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BRIDGE DESIGN MANUAL

SECTION 1: DESIGN STANDARDS

BRIDGE ENGINEERING SECTION

OREGON DEPARTMENT OF TRANSPORTATION

Note: Revisions for April 2024 are marked with yellow highlight. Deleted text is not marked; past editions of the *Bridge Design Manual (BDM)* are available for comparison on the Bridge website.

Preface

PURPOSE:

The purpose of the *Bridge Design Manual (BDM)* is to ensure the design of bridges that are safe, practical, economical, and appropriate for the site through the establishment of standards and best practices.

FORMAT:

Section 1 of the *BDM* is being updated to a two-column format with specifications in the left-hand column, and guidance and commentary on the right. For more information about the BDM format, see the *BDM Format Guide*.

For any portions that have not been updated to the two-column format, consult with the relevant section owners to clarify whether statements are requirements or guidance.

BACKGROUND:

The *BDM* documents design standards and construction practices that have proven to be successful in Oregon over the years. The *BDM* has been consistently updated to draw alongside the advancement of materials and construction techniques and is not intended to prevent the designer from exercising sound engineering judgment. The *BDM* was developed to address the design of conventional bridges. Develop project specific design criteria for non-conventional bridges.

For state-owned bridges the specifications column of the *BDM* is intended to modify and supplement national Bridge Design Specifications (*BDM 1.2.1*). Where in conflict, the *BDM* controls.

DESIGN DEVIATIONS, DESIGN EXCEPTIONS, AND UPDATING:

Adhere to requirements in the specification column unless the section owner technical resource approves the modification. When varying from the *BDM*, communicate with the relevant section owner technical resource regarding the need for a deviation or exception (*BDM 1.2.2*).

If any *BDM* section is found to be inadequate or out of date, consider requesting a *BDM* update from the section owner technical resource. The form to request an update is on the bridge website.

LOCAL AGENCIES:

Local Agency projects use the *BDM* as a guidance document and the primary bridge design specification for the local agency is the *AASHTO LRFD Bridge Design Specifications*. If there is a conflict with the *BDM*, then the *AASHTO LRFD Design Specifications* governs. For design deviation or exception see *BDM 1.2.2*.

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Note: Revisions for April 2024 are marked with yellow highlight. Deleted text is not marked; past editions of the *Bridge Design Manual (BDM)* are available on the Bridge Engineering Section website for comparison.

1.1 Introduction

BDM Section 1 contains standards, practices, and design guidance pertinent to highway bridges and structures design.

See *BDM Section 2* for practices pertinent to design procedures and quality processes for completing highway bridge and structure design.

See *BDM Section 2* for the following information:

- Structural design software.
- Overview of design procedures.
- Roles & responsibilities.
- Quality.
- QPL / Research.
- Preliminary design / TS&L.
- Final design / PS&E.
- Advertisement and award.
- Construction support.
- Other discipline coordination.

See *Bridge CAD Manual (BCM)* for standards and practices pertinent to detailing of highway bridges and structures.

Submit suggested updates for future BDM revisions to Emily Clyburn using the BDM Revision Request Form on the website.

1.2 Bridge Design, General

1.2.1 Bridge Design Standards

1.2.2 Bridge Design Deviations

1.2.3 Bridge Design Categories

1.2.4 Bridge Location and Environment

1.2.5 Structure Appearance and Aesthetics

1.2.6 Bridge Types & Selection Guidance

1.2.7 Bridge Layout

1.2.8 Bridge Security Design Considerations

1.2.9 Bridge Name Plates & Markers

1.2.10 Safety & Accessibility

1.2.1 Bridge Design Standards

1.2.1.1 Standard Specifications and Standard Drawing Manuals

- *LRFD Bridge Design Specifications (9th Edition, 2020)* published by the American Association of State Highway and Transportation Officials (AASHTO).
- *AASHTO Guide Specifications for LRFD Seismic Bridge Design (2nd Edition, 2011 with 2012, 2014, 2015 and 2022 interims)*.
- *AASHTO Guide Specifications for Seismic Isolation Design, 4th Edition (2014).*
- *Guide Specifications for Bridges Subject to Tsunami Effects (1st Edition, January 2022)* published by *AASHTO*.
- *AASHTO Guide Design Specifications for Bridge Temporary Works, 2nd Edition, 2017.*
- *AASHTO LRFD Guide Specifications for Accelerated Bridge Construction (1st Edition, 2018)*.
- *LRFD Guide Specifications for the Design of Pedestrian Bridges (2nd Edition, December 2009 with 2015 interims)* published by *AASHTO*.
- *Manual for Railway Engineering of the American Railway Engineering and Maintenance-of-Way Association (AREMA*) as modified by the individual requirements of each railroad company.
- *Oregon Standard Specifications for Construction*, published by ODOT and pertinent special provisions (for all construction except bridges carrying railways).
- *Oregon Standard Drawings*, published by ODOT, Standards Engineer.
- *AASHTO Guide for Design and Construction of Near-Surface Mounted Titanium Alloy Bars for Strengthening Concrete Structures*
- *AASHTO Guide Specifications for Design of Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements, First Edition*

1.2.1.2 Use of Oregon Standard Drawing and Standard Details

The Standard drawings and standard details prepared by ODOT have been developed through a long history of collaboration with Oregon contractors and fabricators. Consider impacts to both when making modifications.

Use Oregon standard drawings without significant change, as determined by the drawing Engineer of Record. Where a significant change to a standard drawing is needed, submit a design deviation request to the State Bridge Engineer. Where an equivalent ODOT standard drawing or accompanying design detail exists, do not use standard drawings or design details from another state or agency without approval of a design deviation from the State Bridge Engineer.

The standard details do not include the seal of the technical owner. The designer or the Engineer of Record (EOR) is responsible for sealing the contract plans with the standard details, and the design calculations which is specific to the project where the standard details are used.

1.2.2 Bridge Design Deviations (DD) and Roadway Design Exceptions (DE)

The bridge design field is an art that is constantly changing. It is understood that designers will occasionally use innovative details or methods that differ from those contained in this manual and on the standard drawings. Designers having experience in other states may also want to introduce details and methods which have worked well in those states. In addition, context-sensitive design requires the exercise of engineering judgement and sometimes leads to details or methods that satisfy the intent of this manual or the standard drawings, but do not meet the **letter** of these documents.

Submit a request for a design deviation before replacing an established drawing or method from this manual. The two-column layout visually separates standards from guidance with established standards stated in the left column. Deviating from language in the left column of the *BDM* requires a design deviation unless stated otherwise. Replacing standards includes design methods or details established in other states, presented in research reports, or developed by designers. Engineers are encouraged to exercise good engineering judgment, which may result in innovative solutions, including new materials or techniques. Design deviations allow the opportunity for these ideas to be documented and shared, potentially resulting in revisions to standard practice in the *BDM*.

In cases where a standard drawing or *BDM* design requirement is not applicable to the project circumstances and must be modified, a design deviation is necessary. This commonly occurs to meet a project goal, incorporate new technology or to meet a technical requirement e.g. significant modification to accommodate project geometry; ADA requirements; any modified attachments; existing elements; local agency standards; or aesthetics. Modifications made in these instances cannot be based on preference or economy. They must be justified by structure geometry, configuration, constructability, and intended purpose. Use a standard drawing when it satisfies

the need. When modifying a standard drawing, comply with appropriate design specifications (*LRFD*, *BDM*, etc.) to the fullest extent possible.

Modifications to standard drawings on repair and rehabilitation projects do not require Design Deviations, except on bridge rail drawings or when new design techniques or materials are used. This is due to the often unique needs and project specific circumstances on rehab and repair projects. These projects must still comply with *LRFD* and *BDM* design requirements or a design deviation is necessary. The Bridge Reviewer should devote extra attention to modified drawings and raise concerns to the standard drawing owner when appropriate.

Some elements of bridge work may require a Roadway Design Exception (refer to *ODOT Highway Design Manual, Part 1000* for more information about Roadway Design Exceptions). All Design Exceptions related to bridge rail must be signed by the State Bridge Engineer prior to approval by the State Roadway Engineer. In addition, when Design Exceptions involve structural work, such as sidewalk widening, the State Bridge Engineer is expected to concur. A design deviation is not required for project elements where a Design Exception is submitted. When it is unclear whether a design deviation or exception is required, contact the relevant bridge technical resource as early in the design process as possible.

1.2.2.1 Design Exceptions

When the State Bridge Engineer needs to provide concurrence on a Roadway Design Exception, modify the Roadway Design Exception process as follows:

1. Complete the Roadway Design Exception Form. Get information from both Roadway and Traffic designers to complete the form. Contact the ODOT Design Exception Mailbox (mailto:ODOTDesignExceptions@odot.oregon.gov) to get a control number.
2. Revise the first two concurred by signature lines to be for the **State Bridge Engineer** and the **ODOT Region Tech Center Roadway Manager**.
3. Send a link to the draft Design Exception to the relevant bridge technical resource and all necessary Region resources, for review prior to signing.
4. After addressing comments, save the Design Exception as a PDF, create the necessary signature fields, and sign the file.
5. Submit a link to the file for Region signatures according to appropriate Region process.
6. After Region signatures are complete, the Design Exception should be submitted to the ODOT Design Exception Mailbox for final processing. Depending on the Region, this step may be the responsibility of the designer.
7. The form will be sent to Bridge Section for the State Bridge Engineer's signature as part of final processing.
8. The submitter will be notified once signing of the Design Exception is complete.

1.2.2.2 Design Deviations

Prior to submitting a design deviation request, contact the relevant bridge technical resource for guidance. They can help discern when a design deviation is necessary; an email inquiry about the proposed modification suffices for confirmation on whether a formal design deviation is necessary or not.

A design deviation form is available on the ODOT Bridge Engineering website. In the request, include a brief description of the project, an explanation of the issues, what is being proposed, a justification for the proposed design deviation, and any supporting documents. The request may be submitted by email. Send design deviation requests to both:

Ray Bottenberg, State Bridge Engineer, Raymond.D.BOTTENBERG@odot.oregon.gov

Emily Clyburn, Bridge Design Manual Standards Engineer, Emily.Clyburn@odot.oregon.gov

The request will be distributed to and evaluated by the BDM technical resources. On local agency projects include the Senior Local Agency Bridge Standards Engineer for review. The State Bridge Engineer makes the final decision to accept or reject a request for design deviation. A response to each request will be returned by email within 10 business days.

1.2.2.3 Technical Bulletins

When technical issues arise between scheduled *BDM* updates which require urgent distribution of guidance to the design community, a Technical Bulletin is issued. Check the ODOT Technical Guidance web page for Technical Bulletins.

1.2.3 Bridge Design Categories

1.2.3.1 Bridge Design Categories, General

Bridge design categories determine the design criteria and requirements for projects on bridges. These categories include the following:

- Modernization.
- Retrofit.
- Preservation.
- Maintenance.
- Local Agency.

For projects involving multiple bridges, determine the category per bridge. Bridge Program will complete an initial categorization at desk scoping, when possible. Validate the categorization during field scoping. For projects initiating outside Bridge Program, establish the bridge design category at field scoping. Contact the Bridge Design Manual Standards Engineer with questions regarding the bridge design category. Document the identified category in the Bridge Design Criteria. When project need's change after scoping, contact the Program Manager according to *BDM 2.9.5* prior to changing the design category for any bridge.

These categories operate independently from the roadway design standards identified in the *Highway Design Manual Section 100*. A project involving bridges will have both a roadway design standard and at least one bridge design category. For projects initiating outside of Bridge Program, the bridge design category may be based on the work required by the roadway design standard; however the bridge design category must still be determined.

The design requirements governed by these categories include:

- The level of alternatives analysis required and type of documentation, i.e.:
  - Bridge TS&L Report.
  - Bridge Strategy Memo.
  - Bridge Design Criteria and Standards Assessment (DCSA).
- The extent of bridge rail work.
- The level of seismic retrofit.
- The amount of Bridge Section review required.

All projects, regardless of design category, require a minimum of one site visit to be performed. Identify and document existing conditions and defects that may exist, to tailor solutions to site conditions.

The EOR, or assigned designee, will perform the site visit. Collaborate with other disciplines involved in the project at the site visit to discuss overlapping design elements. Coordinate site visits with the Region Bridge Inspector and Bridge Maintenance to facilitate access, where appropriate, and to assure all defects are identified.

Perform the site visit early enough in the project delivery process to allow time for incorporation of site visit findings into project DAP documents.

Roadway design standards and other agreements govern work outside of the bridge footprint, including approach slabs, drainage features and bridge rail transitions.

While site visits are required on all projects, they are more crucial on rehabilitation, preservation, or retrofitting of existing structures. These are projects where existing conditions often dictate design elements and solutions.

Consider performing an initial scoping site visit prior to TS&L, as well as a site visit near the end of project delivery to ensure site conditions have not changed.

1.2.3.2 Bridge Modernization

The bridge modernization design category applies to the following project types:

- New bridge construction.
- Bridge replacement.
- New bridge deck or superstructure.
- Phase 2 seismic retrofit projects.
- Bridge widening requiring the addition of one or more girder lines.

On bridge modernization work, use full current bridge design standards for the entire bridge, except where practical design opportunities or innovations are available. In these cases, document all variations from bridge standards in design deviations or exceptions, as described in *BDM 1.2.2*. Per *BDM 2.9.5*, submit a Bridge TS&L Report for all bridge modernization category work.

1.2.3.3 Bridge Retrofit

The bridge retrofit design category includes any work that upgrades the functional characteristics of a bridge, including load-carrying capacity, except that covered by modernization. A bridge retrofit project can include multiple work elements. Potential work elements include:

- Bridge strengthening.
- Structural concrete overlays and inlays.
- Overhang or sidewalk widening.
- Bridge raising. See *BDM 1.16.1*.
- Bridge rail replacement.

When evaluating projects that include bridge retrofit work during the scoping and Project Initiation phases, consider the bridge holistically. Consider what other related elements of the bridge should also be updated. As an example, retrofitting a bridge rail may be appropriate during a structural concrete overlay, or a strengthening may also include phase 1 seismic retrofit. Clearly document modifications to scope from this process in the project documents (Scoping Notes, Project Charter, CMR or DCSA, depending on project phase) and solicit Program Manager approval prior to proceeding.

Identify required design deviations and Design Exceptions in the DCSA. Use a Bridge Strategy Memo, according to *BDM 2.9.6*, to communicate these decisions to the project team.

1.2.3.4 Bridge Preservation

The bridge preservation design category applies to work that does not change the functional characteristics of a bridge, including load-carrying capacity. Relevant work includes:

- Bridge coating.
- Joint repairs and replacements.
- Protective screening.
- Phase 1 seismic retrofit.

- Non-structural concrete overlays (that do not trigger updating the load rating per *ODOT LRFR 1.7*).

The following qualify as bridge preservation, unless other work on the bridge requires another category:

- Bridge rail height modifications using *BR285* or similar.
- Strengthening required to carry temporary construction loads, i.e. containment.

This category of work does not require design deviations. Instead, use the DCSA to document project decisions, such as overlay type, and as documentation of communication with the relevant technical specialist. Use the DAP Narrative to communicate all bridge work with the project team, without requiring a Bridge TS&L Report or Bridge Strategy Memo.

When using innovative methods on bridge preservation projects, submit a BDM Revision Request form so the method can be evaluated and possibly adopted into the *BDM*. Thoroughly describe the method and how it applied to the project(s) on which the innovation was used.

1.2.3.5 Bridge Maintenance

The bridge maintenance category includes those projects, of any scale, that are funded exclusively with Major Bridge Maintenance (MBM) funds or other similar sources of state funding. Design these projects according to the *Bridge Maintenance Design Manual* and coordinate with the Bridge Maintenance Engineer and the District Bridge Supervisor. Refer to TSB22-01(B) for MBM work related to roadside safety feature design (i.e. bridge rails).

1.2.3.6 Local Agency

The local agency category includes projects with work on local agency structures. *BDM* standards apply to all local agency projects, both certified and non-certified, except as modified for local agency projects in the following *BDM* sections:

- *BDM 1.9.2.1 Deck Slabs*
- *BDM 1.9.2.3 Precast Deck Panels*
- *BDM 1.9.3.1.6 Asphaltic Concrete Overlays*
- *BDM 1.9.3.1.6.1 Waterproofing Membranes*
- *BDM 1.9.4.2 Deck Closure Pours*
- *BDM 1.13.1 Bridge Rail*.
- *BDM 1.17.2.1 General Considerations, Seismic*.

Refer to *Local Agency Guidelines (LAG)* and *ODOT Delivery of Local Agency Projects (ODLAP)* for additional guidance on non-technical design content.

Local agency bridges on the National Bridge Inventory (NBI) require bridge design deviations. Local agency bridges not on the NBI, do not require a design deviation and the Local Public Agency (LPA) has approval authority.

1.2.4 Bridge Location and Environment

The level of effort to prevent corrosion depends mainly on the potential for exposure to a corrosive environment. Refer to the appropriate *BDM* sections for the appropriate material selection and element treatment based on the bridge location and environment as defined in the following sections.

1.2.4.1 Marine Areas

Marine areas are typically the most highly corrosive conditions and require the highest level of protection against deterioration and corrosion. For the purposes of determining when the specified corrosion protection is required a marine area is defined as any of the following:

- A location in direct contact with ocean water, salt water in a bay, or salt water in a river or stream at high tide (substructure).
- A location within ½ mile of the ocean or salt water bay where there are no barriers such as hills and forests that prevent storm winds from carrying salt spray generated by breaking waves.
- A location crossing salt water in a river or stream where there are no barriers such as hill and forests that prevent storm winds from generating breaking waves.

1.2.4.2 Coastal Areas

Coastal areas are defined as locations within 25 air miles of the Pacific Ocean, and not otherwise classified as Marine area.

1.2.4.3 Snow and Ice Areas

Snow and ice areas are defined as all areas of central and eastern Oregon, the Columbia River Gorge, Jackson County, and any other areas above 1500 feet elevation or otherwise identified by the associated maintenance district. These areas are intended to include all areas with the potential to receive periodic application of deicing chemicals.

1.2.4.4 Mild Areas

Mild areas are defined as all areas not in a marine area, coastal area, or snow and ice area. This includes all of western Oregon below 1500 feet elevation that is not within 25 miles of the Pacific Ocean.

1.2.5 Structure Appearance and Aesthetics

1.2.5.1 General

Consider the structure appearance with respect to its surroundings and the context of the site.

ODOT has no general directive or mandate on aesthetics or aesthetic design.

For bridges, appearance is best when elements are few and simple.

Bridge elements are pleasing when the structural intent is clear with respect to the size and shape of the element. Elements forced into a non-structurally responsive shape for decoration are not considered aesthetically pleasing and may be a significant distraction and a safety hazard. Decorations on bridges that are not part of the structural support system may not be maintained to the same level as the structural portions of the bridge. A separate IGA can be executed for maintenance of non-structural features.

Aesthetics and environmental considerations may have apparent conflicts. Historic, ADA and environmental issues may impact the bridge rail type, structure configuration, type of foundation or bent placement. Start the permit application and coordination process for historic structures as early as possible in the design stage. Aesthetics concerns, especially within an existing documented site context, are valid issues that can and should impact resource agencies permitting considerations. Balancing historic needs and ADA requirements requires early coordination and requirements are adhered to the maximum extent feasible.

