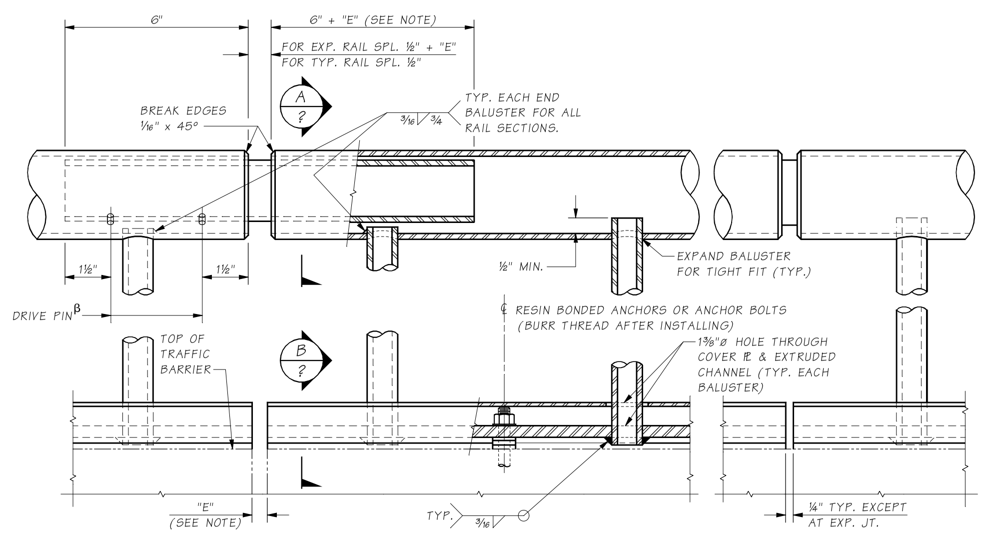
SR
JOB NO.
SHEET


PLAN - TRAFFIC BARRIER
TYPICAL AT END OF MODIFIED TRAFFIC BARIER 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
THRIE BEAM OR W-BEAM GUARDRAIL
END OF BRIDGE OR WALL
2 ~ 10' STEEL CONDUITS
*JUNCTION BOX & PULL BOX (TYP.) FOR SIZE & LOCATION, SEE JUNCTION BOX TABLE
CURB LINE
CONDUIT DEFLECTION FITTING A WITH INTERNAL BONDING JUMPER. SEE DETAIL THIS SHEET
2 ~ STEEL CONDUIT TO PVC ADAPTOR IN STATIONARY FORM BARRIER WHERE PVC PIPE INSTALLED IN BARRIER
TRAFFIC BARRIER ON STRUCTURE
CONDUIT EXPANSION FITTING (TYP. AT EXPANSION JOINT)
* NEMA 4X IN STATIONARY FORM BARRIER, OR NEMA 3R IN SLIP FORM BARRIER. MOUNT JUNCTION BOX SO COVER IS FLUSH WITH BARRIER, CAN BE RECESSED UP TO ⅛".
2'-0"
10'-0" MIN. TO 20'-0" MAX.
JUNCTION BOX SIZED PER ELECTRICAL PLANS. LABEL JUNCTION BOX LIDS AS SHOWN (TYP.)
TS
LT
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
ELEVATION - CONDUITS & J-BOX IN TRAFFIC BARRIER
LABEL JUNCTION BOX COVER IN ACCORDANCE WITH STANDARD SPECIFICATION 9-29.2(4). 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.
℄ J-BOX & BARRIER PANEL
180'-0" MAX.
8'-0" MIN. (L TYP.)
L (TYP.)
10' STEEL CONDUIT SECTION BETWEEN CONDUIT DEFLECTION FITTING A AND PVC CONDUIT IN STATIONARY FORM BARRIER
SECURE CONDUIT AND BOX TO REBAR TO PREVENT MOVEMENT
TOP OF ROADWAY AT CURBLINE
BUNDLE R1 #5, R2 #5 OR R9 #5 ADJACENT TO EACH END OF JUNCTION BOX (TYP.)
#4 x 3'-6" (TYP.)
BLOCKOUT FOR CONDUITS
CONDUIT DEFLECTION FITTING A
STEEL CONDUIT TO PVC ADAPTOR IN STATIONARY FORM BARRIER WHERE PVC PIPE INSTALLED IN BARRIER
STEEL CONDUIT SECTION BETWEEN CONDUIT DEFLECTION FITTING A AND JUNCTION BOX
2"⌀ CONDUIT PIPES OR SIZE CONDUIT PER WIRING SCHEDULE
CONDUIT EXPANSION FITTING
CONDUIT FITTING - (TYPE AX FOR MOVEMENT OF ± 2") AT BRIDGE EXPANSION JOINTS
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
BONDING JUMPER
TIES ~ SPACE @ 2½" MAX.
CONDUIT DEFLECTION FITTING A
CONDUIT FITTING - TYPE DX FOR DEFLECTION OF 30° AND ¾" MOVEMENT. PLACE AT CONDUIT PIPE EXIT FROM STRUCTURE AND SHALL BE IN NEUTRAL STATE AFTER INSTALLATION.
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.
3" MIN., 6" MAX. (CONDUIT AND THREADS CAST OUTSIDE STRUCTURE)
CONDUIT DEFLECTION FITTING WITH INTERNAL BONDING JUMPER
CONDUIT PIPE
CONCRETE
SOIL
CONDUIT DEFLECTION FITTING B
CONDUIT FITTING - TYPE DX FOR DEFLECTION OF 30° AND ¾" MOVEMENT. CONDUIT PIPES PLACED THROUGH RETAINING WALL TRAFFIC BARRIER SHALL BE FITTED WITH DEFLECTION FITTINGS AT A MAXIMUM SPACING OF 120'. THE DEFLECTION FITTINGS SHALL BE PLACED AT THE TRAFFIC BARRIER OPEN JOINT THAT COINCIDES WITH THE RETAINING WALL EXPANSION JOINT.
½" PREMOLDED JOINT FILLER SEE STANDARD PLANS
CONCRETE
CONCRETE
1'-0"
TIES @ 2½"
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
SECTION B
¾" POLYSTYRENE FOAM. WRAP 1½ TIMES AROUND CONDUIT AND CONDUIT FITTING
START
END
SECTION A
2 ~ STAINLESS STEEL MOUNTING TABS (TOP & BOT.)
*8" x 8" x 18" JUNCTION BOX
BUNDLE 2 R1 #5, 2 R2 #5 OR 2 R9 #5 ADJACENT TO EACH END OF JUNCTION BOX (TYP.)
⅜"⌀ POLYETHYLENE OR COPPER PIPE DRAIN
1'-2"
Bridge Design Engr.
Supervisor
Designed By
Checked By
Detailed By
Bridge Projects Engr.
Prelim. Plan By
Architect/Specialist
M:\STANDARDS\Traffic Barriers\Rehab\TB3-R.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 REHABILITATION - DETAILS 3 OF 3
BRIDGE SHEET NO.
SHEET
OF
SHEETS
Thu Dec 19 14:35:36 2013
SR
JOB NO.
SHEET



ELEVATION

BALUSTER AND GUARDRAIL SECTION ATTACHMENT DETAILS NOT SHOWN.



GENERAL DETAIL

NOTES

1. PIPE RAILING, PIPE RAILING SPLICES, COVER PLATES AND BOTTOM EXTRUDED CHANNELS SHALL BE BENT TO THE HORIZONTAL CURVE WHERE THE RADIUS OF CURVATURE IS LESS THAN 200'. THESE ITEMS MAY BE HEATED TO NOT MORE THAN 400°F FOR A PERIOD NOT TO EXCEED 30 MINUTES TO FACILITATE FORMING OR BENDING TO HORIZONTAL CURVATURE.
2. SHOP DRAWINGS OF RAILING SHALL BE SUBMITTED FOR APPROVAL SHOWING COMPLETE DIMENSIONS AND DETAILS OF FABRICATION AND INCLUDING AN ERECTION DIAGRAM. MATERIAL SPECIFICATIONS SHALL BE PROVIDED IN THE SHOP DRAWINGS FOR ALL COMPONENTS.
3. CUTTING SHALL BE DONE BY SAWING OR MILLING AND ALL CUTS SHALL BE TRUE AND SMOOTH. FLAME CUTTING WILL NOT BE PERMITTED.
4. WELDING OF ALUMINUM SHALL CONFORM TO STD. SPEC. SECTION 9-28.14(3).
5. ALL ALUMINUM PARTS SHALL BE GIVEN A *[CLEAR OR BRONZE] ANODIC COATING OF AT LEAST 0.0006" THICK AND SEALED TO MEET THE REQUIREMENTS OF ASTM B 580 WITH A UNIFORM FINISH.
6. PIPE RAILING, PIPE BALUSTERS AND PIPE RAILING SPLICES SHALL BE ADEQUATELY WRAPPED TO INSURE SURFACE PROTECTION DURING HANDLING AND TRANSPORTATION TO THE JOB SITE.

* NOTE TO DESIGNER:
Designer to choose color for their project in consultation with the Bridge Architect.

α PROVIDE EXPANSION RAIL SPLICE AT TRAFFIC BARRIER EXPANSION JOINTS. RAIL SPLICE JOINTS ARE NO REQUIRED AT TRAFFIC BARRIER DUMMY JOINTS.

β LOCATE ON OPPOSITE SIDE OF TRAFFIC. DRIVE PINS SHALL BE DRIVEN FLUSH WITH THE OUTSIDE FACE OF THE RAILING.

NOTE:
"E" DIMENSION SHALL BE EQUAL TO OPENING OF TRAFFIC BARRIER EXPANSION JOINTS.

Bridge Design Engr.	M:\STANDARDS\BP Rails\Ped_Rail1.MAN				
Supervisor					
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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
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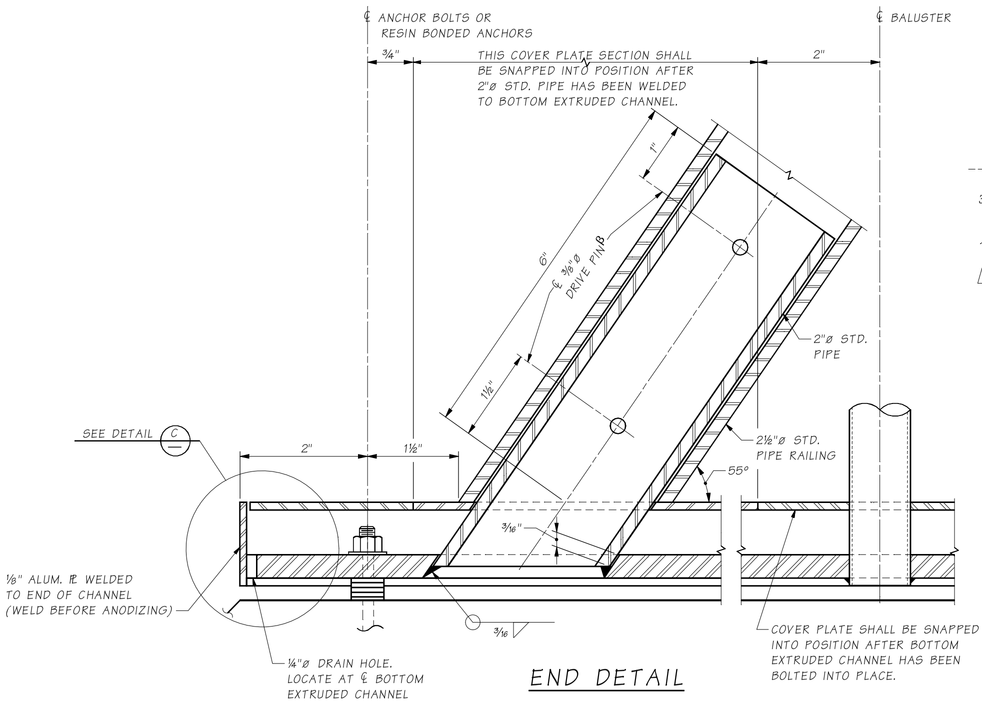
Washington State
Department of Transportation

STANDARD RAILINGS

BRIDGE RAILING TYPE PEDESTRIAN
DETAILS 1 OF 2

Thu Sep 02 07:40:09 2010



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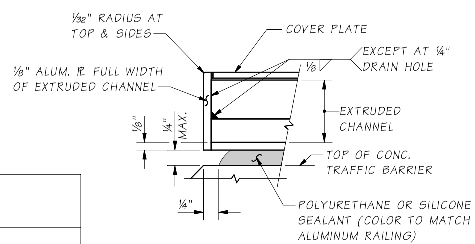
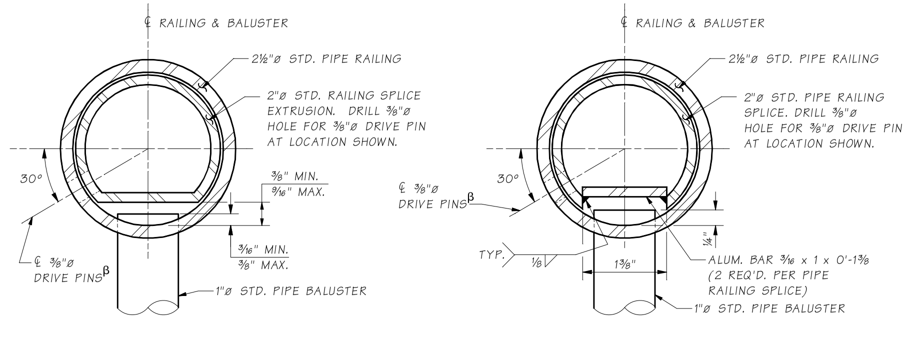


DETAIL C —
DRAIN HOLES NOT SHOWN

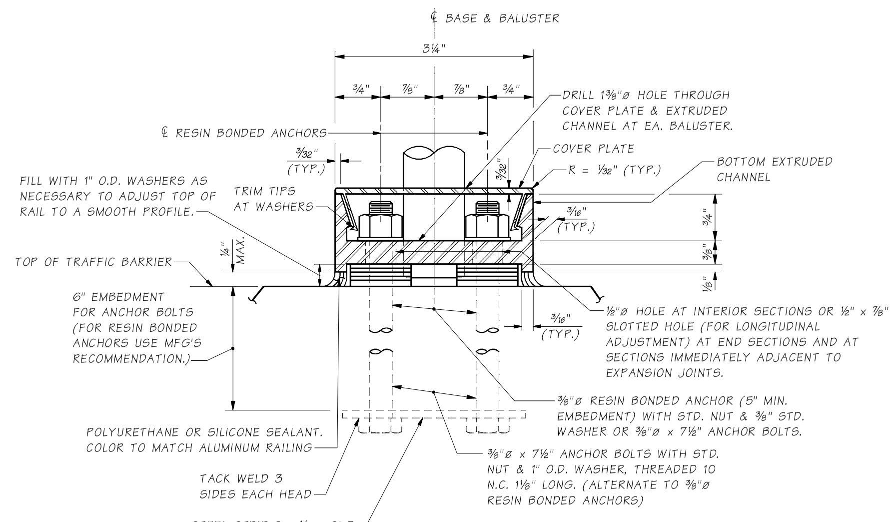


SECTION A ? — OPTION #1

β LOCATE ON OPPOSITE SIDE OF TRAFFIC. DRIVE PINS SHALL BE DRIVEN FLUSH WITH THE OUTSIDE FACE OF THE RAILING.