There is a misconception that improving appearance always costs more. The challenge to the engineer is to use creativity and ingenuity to improve the appearance without increasing cost. When people think that improved appearance is going to add costs, they are generally thinking in terms of add-ons, special ornamental features or special colors. The greatest aesthetic impact can be made by the structural elements themselves. These are seen first, and at the greatest distance. The bridge can be made attractive if these major elements are well shaped, and if they fit in well with the surroundings.

The following topics are commonly known to assist in producing visually pleasing structures. They are discussed in more detail in the following sub-sections.

- Location and surroundings.
- Horizontal and Vertical Geometry.
- Superstructure Type and Shape.
- Bent Shape and Placement.
- End bent Shape and Placement.
- Parapet and Railing Details.
- Colors.
- Textures.
- Ornamentation.

1.2.5.2 Location and Surroundings

When determining the appearance of a bridge, the bridge must be considered in context with its surroundings. Decisions need to be made regarding what color, shape and type of bridge will look best at a given location. The surrounding area may be industrial, urban, or rural. A bridge that looks pleasing in a rural setting may look totally out of place in an urban area.

Individual bridges that span a major land area or body of water, because of their large size, dramatic location, and carrying capacity, will tend to dominate their surroundings. While these structures must harmonize with the surroundings, their importance and size requires that the aesthetic qualities of the structure stand on their own. Multiple bridges seen in succession create a cumulative aesthetic impact on the landscape that must be considered. In these situations, there is more reason for uniformity, and there should be no noticeable differences between structures, without an obvious reason. A specific theme for a particular route, such as a parkway, is often appropriate.

Routine bridges, such as highway overpasses and stream crossings, should be simple, with minimal changes, and with all of the elements in clear relationship with one another. Since many of these bridges are viewed in elevation by those traveling on a roadway below, the structure type, span lengths, and proportions, as viewed in elevation, should be carefully considered.

Bridges that are infrequently viewed, such as those on lightly traveled roadways, are rarely seen by anyone. In these cases, attention to the elements that can be seen from the roadway surface such as parapets, railings, transitions, and road surface, are important.

1.2.5.3 Horizontal and Vertical Geometry

Geometric design standards often dictate the orientation of a bridge. The emphasis is on the need for safe, convenient driving and providing a more attractive highway system. Bridges must adapt to the highway alignment. Thus, they often lie within the curvature of the road and follow the slopes or curvature in elevation. Large curvature is not only desirable from a safety standpoint, but also for aesthetics.

With skewed structures, when it is necessary to orient the substructure parallel to the feature crossed, a wide bridge presents a greater visual impact. The use of natural surfaces that blend in with the surrounding environment may lessen the visual impact. Bents and end bents in waterways that lie parallel to the river's banks look better than those placed perpendicular to the crossing road.

If an alignment requires a curved bridge, then the external longitudinal lines, traffic barriers, and fascia lines of the structure should follow the curved centerline to provide a smooth visual flow. A smooth transition helps the structure fit in with the local topography. Parallel lines should be maintained by matching barrier, sidewalk, curb and fascia depth across the structure.

1.2.5.4 Superstructure Type and Shape

The appearance of a bridge is greatly influenced by different aspects of the superstructure. These include the superstructure type, depth, overhang width, number of spans, and span lengths. One way to make the structure light and slender, without making it appear weak and unsafe, is to use a favorable visible slenderness ratio (the ratio of span length to the visible structure depth, including the decking and any concrete traffic barrier or steel railing). The typical visible slenderness ratio will vary from approximately 10 to 40 depending on the type of superstructure chosen.

A girder depth that is too shallow gives the appearance that the bridge is not structurally safe. A girder that is too deep makes the bridge look bulky and overpowering. Bridges with a well-proportioned slenderness ratio denote strength without excessive materials.

An additional guideline that enhances the appearance of multiple spans is to avoid changing girder depths from one span to another. This would give a very awkward appearance and would not allow the structure to flow evenly across the bridge. From an aesthetic standpoint, deck overhang should be proportional to the girder depth; a desirable overhang would be about 2/3 the girder depth. Vertical stiffeners make steel girders seem heavier, and should be avoided on the fascia side of fascia girders. Haunched girders can make a bridge look more slender, and help demonstrate the flow of forces in the bridge. Fishbelly girders create a heavy look, and could tend to look awkward. Some structure types are more visually elegant than others, such as trapezoidal box girders and concrete segmental bridges. An arch bridge is one of the most natural bridge types, and generally considered one of the most pleasing. Both thru and deck arches can be considered.

1.2.5.5 Bent Shape and Placement

The visual impression that a person gets from a bent is primarily influenced by the proportions, the relative width and height, and the configuration of the bent cap with respect to the bent columns. Bent proportion, in turn, is determined by the bridge geometry and superstructure type and shape. Bents can broadly be classified as either short or tall. Short bents are typically more difficult to design with aesthetic proportions. Care should be taken in proportioning a bent to make sure that horizontal lines of the superstructure are not interrupted. Large bents may direct attention away from the superstructure. Bents that are too slender may convey a feeling of instability.

However, there are aesthetic issues that are common to all bent types involving the shape of the columns and the bent caps. The selection of the proper bent type can be dictated by the site, bridge geometry and design considerations.

The shape and location of the columns affect the appearance of the bents. The light reflecting from the surface often controls how the viewer perceives it. A square or rectangular column with beveled corners will appear more slender due to the edge lines and varying shades of reflected light. The designer needs to assure that the treatments used are in harmony with the

rest of the structure. Bent caps, cantilevered ends, and column spacing can be designed to make the bent appear more graceful. For hammerhead bents, the stem width and height, and the cantilever length and depth should be carefully balanced, and in pleasing proportion. Solid bents can be battered to improve their appearance. The batter should be determined by the bent height and the relative dimensions at the top and bottom of the bent. Gradual lines are important. While tall bents are less common than short bents, they allow a greater opportunity for aesthetic treatment.

1.2.5.6 End Bent Shape and Placement

For most simple span bridges and some multi-span bridges, the end bents are the most visible elements. While the end bent's function is to support the superstructure and transfer loads to the ground, it is important to maintain proper proportion in order to create a good appearance. Good proportions between various elements of the bridge give character to the bridge. For the end bents it is important to consider the relationships between the exposed end bent height and length, the size and type of wingwalls, and the superstructure depth. An attempt should be made to achieve a balance between these elements.

The designer must maintain order between the lines and edges of the structure. Too many lines, or lines that are close to but not parallel to each other, can disrupt the eye and diminish the appearance of the bridge. The monotony of a large flat wingwall can be broken up using textures such as scoring, recessing, or grooving. Surface textures, either by using or simulating natural stone around the area of the bridge, can be used to integrate the structure with its surroundings.

The orientation of the end bents to the feature crossed will create different visual appearances. End bents on severe skews can have very long stems and wingwalls. Consideration should be given to the aesthetic impact of those concrete surfaces. Wingwalls are often very predominating features. The orientation of the wingwalls allows for more or less visual impact. On divided roadways, consider the view presented from the opposite direction of travel.

1.2.5.7 Parapet and Railing Details

The railings or barriers, along with the deck fascia and fascia girders, are sometimes the most dominant visual aspect of the bridge. The railings are viewed by people traveling under the structure who see them in elevation and by people in vehicles on the bridge traveling parallel to them. When vehicle speeds are high, the railing or barrier should have simple and pronounced details because passengers cannot notice fine details. The shape of the railing or barrier system should relate to its function and the overall aesthetic design of the bridge.

The design and appearance of any fencing to be placed on the bridge should be consistent with the railing or barrier system. The vertical supports of the screening should align with the railing post spacing. Fencing on concrete barriers should be detailed to match the construction joints and the ends of the barriers.

See *BDM 1.13.3* for additional information.

1.2.5.8 Colors

When there is a reason to color the concrete, steel, or railings, a decision should be made whether the color should complement or contrast with the surrounding environment. Strong consideration should be made to the fact that colored concrete or steel will require a high level of maintenance. The designer should also consider the appearance if regular maintenance is not performed (e.g., peeling paint, rust spots, etc.).

Coloring agents are not allowed in concrete because of complicated quality control, difficulty in matching colors in each batch, and the high cost of materials. It is nearly impossible to get an identical color of concrete from one pour to the next, or over a period of time between placements. Staining concrete can create a mottled appearance when appropriate to match natural stone, and can be effective if a trial section is used to qualify the process. External coatings are allowed, and when applied correctly can achieve the desired appearance. However, they have durability limitations, and must be used with caution due to concern regarding the owner's ability to maintain the coating.

1.2.5.9 Textures

Texturing concrete can be achieved through form liners, panels, stone or brick veneer, or acid washing. Any texturing should fit in within the overall design and proportions of the structure.

Several types of commercial form liners are available. Natural stone or brick facades can also be used. Stone is most often used for parkway bridges and those in rural settings. The cost of stone covering can be quite high; and should therefore be limited to areas of high visibility and established contextual settings. When a concrete cap is used on the top of a wingwall or retaining wall, it should be visually proportioned to the wall itself.

1.2.5.10 Ornamentation

Ornamentation can be added to a bridge in special circumstances. The additional cost of add-ons is rarely justified except in cases of importance to the community (such as a gateway to a city) or of historical significance. Details such as ornamental light posts, columns or pylons, real or simulated gatehouses, commemorative plaques or reliefs may be added. The designer should consider these details carefully since it is just as easy to detract from the overall appearance of the bridge, as it is to improve it.

Such details are secondary to the primary purpose of the structure, which is to provide a safe and efficient crossing to the public. Ornamental and non-structural details require additional coordination, sketches and drawings to ensure that the details will add to the aesthetic characteristics of the structure in a way acceptable to all concerned. ADA requirements need to be reviewed carefully to ensure these items do not create protruding objects or restrictions in the clear width available to pedestrians.

Local stakeholders sometimes request ornamental screening and features on overpass structures to showcase local attractions as a gateway to their community. Ornamental protective screening

should not be a distraction to drivers, and must not cause sight distance or clearance problems. Treatments must not reflect a commercial interest. See *BDM 1.13.11* for additional screening requirements.

1.2.6 Bridge Types & Selection Guidance

1.2.6.1 Bridge Types and Economics

Bridge superstructure type is generally the most important factor influencing bridge costs. Substructure cost is normally included in bridge deck area unit cost. In some situations, the substructure cost can be greater than 50 percent of the unit cost, when significant seismic design and details are required for the bridge. Each project site is unique and should be evaluated for conditions that alter the usual cost expectations. For usual cost expectation of bridge deck area unit cost, refer to Bridge Section's annual Bridge Cost Data for bridges constructed in Oregon and the FHWA website (https://www.fhwa.dot.gov/bridge/nbi/sd.cfm) for structures in other states in the national bridge inventory. Use the last three years average unit cost as a basis for comparison.

If an estimated deck area unit cost for a bridge with a typical substructure is more than 15 percent higher than the average unit cost shown in both the *Bridge Section's Cost Data* and the FHWA bridge replacement unit cost, inform the design lead and design team as soon as possible including the reasons for the higher unit cost. Re-evaluate the alternatives with the design team for their effectiveness at meeting the project's basic goals and look for innovative solutions to address the high cost.

Various types of bridge superstructure provide efficient solutions for different span arrangements. There are many reasons for choosing particular span length(s) for a bridge, some of which are discussed below. There is generally significant overlap for common span ranges, so multiple bridge types are viable. *Table 1.2.6.1-1* shows various bridge types categorized by construction material and method of construction with the design span ranges.

Table 1.2.6.1-1 Bridge Type Span Range

Bridge Type	Span Range (feet)
Precast concrete slabs	up to 83
Precast concrete box beams	up to 120
Cast-in-place concrete slabs	up to 66
Precast integral deck concrete girder	up to 130
Precast concrete girder, BT72	up to 140
Precast concrete girder, BT84	up to 160
Precast concrete girder, BT90 & BT96	up to 183 *
Cast-in-place box girder	up to 160 **
Cast-in-place post-tensioned box girder	up to 300 **
Steel girder	40 to 400
Steel box	200 to 400**
Steel truss	greater than 300
* Length for BT90 & 96 is limited by prestressing bed capacity for Oregon precasters.	
** Normally used for longer, multi-span continuous bridges.	

When using precast or prefabricated girders, verify that there is an acceptable route for shipping. As girder lengths increase, shipping becomes more difficult on roadways with sharp curves, high superelevation or load-restricted bridges.

Timber bridges up to 30 feet of length may be considered for special situations (See *BDM 1.8.1*). The cost of a timber bridge may be more than concrete and steel bridges of the same length.

Do not use cast-in-place concrete slabs with any span greater than 66 feet. Cast-in-place concrete slab superstructures have significant dead load deflections. Even if actual deflections match estimated deflections, it will likely take 10 to 15 years for creep deflection to diminish. For longer span lengths, the ride quality would be unacceptable while waiting for the creep deflection to occur.

Do not use voids in cast-in-place concrete slab superstructures. Although such designs are effective at reducing the structure weight and dead load deflections, it is very difficult to secure the voids in the field. The potential for failure is unacceptably high.

When cast-in-place slabs are used, ensure the edge beam requirements in *LRFD 4.6.2.1.4, 5.14.4.1* and *9.7.1.4* are met.

Where a design deviation is approved by the State Bridge Engineer for use of voids in a cast-in-place concrete slab superstructure, apply the edge beam requirements listed above to this type of bridge.

Use HPC concrete in cast-in-place concrete slab superstructures. Place concrete full-depth of the slab (i.e., no horizontal construction joints). For cast-in-place slab superstructures having any span greater than 40 feet, apply a deck sealer product (from the QPL) at least 60 days after placement of the slab.

1.2.6.1.1 Precast Concrete versus Cast-in-Place Concrete

Formwork is the key to concrete structure costs. Use of standard forms or repeated use of specially built forms means lower costs. For smaller bridges in remote areas, precast or shop-fabricated elements usually lead to the most economical solution. Also see *BDM 1.16.2, Accelerated Bridge Construction*, for more guidance in the use of precast elements.

Precast concrete slabs have the following advantages:

- Good for shorter stream crossings, low-volume roads, and remote locations.
- No falsework required in roadway or stream.
- Fast, simple installation, saving construction time.
- Shallow depth providing greater clearance to stream or roadway surfaces below.

However, they have problems with:

- Providing smooth riding surfaces. (Wearing surface is required to level up except for low-volume roads.)
- Accommodating horizontal curves, gradelines, or superelevations. (Thickness of ~~AC~~ wearing surface to accommodate superelevation can become excessive.)

Precast concrete box girders, and deck Bulb-T girders have most of the same positive and negative points as precast concrete slabs. They can accommodate longer spans, but they do have deeper depths resulting in less clearance to stream or roadway surfaces below.

In general, cast-in-place concrete spans are a good choice for:

- Accommodating horizontal curves, gradelines, or superelevations.
- Longer spans.

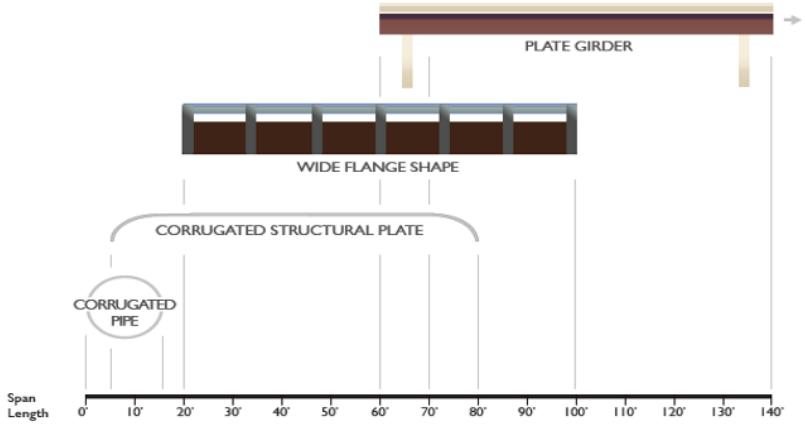
However, three drawbacks are:

- Falsework is required.
- Falsework in the roadway below a grade crossing creates traffic hazards.
- Settlement of falsework before post-tensioning begins is a potential problem.

1.2.6.1.2 Short Span Steel Bridges

Steel provides an excellent solution for short span bridges because steel is often lighter than other materials, resulting in smaller or fewer erection cranes and smaller substructures. In addition, short span steel bridges can be fabricated off-site in a controlled environment and be ready to erect when it reaches the bridge site. Several section options are available depending on the length of the bridge, including buried plate structures, wide flange shape/rolled beams and plate girders for span length from 20 feet to 140 feet as shown *Figure 1.2.6.1.2-1*.

Figure 1.2.6.1.2-1 Short Span Steel Bridge Alternatives



1.2.6.1.3 Composite Steel Girder Bridges

Steel construction extends the span length range and usually does not require falsework in the roadway or stream. Used for simple spans up to 260 feet and for continuous spans from 120 feet to 400 feet. This bridge type has relatively low dead load when compared to a concrete superstructure which makes it an asset in areas where foundation materials are poor. Shipping and erecting of large sections must be reviewed.

1.2.6.1.4 Bridge Widening

Generally, a type of construction that matches the existing bridge should be considered for the widened portion. It is desirable to design the widened portion to have a similar appearance to the existing. With these considerations, similar stiffness between the existing and widened structures can be achieved.

See *BDM 1.3.3.3* for structure widening loading.
See *BDM 1.5.6.2.6* and *1.5.7.11* for concrete structure widening.
See *BDM 1.6.2.18* for steel structure widening.
See *BDM 1.11.3.3* for substructure widening.

1.2.6.1.5 Design Criteria for Major or Unusual Bridges

Some elements of design criteria for major and unusual bridges may not be appropriate for normal bridges and may be dependent on the location and expected service level. For those bridges the design criteria will be established specifically for each bridge in a collaborative effort between ODOT Bridge Engineering Section and the Region. Early coordination is required to allow time to establish the design criteria. See *BDM 1.2.6.3.2* for further guidance regarding unusual bridges.

1.2.6.1.6 Maintenance and Provisions for Inspection of Bridges

- Formal constructability and maintainability reviews by representatives of the Construction and Maintenance Sections are required for most bridges to determine the practicality and feasibility of erection/construction of the bridge as assumed in the design as well as adequacy for future maintenance.
- Preparation of an Inspection and Maintenance Guide for the future operation of each major or unusual bridge (see *BDM 2.10.8*).
- Consider designing for the possibility of future bearing replacement. Bearing replacement requires the use of jacks to lift the superstructure off the bearings to be replaced. Indicate the position of these jacks, and allowable jacking loads, on the drawings (per *BDM 1.3.4*). Provide distribution reinforcement to accommodate the jack loads in the top of the piers and the soffit of the superstructure. Further, consider the relocation of the reactions in the transverse analysis of the superstructure when the jacks are engaged to replace the existing bearings.
- Bridges fabricated from coated structural steel should be designed for future recoating according to *BDM 1.6.4.4.1*.

1.2.6.2 Substructure Guidance

See *Section 1.11* for information and design guidelines for end and interior bents and wingwall layout.

1.2.6.3 Special Considerations for Federal-Aid Projects

1.2.6.3.1 Alternate Designs

According to the Federal Highway Administration (FHWA), the practice of providing alternate designs for major bridges results in substantial savings in bridge construction costs. Current FHWA policy states that use of alternate designs is optional and at the discretion of State highway agencies. If alternate designs are appropriate, consider the following:

- Utilize competitive materials and structural types. Prepare each alternate design using the same design philosophy. (That is, LRFD design, finite element analysis, etc.) Ensure the design/construction requirements for the entire bridge (foundation, substructure, deck) are compatible.
- Prepare estimates for all Alternate Designs during the TS&L design phase.

Note: Do not confuse this **Alternate Designs** with the TS&L **Alternatives Study**. This Alternate Designs is the actual preparation of two or more designs, and plan sheets, to be included in the bid documents.

1.2.6.3.2 Unusual Structures

FHWA policy requires **unusual bridges** to be approved by FHWA before being designed. An u**nusual bridge** may have:

- Difficult, new or unique foundation elements or problems.
- A new or complex design concept involving unique operational or design features.
- Design procedures which depart from current acceptable practice.

Examples of unusual bridges include:

- Cable-stayed, suspension, arch, segmental concrete, moveable, or truss bridges, and other bridge types which deviate from *AASHTO Design Specifications or Guide Specifications.*
- Bridges requiring abnormal dynamic analysis for seismic design.
- Bridges designed using a three-dimensional computer analysis.
- Bridges with span lengths exceeding 500 feet.
- Bridges with major supporting elements of ultra-high strength concrete or steel.

Other unusual structures include:

- Tunnels.
- Geotechnical structures featuring new or complex wall systems or ground improvement systems.
- Hydraulic structures that involve complex stream stability countermeasures.
- Designs or design techniques that are atypical or unique.

Where unusual bridges are identified, seek FHWA involvement at Project Initiation. Do not advance the design beyond TS&L without FHWA approval.

1.2.6.3.3 Experimental Features Program

An experimental feature is a material, process, method, or equipment item that:

- Has not been sufficiently tested under actual service conditions to be accepted without reservation in normal highway construction, or
- Has been accepted, but needs to be compared with acceptable alternatives for determining relative merits and cost effectiveness.

Although the Experimental Features Program is normally used in conjunction with Federal-Aid projects, the program format has occasionally been followed for projects funded entirely with State funds. In some cases, FHWA has paid part of the research cost for basically a State-funded experimental program.

The intent of the Federal-Aid Experimental Features Program is to allow ODOT time to develop, test, and evaluate specifications for new, innovative, or untried products or processes.

1.2.6.3.4 Specifying Proprietary Items

To encourage competitive prices from manufacturers and suppliers, FHWA has established a policy for specifying proprietary products or processes for Federal-Aid projects. Generally, **proprietary** means:

- Calling out a product on plans or in specifications by brand name.
- Using specifications written around a specific product in such a way as to exclude similar products.

The policy basically says:

- You must use two, preferably three, products when specifying by name brand.
- You can use generic specifications patterned after a specific item if at least two manufacturers can supply the item.

On the other hand, specifying one proprietary item is allowed only:

- If it qualifies for the experimental features program.
- If, with written justification from ODOT, FHWA specifically approves in advance a single product, which is essential because of compatibility with an existing system, or the only suitable product that exists.

1.2.6.3.5 Use of Debris from Demolished Bridges and Overpasses

Public Law 109-59 Section 1805, dated August 10, 2005, mandates that for Federal-Aid bridge replacement and rehabilitation projects, States are **directed to first make the debris from the bridge demolition available for beneficial use by the Federal, State or local government, unless such use obstructs navigation.** Links are provided for more information:

- Public Law 109-59 August 10, 2005.
- FHWA Memorandum of March 7, 2006.

Environmental regulations may prohibit the use of debris in waterways.

1.2.6.4 Use of Salvage Materials

ODOT Bridge Engineering Section does not prefer the use of used bridge items. Use new materials for new bridges, replacement bridges, and for added portions of widened bridges. Incorporation of used materials requires an approved design deviation (see *BDM 1.2.2*). Include the following issues in a design deviation request:

- Locate and include in the project records for the new bridge all original material certifications and documentation of material properties.
- Document the condition of the used materials.
- Locate and include a copy of applicable portions of the original calculation book in the project records for the new bridge. The copied portions may be scanned and transmitted electronically to the design engineer. Hard copies should be made and included in the calculation book for the new bridge.

- Prepare a new calculation book for the new bridge.
- Document agreement from FHWA (on Federal projects) with a Public Interest Finding processed through Roadway Section.
- Designate on the new plans the portions of the new bridge that are built with salvaged materials.

1.2.7 Bridge Layout

At the start of the Preliminary Design Phase, after collecting and reviewing available project data, start identifying site constraints that will impact or affect the bridge layout. Suggested items to discuss with respective project team members (list may not include all applicable items):

- Right of way.
- Geology; poor soils.
- Known buried hazardous materials.
- Waterway; thalwag, potential scour areas.
- Floodplains.
- Riparian zones.
- Wetlands.
- Historic resources.
- Archeological sites.
- Buildings.
- Parks.
- Air space envelope.
- Fluvial envelope.
- Railroad envelope.
- ADA improvements.

Consider these items early in the bridge layout process. Learning of these constraints later may cause rework that can affect both schedule and budget.

1.2.7.1 ADA Considerations

ADA design exceptions are rare in cases where new infrastructure is being built. Plan for ADA requirements in the design phase. On bridge retrofit projects expect to address ADA improvements as part of overhang or sidewalk widening.

Review the ODOT CQCR asset layer for ADA upgrades to be incorporated into the project as a result of a citizen complaint. For more information about the CQCR process refer the *Highway Design Manual, Part 800*.

Review *BDM Appendix B*, for determining curb ramps triggered with paving overlays and *Section 1.2* for ADA compliance. Refer to ODOT operational notice *MG100-107* for what is determined to be maintenance activities versus an alteration that would require curb ramps to

be reconstructed. Maintenance activities shall not be modified for the purpose of avoiding triggering ADA upgrades.

1.2.7.2 Spans and Proportions

COLUMN LOCATIONS

Column locations, which of course affect span lengths, are subject to clearance requirements of *BDM 2.14.4.2, AASHTO* standard clearances, and hydraulic considerations. After these conditions are met, spans lengths may also be governed by environmental issues, economics and aesthetics. Consider alternate structure types to best fit the needs of the site.

Consider the effects of columns in waterways when locating columns and setting span configurations. Consider the possibility for scour or difficulty in inspecting a column that is in the highest flow area of a river. Avoid placing the column directly in the middle of the river.

Protect columns located in the median of a divided highway and within the clear zone (as determined by the roadway designer), from traffic by a guardrail or concrete barrier. Check with the roadway designer regarding which barrier will be used. It will affect the bridge's appearance and may influence the type of column selected. Design according to *BDM 1.3.5*.

STRUCTURE DEPTH

Structure depth including deck (also referred to as superstructure depth) is generally controlled by span length and clearance limitations. Although a minimum depth structure may be aesthetically appealing, it may not be the optimal solution for the site.

For steel superstructures, use the minimum depth recommended in *LRFD Table 2.5.2.6.3-1* for estimating purposes. Girder depths for haunched girders made continuous may be reduced up to 20 percent. For haunched girder, use minimum depth of L/40 at center of span and L/20 at intermediate bent.

For concrete superstructures with continuous spans, use the minimum depths given in *Table 1.2.7.2-1*.

Table 1.2.7.2-1 Minimum Superstructure Depth for Concrete Superstructures with Continuous Spans

Reinforced Concrete Superstructures:	**Minimum Depth:**
Balanced 3-span cast-in-place slabs with main reinforcement parallel to traffic	d = 0.542 + L/48
T-Beams	d = L/19
Box Girders, constant depth	d = L/21
Post Tensioned Box Girders:	**Minimum Depth:**
Continuous, uniform depth	d = L/29
Precast Prestressed Concrete Superstructures:	**Minimum Depth:**
Slabs and Boxes	d = L/40
Deck Bulb-T Girders	d = L/33
Bulb-I and Bulb-T girders	d = L/25
d = depth of constant depth members or depth at midspan of haunched member	
L = span length between centerlines of bearings for a simple span bridge or longest span between centerlines of bents for a continuous span bridge	

Increase depths given in *Table 1.2.7.2-1* for simple span bridges by 10 percent.

Depths shown are for constant-depth sections. Depth at midspan of haunched members may be reduced 15 percent for beams with continuous parabolic haunches or with straight haunches equal to ¼ the span where the total depth at the haunch is 1.5d.

Where minimum depth requirements, given above, are satisfied, the optional live load deflection criteria in *LRFD 2.5.2.6.2* will not be required. When minimum depth requirements are not satisfied, verify that the live load deflection does not exceed the limits recommended in *LRFD 2.5.2.6.2*.

When both minimum depth and live load deflection requirements are not satisfied, submit a request for a design deviation (see *BDM 1.2.2*). As justification for the request, document girder and deck service stress levels, live load deflection, and provide evidence of similar structures already in service with satisfactory performance.

GIRDER SPACING

Girder spacing is normally dependent on girder capacity. As span length increases, girder spacing should decrease. Limit deck overhangs to no more than one-half the girder spacing. Long deck overhangs tend to sag over time (even decks post-tensioned transversely).

1.2.7.3 Bridge Length

Determine the bridge length by referring to the following, as applicable:

- *BDM 1.11.2.1*, *Determining Bridge Length.*
- Bridge Standard Drawing *BR115*, *Standard Slope Paving.*
- *BDM 2.14.7.2*, *Hydraulic Requirements for Stream Crossings.*
- *BDM 2.14.11.3*, *Railroad Clearances.*
- *BDM 2.14.8*, (Wildlife passage requirements that may add structure length).
- The remainder of *BDM 1.2.7.3*.

WIDTH AND CROSS SECTION OF LOWER ROADWAY

For horizontal clearances, see *BDM 2.14.4.2*. Choose the back-slopes as follows:

- Use 2:1 end fill slopes for all bridges unless the geotechnical designer recommends otherwise.
- 1.5:1 end fill slopes are common for county roads and less-traveled highways. Review the *ODOT Highway Design Manual Figure 300-22, Standard Sections for Highways Other Than Freeways*, but do not use a slope steeper than 2:1 unless a steeper slope is recommended in the *Geotechnical Report*.

STOCK PATHS AT STREAM CROSSINGS

Provisions for stock to cross the roadway should be located away from the bridge crossing to reduce concentration of pollutants in the stream. However, if a stock path running under the bridge parallel to the stream is required, additional bridge length will be needed to accommodate:

- Sufficient horizontal space and vertical clearance to construct a benched section for a path above ordinary high water.
- A fence to keep stock out of the stream.

Stock passes are also discussed in the *ODOT Highway Design Manual*.

1.2.7.4 Substructure Guidance

Read the *Geotechnical Report* for information and recommendations about type of foundation required, or talk to the geotechnical designer when the *Geotechnical Report* is not yet available. For stream crossings, recommendations for scour and riprap protection are contained in the *Hydraulics Report*.

1.2.8 Bridge Security Design Considerations

1.2.8.1 Bridge Security Design Considerations

Consider project-specific countermeasures during the Scoping Phase for those structures which ODOT management determines need specific attention.

Potential bridge security threats include: **carried and placed** bombs, vehicle bombs, intentional vehicle or ship collisions, intentional fires, and other intentional and unintentional threatening activities. This section tells when and how to consider potential bridge security threats during the design of:

- New bridges.
- Bridge widenings.
- Bridge rehabilitation projects.

1.2.8.1.1 Countermeasures

Four countermeasures can help protect structures against potential security threats:

- Deter: Prevent an aggressor from attacking the structure by making the security presence known such as police or other authorized personnel.
- Deny: Prevent an aggressor from entering an unauthorized zone by a physical barrier such as security fencing, secure hatches or locked doors.
- Detect: Observe unauthorized personnel in a restricted area by means such as cameras or sensors.
- Defend: Provide **hardening** measures to protect a component from attack.

1.2.8.1.2 Process

Assess the probable structure specific security risks:

- Remote.
- Possible.
- High.
- Critical.

Remote: Only applies to structures on remote, low volume AADT facilities. Implementation of security countermeasures normally not warranted.

Possible: Applies to structures on the non-freeway State Highway System. Consider implementing security countermeasures associated with Deterring and Denying access to the structure. Ideas to consider include:

- Locate box girder soffit access openings away from abutments, requiring a ladder or other mechanical means to gain access.
- Provide shielded locking mechanisms on all access openings.
- Place secure screens at soffit vents near abutments.

- Prevent access to maintenance walkways and girder flanges at abutments.
- Post warning signs on the bridge approaches and below the structure.
- Deny access to critical structural components.
- Prevent vandalism, graffiti artists, or camping.

High: Applies to structures on the Interstate Highway System. Consider implementing security countermeasures associated with Deterring, Denying, Detecting, and Defending the structure. In addition to the items listed under **Possible**, include the following:

- Establish guidelines for standoff distance
- Eliminate access to small confined spaces

Critical Structures: These are structures that have been determined to be the most vulnerable structures in the State of Oregon.

Some bridges, due to their complex and unique nature, will require project-specific countermeasures along with those countermeasures that apply to all structures. These are bridges considered **critical** to the transportation network. The most critical bridges will also require site-specific operational security plans. The ODOT Emergency Preparedness Committee identified critical bridges and their potential vulnerabilities. To find out more, contact the Statewide Emergency Operations Manager in the ODOT Office of Maintenance and Operations.

Consider the need for security countermeasures during the Scoping Phase to ensure that added costs are included in the project budget. Define countermeasures and security plans and include in the TS&L Report. The bridge designer is to consult with the Bridge Operations Engineer for security guidance and to maintain consistency statewide.

If the Bridge Operations Engineer decides a critical bridge needs specific mitigation measures, consider these strategies first:

- Locate piers and towers so vehicular access is prevented.
- Design redundancy with critical elements.
- Place barriers to provide standoff distance when critical structural elements cannot be located away from vehicular traffic. If this cannot be achieved, the critical structural member or mechanical system should be analyzed and hardened against the design threat.
- Install locks, caging, and fencing to deny access to key points of vulnerable structural and mechanical systems.
- Install motion detectors or security cameras, and plan for communications to security response entities, to minimize **time-on-target**.

When cost-effective, consider selective protection of the structural integrity of key members against collapse. Ways to do this include strengthening key substructure members, adding redundancy, and use of blast hardening.

Again, consider project-specific countermeasures during the Scoping Phase for those structures which ODOT management determines need specific attention.

1.2.8.2 Placing Buildings Beneath ODOT Bridges

The placement of buildings beneath ODOT bridges is strongly discouraged. However, if local public agencies request and are given approval to place buildings below ODOT bridges, satisfy the following requirements:

- Maintain the structural integrity of the bridge:
    - Shore excavations that extend below the bottom of bridge footings adjacent to the proposed building according to Standard Specifications *SP 00510.44*.
    - Replace any soil removed within the vicinity of a bridge footing and compact according to Standard Specifications *SP 00510.46(a)*.
- Bridge maintenance provisions:
    - Provide 10 feet of vertical clearance between roof and superstructure for operation of snooper cranes, or for hanging scaffolds; or
    - Design the building's roof system to act as a work platform for maintenance or construction activities. Provide 3 feet minimum vertical clearance between roof and superstructure. Design the roof sheathing and purlins for a working load of 250 pound point load or 100 psf, whichever controls. Extend the design area 10 feet beyond the shadow of the structure. Design members below the purlin level for a working load of 50 psf over an area of 10' x 20'.
- Future seismic retrofit provisions:
    - Place the building to allow for increasing the size of the existing footing or footings by 50 percent plus an allowance of 5 feet for work area.
    - Make the building owners aware that future footing excavations or pile driving could cause vibrations in the building with a potential for damage to the building or contents. And that the State will not be responsible for any damage to the building or contents caused by such construction.
- Future bridge replacement or widening provisions:
    - Evaluate the need for a new bridge or future widening of the bridge. If the potential exists, allow for increasing the bridge width and construction of new footings. Allow 5 feet around the future footings for work area.
    - Make the building owners aware that future footing excavations or pile driving could cause vibrations in their building with a potential for damage to the building or contents. And that the State will not be responsible for any damage to the building or contents caused by such construction.
- Falling object protection:
    - Place protective fencing on the bridge above the building to cover the limits of any ground activity below the bridge.
    - Make the building owners aware that the State will not be responsible for any damage to the building or content caused by falling objects.

- Bridge fire protection:
    - The building shall be constructed of non-flammable materials and be equipped with an automatic sprinkler system.
    - The building shall not be used to store large quantities of flammable materials.
- Right of Access:
    - ODOT and or contractor employees shall be given access to the property and building as needed to perform any construction or maintenance activities.

Submit proposals to the District Manager and the Bridge Operations Engineer for review and approval. Include a drawing or drawings showing the existing bridge with all pertinent members dimensioned, and showing the proposed building with all pertinent dimensions, clearances, materials and roof design loads. The drawing or drawings shall be prepared, signed, and stamped with a seal of an engineer registered to practice in the State of Oregon.

1.2.9 Bridge Name Plates & Markers

1.2.9.1 Existing Bridge Name Plates

Specify that existing bridge name plates be salvaged and delivered to the office of the ODOT Construction Project Manager.

1.2.9.2 Bridge and Structure ID Markers

Specify bridge identification (ID) markers according to *BR195* on all bridges, and culverts defined as a bridge per *BDM 1.21.1*.

Specify bridge ID markers on all new bridges, widening projects and when existing markers will be impacted by the project. Replace existing bridge ID markers in poor condition or when the phone number on the bridge ID marker is out of date.

Include bridge identification markers in the bid schedule of items for contractor installation.

Show bridge ID marker placement locations in the contract plans, typically on the Deck Plan using *Detail Reference Number 80.* See *SP 00842* for mounting instructions.

Place bridge ID markers in the bridge rail transition at both ends of the bridge regardless of the direction of traffic. Place ID markers facing on-coming traffic, per *SP 00842*. When the structure is located over another public roadway, place additional bridge identification markers on the face of the bridge bent, immediately adjacent to and on both sides of the under-crossing roadway,

C1.2.9.2

Bridge ID markers are optional for local agency projects.

Coordinate with the Hydraulics Engineer to determine if a culvert field mark is appropriate in lieu of a bridge ID marker.

All bridges should have current bridge ID markers as part of a project.

When the Region has an arrangement with District Maintenance, installation by Agency staff is acceptable.

Bridge ID markers are not part of the project signing and should not be shown in the sign plans.

facing on-coming traffic. When the structure is located over another public roadway and the bridge bents are not accessible or visible, place additional bridge ID markers on a delineator immediately adjacent to and on both sides of the under-crossing roadway, facing on-coming traffic.

Configure each bridge ID marker in accordance with *Table 1.2.9.2-1*. Show this information in a table, or on a sign image, in the bridge plans.

BR195 shows dimensions, text, colors and other requirements of the marker for inclusion in the contract plans.

Table 1.2.9.2-1 Bridge ID Marker Legend

Bridge ID Marker Element	**Explanation**
Telephone number of the appropriate agency Dispatch Center:	For state owned bridges, find telephone numbers for the appropriate dispatch at https://www.oregon.gov/ODOT/Pages/Report-Hazard.aspx. Dispatch center boundaries may not correspond to Region boundaries. Consultants contact the ABR for correct contact number.
US or OR Route Number:	e.g. I-5, US97, OR42
State Highway Number:	e.g. 001, 004, 0025
Milepoint Number:	
Bridge Number:	
Name of the Structure:	Provide when a structure has a commemorative, historic, local, or interchange name as noted in parentheses of the title. Otherwise leave blank. Limit to three rows.