SECTION A ? — OPTION #2

β LOCATE ON OPPOSITE SIDE OF TRAFFIC. DRIVE PINS SHALL BE DRIVEN FLUSH WITH THE OUTSIDE FACE OF THE RAILING.



SECTION B ?

ANCHOR BOLTS SHALL BE POSITIONED IN A JIG DURING WELDING.

MAT'L	PART	MATERIAL SPECIFICATION
ALUMINUM	PIPES	ASTM B 221- 6005-T5 OR 6005A-T5 ASTM B 241 OR B 429 6061-T6
ALUMINUM	EXTRUDED CHANNELS, COVER PLATES & BAR	ASTM B 221- 6005-T5
STEEL	ANCHOR BOLTS, NUTS & WASHERS	STANDARD SPECIFICATION SECTION 9-06.5(4) (GALVANIZE IN ACCORDANCE WITH AASHTO SPECIFICATION M 232)
STEEL	STRIP	ASTM A 36
STEEL	DRIVE PINS	ASTM A 276 TYPE 302 STAINLESS STEEL

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10	WASH.			
JOB NUMBER				

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

STANDARD RAILINGS

BRIDGE RAILING TYPE PEDESTRIAN DETAILS 2 OF 2

Thu Sep 02 07:40:14 2010

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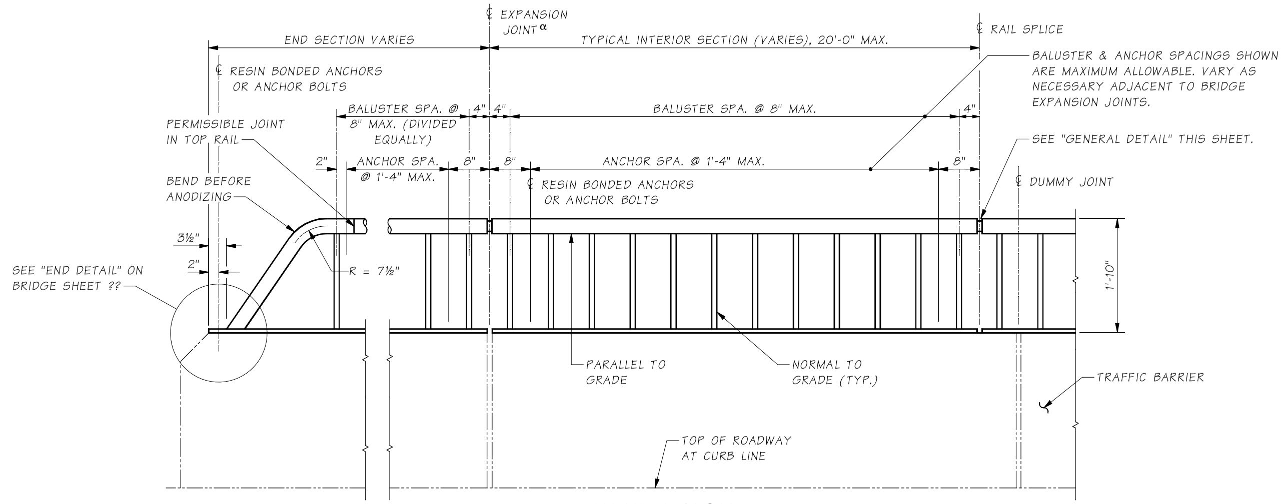
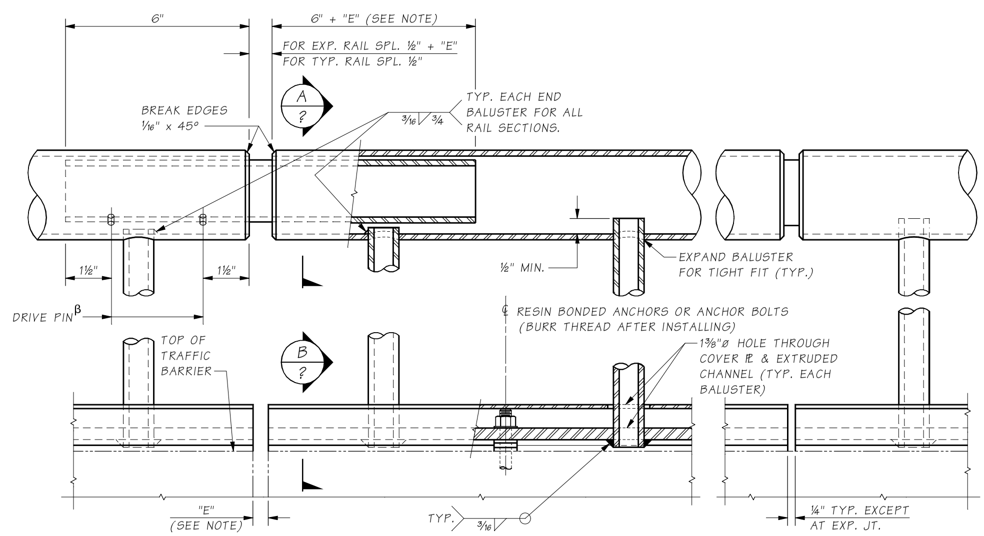


ELEVATION

BALUSTER AND GUARDRAIL SECTION ATTACHMENT DETAILS NOT SHOWN.



GENERAL DETAIL

NOTES

1. PIPE RAILING, PIPE RAILING SPLICES, COVER PLATES AND BOTTOM EXTRUDED CHANNELS SHALL BE BENT TO THE HORIZONTAL CURVE WHERE THE RADIUS OF CURVATURE IS LESS THAN 200'. THESE ITEMS MAY BE HEATED TO NOT MORE THAN 400°F FOR A PERIOD NOT TO EXCEED 30 MINUTES TO FACILITATE FORMING OR BENDING TO HORIZONTAL CURVATURE.
2. SHOP DRAWINGS OF RAILING SHALL BE SUBMITTED FOR APPROVAL SHOWING COMPLETE DIMENSIONS AND DETAILS OF FABRICATION AND INCLUDING AN ERECTION DIAGRAM. MATERIAL SPECIFICATIONS SHALL BE PROVIDED IN THE SHOP DRAWINGS FOR ALL COMPONENTS.
3. CUTTING SHALL BE DONE BY SAWING OR MILLING AND ALL CUTS SHALL BE TRUE AND SMOOTH. FLAME CUTTING WILL NOT BE PERMITTED.
4. WELDING OF ALUMINUM SHALL CONFORM TO STD. SPEC. SECTION 9-28.14(3).
5. ALL ALUMINUM PARTS SHALL BE GIVEN A *[CLEAR OR BRONZE] ANODIC COATING OF AT LEAST 0.0006" THICK AND SEALED TO MEET THE REQUIREMENTS OF ASTM B 580 WITH A UNIFORM FINISH.
6. PIPE RAILING, PIPE BALUSTERS AND PIPE RAILING SPLICES SHALL BE ADEQUATELY WRAPPED TO INSURE SURFACE PROTECTION DURING HANDLING AND TRANSPORTATION TO THE JOB SITE.

* NOTE TO DESIGNER:
Designer to choose color for their project in consultation with the Bridge Architect.

α PROVIDE EXPANSION RAIL SPLICE AT TRAFFIC BARRIER EXPANSION JOINTS. RAIL SPLICE JOINTS ARE NO REQUIRED AT TRAFFIC BARRIER DUMMY JOINTS.

β LOCATE ON OPPOSITE SIDE OF TRAFFIC. DRIVE PINS SHALL BE DRIVEN FLUSH WITH THE OUTSIDE FACE OF THE RAILING.

NOTE:
"E" DIMENSION SHALL BE EQUAL TO OPENING OF TRAFFIC BARRIER EXPANSION JOINTS.

Bridge Design Engr.
Supervisor
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Bridge Projects Engr.
Prelim. Plan By
Architect/Specialist

M:\STANDARDS\BP Rails\BP1.man

DATE | REVISION | BY | APP'D

REGION NO. | STATE | FED. AID PROJ. NO. | SHEET NO. | TOTAL SHEETS
10 | WASH.
JOB NUMBER

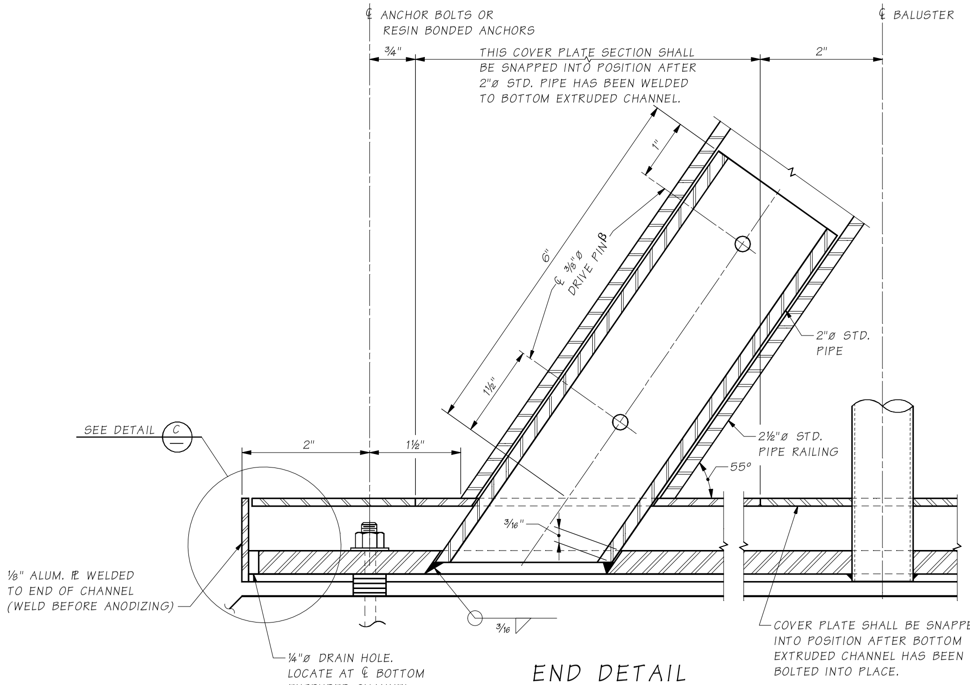
BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

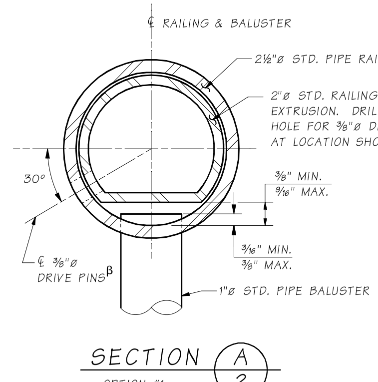
STANDARD RAILINGS

BRIDGE RAILING TYPE BP
DETAILS 1 OF 2

Thu Sep 02 07:40:24 2010

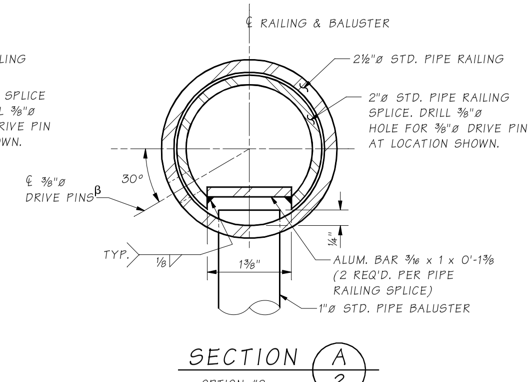


END DETAIL



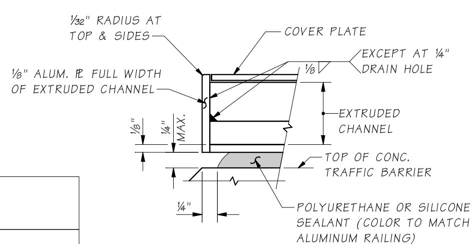
SECTION A
OPTION #1

β LOCATE ON OPPOSITE SIDE OF TRAFFIC. DRIVE PINS SHALL BE DRIVEN FLUSH WITH THE OUTSIDE FACE OF THE RAILING.

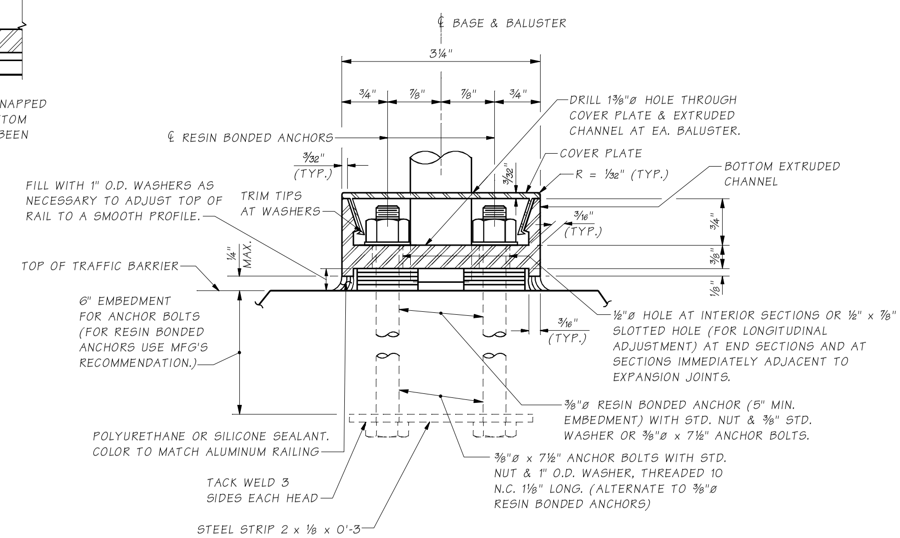


SECTION A
OPTION #2

β LOCATE ON OPPOSITE SIDE OF TRAFFIC. DRIVE PINS SHALL BE DRIVEN FLUSH WITH THE OUTSIDE FACE OF THE RAILING.



DETAIL C
DRAIN HOLES NOT SHOWN



SECTION B

ANCHOR BOLTS SHALL BE POSITIONED IN A JIG DURING WELDING.