1.2.10 Safety and Accessibility

1.2.10.1 Accessibility Standards

Use the *2010 ADA Standards for Accessible Design* for the design of all Federal-Aid projects.

Design pedestrian overpass and underpass ramps to not exceed a 7.5 percent slope so a finished grade of 1:12 maximum if achieved and locate level resting areas every 30 feet. Design other features, such as handrails and stairs, to comply with the *Oregon Structural Specialty Code.* Review the standard drawings for handrailing and stair requirements in the roadway series.

For pedestrian structures, use FHWA publication *Guidelines for Making Pedestrian Crossing Structures Accessible (FHWA-I-84-6)*.

C1.2.10.1

Refer to *BDM 1.15.5* for inspection and maintenance accessibility requirements.

1.2.10.2 ADA Compliance for Bridge Work

The *Americans with Disabilities Act of 1990 (ADA)* prohibits discrimination and ensures equal opportunity for persons with disabilities in employment, state and local government services, public accommodations, commercial facilities, and transportation. Implementation guidelines, standards, and court decisions provide guidance on how to comply with the law.

Although these sources are applied to transportation construction and maintenance decisions, there is limited guidance about specific work activities on bridges. This document provides guidance on the ADA application to the unique situations created by bridge rehabilitation and maintenance projects.

1.2.10.2.1 General Guidance

The ADA regulation prohibits discrimination against people with disabilities by government entities. Government entities must make services accessible and usable to all people, including people with disabilities. For the purpose of this document, the sidewalk of a bridge is a facility that, when provided, must be accessible and usable by people with disabilities, regardless of whether the bridge is in an urban or rural setting.

The Oregon Department of Transportation (ODOT) has an obligation to ensure that a bridge infrastructure is accessible and usable, including bridge sidewalk facilities. ODOT addresses that obligation in several ways. Through the ODOT ADA Title II Transition Plan, all pedestrian facilities are brought into compliance over time. Specific work will also trigger a requirement to

bring certain facilities into compliance at the same time as the work is performed. Examples of specific work that triggers accessibility improvements include when a new bridge is constructed, when a new sidewalk is added to a bridge, or when the level of work on a bridge is considered to be an **alteration**. Activities that result in below-standard sidewalks require a roadway design exception. Additionally, a project's scope of work must not be modified solely to avoid triggering accessibility upgrades.

ALTERING ACCESS OR USABILITY OF A PEDESTRIAN ACCESS ROUTE

Refer to the *HDM 800* for guidance and standards on walkways. When alterations affect access to a sidewalk on a bridge, it is necessary to ensure that the sidewalk can be accessed from the approaching shoulder or sidewalk utilizing a curb ramp design. When alterations affect the usability of the sidewalk itself, it is necessary to ensure that the sidewalk is readily accessible to people with disabilities to the maximum extent feasible unless it is technically infeasible to do so. Explore alternatives to improve the level of accessibility of an existing sidewalk when it is below ODOT standards. Address accessibility barriers which include narrow passages, obstructions or routes lacking sufficient passing space. Alterations must follow current ODOT policy.

Where sidewalks are not present, pedestrian travel may be expected on or along the roadway except where prohibited. Pedestrians travel in the shoulder or in the roadway when there is no walkway provided. When a sidewalk is not present, treat the existing shoulder as a pedestrian access route when determining the impact of alterations.

1.2.10.2.2 Work Activity Triggers

The US DOJ/FHWA memorandum defines an **alteration** as: A change that affects or could affect the usability of all or part of a building or facility. Alterations of streets, roads, or highways include activities such as reconstruction, rehabilitation, resurfacing, widening, and projects of similar scale and effect. Activities defined as maintenance by the US DOJ/FHWA on streets, roads, or highways, such as filling potholes, are not alterations. Upgrade each facility or part of a facility that is altered to be readily accessible and usable by individuals with disabilities to the maximum extent feasible.

Examples of alterations of bridges include rail retrofit, widening, bridge deck concrete overlays, and asphalt concrete overlays when part of a full width paving project that extends beyond the bridge. Further, when changes are made that affect the accessibility or usability of the pedestrian access route, accessibility requirements need to be addressed. An existing bridge is considered to have a sidewalk, or pedestrian access route, when the horizontal surface is at least 32 inches wide, and that horizontal surface is intended for pedestrian use. See *HDM* for additional information. Point constraints less than 32 inches do not negate the primary use for pedestrian activity and walkway determination. Many existing walkways have had alterations to the pedestrian railing to provide safety improvements for vehicular departures, and therefore reduced the intended sidewalk pedestrian access route at bridge rail transitions.

The work types in the list below are not considered **alterations** and are considered maintenance activities. The list is not exhaustive. Evaluate additional activities on the basis of their impact to the usability of the pedestrian facility; activities that affect the usability are considered **alterations**. Consult the Traffic Roadway Senior ADA Standard Engineer for assistance in determining alteration activities.

Individual activities that do not result in changes to usability of the pedestrian facility:

- Structural deck overlays, thin bonded polymer system overlays, ACP overlays and deck seals that do or do not overlap the pedestrian facility (sidewalk, shoulder, and crosswalk).
- Deck repair patching, repair of deck soffit, full depth deck repair when these activities do not span the length of the bridge.
- Under deck superstructure repair.
- Repair of sidewalk soffit, repair of longitudinal sidewalk beams, repair of cantilevered sidewalk supports.
- Bridge Painting, cathodic protection.
- Repair in-kind of existing bridge rails.
- Concrete patching of superstructure elements, prestressed components, concrete surface finishing.
- Bonded structural strengthening (FRP, titanium, other metals, surface or near-surface), resin injection of concrete cracks.
- Seismic retrofits.
- Bearing device cleaning and lubrication.
- Substructure or Foundation repair.

While the following are maintenance activities, the pedestrian facility is being altered with a limited scope or incremental improvement. The repair work needs to meet the ADA requirements to the maximum extent feasible. This includes but is not limited to:

- Spot sidewalk repair.
- Expansion joint repair or replacement.
- Deck drainage improvements.

Review BDM Appendix B: ADA Bridge Work Examples for surface alterations that trigger curb ramp reconstruction.

(1) EXISTING BRIDGES & SAFETY CURBS

Existing bridges may support pedestrian facilities, either by providing a sidewalk or by providing adequate shoulder width. Do not reduce existing pedestrian facilities below applicable standards.

Existing bridges that have **safety curbs** do not have sidewalks. **Safety curbs** are features with similar appearance as sidewalks, but are provided as part of a bridge rail system and are not

intended to provide access for pedestrians. Some pedestrian may still be able to traverse them. The clear width of the horizontal surface of a **safety curb** is less than 32 inches.

Figure 1.2.10.2.2-1 Existing Bridge Rail, Section with Safety Curb

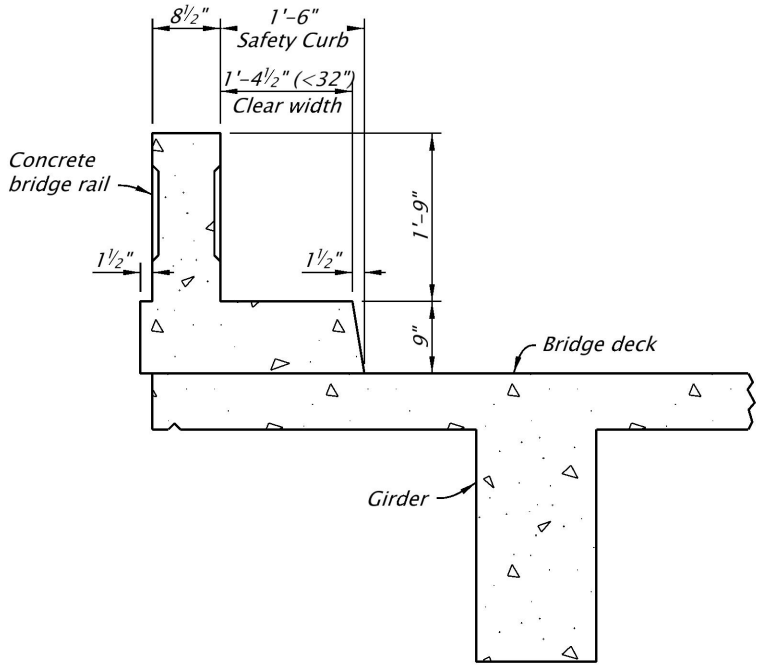


Figure 1.2.10.2.2-2 Existing Bridge, Section with Sidewalk

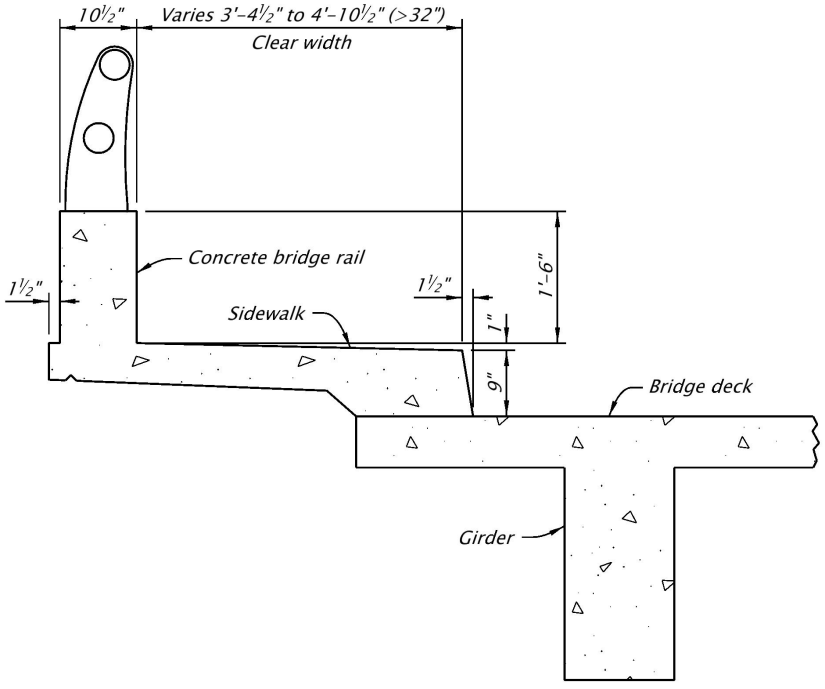
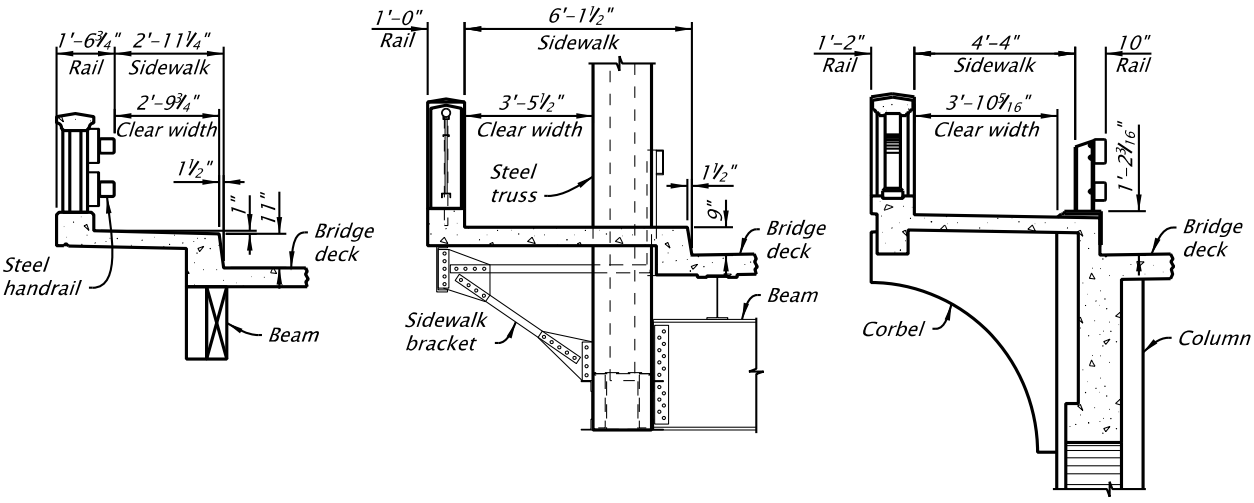


Figure 1.2.10.2.2-3 Existing Bridge, Clear Width Examples



When work on an existing bridge with sidewalk is an alteration as described in *BDM 1.2.10.2.2*, upgrade accessibility to meet full standards or to the maximum extent feasible. Refer to *HDM 810.7* for sidewalk requirements.

Feasibility of upgrades depends on numerous factors including:

- Structural capacity of the bridge to accept additional dead load and pedestrian load. Perform load rating to determine when the main structural system has additional capacity to support the upgrade.
- Additional strengthening required to support the proposed upgrade. Determine when the project may include girder strengthening.
- Structural system.
    - Thickness and reinforcement of existing concrete cantilevered sidewalks (when the existing sidewalk is thin and lightly reinforced, strengthening of the existing sidewalk might not be feasible, for example).
    - Spacing and configuration of sidewalk brackets and edge beams supporting a sidewalk.
    - Bridge deck thickness.
    - Girder spacing (may limit reasonable widening).
    - Presence of truss or arch members above deck.
- Interaction of the proposed upgrade with future program work such as rail retrofit or bridge deck widening (when a major rail or widening project is planned or needed, it may be most reasonable to improve accessibility at the same time).
- Historic preservation needs (some accessibility modifications may impact the

significance of historic features of the bridge, while others may be acceptable due to limited scope of impacts or reversibility). See *BDM 1.2.10.2.3* for historic bridges.

- Roadway geometrics or roadway cross section. Roadway geometrics and cross section elements that can be reconfigured to increase accessibility of the pedestrian access route or sidewalk on one or both sides of the bridge need to be evaluated.

Document the feasibility of upgrades or reasonable upgrades/options considered. Document the rationale (i.e., the justification) for why the upgrade(s) or option(s) was (were) not feasible.

(2) RAIL RETROFIT OR REPLACEMENT

Bridge rail replacement requires upgrade of existing pedestrian facilities unless a roadway design exception is obtained. For installation of bridge rail retrofit or bridge rail transitions at bridge ends, do not decrease the width of an existing sidewalk. Submit a roadway design exception when the pedestrian accessible route does not meet requirements or is less than ODOT standard widths. Review the *Highway Design Manual* for applicable standards and coordination with Traffic Roadway is required.

(3) INTERSECTIONS ON BRIDGE STRUCTURES

In some cases, intersections are located on bridge structures or at the bridge ends. The ADA requires that walkways are accessible when provided. This typically occurs by providing curb ramps at intersections. When projects provide sidewalk or alter streets, roadways, or highways and span from one intersection to another curb ramps are required to meet ODOT standards.

When a bridge alteration project includes an intersection, it triggers the obligation to provide or upgrade curb ramps at the intersection. Coordination with the roadway designer is critical in preparing curb ramp details for construction and is to be reviewed by both disciplines for constructability. Without cross discipline review and coordination, the curb ramp is at high risk for construction failure and acceptance. *BDM* Appendix A: ADA Design of Bridge Curb Ramps or *BDM* Appendix B: ADA Bridge Works Examples are posted on the Bridge Standards website for general guidance. Coordination with Traffic Roadway is required and review the *Highway Design Manual* for applicable standards for curb ramp design.

(4) TEMPORARY TRAFFIC CONTROL AND WORK ZONES

For construction or maintenance work, a temporary accessible route must be provided that is equal to or better than existing conditions. For example, when an existing shoulder across a structure is four feet wide, provide a four-foot temporary pedestrian access route for pedestrian traffic. See Temporary Pedestrian Accessible Routes (TPAR) for more guidance.

1.2.10.2.3 Design Considerations

(1) HISTORIC BRIDGES

For bridges that are considered a historic resource, where accessibility modifications may impact the significance of historic features of the bridge, consult with the Region Environmental Coordinator and the Historic Resource Coordinator to ensure the State Historic Preservation Office (SHPO) comments are addressed. Where upgrading accessibility would conflict with federal law regarding historic preservation, neither law supersedes the other.

When there is a conflict between federal historic preservation requirements and accessibility requirements, a Design Exception is required to validate and justify any deviation from meeting the accessibility standards. With the supporting document in the Design Exception, include a letter from SHPO declaring that an adverse effect would be caused by meeting the full requirements of the ADA. Ensure the letter describes whether alternative designs or incremental ADA improvements would create adverse effects. Consider and document mitigations that will be used to minimize the impact to pedestrians with disabilities. Mitigating options might be posted minimum widths or possible accessible detours.

(2) TECHNICAL INFEASIBILITY AND DESIGN EXCEPTION

ADA compliance requirements are not the same as ODOT geometric design standards. The Roadway design exception process is used when geometric standards cannot be achieved. ODOT also uses the Roadway design exception process to document when it is technically infeasible to meet all the ADA criteria. Early coordination before DAP with the ODOT Roadway Engineering Unit regarding impacts to either the ODOT geometric standards or the ADA requirements is especially important. Justification for not meeting standards is a required section of the design exception request. While costs can be used to justify exceptions to geometric standards it is not a justification for ADA criteria. See Part 1000 of the ODOT HDM – Design Exceptions for the design exception process.

- 4R Standard: 4R or modernization projects are considered the highest level of design and as such ODOT geometric standards for horizontal and vertical alignments, super elevation, lane width, shoulder width, sidewalk width are expected to be used. Document the justification for any non-standard feature not meeting the geometric design standards and the options considered in a design exception request. ODOT geometric standards for sidewalk width are wider than the minimum ADA compliance width but the ODOT geometric standard of 7 foot wide sidewalks are required unless there is an approved design exception.
- 3R Standard: 3R or Rehabilitation projects are more common than 4R projects. These projects are able to retain many of the existing geometric features even when they do not meet full ODOT geometric design standards. On a 3R project a sidewalk that does not meet the full 7 foot width but does meet the accessibility criteria of a 5 foot wide sidewalk does not require a design exception. When the sidewalk width is below 5 foot, a design exception is required and 5 foot by 5 foot passing spaces must be provided

every 200 feet along the sidewalk. Other geometric features that are non-standard need to be evaluated by the Roadway designer for design exception requirements.

- 1R Road Paving: 1R or Resurfacing projects are the most common project type. These projects have a narrow scope defined to just surface treatments. When the resurfacing is classified as an alteration and road pavement surfacing extends from one intersection to another, provide or upgrade curb ramps at all intersections and crossings. Utilize the ODOT curb ramp process per *HDM Appendix G* for upgrading curb ramps.
- Single Function Standard: Single Function projects are very limited in scope. The feature addressed in a single function project is to use 4R design standards for that specific feature. Design exceptions are not required for non-standard geometric elements not impacted by the single function work. When the sidewalk is not being modified, but the railing is being modified, which reduces the width of the sidewalk below the minimum 4 foot width, a design exception is required. When the reduction in usable sidewalk width retains the minimum 4 foot width, but not the ODOT standard width, a design exception is required for not meeting the ODOT standard.

1.3 Loads and Distributions

1.3.1 Ductility, Redundancy and Operational Importance

Apply the ductility per *LRFD 1.3.3* without change.

Modify *LRFD 1.3.4* redundancy factor as follows:

- Do not use a redundancy factor less than 1.0.

Modify *LRFD 1.3.5* operation importance factors as follows:

- Consider all bridges as **typical,** ($\eta^I = 1$) for the operational importance factor.

C1.3.1

1.3.2 Dead Loads

Provide the dead load allowances shown in *Table 1.3.2-1* for all vehicular bridges.

C1.3.2

See *AASHTO 3.5.1* for dead loads not listed.

The utilities load stated is a minimum utility design load for future use on the bridge superstructure to be assumed when the weights of proposed utilities are unknown. There will be situations where utilities are heavier. Use project specific utility loads when they are defined. See *BDM 1.12.6*.

Table 1.3.2-1 Design Dead Load Unit Weights

Load Type	Loading	Comments
Asphaltic Concrete Pavement (ACP)	0.150 kcf	
Bridge Rails		See *BDM 1.13.5, 1.13.6.2* and *ODOT LRFR Manual*
Concrete (unreinforced)	0.145 kcf	For cast-in-place and precast
Concrete (CIP, HPC, HPC-IC)	0.150 kcf	Reinforced. See *BDM 1.5.1*
Concrete (Precast Prestressed)	0.155 kcf 0.160 kcf	Reinforced, f'c less than or equal to 8.0 ksi Reinforced, f'c greater than 8.0 ksi
Concrete (UHPC)	0.155 kcf	Unreinforced
PPC Overlay	0.135 kcf	See *BDM 1.9.3.1.5*
Protective Screening		Per standard drawings
SIP Deck Forms	0.010 ksf	Precast Prestressed – See *BDM 1.5.6.2.1.* CIP - Use when deck forms are not required to be removed. See *BDM 1.5.7.7*
Future Utilities	0.090 klf	Based upon a 12 inch diameter water filled pipe. See *BDM 1.12.6.*

1.3.2.1 Wearing Surface

Provide the minimum present wearing surface (PWS) and future wearing surface (FWS) allowances shown in *Table 1.3.2.1-1* for all new vehicular bridges.

C1.3.2.1

Generally, FWS is not required for pedestrian structures. When the width of the pedestrian bridge allows trucks to pass regularly, FWS may be warranted. Consult the BDM Deck Specialist and *BDM 1.5.1*.

Table 1.3.2.1-1 Wearing Surface Dead Loads

Bridge Type	Present Wearing Surface (ksf)	Future Wearing Surface (ksf)
All new bridges with CIP concrete decks	0	0.025, (2 inches)
Side-by-side construction with PPC overlay	0.012, (1 inch) *	0.025, (2 inches)
Side-by-side construction with ACP (Local Agency projects only)	0.025 (2 inches)* **	0.040, (3 inches)
* Provide additional PWS allowance as needed to account for camber and superelevation buildup. Determine actual thickness from roadway profile. ** Use a minimum 0.050 (4 inches) thickness when a rolled membrane is used.		

1.3.2.2 Load Rating of Existing Bridges

See *ODOT LRFR Manual 1.7* to determine when an existing load rating needs to be updated. Review the latest load rating and when needed conduct a pre-design load rating per *BDM 2.10.8.1*. When the existing load rating is not in accordance with LRFR standards, consult with ODOT's load rating unit before using.

For all non-load-path-redundant steel truss bridges, verify that the stress levels in all structural elements, including gusset plates, remain within applicable requirements whenever planned modifications or operational changes may increase stresses.

C1.3.2.2

It is critical to know the rating factor of each member anytime there is a change in capacity, condition, or dead load.

When the load rating of the existing structure is available, check the latest Bridge Inspection Conditions' Rating Report against condition rating used for load rating. When the current superstructure or substructure condition factors are below those used in the load rating, the current load capacity of the bridge may not be accurately reflected in the load rating. Review the asphalt thickness used (if any) in the bridge load rating versus the values in the inspection report. Also review the rating

date and identify if any bridges updates have occurred since this date.

A load rating that reflects the current condition of each bridge is valuable and is used to identify when a bridge needs load posting or strengthening. Rating of a structure decreases with an increase in dead load and may result in a bridge load posting.

See *BDM 2.10.8* for additional information.

Contact the Senior Load Rating Engineer for assistance in a load rating.

1.3.3 Live Loads

1.3.3.1 New Vehicular Traffic Structure

Design by *AASHTO LRFD Bridge Design Specifications* using all of the following loads:

- Service and Strength I Limit States:
  - HL-93: Design truck (or trucks per *LRFD 3.6.1.3*) or the design tandems and the design lane load.
- Fatigue truck per *LRFD 3.6.1.4.*
- Strength II Limit State:
  - ODOT OR-STP-5BW permit truck.
  - ODOT OR-STP-4E permit truck.
  - EV3.

C1.3.3.1

ODOT permit loads are shown in ODOT LRFR Manual *1.5.1.5B*.

Figures 1.3.3.1-1 to *1.3.3.1-4* show live load effects for single span bridges with prismatic girders without live load distribution factors. A load factor of 1.75 (Strength I) was applied to HL-93 design truck loads and a load factor of 1.35 (Strength II) was applied to ODOT OR-STP-4D, ODOT OR-STP-4E, ODOT OR-STP-5BW, and EV3 truck loads. The plots may be used to determine which design truck could control during a preliminary design given a span length for the project.

Figure 1.3.3.1-1 Live Load Plus Impact for Single Span Prismatic Members – Bending Moment at Mid-Span, (span lengths 30 – 150 feet)

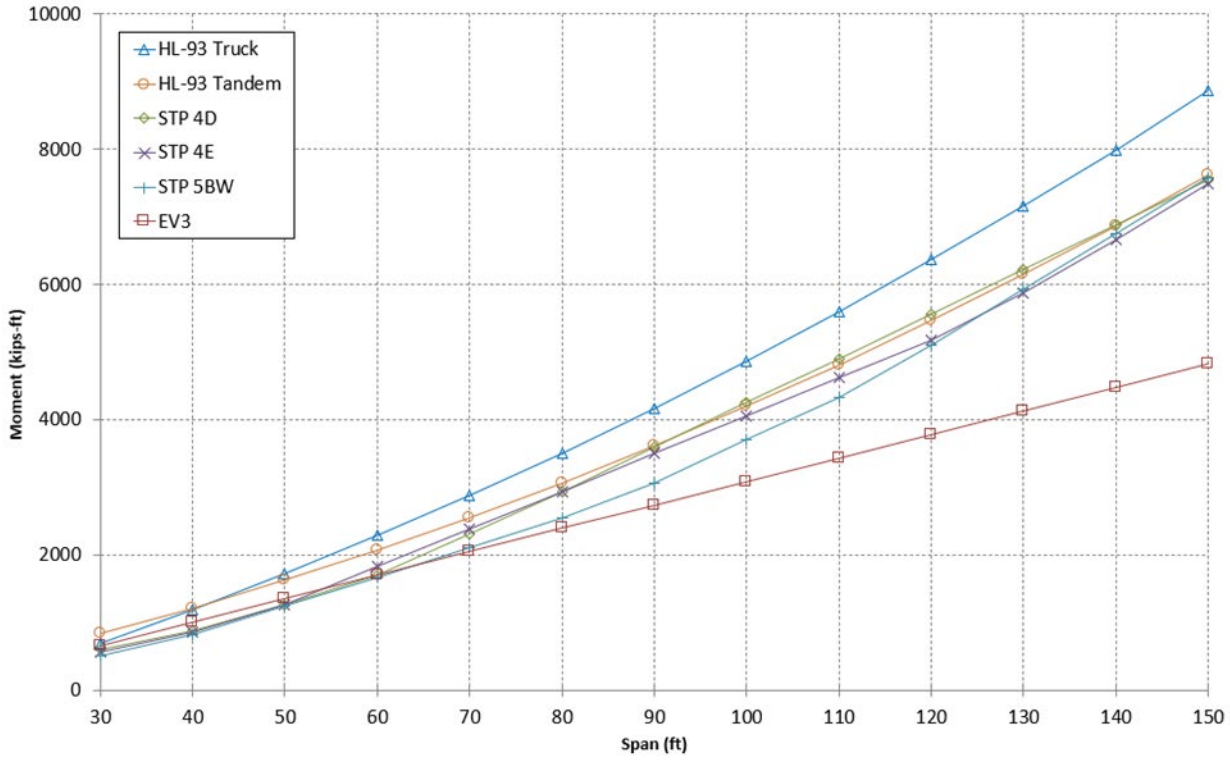


Figure 1.3.3.1-2 Live Load Plus Impact for Single Span Prismatic Members – Bending Moment at Mid-Span, (span length 150 – 300 feet)

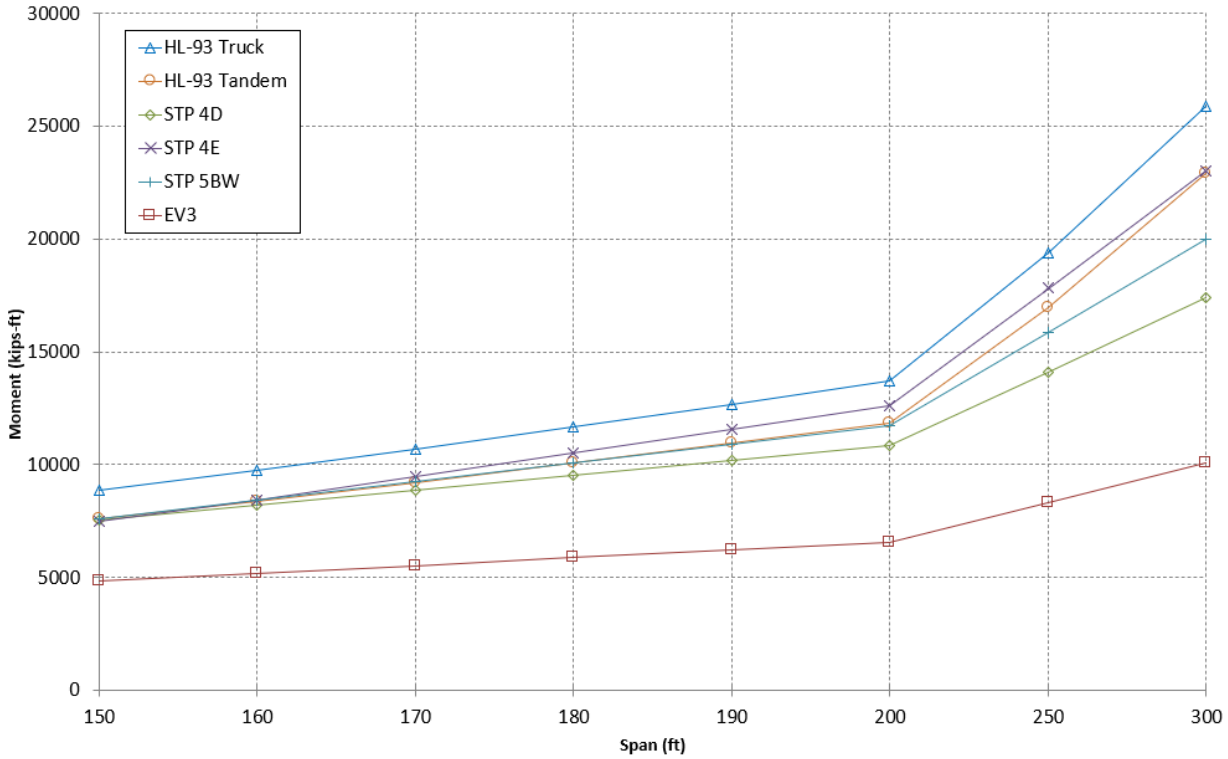


Figure 1.3.3.1-3 Live Load Plus Impact for Single Span Prismatic Members – Maximum Shear, (span length 20 – 150 feet)

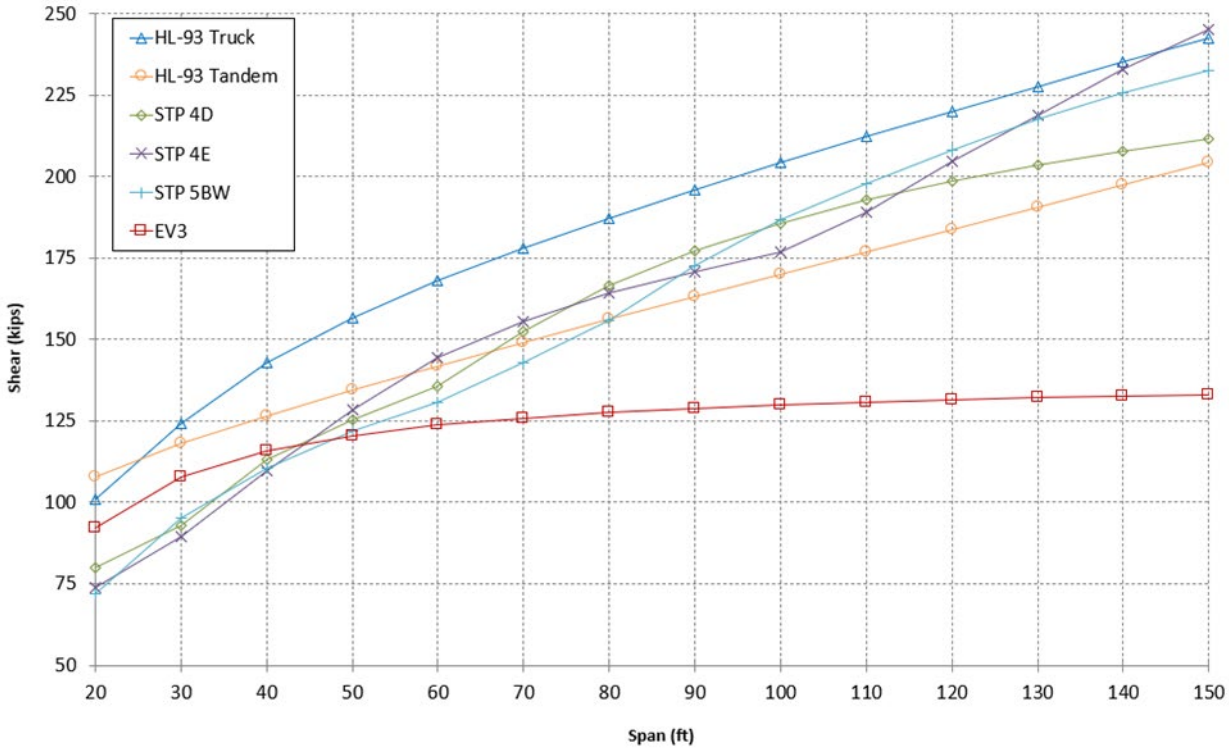
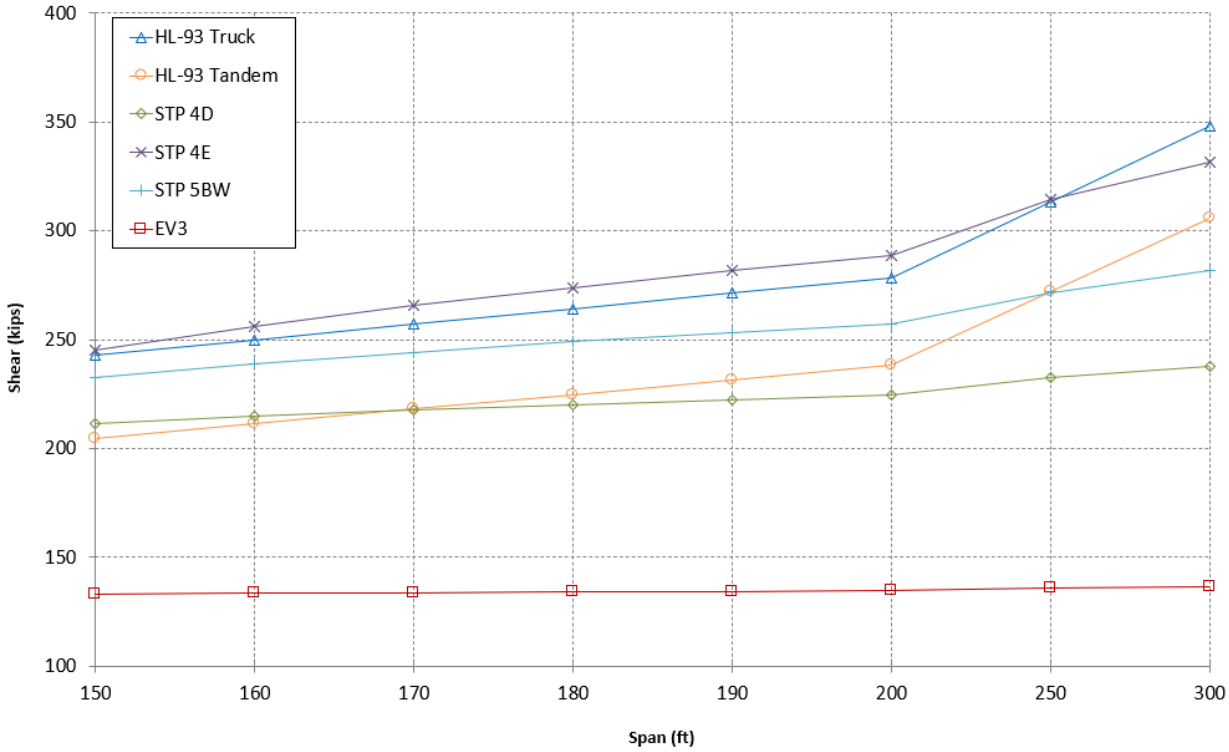


Figure 1.3.3.1-4 Live Load Plus Impact for Single Span Prismatic Members – Maximum Shear, (span lengths 150 – 300 feet)



1.3.3.2 Pedestrian Structures

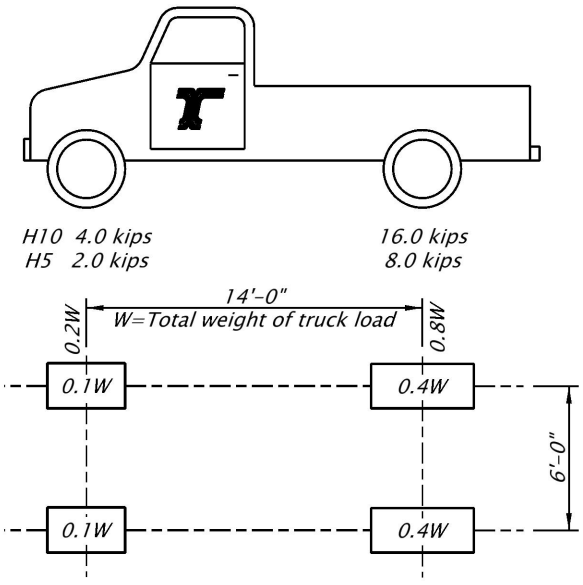
For bridges designed for pedestrian or bicycle traffic only, use a live load of 0.090 ksf. When an agency design vehicle is not specified, use *AASHTO* Standard H-5 or H-10 Truck loading as shown in *Figure 1.3.3.2-1* to check the longitudinal beams. A vehicle impact allowance is not required.