MAT'L	PART	MATERIAL SPECIFICATION
ALUMINUM	PIPES	ASTM B 221- 6005-T5 OR 6005A-T5 ASTM B 241 OR B 429 6061-T6
ALUMINUM	EXTRUDED CHANNELS, COVER PLATES & BAR	ASTM B 221- 6005-T5
STEEL	ANCHOR BOLTS, NUTS & WASHERS	STANDARD SPECIFICATION SECTION 9-06.5(4) (GALVANIZE IN ACCORDANCE WITH AASHTO SPECIFICATION M 232)
STEEL	STRIP	ASTM A 36
STEEL	DRIVE PINS	ASTM A 276 TYPE 302 STAINLESS STEEL

Bridge Design Engr. M:\STANDARDS\BP Rails\BP2.man
Supervisor
Designed By
Checked By
Detailed By
Bridge Projects Engr.
Prelim. Plan By
Architect/Specialist
DATE REVISION BY APP'D
REGION NO. 10 STATE WASH. FED. AID PROJ. NO.
JOB NUMBER

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

STANDARD RAILINGS

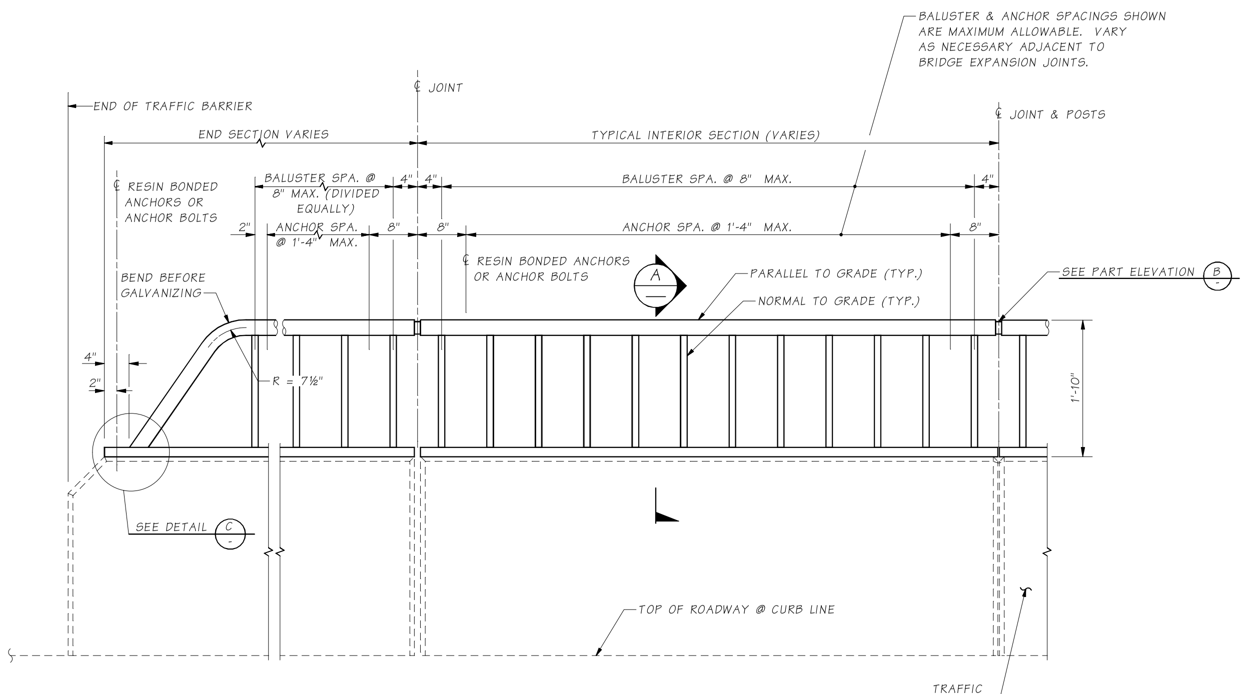
BRIDGE RAILING TYPE BP DETAILS 2 OF 2

Thu Sep 02 07:40:30 2010

MAT'L	PART	MATERIAL SPECIFICATION
STEEL	PIPES	ASTM A53 GRADE B SCHEDULE 40 (STD. PIPE). GALVANIZED IN ACCORDANCE WITH AASHTO M 111.
STEEL	WASHERS	ASTM A 193 GRADE B7 GALVANIZED IN ACCORDANCE WITH AASHTO M 232.
STEEL	PLATES/BAR CHANNELS	ASTM A 36 GALVANIZED IN ACCORDANCE WITH AASHTO M 111
STEEL	DRIVE PINS	ASTM A 276 TYPE 302 STAINLESS STEEL
STEEL	ANCHOR BOLTS	ASTM F 593 TYPE 302 STAINLESS STEEL
	NUTS	TAMPER-PROOF TYPE OF EITHER CLEAR ANODIZED ALUMINUM OR ZINC ALLOY

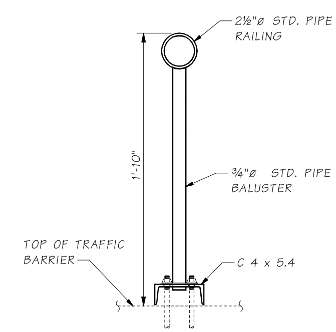
NOTES

1. PIPE RAILING, PIPE RAILING SPLICES, AND CHANNELS SHALL BE BENT TO THE HORIZONTAL CURVE WHERE THE RADIUS OF CURVATURE IS LESS THAN 200 FEET.
2. SHOP DRAWINGS OF RAILING SHALL BE SUBMITTED FOR APPROVAL SHOWING COMPLETE DIMENSIONS AND DETAILS OF FABRICATION, GALVANIZING AND INCLUDING AN ERECTION DIAGRAM. MATERIAL BEING USED SHALL BE SPECIFIED IN THE SHOP DRAWINGS.
3. THE BRIDGE RAILING SHALL BE HOT DIP GALVANIZED AFTER FABRICATION PER AASHTO M 111. ALL COMPONENTS OF RAIL PANEL, POSTS, AND SPLICE SLEEVES SHALL HAVE ALL SURFACES HOT DIP GALVANIZED PRIOR TO ASSEMBLY.
4. CUTTING SHALL BE DONE BY SAWING OR MILLING AND ALL CUTS SHALL BE TRUE AND SMOOTH.
5. PIPE RAILING, PIPE BALLUSTERS, RAILING SPLICE AND CHANNELS SHALL BE ADEQUATELY WRAPPED TO ENSURE SURFACE PROTECTION DURING HANDLING AND TRANSPORTATION TO THE JOB SITE.
6. WELDING SHALL BE DONE IN ACCORDANCE WITH AWS D1.1.
7. PIPE RAILING AND SPLICES MAY BE HEATED TO FACILITATE FORMING OR BENDING.
8. THE CONTRACTOR SHALL REPAIR ALL GALVANIZED STEEL SURFACES DAMAGED BY FIELD OPERATIONS, BY PAINTING THE DAMAGED AREAS WITH TWO COATS OF PAINT CONFORMING TO FORMULA A-9-73 AS SPECIFIED IN SECTION 9-08.2 OF THE WASHINGTON STATE STANDARD SPECIFICATIONS FOR ROAD, BRIDGE AND MUNICIPAL CONSTRUCTION.



ELEVATION

BALUSTER ATTACHMENT DETAILS NOT SHOWN



SECTION A

Bridge Design Engr.	M:\STANDARDS\BP Rails\STEEL RAILING BP-1.man	REGION NO. 10	STATE WASH.	FED. AID PROJ. NO.	JOB NUMBER
Supervisor					
Designed By					
Checked By					
Detailed By					
Bridge Projects Engr.					
Prelim. Plan By					
Architect/Specialist	DATE	REVISION	BY	APP'D	

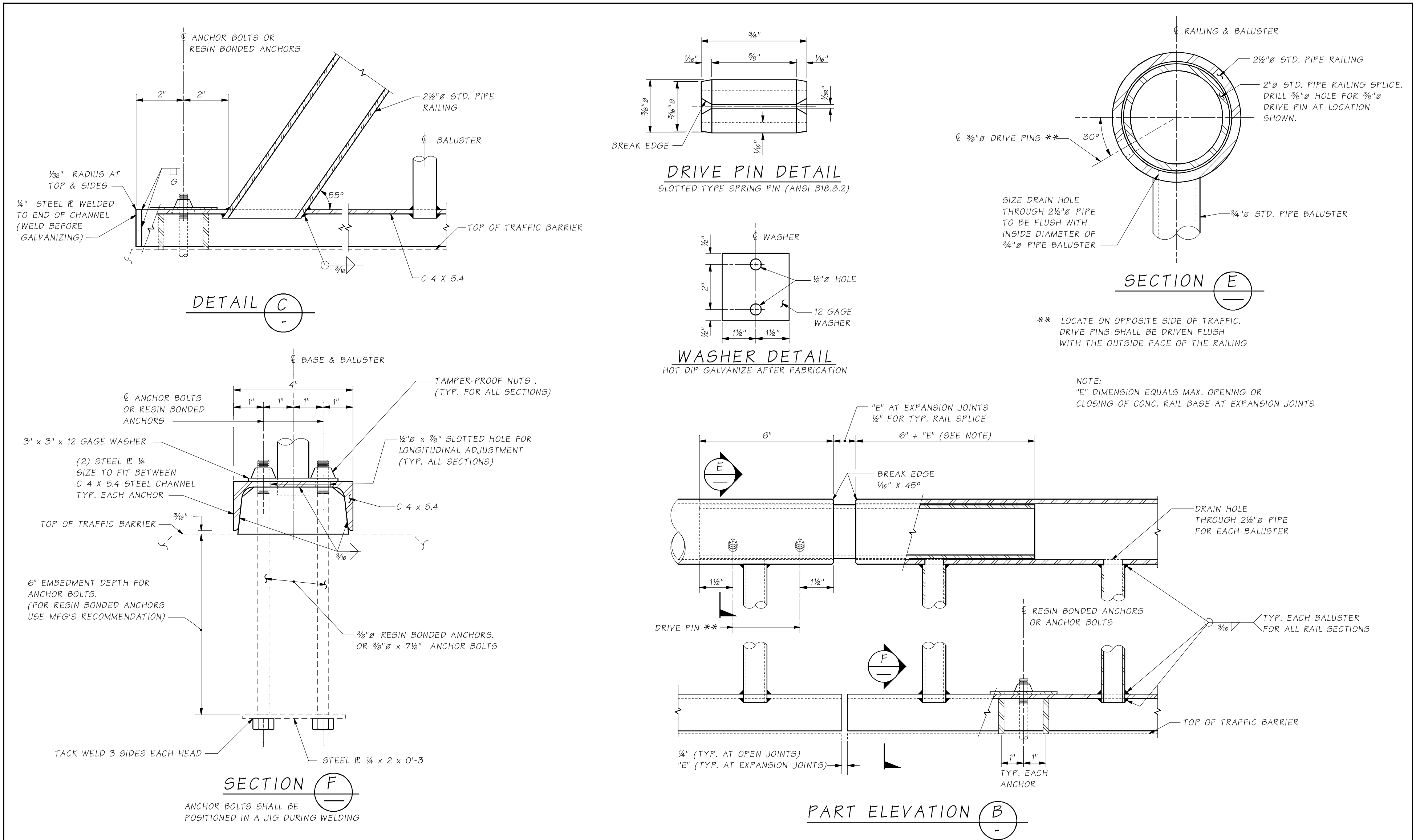
BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

STANDARD RAILINGS

BRIDGE RAILING TYPE S-BP DETAILS 1 OF 2

Thu Sep 02 07:39:56 2010



Bridge Design Engr.	Charles C. Ruth	M:\STANDARDS\BP Rails\STEEL RAILING BP-2.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

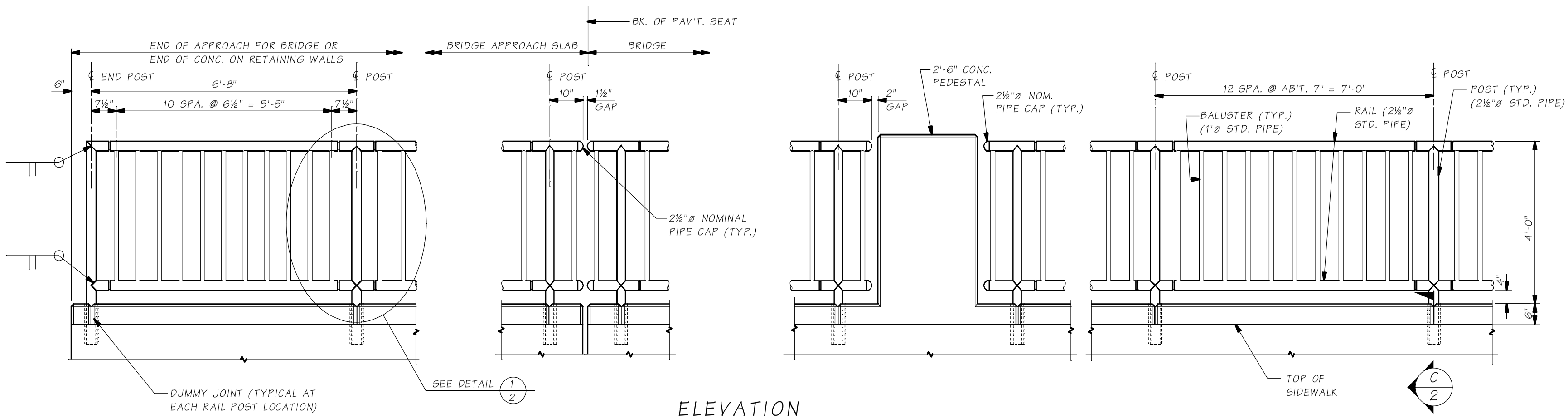
Washington State Department of Transportation

STANDARD RAILINGS

BRIDGE RAILING TYPE S-BP
DETAILS 2 OF 2

Thu Sep 02 07:40:00 2010

JOB NO.
SR
SHEET



ELEVATION

BALUSTERS NORMAL TO GRADE.
TOP & BOTTOM RAILS PARALLEL TO GRADE.

NOTES

1. PIPE RAILING, AND PIPE RAILING SPLICES, SHALL BE BENT TO THE HORIZONTAL CURVE WHERE THE RADIUS OF CURVATURE IS LESS THAN 200 FEET.
2. SHOP DRAWINGS OF RAILING SHALL BE SUBMITTED FOR APPROVAL SHOWING COMPLETE DIMENSIONS AND DETAILS OF FABRICATION AND INCLUDING AN ERECTION DIAGRAM. MATERIAL BEING USED SHALL BE SPECIFIED IN THE SHOP DRAWINGS.
3. PIPE RAILING, AND PIPE RAILING SPLICES, MAY BE HEATED TO NOT MORE THAN 400°F FOR A PERIOD NOT TO EXCEED 30 MINUTES TO FACILITATE FORMING OR BENDING HORIZONTAL CURVATURE.
4. CUTTING SHALL BE DONE BY SAWING OR MILLING AND ALL CUTS SHALL BE TRUE AND SMOOTH. FLAME CUTTING WILL NOT BE PERMITTED.
5. WELDING OF ALUMINUM SHALL CONFORM TO STD. SPEC. SECTION 9-28.14(3).
6. AFTER FABRICATION, POSTS SHALL BE HEAT TREATED IN ACCORDANCE WITH SECTION 6.5 OF THE AASHTO STANDARD SPECIFICATIONS FOR STRUCTURAL SUPPORTS FOR HIGHWAY SIGNS, LUMINAIRES, AND TRAFFIC SIGNALS DATED 2001 AND INTEREMS THROUGH 2003.
7. ALL ALUMINUM PARTS SHALL BE GIVEN A (CLEAR OR BRONZE)* ANODIC COATING OF AT LEAST 0.0006 INCHES THICK AND SEALED TO MEET THE REQUIREMENTS OF ASTM B 580 WITH A UNIFORM FINISH.
8. PIPE RAILING, PIPE BALUSTERS, PIPE RAILING SPLICES, SHALL BE ADEQUATELY WRAPPED TO INSURE SURFACE PROTECTION DURING HANDLING AND TRANSPORTATION TO THE JOB SITE.