For a pedestrian or bikeway bridge clear deck width less than 7 feet do not consider the maintenance truck. See also the *AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges.*

C1.3.3.2

Figure 1.3.3.2-1 H-5 or H-10 Truck Loading

Clear deck width 7 to 10 feet	10,000 lb. (H5 Truck)
Clear deck width over 10 feet	20,000 lb. (H10 Truck)



1.3.3.3 Widening of Vehicular Traffic Structures

When widening an existing structure, design the widening using the loading in *BDM 1.3.3.1*.

Do not use live loading less than the design live load for the existing structure.

C1.3.3.3

1.3.3.4 Structure Strengthening

Design strengthening projects for the maximum load effect from the following permit trucks using the *AASHTO LRFD Bridge Design Specifications* Strength II Limit State:

- ODOT OR-STP-4D.
- ODOT OR-STP-5BW.
- ODOT OR-STP-4E.
- EV3.

For strengthening of prestressed concrete structures, ensure the requirements of Service I and III Limit States are satisfied using HL-93 loading.

Do not include FWS in strengthening design. When strengthening, determine existing wearing surface loads according to *ODOT LRFR Manual 2.2.8*.

C1.3.3.4

For design loads for structural repair projects, refer to the *Bridge Maintenance Design Manual*. See BDM 1.15.1 and 1.15.2 for additional criteria for repair and strengthening bridges.

See *BDM 1.3.3.1* and *ODOT LRFR Manual 1.5* for vehicle descriptions and *LRFD Table 3.4.1-1* for Load Factors.

The EV3 truck only controls the strengthening design on shorter span structures (less than 50 feet) and is not meant to impact every strengthening design.

1.3.4 Construction and Temporary Loading

Specifications dictate the contractor to design temporary works, except seismic loading, according to the *AASHTO Guide Design Specifications for Bridge Temporary Works, 2nd Edition, 2017*, unless specified otherwise therein.

Specifications dictate the contractor to construct temporary works according to *AASHTO Construction Handbook for Bridge Temporary Works, 2nd Edition, 2017*, unless included in the *Oregon Standard Specifications for Construction and Special Provisions* or specified therein.

Demonstrate at least one feasible means of construction, i.e. the incomplete structure is stable and not overstressed throughout a commonly used construction process relying on available equipment. Document the method in the calculation book but do not show results in contract documents, except as noted below:

- Jacking plans typically show estimated jacking loads. See *BDM 1.3.4.1*.
- Work platform and containment contract documents typically offer loading limits that will not overload the existing structure, and only if these limits are exceeded require stamped calculations showing that the proposed work platform & containment doesn't overstress the structure. See *BDM 1.3.4.3*.
- Geotechnical shoring plans may show estimated loading. See *GDM 15*.

C1.3.4

The intent of this section is to provide guidance to designers so they can effectively specify design parameters and review contractor designed temporary works submittals.

See *BDM 1.16.2* for Bridge Temporary Works.

See *SP 00560.04* for information on wind loading during construction. When reviewing shop drawings for erection plans, refer to *AASHTOs Guide Specifications for Wind Loads on Bridges During Construction.*

When design-bid-build plans are advertised, the Agency is warranting that the contract documents represent a buildable design.

Longstanding industry practice requires the Agency to avoid dictating the contractor's means and methods, except when specific means and methods are necessary. Stability of the structure during active construction is the contractor's responsibility.

It is not practical to require a contractor to provide a structural analysis of an existing structure.

Refer to *AASHTO 2.5.3* for additional clarification on constructability.

- Proposed temporary bracing for prestressed girders typically is detailed in contract plans.
- Contract documents may show limits for crane loading on existing structures.
- Instances where constructability dictates it is necessary.

1.3.4.1 Jacking Forces

Provide a jacking load diagram on the plans that identifies proposed jack locations and sizes. Provide a jacking load table on the plans that identifies the loads, load combinations, and other load requirements for the development of the jacking plan. An example jacking load table is shown in *Table 1.3.4.1-1* but may need to be modified for project specific designs.

C1.3.4.1

The loads specified in the jacking load table are intended to be used by the contractor's engineer to design the temporary work structures.

Determine forces generated by jacking and a jacking location during design. The existing structure elements must be able to resist the jacking forces. When the jacking location is limited by the existing structure situation, propose a shoring structure extending from the existing structure or supported by a designed foundation for jacking.

Consider uplift on structures with short end spans.

Table 1.3.4.1-1 Jacking Load Table

Jacking Load Table			
Location	**Girder Reactions (kips/girder)**		
	Permanent Load	**Live Load**	**Modified Strength II**
Bent 1			
Bent 2			

Modify *AASHTO 3.4.3.1* as follows:

Specify design forces for jacking when the structure is not in service as 1.5 times the permanent load reaction at the bearing, adjacent to the point of jacking.

Do not include FWS in jacking design. When jacking, determine existing wearing surface loads according to *ODOT LRFR Manual 2.2.8.*

A structure is considered in service when it is open to traffic during the jacking operation.

Permanent loads consist of all dead loads, self-weight, bridge rails, wearing surface, etc.

When structure is not in service, designers may assume construction live loading does not exceed traffic live loading as specified in *BDM 1.3.4.1*.

Specify design forces for jacking in service at the bearing, adjacent to the point of jacking, for lanes in service, using permit trucks from *BDM 1.3.3.4* as *AASHTO* Strength II Limit State modified as follows:

- 25 percent IM load allowance.

Determine live load forces consistent with the traffic control plan.

1.3.4.2 Bridge Raising

C1.3.4.2

Use the loading criteria and jacking load table specified in *BDM 1.3.4.1* for designing bridge raising operations, except as modified in this section.

Specify lateral design forces as 5 percent of the permanent load reaction and include in the jacking load table.

Permanent loads consist of: superstructure and substructure (above the chip-in point when using the chip-in method), superimposed dead loads (utilities, signs and other dead loads that will remain on the bridge during the raising operation). Field verify all dead loads at Project Initiation.

Wind loads during construction are considered low risk and are typically ignored since operations are low to the ground.

Close the bridge to traffic during active bridge raising operation.

Consider whether the bridge should be open to traffic during inactive construction periods. It is preferred to close the bridge to traffic for the duration of construction.

See *BDM 1.16.1* for additional information on bridge raising. Modify *SP 00255* as needed to incorporate bridge raising operations.

1.3.4.3 Falsework

When falsework falls within the live-load path, use the loading criteria and load table specified in *BDM 1.3.4.1* for designing falsework, except as modified in *SP 00540*.

Specify lateral design forces for falsework as 5 percent of the permanent load reaction, adjacent to the point of jacking.

C1.3.4.3

Refer to *SP 00540* for additional loading requirements at highway and railroad traffic openings.

Refer to the *Glossary* for a definition of Falsework. See *BDM 1.16.3.3* for additional information on Falsework.

Where wind loads during construction are expected to exceed this lateral load, consider increasing the lateral load to account for relevant wind loads.

1.3.4.4 Work Access Platforms

When work access platforms are necessary for more than five consecutive shifts:

- Evaluate a single configuration of work platform loading.
- Include reasonable limits for the work platform on the plans.
- Specify allowable point loads, and where they can be applied in *SP 00253.09*.
- Evaluate load rating to determine the remaining capacity of the structure that can be used for containment or work platforms.

When wind loading is considered, specify the maximum allowable containment size in *SP 00253*.

When evaluating a load rating, obtain the System Factor, Condition Factor, and Evaluation Live Load Factor from the most recent load rating when available. When

C1.3.4.4

The contractor does not need to evaluate the existing structure when they remain within the prescribed limits. Evaluate the work platform to ensure a single configuration is constructible. It is not feasible to consider every option available to a contractor.

Contact the Senior Load Rating Engineer with questions. Refer to *TM671* for wind speeds.

there is not an existing load rating, complete a load rating to ensure the existing structure can carry the work access platform or containment. When necessary, update an existing load rating per *BDM 1.3.2.2*.

For non-load rated elements, evaluate the structure for the ability to carry temporary work access platform using Strength II loading. Use a construction live load factor of 1.5 and construction live loading from *AASHTO Guide Design Specifications for Bridge Temporary Works Table 2.3.4.3.3-1*.

Assume work platforms and containment extend 5 feet beyond the sides of the structure and the full length of each span.

An example of very light duty is cable supported corrugated/steel decking.

An example of light duty is frame and tube supported decking system.

Medium and heavy duty systems are typically custom designs.

It may be necessary to separate the work platforms or containment into zones for structural reasons. Carefully size the zones to allow efficient work by the contractor.

1.3.5 Sidewalk Loading

C1.3.5

For sidewalks not separated from traffic by a structural rail, account for the potential for a truck to mount the sidewalk. Design the sidewalk for the greater of:

- 0.075 ksf pedestrian loads considered simultaneously with the vehicular load in the adjacent lane as stated in *3.6.1.6* of the *LRFD Bridge Design Specifications*. Per *LRFD 3.6.2.1*, impact does not apply to pedestrian loads.
- The *LRFD* design truck placed with a line of wheels 2 feet from the face of rail. Do not apply a lane load with the design truck, but do include impact. Consider this load only under the Strength I limit state. Do not consider trucks or vehicle loads in adjacent lanes. Do apply the multiple presence factor for this case.

In addition to the above cases, ensure the supporting member (exterior girder) is adequate for HL-93 loading when the sidewalk is removed and an approved barrier type (*BR290* or *BR208*) is placed at the edge of deck per *BDM 1.13.5*.

1.3.6 Vehicular Collision Forces: CT

Modify *LRFD 3.6.5* as follows:

Where the design choice is to redirect or absorb the collision load, use pier systems with three or more columns. Specify pier protection that consists of a minimum 42 inch high MASH TL-5 rigid barrier with standard pin anchorage to subgrade.
Based on the clear distance from the face of the pier component to the traffic face of the barrier, use minimum column sizes as follows:

- For 0 to 3.25 feet clear distance, use a minimum 48 inch circular (or equivalent square) column with 1 1/8 percent minimum longitudinal steel reinforcement and #4 spiral reinforcement with 4 inch pitch.

- For 3.25 feet or greater clear distance, use a minimum 36 inch circular (or equivalent square) column with 1 1/8 percent minimum longitudinal reinforcement and #4 spiral reinforcement with 4 inch pitch.

C1.3.6

The standard detail for barrier protection of a structure (column, wall, traffic support pole, or other structure) places the curb face of the barrier 4 feet minimum from the face of the structure. This detail dates back at least as far as the mid 1990's. The 4 feet dimension is intended to allow room for **rollover** when a truck impacts the barrier. It should be noted that barrier impacts can create a rollover scenario that exceeds 4 feet. Therefore, consider clear distances exceeding 4 feet when you can without impacting roadway width standards.

AASHTO 9th Edition updated this spacing to 3.25 feet to match current research.

When 4 feet or more clear distance can be provided, the proposed barrier placement detail will meet standards and no special consultation with the roadway designer is necessary. It should be understood that such a detail may pose some risk of structure impact, even when small.

Where the 4 feet clear distance cannot be achieved, consult with the roadway designer to confirm what clear distance can be provided. Any clear distance less than 4 feet results in additional risk for catastrophic impact of the structure being protected. Therefore, consider reducing roadway shoulder width. The desired result should be to find an acceptable balance between roadway risk (due to inadequate shoulder width) and structure risk (impact to the structure being protected).

When evaluating the roadway and structure risk, the following factors should be taken into account:

Alignment of the roadway – straight vs. curved, inside of curve vs. outside of curve.

Length of roadway width reduction (e.g., single sign support vs. long retaining wall).

Ability of the structure to absorb a hit (single column vs. wall abutment).

Traffic volume – higher traffic volume means higher risk of an incident.

Consequences of structure failure.

When clear distance must be minimized, provide 3 inch minimum clear distance from the back of pinned barrier to the face of the structure. Use of this detail includes accepting significant risk. The 3 inch minimum clear distance is intended to

minimize the amount of horizontal impact load that would be transferred from the impacted barrier to the structure being protected. The void between the back of the barrier is typically filled with pea gravel. Note that 3 inch concrete surfacing is often provided at the top of barrier for aesthetic purposes. This surfacing is expected to disintegrate upon impact and so would not be expected to transmit unacceptable forces to the structure being protected.

In cases where reduction of the clear distance width provides an unacceptable risk against rollover, the barrier can be transitioned from a safety shape to vertical. Vertical barrier will reduce vehicle rollover and in some cases can provide an installation with an acceptable level of risk.

Intrusions zones for TL3 and TL4 barrier per *Guidelines for Attachments to Bridge Rails and Median Barriers Midwest Roadside Safety Facility (MwRSF)*; University of Nebraska-Lincoln, Nebraska, 2003 are shown in *Figures 1.3.6-1* and *1.3.6-2*.

Earth mounds are no longer an acceptable method of column protection. At this time, existing earth mounds do not need to be removed.

Figure 1.3.6-1 TL-3 Zones of Intrusion

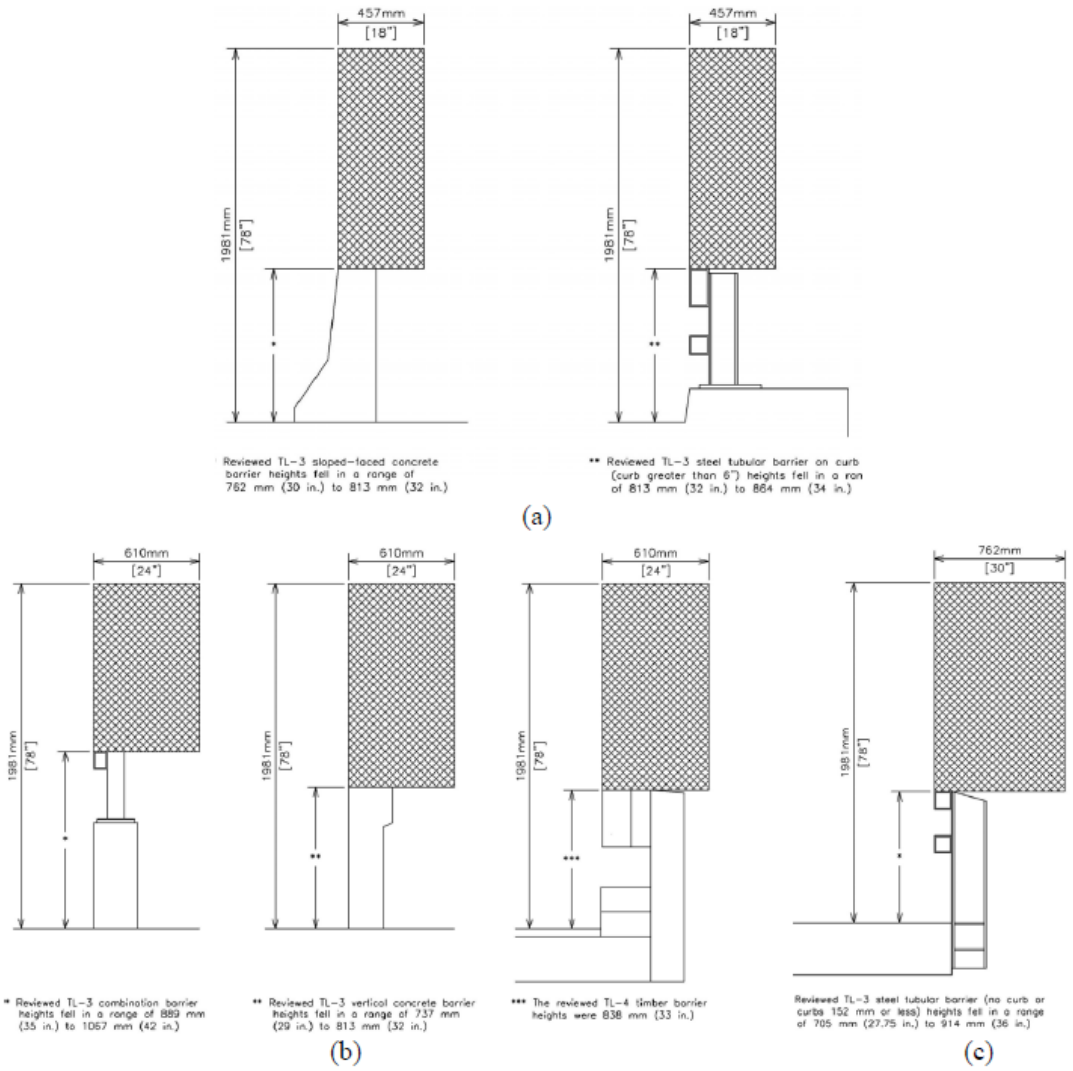
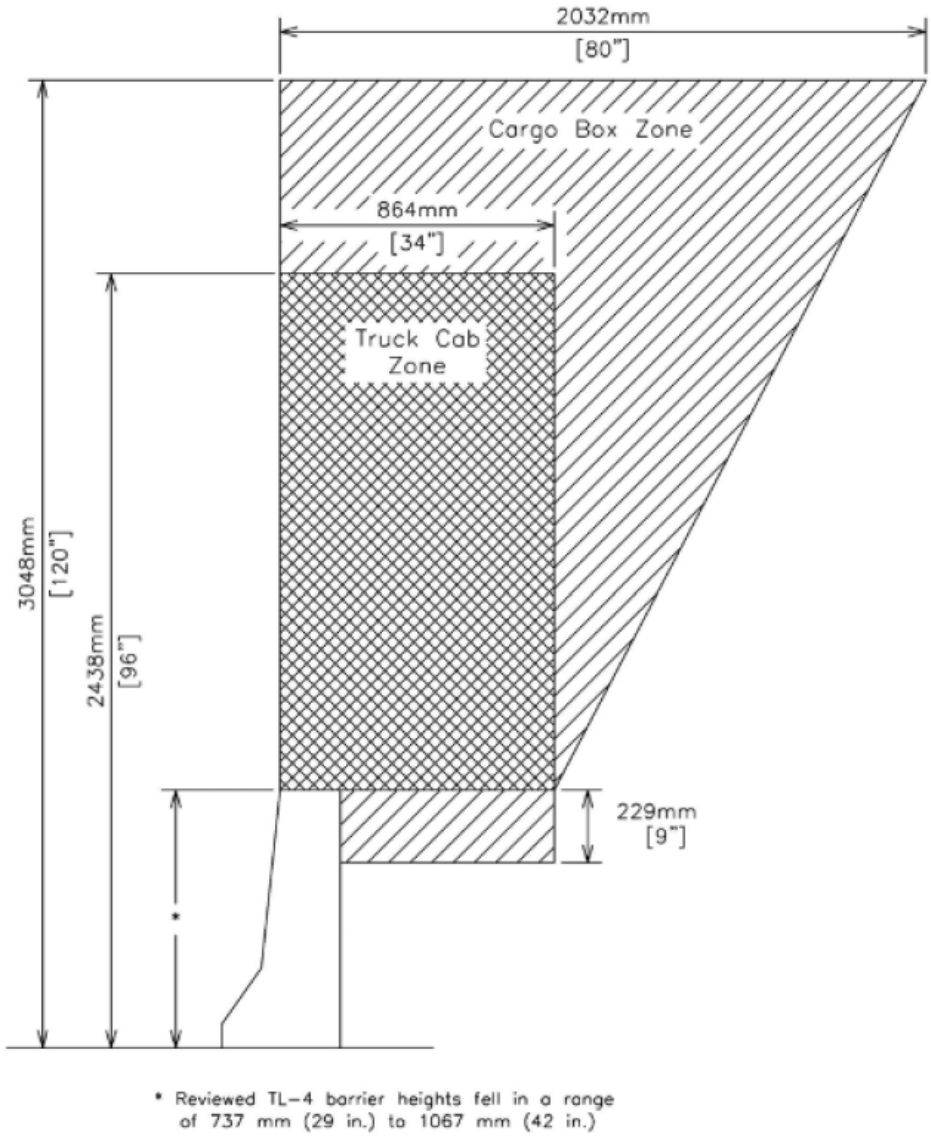


Figure 2.1. TL-3 Zone of Intrusions for (a) Sloped Face Concrete Barrier and Steel Tube Rail on Curbs > 6 inches; (b) Vertical Face Concrete Barrier and Combination Concrete and Steel Rail; and (c) Steel Tube Rail on Curbs > 6 inches (*3*).

Figure 1.3.6-2 TL-4 Zones of Intrusion



1.3.7 Change in Foundations Due to Limit State for Scour

C1.3.7

Follow *LRFD 2.6.4.4.2* and *LRFD 3.7.5.*

See *BDM 1.10.5.8, 2.14.7* and the *GDM* for additional information on hydraulics and scour design information.

1.3.8 Thermal Forces

C1.3.8

Use the temperature ranges shown in *Table 1.3.8-1.* Figure the rise and fall in temperature from an assumed temperature at time of erection.

Table 1.3.8-1 Temperature Ranges for Thermal Forces

Section *	Metal Structures	Concrete Structures	Annual Mean Temperature
Section I	$+10^0$ F. to $+110^0$ F.	$+22^0$ F. to $+72^0$ F.	52^0 F.
Section II	-10^0 F. to $+120^0$ F.	$+12^0$ F. to $+82^0$ F.	52^0 F.
Section III	-30^0 F. to $+120^0$ F.	0^0 F. to $+82^0$ F.	47^0 F.

* Section I designates that portion of the state west of the coast range.
Section II designates the valley region between the coast range and Cascade Mountains.
Section III designates the Cascade Mountains and all of eastern Oregon. For structures in the Columbia River Gorge, use Section III.

1.3.9 Wind Load

C1.3.9

Determine wind load according to *LRFD 3.8.1*. Determine the design 3 second gust wind speed used in the determination of design wind loads on bridges and walls from *Table 1.3.9-2*. Use *Figure 1.3.9-1* to identify areas of special wind regions in *Table 1.3.9-2.*

The wind speed map is adapted from the *2022 Oregon Structural Code* and *2022/2023 CPP Special Wind Region Study for Oregon* to account for locations in the state with special wind regions.

When verified wind speed using the ASCE 7 Hazard Tool. The risk category from building to mean recurrence interval (MRI) is as shown in *Table 1.3.9-1*.

Table 1.3.9-1 Risk Category from Building to Mean Recurrence Interval (MRI)

Building Risk Category	Description	MRI (year)	Wind Speed Correspond to
I	Low hazard to human life in case of failure	300	approximately a 15 percent probability of exceedance in 50 years (Annual Exceedance Probability = 0.00333)
II	Most Residential and Commercial Dwellings	700	approximately a 7 percent probability of exceedance in 50 years (Annual Exceedance Probability = 0.00143)
III	Substantial risk to human life in case of failure	1700	approximately a 3 percent probability of exceedance in 50 years (Annual Exceedance Probability = 0.000588)
IV	Essential Facilities	3000	approximately a 1.6 percent probability of exceedance in 50 years (Annual Exceedance Probability = 0.000333)

Typical bridges are designed for 3-second gust wind speed, at an elevation of 33.0 ft, for wind exposure Category C, with an MRI of 700 years (See *LRFD 3.8.1.1.2*).

In all areas with full exposure to ocean winds, use the highest wind velocity for that MRI category.

In areas in Multnomah and Hood River counties with full exposure to Columbia River Gorge winds, use the highest wind velocity for that MRI category.

Examine mountainous terrain, gorges, and ocean promontories for unusual wind conditions.

For wind acting horizontally on the supports, signs, luminaires, traffic signals, the selection of the MRI accounts for the consequences of failure. A "typical" support could cross the travelway during a failure thereby creating a hazard for travelers (MRI = 700 years). All supports that could cross lifeline travelways are assigned a high risk

category to consider the consequences of failure (MRI = 1700 years). Supports that cannot cross the travelway are assigned a low risk and 300-yr MRI. (See *LRFD Specification for Structural Supports for Highway Signs, Luminaires, and Traffic Signals Section 3.1* and *3.8*).

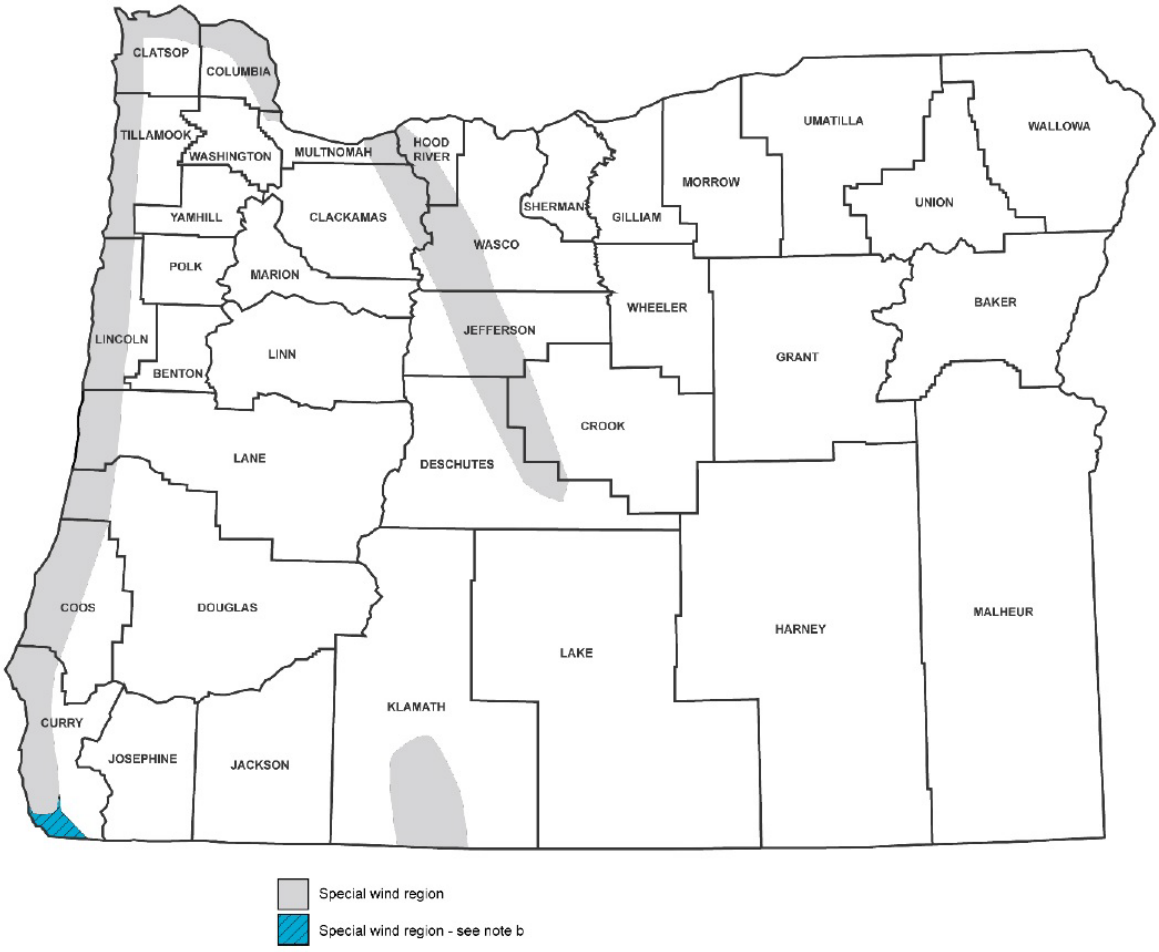
Table 1.3.9-2 Basic Design Wind Speed, V

COUNTY	300-Year MRI Basic Design Wind Speed, V (MPH)	700-Year MRI Basic Design Wind Speed, V (MPH)	1700-Year MRI Basic Design Wind Speed, V (MPH)	3000-Year MRI Basic Design Wind Speed, V (MPH)
Baker	97	103	110	114
Benton	90	96	102	107
Clackamas	92	98	105	109
Clackamas special wind region[a]	92	98	105	109
Clatsop	91	96	102	107
Clatsop special wind region[a]	115	120	130	135
Columbia	91	97	103	107
Columbia special wind region[a]	91	97	103	107
Coos	89	95	101	106
Coos special wind region[a]	115	120	130	135
Crook	93	100	106	111
Crook special wind region[a]	93	100	106	111
Curry	88	94	101	105
Curry special wind region[a]	115	120	130	135
Deschutes	93	99	106	110
Deschutes special wind region[a]	93	99	106	110
Douglas	91	97	103	108
Douglas special wind region[a]	115	120	130	135
Gilliam	94	100	107	111
Grant	95	101	108	113
Harney	94	101	108	112
Hood River	92	98	105	109
Hood River special wind region[a]	92	98	105	109

Jackson	90	96	103	107
Jefferson	93	99	106	110
Jefferson special wind region[a]	93	99	106	110
Josephine	89	95	102	106
Klamath	91	98	104	108
Klamath special wind region[a]	91	98	104	108
Lake	93	99	106	111
Lane	91	98	105	110
Lane special wind region[a]	115	120	130	135
Lincoln	90	96	102	106
Lincoln special wind region[a]	115	120	130	135
Linn	92	98	104	108
Malheur	96	102	109	113
Marion	92	98	104	108
Morrow	94	101	108	112
Multnomah	92	98	105	110
Multnomah special wind region[a]	92	98	105	110
Polk	90	97	103	107
Sherman	93	99	106	111
Tillamook	91	96	102	107
Tillamook special wind region[a]	115	120	130	135
Umatilla	95	102	109	113
Union	96	102	109	113
Wallow[a]	97	103	110	115
Wasco	93	99	106	110
Wasco special wind region[a]	93	99	106	110
Washington	91	97	103	107
Wheeler	94	100	107	111
Yamhill	91	97	103	107

a. Refer to *Figure 1.3.9-1* for mapped special wind regions.

Figure 1.3.9-1 Special Wind Regions - Oregon



a. Sites on the perimeter of the identified special wind regions shall be verified using ASCE 7 Hazard Tool.
b. This portion of the special wind region in Curry County extends 15 miles inland from the Pacific Coast and is not identified on the ASCE 7 Hazard Tool.

1.4 Structural Analysis

See BDM 1.3.1 for Ductility, Redundancy and Operational Importance factors.

1.4.1 Live Load Distribution Factors

Use the live load distribution factors and procedures provided in the *AASHTO LRFD Bridge Design Specifications* to determine load effects on bridge members for new, replacement, and strengthening bridge designs.

Higher level techniques such as finite element analysis or grillage analysis are not acceptable basis for adjustment of live load distribution factors in *LRFD 4.6.2.2.2* and *4.6.2.2.3* for design of new bridges.

Use the following information to support a design deviation for using refined method of analysis per *LRFD 4.6.3*:

- Name, version, and release date of design software used to perform refined method of analysis.
  Proposed table of live load distribution factors for controlling moment and shear at critical locations in each span. This aids in permit issuance and load rating of the bridge for all standard load rating trucks listed in *ODOT LRFR Manual Section 1.5* and report rating factors using *ODOT LRFR Section 11 Load Rating Summary Workbook* (excel).
- Apply an additional 1.10 factor to the distribution factor obtained from the refined method of analysis.

C1.4.1

The required information will aid future load rating and structural evaluation for these bridges.

The 1.10 factor is conservatively included to determine a distribution factor using the refined method of analysis.

- Include in the calculation book a comparison of the moment and shear for an equivalent single girder line with AASHTO distribution factors vs refined method of analysis (with lower distribution factors and lower demands) for both the live load and dead load.

1.5 Concrete

1.5.1 Concrete, General

1.5.2 Concrete Finish

1.5.3 Concrete Bonding Agents

1.5.4 Curing Concrete

1.5.5 Reinforcement

1.5.6 Precast Prestressed Concrete Elements

1.5.7 Cast-In-Place Superstructure

1.5.8 Post-Tensioned Structures

1.5.9 Camber Diagrams

1.5.10 Pour Schedules

1.5.11 Concrete Anchors

1.5.1 Concrete, Design Considerations

Designate the concrete class by the minimum compressive strength at 28 days followed by the maximum aggregate size (e.g., Class 4000 – ¾). Unless otherwise specified, Class 3300 – 1 ½, 1 or ¾ is called for by the *Standard Specifications*. The maximum ultimate strength on which allowable stresses are based is 5000 psi, except for prestressed concrete. Use high performance concrete (HPC) in all cast-in-place concrete decks and approach slabs, with the exception of pedestrian bridge decks. Pedestrian bridge decks do not require HPC unless they are at a location using significant amounts of deicing chemicals.

Table 1.5.1-1 Classes of Concrete (For Design and To Be Shown on Plans)

Class of Concrete	Concrete Location
HPC 4500 – 1-1/2	All poured decks (except box girder decks that require greater strength and Pedestrian Bridge decks) Note: This concrete strength works well with both grade 80 and grade 60 rebar and therefore would facilitate use of grade 80 rebar, but still allow contractors to consider grade 60 rebar without the need to change to a different concrete mix. The use of more coarse aggregate is to achieve more durable decks.
HPC 4500 – 1-1/2	Approach Slabs
4000 – 3/8	Drilled Shafts
XXXX – 3/4	Prestressed members (Does not include poured deck on prestressed members, see above)
XXXX – 1/2 or 3/8	Post-tensioned box girder bottom slab and stem walls
4000 – 3/4	Columns, footings, footing caps, and crossbeams
3300 – 1 or 3/4	All other concrete

Use of internally cured concrete for bridge decks is preferred. Internal curing (IC) is a practical way of supplying additional curing water throughout the concrete mixture. This water can improve the hydration of cement, reduce autogenous shrinkage, and improve durability. Use of IC with lightweight fine aggregate for concrete is allowed to mitigate cracking due to shrinkage in bridge decks.

Contact the Technical Specialist and the Structural Materials Engineer for all new bridge decks to determine when IC or HPC/IC is to be used. Internally cured concrete utilizes lightweight fine aggregate (LWFA) according to *ASTM C1761*. The availability of lightweight fine aggregates (expanded clay or shale) depends on the local suppliers.

By replacing normal weight fine aggregate with lightweight fine aggregate for IC, the unit weight of the concrete is lighter. See *BDM 1.3.2* for concrete dead loads. When using lightweight concrete, adjust reinforcement development length for lightweight concrete per *LRFD 5.10.8.2. SP 02001* is required when internally cured concrete is specified. Curing time before subsequent loading may be shortened.

1.5.1.1 Corrosion Protection System

Provide a corrosion protection system for reinforced concrete girders, pile caps, diaphragms, and crossbeams as shown in *Table 1.5.1.1-1*. See environment definition in *BDM 1.2.4*.

Table 1.5.1.1-1 Corrosion Protection System for Precast and Cast-In-Place Concrete Members

Environment	Marine Areas	Coastal Areas
Precast and cast-in-place concrete	HPC	HPC
Non-prestress reinforcement	Stainless steel	ChromX4100
Prestressing strand	Uncoated, black	Uncoated, black
Minimum concrete clear cover for cast-in-place members	2 inches	2 inches

5.2 Concrete Finish

Concrete finishes are defined in *SP 00540.53* of the *Oregon Standard Specifications for Construction*. The usual finishes are General Surface Finish and Class 1 Surface Finish. Occasionally, Class 2 Surface Finish is used as mentioned in the following paragraph.

Concrete finishes are selected as follows:

- For bridges whose superstructure and substructure can be viewed by the public, such as grade separations and river crossings in or near populated areas, exposed surfaces receive a Class 1 Surface Finish. In special situations of high visibility to traffic or people, use of a Class 2 Surface Finish may be considered. Normally, it is limited to the concrete rail sides facing the roadway/bikeway and the tops.
- For bridges not viewed by large segments of the public, such as stream crossings in sparsely populated areas, exposed surfaces, except portions of the concrete bridge rail, receive a General Surface Finish. The concrete rail sides facing the roadway/bikeway and tops receive a Class 1 Surface Finish.

Review selected surface finish with the design team.

Pedestrian concrete bridge decks and sidewalks do not require Deck Roadway Texturing with saw cutting according to *SP 00540.50(c)*. Instead apply a Deck Sidewalk Finish according to *SP 00540.50(d)*. Ensure that all pedestrian surfaces comply with ADA slip resistance and surface requirements.

Do not use color additives in concrete mixes. Provide color to concrete only by coating with either concrete stain or concrete paint products from the QPL.