* NOTE TO DESIGNER:
Designer to choose color for their project in consultation with the Bridge Architect.

MAT'L	PART	MATERIAL SPECIFICATION
ALUMINUM	PIPES	ASTM B 221-6005-T5 SCHEDULE 40 (STD. PIPE) ASTM B 241 OR B 429 6061-T6
ALUMINUM	BAR	ASTM B 221-6005-T5
STEEL	DRIVE PINS	ASTM A 276 TYPE 302 STAINLESS STEEL

Bridge Design Engr.	M:\STANDARDS\BP Rails\4ftPedRail_Alumin 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 RAILINGS

PEDESTRIAN RAILING DETAILS 1 OF 2

BRIDGE SHEET NO.
SHEET OF SHEETS

Thu Dec 19 14:35:36 2013

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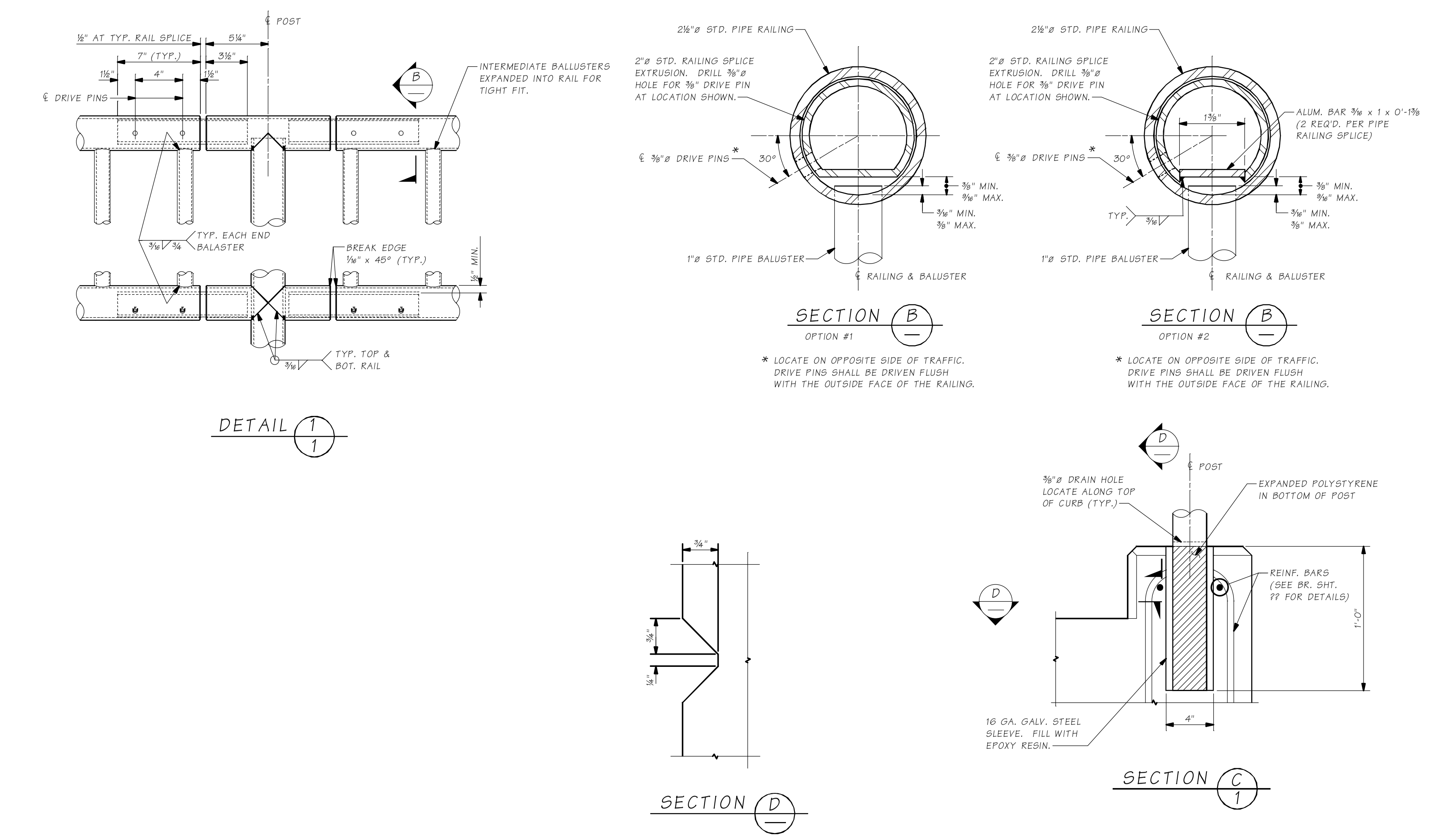

Bridge Design Engr.	M:\STANDARDS\BP Rails\4ftPedRail_Alumin 2.MAN			REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
Supervisor				10	WASH.			
Designed By				JOB NUMBER				
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Detailed By								
Bridge Projects Engr.								
Prelim. Plan By								
Architect/Specialist	DATE	REVISION	BY	APP'D				

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

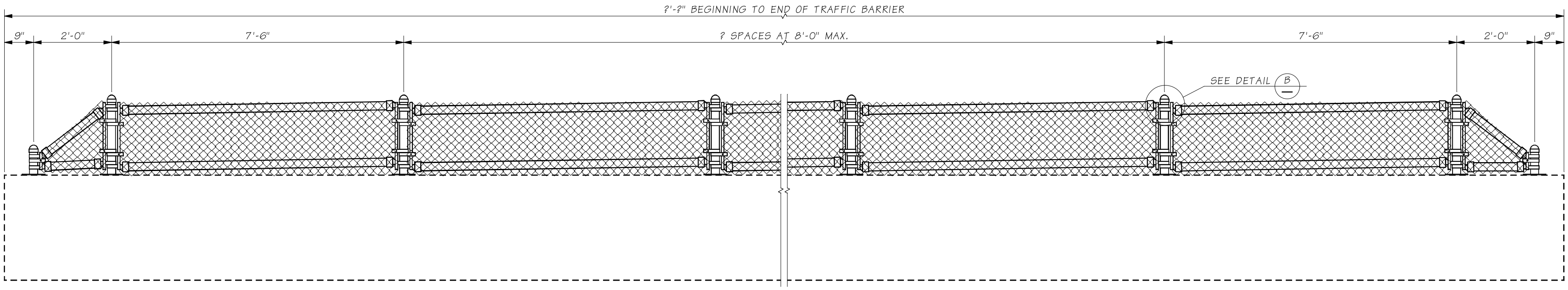
STANDARD RAILINGS

PEDESTRIAN RAILING DETAILS 2 OF 2

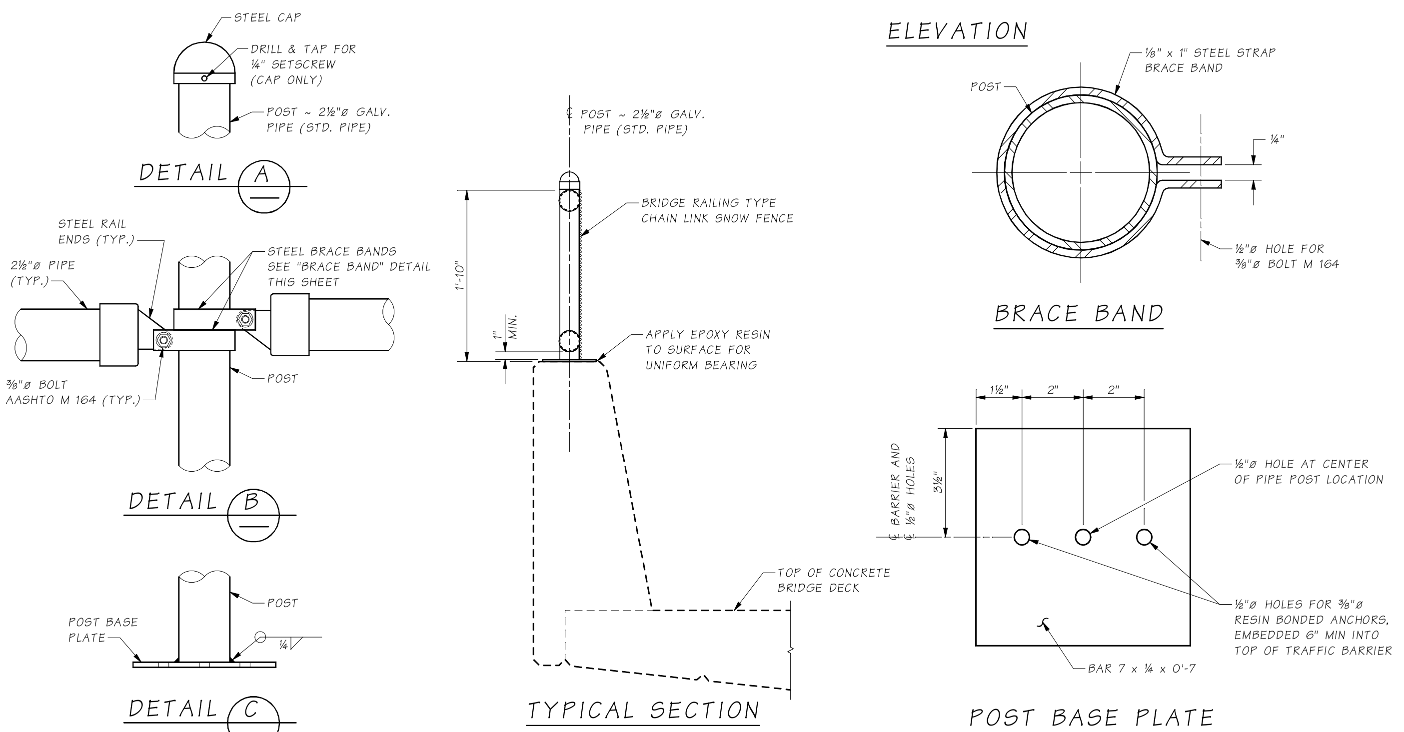
BRIDGE SHEET NO.
SHEET OF SHEETS

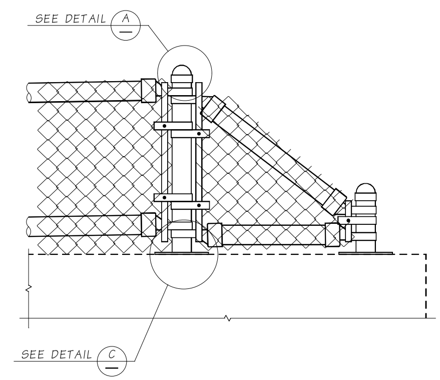
Thu Dec 19 14:35:36 2013

SR JOB NO. SHEET



ELEVATION





RAILING END VIEW

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		REGION NO. 10	STATE WASH.	FED. AID PROJ. NO.
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Detailed By				
Bridge Projects Engr.				
Prelim. Plan By				
Architect/Specialist	DATE	REVISION	BY	APP'D

Thu Sep 02 07:40:04 2010

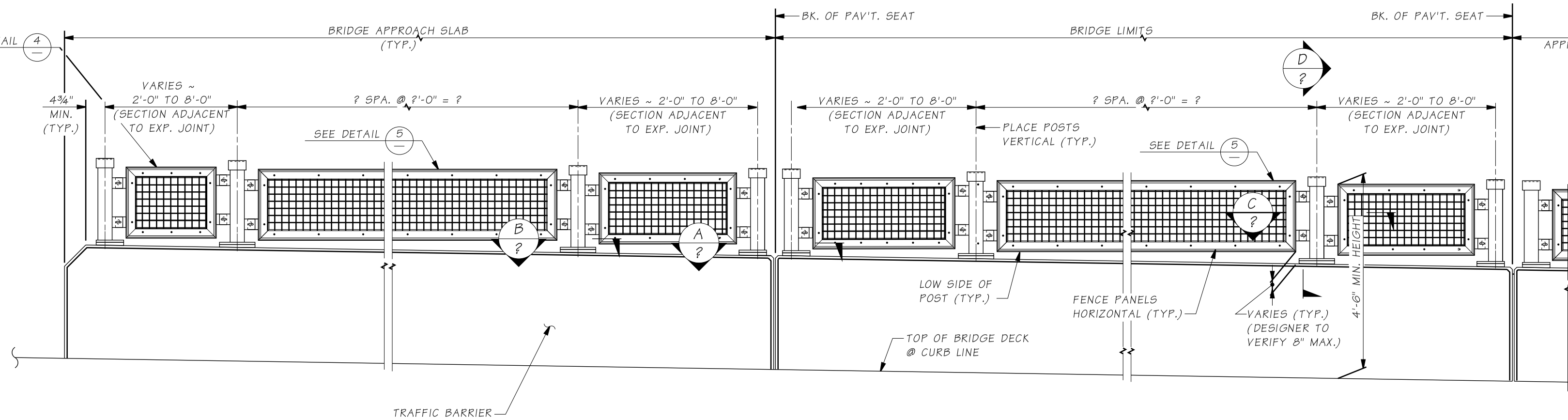
BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

STANDARD RAILINGS

BRIDGE RAILING TYPE CHAIN LINK SNOW FENCE

SR
JOB NO.
SHEET 1



TRAFFIC BARRIER OUTSIDE ELEVATION

ARCHITECTURAL FINISH OMITTED FOR CLARITY.

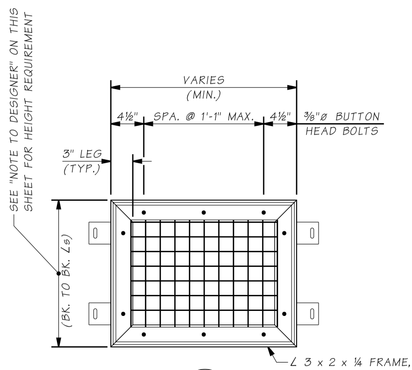
NOTES:

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:
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.

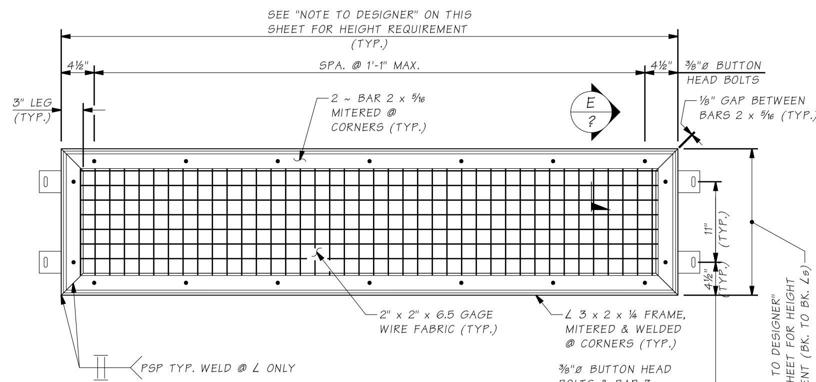
NOTE TO DESIGNER:

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 4

VIEW FROM OUTSIDE ELEVATION
FOR DETAILS NOT SHOWN SEE DETAIL 5



DETAIL 5

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				

Thu Jan 30 13:57:07 2014

BRIDGE AND STRUCTURES OFFICE

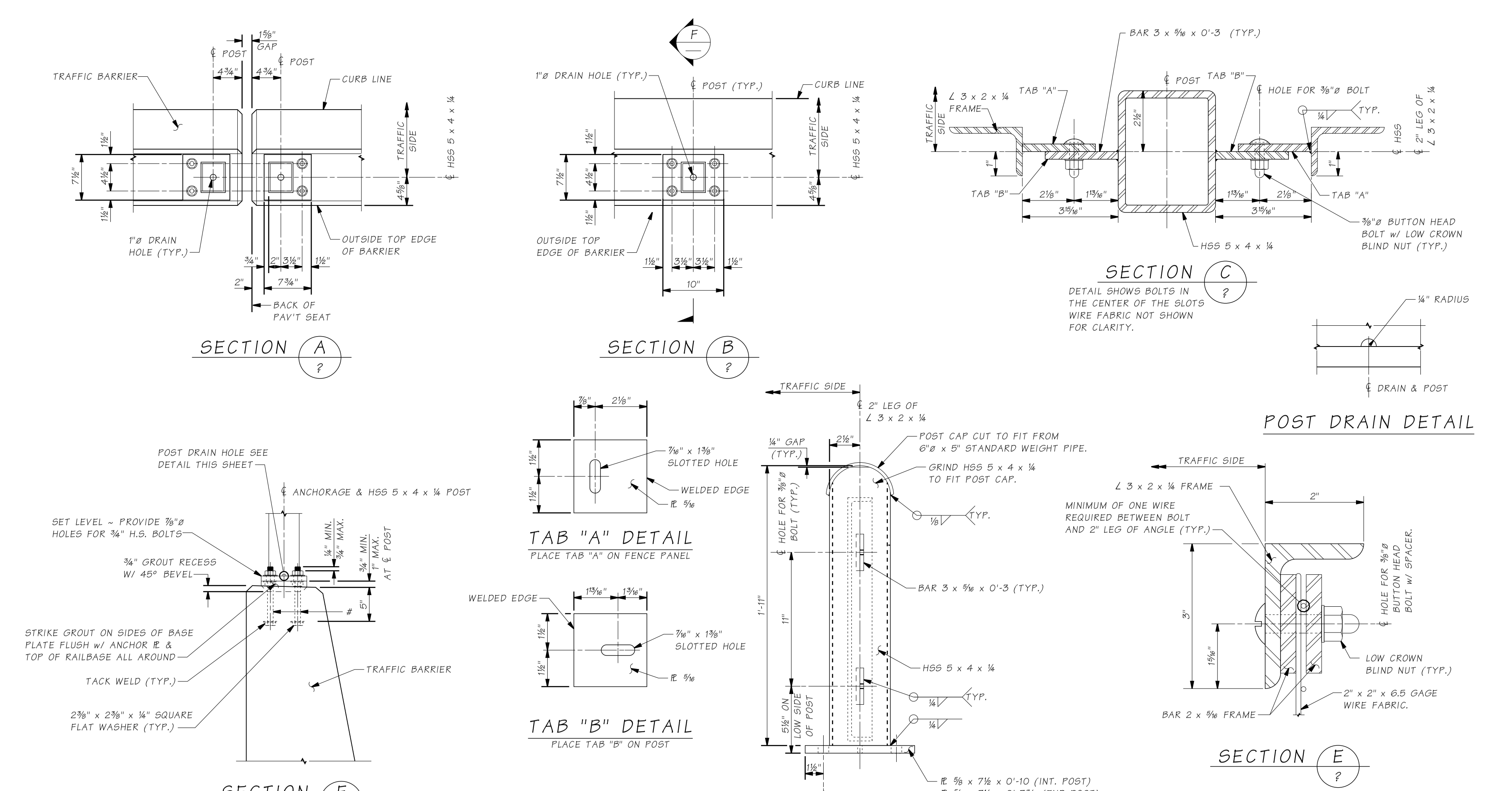
Washington State Department of Transportation

STANDARD RAILINGS

BRIDGE RAILING TYPE SNOW FENCE
DETAILS 1 OF 2

BRIDGE SHEET NO.
SHEET OF SHEETS

SR 5 FILE NO. Job SHEET 1



\# 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.	M:\STANDARDS\BP Rails\BRIDGE RAILING SNOW FENCE 2.MAN					
Supervisor	Sup		REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO. / TOTAL SHEETS
Designed By	Designer	Date	10	WASH.		
Checked By	Checker	Date	JOB NUMBER			
Detailed By	Detailer	Date				
Bridge Projects Engr.						
Prelim. Plan By						
Architect/Specialist	DATE	REVISION	BY	APP'D		

Thu Jan 30 13:58:18 2014

BRIDGE AND STRUCTURES OFFICE

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STANDARD RAILINGS

BRIDGE RAILING TYPE SNOW FENCE DETAILS 2 OF 2

BRIDGE SHEET NO. / SHEET OF SHEETS

Last Revised on: 4/4/13

SR / FILE NO. / Job / SHEET 1

PROTECTIVE SCREENING NOTES

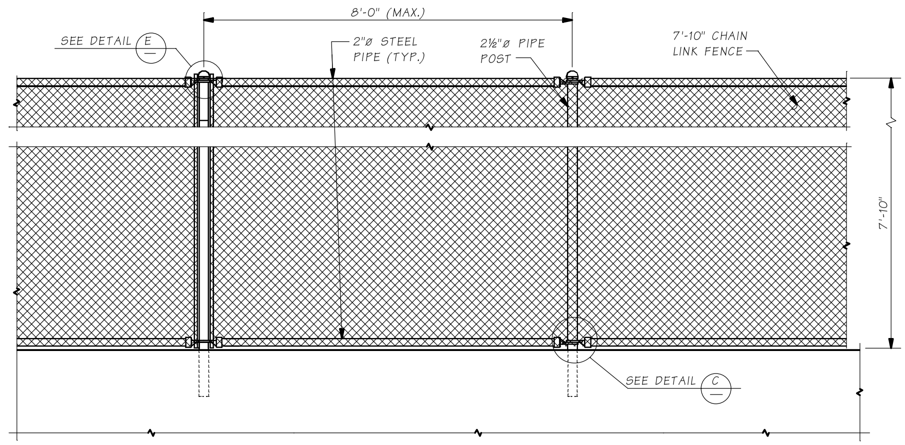
ALL ELEMENTS OF FENCE SHALL BE HOT DIPPED GALVANIZED AFTER FABRICATION.

STEEL PIPES FOR POSTS AND LONGITUDINAL MEMBERS SHALL CONFORM TO ASTM SPECIFICATION A53 GRADE B GALV. PER AASHTO M 111.

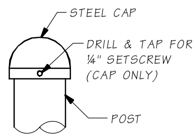
ALL HARDWARE SHALL CONFORM TO AASHTO SPECIFICATION M 183 GALV. PER AASHTO M 111 & M232 UNLESS NOTED OTHERWISE.

FABRIC SHALL BE HEAVY DUTY ALUMINUM OF #9 GAGE WIRE WOVEN IN A 50 CHAIN LINK DIAMOND MESH.

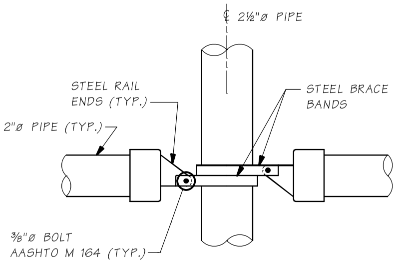
FABRIC TIES SHALL BE INSTALLED TO ALL FRAMES IN ACCORDANCE WITH GOOD TRADE PRACTICES AT 355 CENTERS MAXIMUM SPACING.



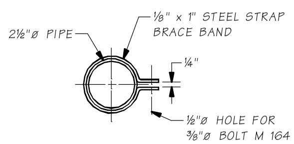
ELEVATION



DETAIL E

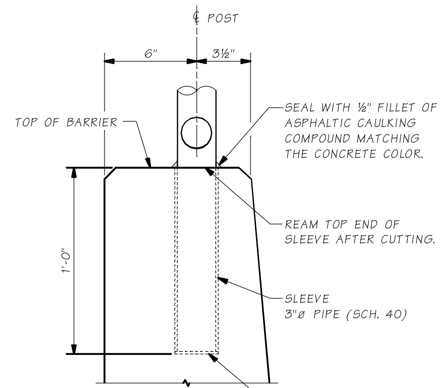


DETAIL C



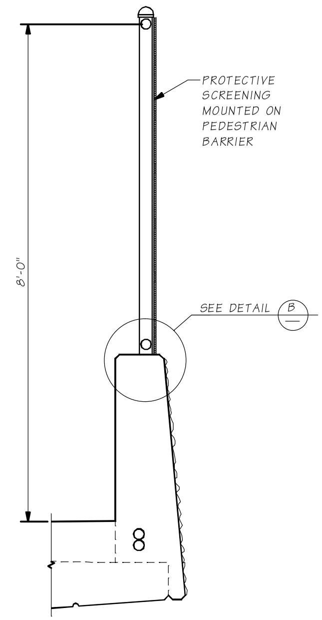
BRACE BAND

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.



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.				
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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

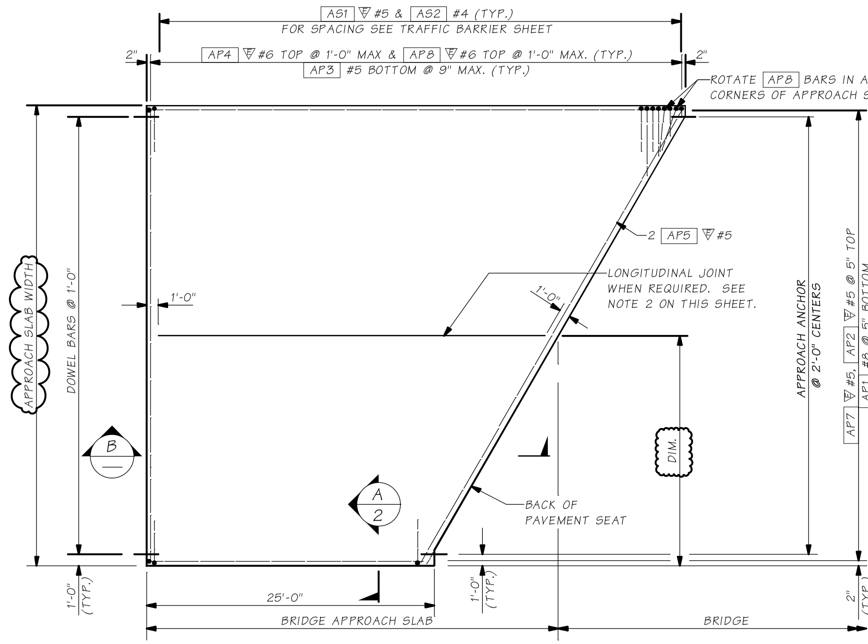
STANDARD RAILINGS

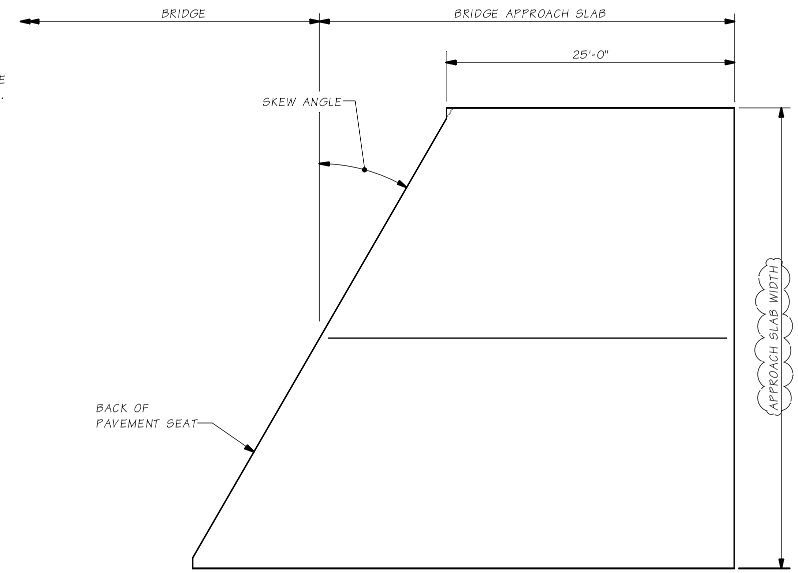
BRIDGE RAILING
TYPE CHAIN LINK FENCE

BRIDGE SHEET NO.
SHEET OF SHEETS

Tue Jul 10 11:50:09 2012

SR JOB NO. SHEET





PLAN

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 .

NOTE:
Designer to remove call outs that are not applicable in note 3.

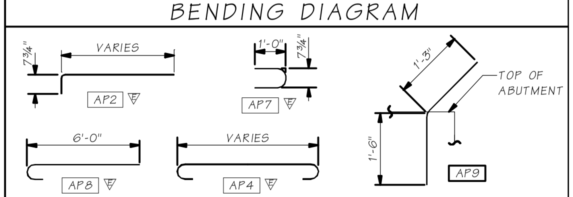
NOTE:
AP9 bars are only applicable in L abutments or approach retrofits. Remove when not used.

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.

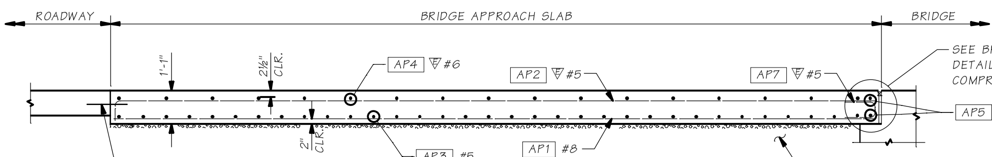
NOTES:

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.
4. FOR TRAFFIC BARRIER DETAILS, INCLUDING ANY BRIDGE APPROACH SLAB BLOCKOUT INFORMATION, SEE TRAFFIC BARRIER SHEETS.

BENDING DIAGRAM



∇ = EPOXY COATED REINFORCING STEEL   NOTE: ALL DIMENSIONS ARE OUT TO OUT



SECTION B

Bridge Design Engr.	M:\STANDARDS\Approach Slabs\Approach Slab 1.MAN			REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
Supervisor				10	WASH.			
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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

APPROACH SLABS

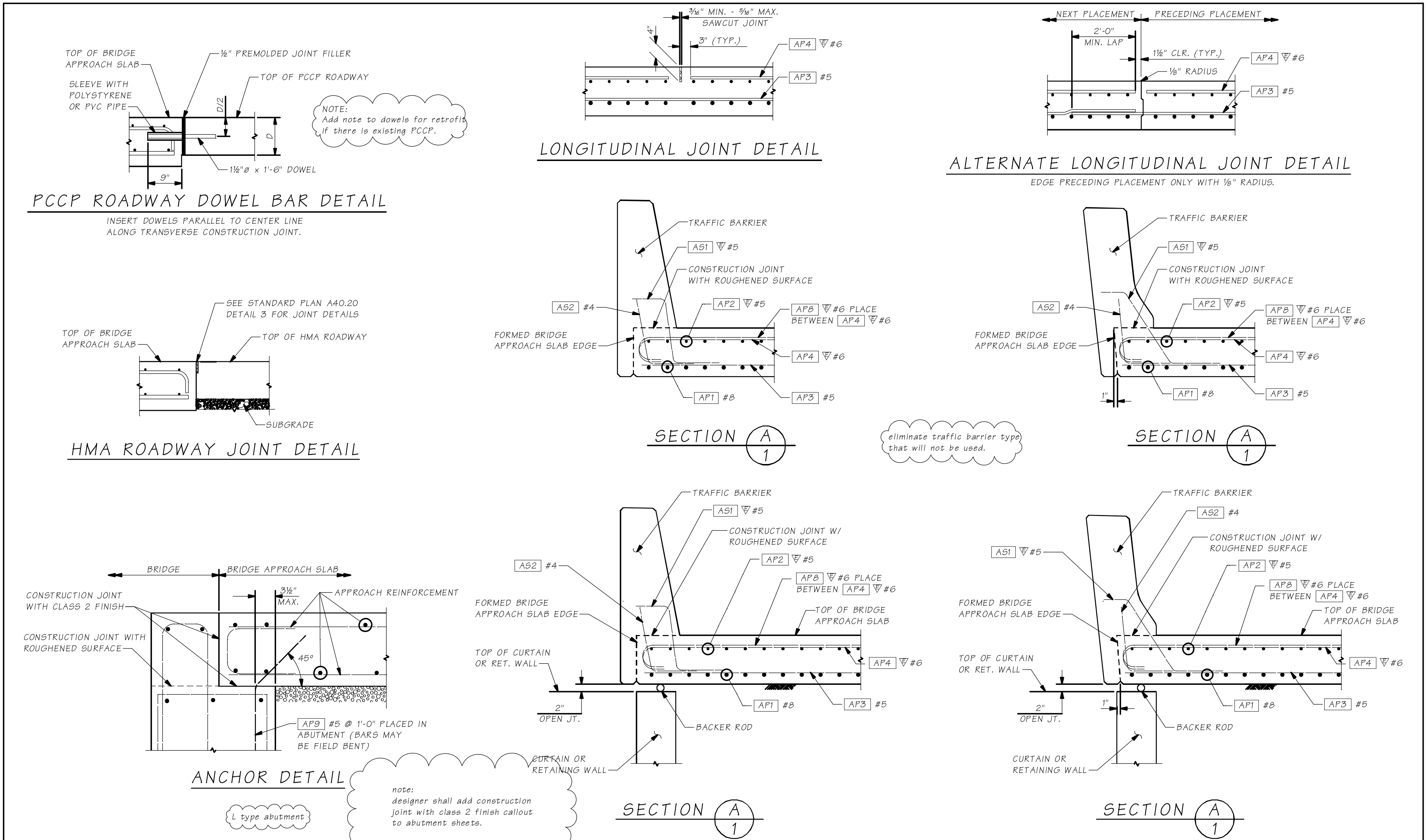
BRIDGE APPROACH SLAB DETAILS 1 OF 3

BRIDGE SHEET NO.
SHEET OF SHEETS

Tue Dec 24 08:52:31 2013

SR   JOB NO.   SHEET 1

10.6-A1-1



Bridge Design Engr.	M:\STANDARDS\Approach Slabs\Approach Slab 2.man				
Supervisor					
Designed By					
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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

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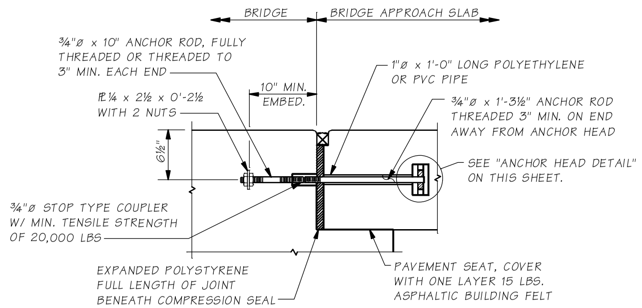
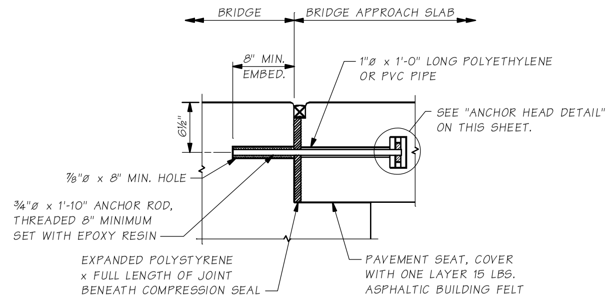
SR | JOB NO. | SHEET



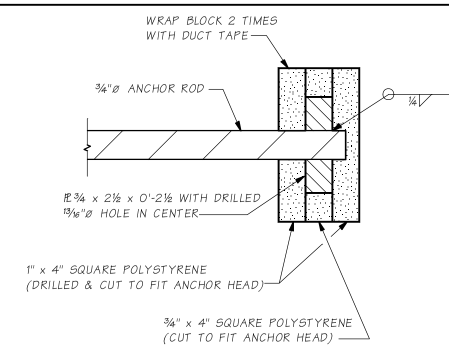
APPROACH ANCHOR - METHOD A

Not for "L" type abutments.



APPROACH ANCHOR - METHOD B

Not for "L" type abutments.



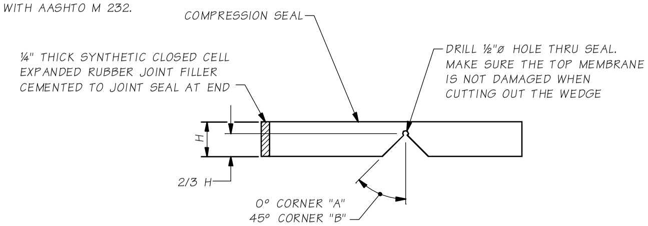
ANCHOR HEAD DETAIL

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

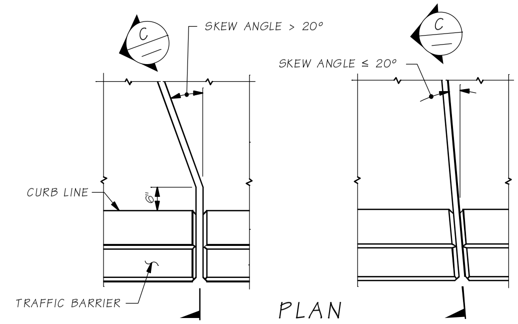
D.S. BROWN		WATSON BOWMAN ACME	
CV-2502	2½	WA-250	2½

TESTING SHALL BE PER AASHTO M-220 PRIOR TO USE.



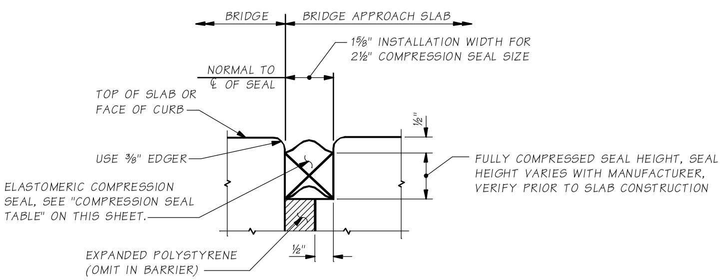
SEAL CUTTING DETAIL

eliminate corner "B" callout for single slope barrrier.



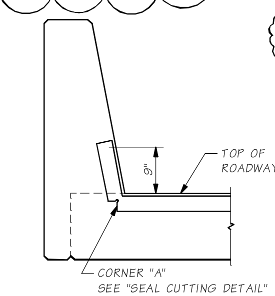
PLAN
EXPANSION JOINT

NOTE:
Remove expansion joint in barrier details when there is no barrier on approach slab.



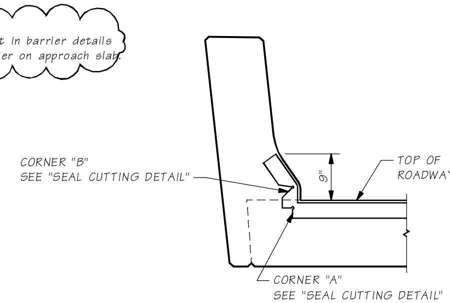
COMPRESSION SEAL DETAIL

EXPANSION JOINT AT BACK OF PAV'T SEAT



SECTION C

eliminate this detail for f shape barrier.



SECTION C

eliminate this detail for single slope barrier.

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Architect/Specialist
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10	WASH.			

JOB NUMBER

BRIDGE AND STRUCTURES OFFICE

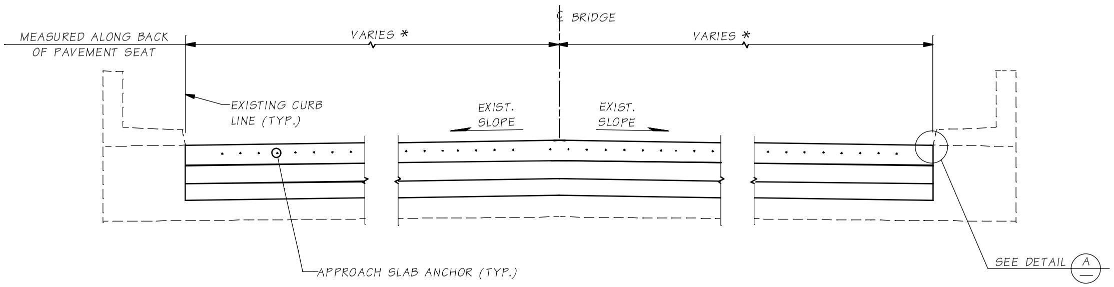
Washington State Department of Transportation

APPROACH SLABS

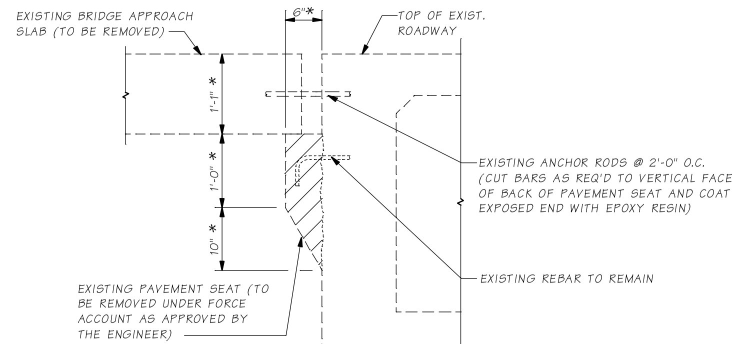
BRIDGE APPROACH SLAB DETAILS 3 OF 3

BRIDGE SHEET NO.
SHEET
OF
SHEETS

SR
FILE NO.
SHEET



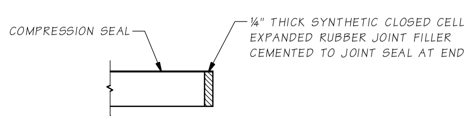
PAVEMANT SEAT END VIEW



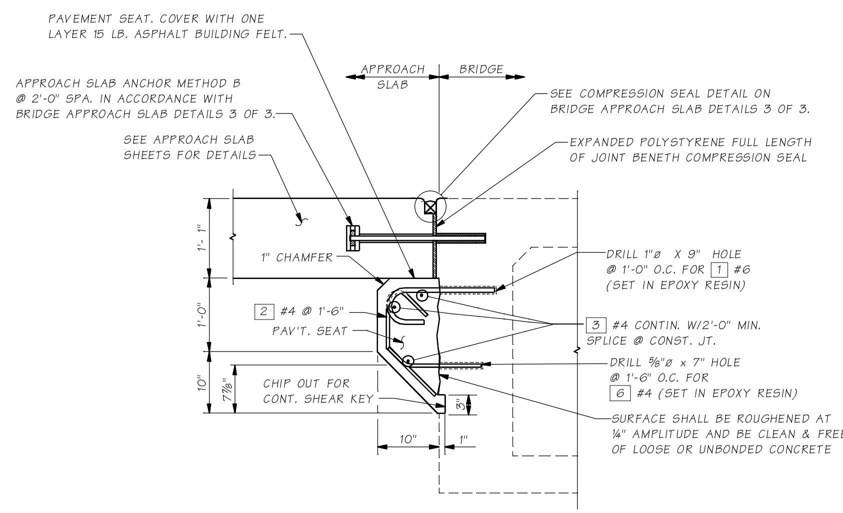
PAVEMENT SEAT

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



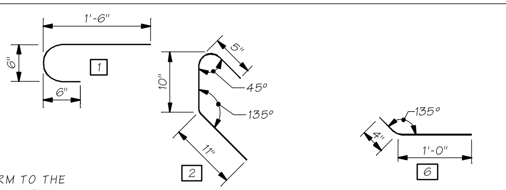
PAVEMENT SEAT

RETROFIT CONDITION

BAR LIST

MARK	SIZE	LENGTH	
1	6	2'-4"	
2	4	2'-4'	
3	4	(A)	STR.
6	4	1'-3"	

BENDING DIAGRAM(ALL DIMENSIONS ARE OUT TO OUT TO POINTS OF INTERSECTION)



(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.	FED. AID PROJ. NO.	SHEET NO. / TOTAL SHEETS
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BRIDGE AND STRUCTURES OFFICE

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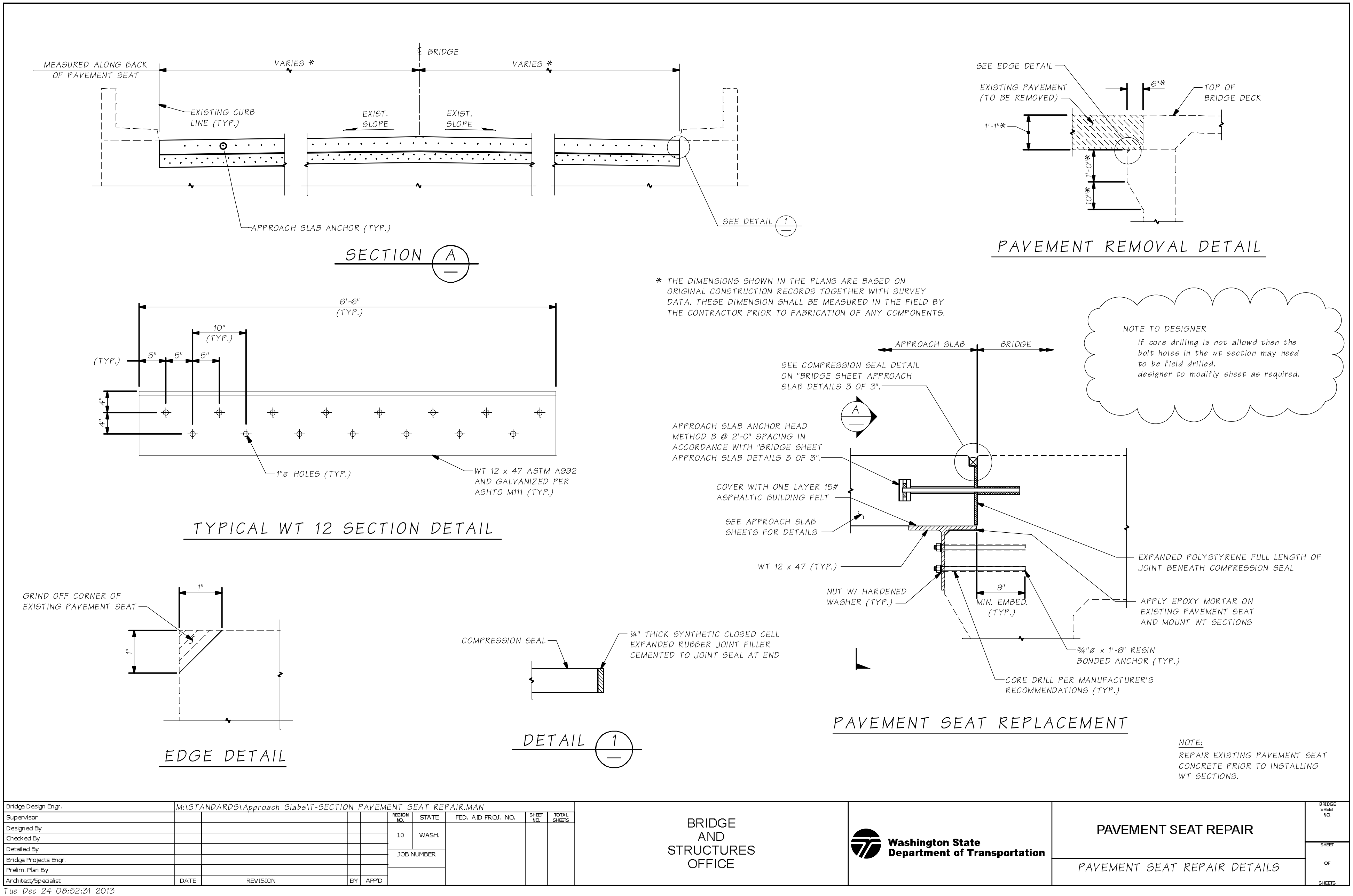
PAVEMENT SEAT REPAIR

PAVEMENT SEAT REPAIR DETAILS

BRIDGE SHEET NO.
SHEET OF SHEETS

SR
JOB NO.
1000 SHEET 3


MEASURED ALONG BACK OF PAVEMENT SEAT
VARIES *
℄ BRIDGE
VARIES *
EXISTING CURB LINE (TYP.)
EXIST. SLOPE
EXIST. SLOPE
APPROACH SLAB ANCHOR (TYP.)
SEE DETAIL 1
SECTION A
SEE EDGE DETAIL
EXISTING PAVEMENT (TO BE REMOVED)
6"*
TOP OF BRIDGE DECK
1'-1"*
1'-0"*
10"*
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 modifiy sheet as required.
6'-6" (TYP.)
10" (TYP.)
(TYP.)
5"
5"
5"
4"
4"
1"ø HOLES (TYP.)
WT 12 x 47 ASTM A992 AND GALVANIZED PER ASHTO M111 (TYP.)
TYPICAL WT 12 SECTION DETAIL
APPROACH SLAB
BRIDGE
SEE COMPRESSION SEAL DETAIL ON "BRIDGE SHEET APPROACH SLAB DETAILS 3 OF 3".
APPROACH SLAB ANCHOR HEAD METHOD B @ 2'-0" SPACING IN ACCORDANCE WITH "BRIDGE SHEET APPROACH SLAB DETAILS 3 OF 3".
COVER WITH ONE LAYER 15# ASPHALTIC BUILDING FELT
SEE APPROACH SLAB SHEETS FOR DETAILS
WT 12 x 47 (TYP.)
NUT W/ HARDENED WASHER (TYP.)
9"
MIN. EMBED. (TYP.)
EXPANDED POLYSTYRENE FULL LENGTH OF JOINT BENEATH COMPRESSION SEAL
APPLY EPOXY MORTAR ON EXISTING PAVEMENT SEAT AND MOUNT WT SECTIONS
¾"ø x 1'-6" RESIN BONDED ANCHOR (TYP.)
CORE DRILL PER MANUFACTURER'S RECOMMENDATIONS (TYP.)
PAVEMENT SEAT REPLACEMENT
GRIND OFF CORNER OF EXISTING PAVEMENT SEAT
1"
1"
EDGE DETAIL
COMPRESSION SEAL
¼" THICK SYNTHETIC CLOSED CELL EXPANDED RUBBER JOINT FILLER CEMENTED TO JOINT SEAL AT END
DETAIL 1
NOTE:
REPAIR EXISTING PAVEMENT SEAT CONCRETE PRIOR TO INSTALLING WT SECTIONS.
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Prelim. Plan By
Architect/Specialist
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BY
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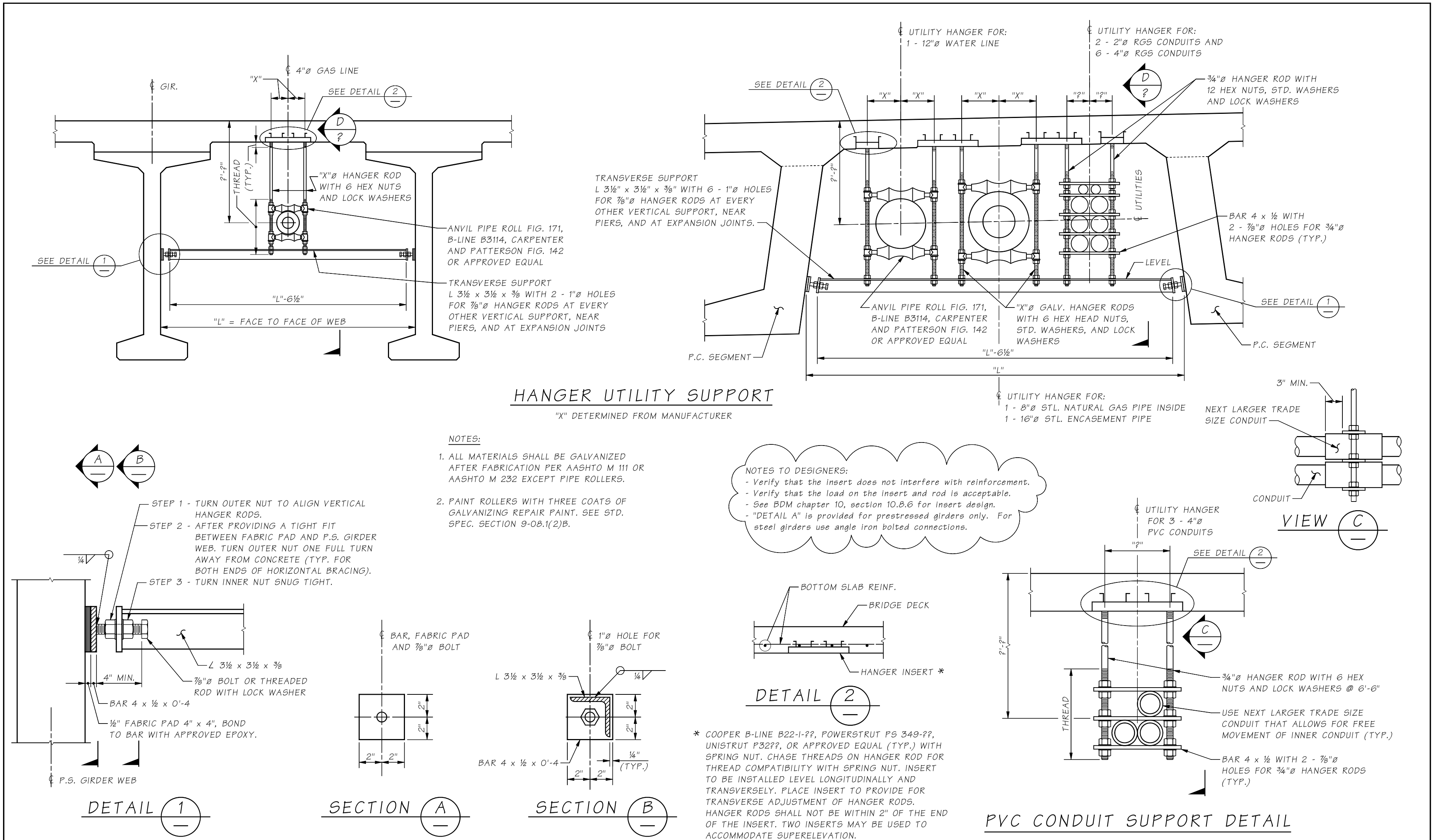
M:\STANDARDS\Approach Slabs\T-SECTION PAVEMENT SEAT REPAIR.MAN
REGION NO.
STATE
FED. AID PROJ. NO.
SHEET NO.
TOTAL SHEETS
10
WASH.
JOB NUMBER
BRIDGE AND STRUCTURES OFFICE
Washington State Department of Transportation
PAVEMENT SEAT REPAIR
PAVEMENT SEAT REPAIR DETAILS
BRIDGE SHEET NO.
SHEET
OF
SHEETS
Tue Dec 24 08:52:31 2013
SR
JOB NO. 1000 SHEET 3



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DATE | REVISION | BY | APP'D

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JOB NUMBER

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

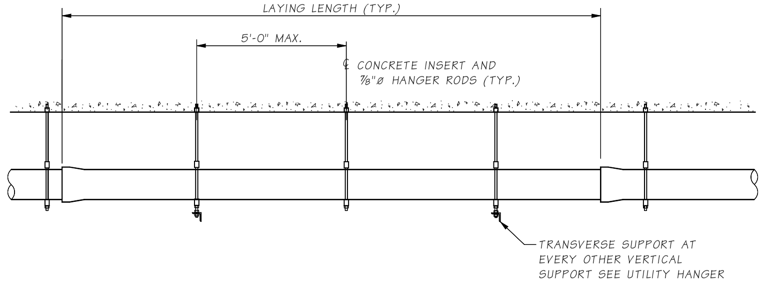
UTILITY INSTALLATION GUIDELINE DETAILS FOR EXISTING BRIDGES

UTILITY HANGER DETAILS

BRIDGE SHEET NO. / SHEET OF SHEETS

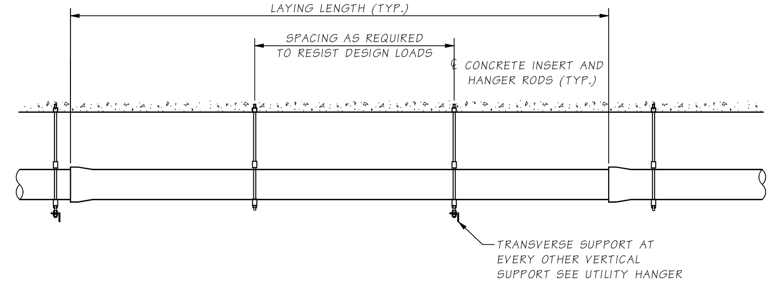
Thu Dec 26 09:16:32 2013

SR JOB NO. SHEET 1



SECTION D/?

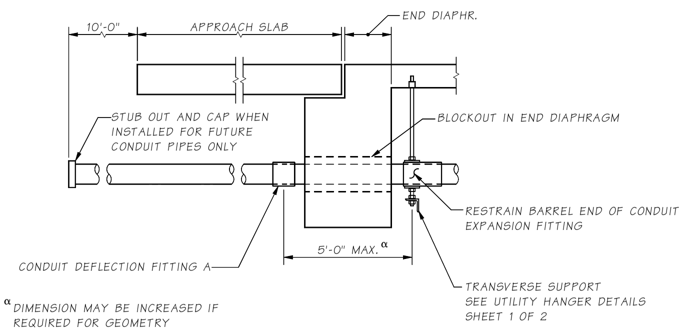
RGS OR PVC CONDUIT



SECTION D/?

UTILITIES OTHER THAN RGS OR PVC CONDUITS

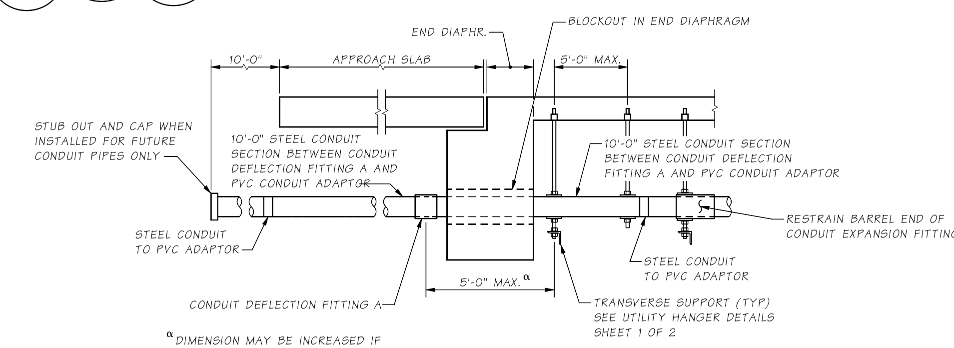
NOTE TO DESIGNER:
COORDINATE WITH UTILITY ENGINEER FOR CONDUIT PLACEMENT DETAILS AT ABUTMENTS FOR UTILITIES OTHER THAN RGS OR PVC.



α DIMENSION MAY BE INCREASED IF REQUIRED FOR GEOMETRY

UTILITY CONDUIT
PLACEMENT DETAIL ~ RGS

SEE BARRIER SHEETS FOR CONDUIT DEFLECTION FITTING A DETAIL.



α DIMENSION MAY BE INCREASED IF REQUIRED FOR GEOMETRY

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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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

Thu Dec 26 09:16:58 2013

Last revised on : 12/24/2013

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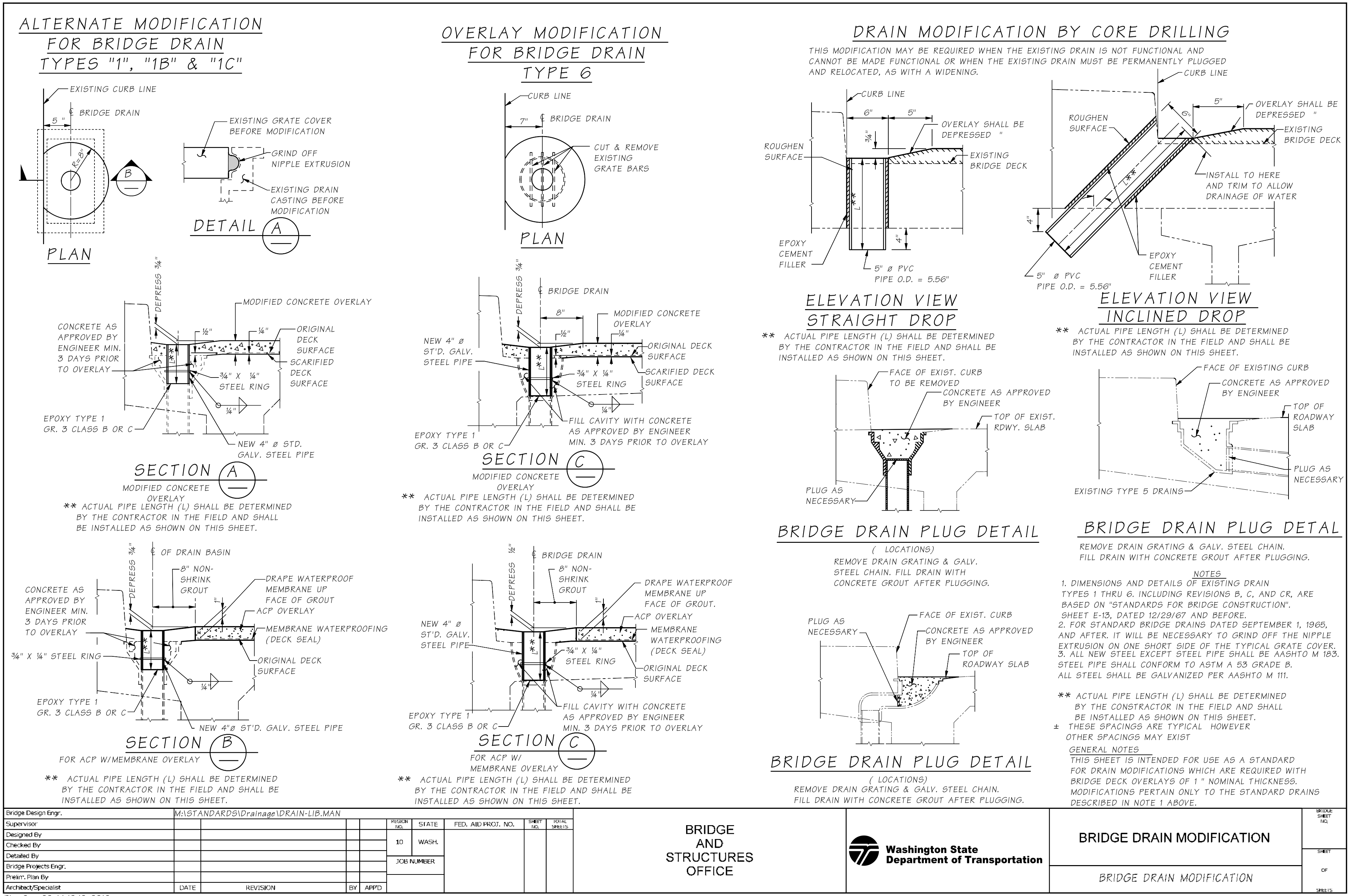
ALTERNATE MODIFICATION FOR BRIDGE DRAIN TYPES "1", "1B" & "1C"
EXISTING CURB LINE
℄ BRIDGE DRAIN
5 "
R=8"
B
PLAN
EXISTING GRATE COVER BEFORE MODIFICATION
GRIND OFF NIPPLE EXTRUSION
EXISTING DRAIN CASTING BEFORE MODIFICATION
DETAIL A
DEPRESS ¾"
MODIFIED CONCRETE OVERLAY
CONCRETE AS APPROVED BY ENGINEER MIN. 3 DAYS PRIOR TO OVERLAY
½"
¼"
ORIGINAL DECK SURFACE
SCARIFIED DECK SURFACE
¾" X ¼" STEEL RING
EPOXY TYPE 1 GR. 3 CLASS B OR C
NEW 4" ø STD. GALV. STEEL PIPE
SECTION A
MODIFIED CONCRETE OVERLAY
** ACTUAL PIPE LENGTH (L) SHALL BE DETERMINED BY THE CONTRACTOR IN THE FIELD AND SHALL BE INSTALLED AS SHOWN ON THIS SHEET.
℄ OF DRAIN BASIN
8" NON-SHRINK GROUT
DRAPE WATERPROOF MEMBRANE UP FACE OF GROUT
ACP OVERLAY
MEMBRANE WATERPROOFING (DECK SEAL)
ORIGINAL DECK SURFACE
¾" X ¼" STEEL RING
NEW 4"ø ST'D. GALV. STEEL PIPE
SECTION B
FOR ACP W/MEMBRANE OVERLAY
** ACTUAL PIPE LENGTH (L) SHALL BE DETERMINED BY THE CONTRACTOR IN THE FIELD AND SHALL BE INSTALLED AS SHOWN ON THIS SHEET.
OVERLAY MODIFICATION FOR BRIDGE DRAIN TYPE 6
CURB LINE
7"
℄ BRIDGE DRAIN
CUT & REMOVE EXISTING GRATE BARS
PLAN
DEPRESS ¾"
8"
MODIFIED CONCRETE OVERLAY
NEW 4" ø ST'D. GALV. STEEL PIPE
ORIGINAL DECK SURFACE
SCARIFIED DECK SURFACE
¾" X ¼" STEEL RING
FILL CAVITY WITH CONCRETE AS APPROVED BY ENGINEER MIN. 3 DAYS PRIOR TO OVERLAY
EPOXY TYPE 1 GR. 3 CLASS B OR C
SECTION C
MODIFIED CONCRETE OVERLAY
** ACTUAL PIPE LENGTH (L) SHALL BE DETERMINED BY THE CONTRACTOR IN THE FIELD AND SHALL BE INSTALLED AS SHOWN ON THIS SHEET.
DEPRESS ½"
8" NON-SHRINK GROUT
DRAPE WATERPROOF MEMBRANE UP FACE OF GROUT.
ACP OVERLAY
MEMBRANE WATERPROOFING (DECK SEAL)
ORIGINAL DECK SURFACE
SECTION C
FOR ACP W/ MEMBRANE OVERLAY
DRAIN MODIFICATION BY CORE DRILLING
THIS MODIFICATION MAY BE REQUIRED WHEN THE EXISTING DRAIN IS NOT FUNCTIONAL AND CANNOT BE MADE FUNCTIONAL OR WHEN THE EXISTING DRAIN MUST BE PERMANENTLY PLUGGED AND RELOCATED, AS WITH A WIDENING.
CURB LINE
6"
5"
ROUGHEN SURFACE
OVERLAY SHALL BE DEPRESSED "
EXISTING BRIDGE DECK
EPOXY CEMENT FILLER
5" ø PVC PIPE O.D. = 5.56"
ELEVATION VIEW STRAIGHT DROP
** ACTUAL PIPE LENGTH (L) SHALL BE DETERMINED BY THE CONTRACTOR IN THE FIELD AND SHALL BE INSTALLED AS SHOWN ON THIS SHEET.
INSTALL TO HERE AND TRIM TO ALLOW DRAINAGE OF WATER
ELEVATION VIEW INCLINED DROP
FACE OF EXIST. CURB TO BE REMOVED
CONCRETE AS APPROVED BY ENGINEER
TOP OF EXIST. RDWY. SLAB
PLUG AS NECESSARY
BRIDGE DRAIN PLUG DETAIL
(LOCATIONS)
REMOVE DRAIN GRATING & GALV. STEEL CHAIN. FILL DRAIN WITH CONCRETE GROUT AFTER PLUGGING.
FACE OF EXISTING CURB
TOP OF ROADWAY SLAB
EXISTING TYPE 5 DRAINS
BRIDGE DRAIN PLUG DETAL
REMOVE DRAIN GRATING & GALV. STEEL CHAIN. FILL DRAIN WITH CONCRETE GROUT AFTER PLUGGING.
NOTES
1. DIMENSIONS AND DETAILS OF EXISTING DRAIN TYPES 1 THRU 6. INCLUDING REVISIONS B, C, AND CR, ARE BASED ON "STANDARDS FOR BRIDGE CONSTRUCTION". SHEET E-13, DATED 12/29/67 AND BEFORE.
2. FOR STANDARD BRIDGE DRAINS DATED SEPTEMBER 1, 1965, AND AFTER. IT WILL BE NECESSARY TO GRIND OFF THE NIPPLE EXTRUSION ON ONE SHORT SIDE OF THE TYPICAL GRATE COVER.
3. ALL NEW STEEL EXCEPT STEEL PIPE SHALL BE AASHTO M 183. STEEL PIPE SHALL CONFORM TO ASTM A 53 GRADE B. ALL STEEL SHALL BE GALVANIZED PER AASHTO M 111.
** ACTUAL PIPE LENGTH (L) SHALL BE DETERMINED BY THE CONTRACTOR IN THE FIELD AND SHALL BE INSTALLED AS SHOWN ON THIS SHEET.
± THESE SPACINGS ARE TYPICAL HOWEVER OTHER SPACINGS MAY EXIST
GENERAL NOTES
THIS SHEET IS INTENDED FOR USE AS A STANDARD FOR DRAIN MODIFICATIONS WHICH ARE REQUIRED WITH BRIDGE DECK OVERLAYS OF 1 " NOMINAL THICKNESS. MODIFICATIONS PERTAIN ONLY TO THE STANDARD DRAINS DESCRIBED IN NOTE 1 ABOVE.
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FED. AID PROJ. NO.
SHEET NO.
TOTAL SHEETS
10
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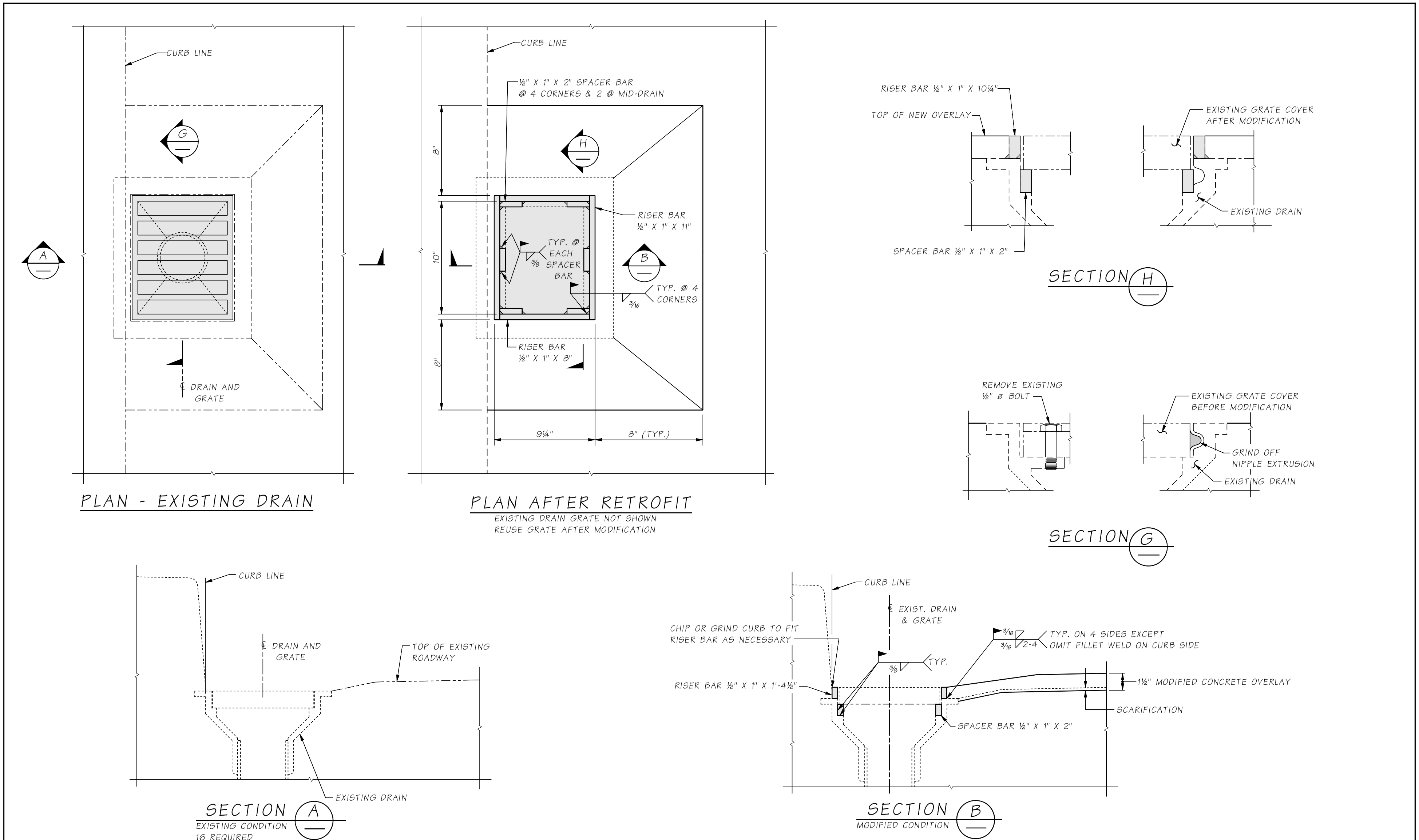
BRIDGE AND STRUCTURES OFFICE
Washington State Department of Transportation
BRIDGE DRAIN MODIFICATION
BRIDGE DRAIN MODIFICATION
BRIDGE SHEET NO.
SHEET
OF
SHEETS
SR
JOB NO. 000
SHEET 1
Thu Sep 02 14:12:10 2010



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Supervisor				REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
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Prelim. Plan By								
Architect/Specialist	DATE	REVISION	BY	APP'D				

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BRIDGE DRAIN MODIFICATION

BRIDGE DRAIN TYPES 2 THRU 5 MODIFICATION FOR OVERLAY

BRIDGE SHEET NO.

SHEET OF SHEETS

SR JOB NO. SHEET 1

Thu Sep 02 14:12:07 2010