Include details similar to *Figures 1.5.2-1, 1.5.2-2,* or *1.5.2-3* for all contract plans:

Figure 1.5.2-1 Concrete Finish Detail for Box Girder Bridges

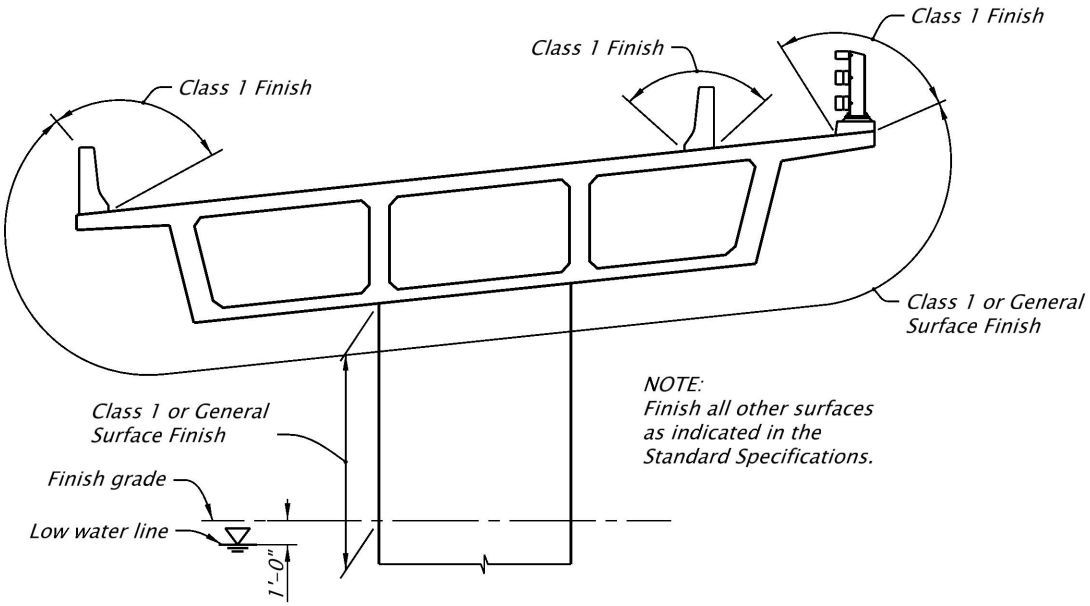


Figure 1.5.2-2 Concrete Finish Detail for Precast/Prestressed Concrete Girder Bridges

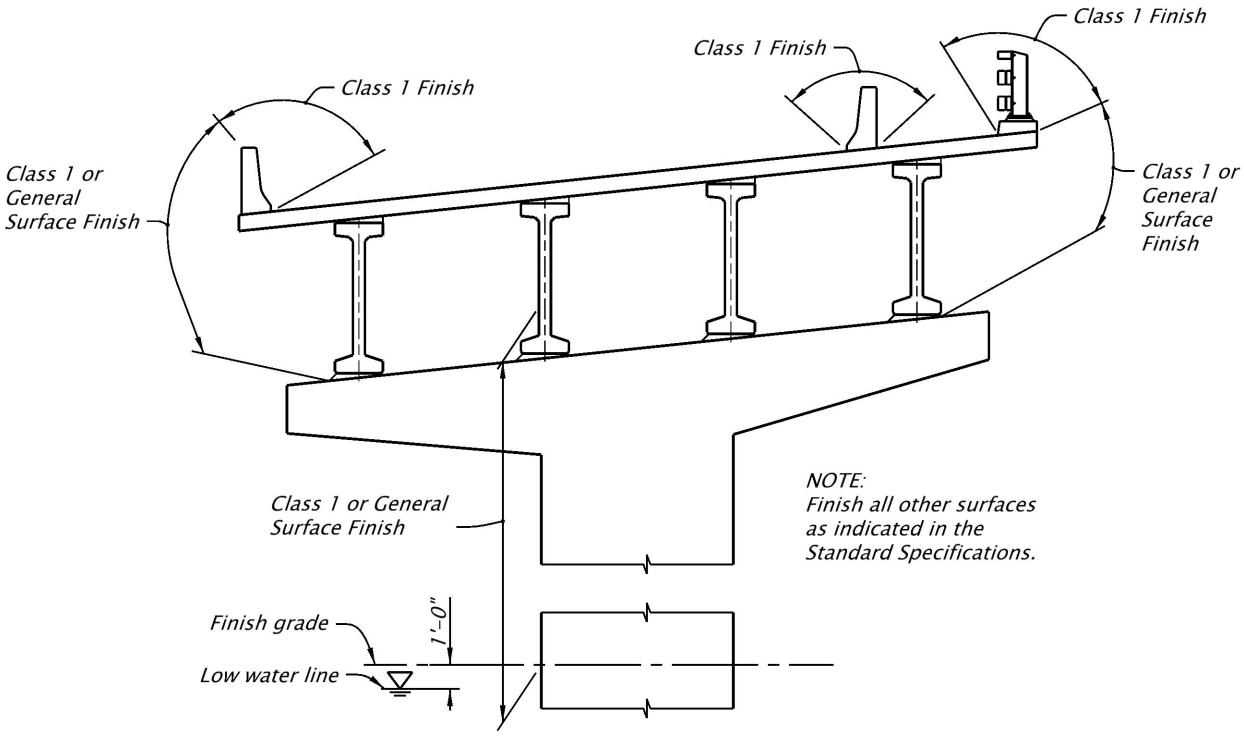
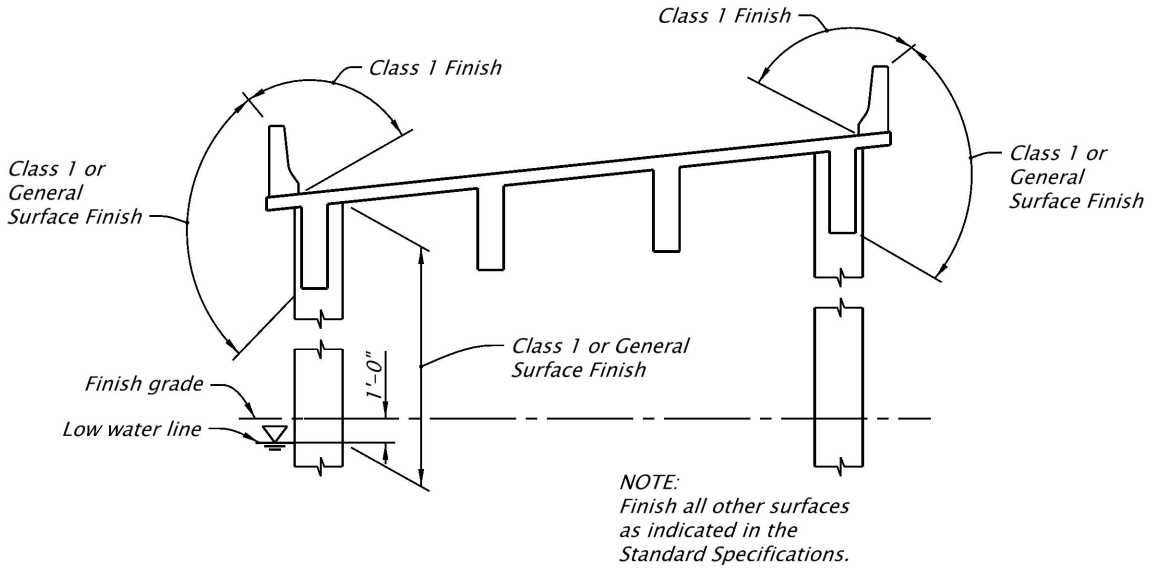


Figure 1.5.2-3 Concrete Finish Detail for Reinforced Concrete Deck Girder Bridges



1.5.3 Concrete Bonding Agents

Bonding agents are used to help new concrete adhere to existing concrete. To obtain better bond with agents the existing surface must be clean, dry and at proper temperature. The surfaces must also be well exposed to facilitate brush application of the bonding agent. Two principal bonding agents are in use today:

- Epoxy - These agents provide the best bond when properly applied. However, they are highly volatile and when the agent is allowed to dry before placement of the new concrete, a bond breaker may be formed. For this reason restrict the use of epoxy agents to critical situations where control can be guaranteed.
- Concrete - These agents have longer pot life and improved bond. They may be applied with greater lead time, but have the same application requirements as epoxy agents.

At normal construction joints, a bonding agent is not generally needed. Mating surfaces prepared to the specifications are considered sufficient to provide acceptable bond and shear transfer through the roughened surface and rebar holding a tight joint.

1.5.4 Curing Concrete

SP 00540.51 in the *Standard Specifications* require cast-in-place concrete to be cured with water. Design all structures assuming concrete is cured using the ODOT standard. Acting as EOR, assure that alternate curing methods are not allowed without prior approval of the Structural Materials Engineer.

Bridge decks must also be cured with water. Although ODOT does use curing compounds for some pavement and sidewalk applications, curing compounds are not be allowed on bridge decks. ODOT experimented with curing compounds in the early 1990s. The results were not consistent from batch to batch. Also, more recent experiments with curing compounds revealed that cylinders cured with a curing compound achieved only 80 percent compressive strength compared to water cured cylinders.

The ODOT water cure requirement also applies to bridge columns, abutments and retaining walls. Since it is difficult to keep vertical surfaces saturated during the cure period, vertical forms must often be left in place for the entire cure period. Contractors will often request to use a curing compound so that forms can be stripped sooner and production increased. However, due to the negative impacts of curing compounds, their use is rarely permitted.

For applications that receive a coating, use of curing compounds can inhibit adherence of the coating. Generally, curing compounds must be removed by sandblasting before subsequent coatings can be applied. Removal of a curing compound would be even more problematic on textured surfaces.

In summary, do not use curing compounds. Exceptions require approval from the Structural Materials Engineer, but do not require a design deviation.

1.5.5 Reinforcement

Ensure there is enough room for bars to fit and to place concrete. Ensure steel can be placed and supported. Show bolster bars on reinforcement details when needed.

Table 1.5.5-1 Standard Bar Chart

Bar #	Nominal Dia. (inch)	Area (in^2)	Weight (lb/ft)
3	0.375	0.11	0.376
4	0.500	0.20	0.668
5	0.625	0.31	1.043
6	0.750	0.44	1.502
7	0.875	0.60	2.044
8	1.000	0.79	2.670
9	1.128	1.00	3.400
10	1.270	1.27	4.303
11	1.410	1.56	5.313
14	1.693	2.25	7.650
18	2.257	4.00	13.60

Provide a minimum 2 inch clear cover for situations that are not listed in *Table 1.5.5-2.* When using stainless steel reinforcing, non-ferrous reinforcing or ChromX9100 use *AASHTO Table 5.10.1-1 Category C* cover requirements.

Table 1.5.5-2 Minimum Clear Cover for Coated or Uncoated Reinforcement

Location:	**Cover (inch)**
Top of deck slab (main reinforcing)*	2.5
Bottom of deck slab*	1.5
All superstructure surfaces exposed to the effects of a marine area, *BDM 1.9.2.1.3*	2
Stirrups and ties in T-beams, bottom rebar of slab spans, and curbs and rails *	1.5
Stirrups in box girder stems with non-bundled ducts **	2.5
Stirrup ties in box girder stems with non-bundled ducts **	2
Bottom slab steel in box girders	1
All faces in precast members (slabs, box beams and girders)	1
All cast-in-place members	2
Pier and column spirals, hoops or tie bars+ (increase to 4 inch when exposed to marine area or concrete is deposited in water)	2.5
Footing mats for dry land foundations (use 6 inch when ground water may be a construction problem)	3
Footing mats for stream crossing foundations	6
* Except in coastal areas use 2 inch cover. **For box girder stems with bundled ducts, provide 3 inches clearance to ducts and place stirrups directly against ducts. +Cover over supplementary crossties may be reduced by the diameter of the tie.	

1.5.5.1 Reinforcement for Shrinkage and Temperature

Provide reinforcement for shrinkage and temperature stresses near exposed surfaces and in structural mass concrete according to *LRFD 5.10.8*. Use an area of reinforcement per surface of at least 0.0008 times the gross concrete area with a minimum of #4 at 18 inch centers. Space the reinforcement no farther apart than three times the concrete thickness or a maximum of 18 inch centers.

Table 1.5.5.1-1 Temperature and Shrinkage Reinforcement Spacing

Thickness (inches)	**A_s (in^2/foot)**	**Maximum Bar Size and Spacing for One Surface (inches)**			
6 9 12	0.062 0.091 0.118	#4 @ 18 #4 @ 18 #4 @ 18			
15 18	0.144 0.170	#4 @ 15 #4 @ 12			
21 24 27	0.194 0.217 0.239	#4 @ 12 #4 @ 10 #4 @ 10	#5 @ 18 #5 @ 15 #5 @ 15		
30 36	0.260 0.300		#5 @ 12 #5 @ 12	#6 @ 18 #6 @ 15	
48 60	0.371 0.433		#5 @ 10	#6 @ 12 #6 @ 12	#7 @ 18 #7 @ 15

Since the amount of reinforcement is somewhat empirical, convenient spacing can be assumed as shown in *Table 1.5.5.1-1*. *Table 1.5.5.1-1* is intended for preliminary purposes only. It is based on a least width dimension of 10 feet.

1.5.5.2 Spacing of Shear Reinforcement

Where shear reinforcement is required and placed perpendicular to the axis of the member, spacing is not to exceed 18 inches.

1.5.5.3 Negative Moment Reinforcement

For cantilever crossbeams with wide bents, extend at least half of the negative reinforcement the full length of the crossbeam.

1.5.5.4 Minimum Bar Spacing

Table 1.5.5.4-1 Minimum Bar Spacing

Bar #	Nominal Dia.(d_b) (inch)	2.5 x d_b or 1.5+d_b (inch)	(1.5x1.5) + d_b for 1.5 agg. (inch)	(1.5x0.75)+ d_b for 0.75 agg. (inch)
3	0.375	1-7/8	2-5/8	1-1/2
4	0.500	2	2-3/4	1-5/8
5	0.625	2-1/8	2-7/8	1-3/4
6	0.750	2-1/4	3	1-7/8
7	0.875	2-3/8	3-1/8	2
8	1.000	2-1/2	3-1/4	2-1/8
9	1.128	2-7/8	3-3/8	2-1/4
10	1.270	3-1/4	3-1/2	2-3/8
11	1.410	3-5/8	3-5/8	2-1/2
14	1.696	4-1/4	4	2-7/8
18	2.257	5-5/8	4-1/2	3-3/8

1.5.5.5 Modified Tension Development Length - GRADE 60 – Uncoated Deformed Bars

Provide details to achieve $\lambda rc = 0.4$ reference to *LRFD 5.11.2.1*. The following modified tension development length is calculated using λrc, reinforcement confinement factor, equal to 0.4.

Table 1.5.5.5-1 Modified Tension Development Length – Grade 60 – Uncoated Deformed Bars - Top bars are horizontal bars placed so that more than 12 inches of fresh concrete is cast below the reinforcement.

Bar #	L_d (in) f'c =3.3 ksi	L_d (in) f'c =4.0 ksi	L_d (in) f'c =4.5 ksi	L_d (in) f'c =5.0 ksi	L_d (in) f'c =6.0 ksi
3	15.46	14.04	13.24	12.56	12.00
4	20.61	18.72	17.65	16.74	15.28
5	25.76	23.40	22.06	20.93	19.11
6	30.92	28.08	26.47	25.12	22.93
7	36.07	32.76	30.89	29.30	26.75
8	41.22	37.44	35.30	33.49	30.57
9	46.50	42.23	39.82	37.77	34.48
10	52.35	47.55	44.83	42.53	38.82
11	58.12	52.79	49.77	47.22	43.10
14	69.79	63.39	59.76	56.69	51.75
18	93.03	84.50	79.67	75.58	69.00

Table 1.5.5.5-2 Modified Tension Development Length – Grade 60 – Uncoated Deformed Bars – Other Horizontal Bars

Bar #	L_d (in) f'c =3.3 ksi	L_d (in) f'c =4.0 ksi	L_d (in) f'c =4.5 ksi	L_d (in) f'c =5.0 ksi	L_d (in) f'c =6.0 ksi
3	12.00	12.00	12.00	12.00	12.00
4	15.85	14.40	13.58	12.88	12.00
5	19.82	18.00	16.97	16.10	14.70
6	23.78	21.60	20.36	19.32	17.64
7	27.74	25.20	23.76	22.54	20.58
8	31.71	28.80	27.15	25.76	23.52
9	35.77	32.49	30.63	29.06	26.53
10	40.27	36.58	34.48	32.71	29.86
11	44.71	40.61	38.29	36.32	33.16
14	53.68	48.76	45.97	43.61	39.81
18	71.56	65.00	61.28	58.14	53.07

1.5.5.6 Modified Tension Development Length - GRADE 60 – Epoxy Coated Deformed Bars

Provide details to achieve $\lambda rc = 0.4$ reference to *LRFD 5.11.2.1*. The following modified tension development length is calculated using λrc, reinforcement confinement factor, equal to 0.4.

Table 1.5.5.6-1 Modified Tension Development Length – Grade 60 – Epoxy Coated Deformed Bars - Top bars are horizontal bars placed so that more than 12 inches of fresh concrete is cast below the reinforcement.

Bar #	**L_d (in) f'c =3.3 ksi**	**L_d (in) f'c =4.0 ksi**	**L_d (in) f'c =4.5 ksi**	**L_d (in) f'c =5.0 ksi**	**L_d (in) f'c =6.0 ksi**
3	20.21	18.36	17.31	16.42	14.99
4	26.95	24.48	23.08	21.90	19.99
5	33.69	30.60	28.85	27.37	24.98
6	40.43	36.72	34.62	32.84	29.98
7	47.17	42.84	40.39	38.32	34.98
8	53.90	48.96	46.16	43.79	39.98
9	60.80	55.23	52.07	49.40	45.09
10	68.46	62.18	58.62	55.61	50.77
11	76.00	69.03	65.09	61.75	56.37
14	91.26	82.89	78.15	74.14	67.68
18	121.66	110.50	104.18	98.84	90.23
* Cover less than 3db or clear spacing between bars less than 6db.					

Table 1.5.5.6-2 Modified Tension Development Length – Grade 60 – Epoxy Coated Deformed Bars – Other Horizontal Bars

Bar #	L_d (in) f'c =3.3 ksi	L_d (in) f'c =4.0 ksi	L_d (in) f'c =4.5 ksi	L_d (in) f'c =5.0 ksi	L_d (in) f'c =6.0 ksi
3	17.84	16.20	15.27	14.49	13.23
4	23.78	21.60	20.36	19.32	17.64
5	29.73	27.00	25.46	24.15	22.05
6	35.67	32.40	30.55	28.98	26.45
7	41.62	37.80	35.64	33.81	30.86
8	47.56	43.20	40.73	38.64	35.27
9	53.65	48.73	45.94	43.59	39.79
10	60.40	54.86	51.73	49.07	44.80
11	67.06	60.91	57.43	54.48	49.73
14	80.52	73.14	68.95	65.42	59.72
18	107.35	97.50	91.93	87.21	79.61
* Cover less than 3db or clear spacing between bars less than 6db.					

Table 1.5.5.6-3 Modified Tension Development Length – Grade 60 – Epoxy Coated Deformed Bars - Top bars are horizontal bars placed so that more than 12 inches of fresh concrete is cast below the reinforcement

Bar #	L_d (in) f'c =3.3 ksi	L_d (in) f'c =4.0 ksi	L_d (in) f'c =4.5 ksi	L_d (in) f'c =5.0 ksi	L_d (in) f'c =6.0 ksi
3	18.55	16.85	15.88	15.07	13.76
4	24.73	22.46	21.18	20.09	18.34
5	30.92	28.08	26.47	25.12	22.93
6	37.10	33.70	31.77	30.14	27.51
7	43.28	39.31	37.06	35.16	32.10
8	49.46	44.93	42.36	40.18	36.68
9	55.80	50.68	47.78	45.33	41.38
10	62.82	57.06	53.80	51.03	46.59
11	69.74	63.35	59.73	56.66	51.72
14	83.74	76.06	71.71	68.03	62.11
18	111.64	101.40	95.60	90.70	82.79
* Cover not less than 3db and clear spacing between bars not less than 6db.					

Table 1.5.5.6-4 Modified Tension Development Length – Grade 60 – Epoxy Coated Deformed Bars – Other Horizontal Bars

Bar #	L_d (in) f'c =3.3 ksi	L_d (in) f'c =4.0 ksi	L_d (in) f'c =4.5 ksi	L_d (in) f'c =5.0 ksi	L_d (in) f'c =6.0 ksi
3	14.27	12.96	12.22	12.00	12.00
4	19.02	17.28	16.29	15.46	14.11
5	23.78	21.60	20.36	19.32	17.64
6	28.54	25.92	24.44	23.18	21.16
7	33.29	30.24	28.51	27.05	24.69
8	38.05	34.56	32.58	30.91	28.22
9	42.92	38.98	36.75	34.87	31.83
10	48.32	43.89	41.38	39.26	35.84
11	53.65	48.73	45.94	43.59	39.79
14	64.42	58.51	55.16	52.33	47.77
18	85.88	78.00	73.54	69.77	63.69
* Cover not less than 3db and clear spacing between bars not less than 6db.					

1.5.5.7 Class B Tension Lap Splice (in) - GRADE 60 – Uncoated Deformed Bars

Provide details to achieve $\lambda rc = 0.4$ reference to *LRFD 5.10.8.2.1c*. The following modified tension development length is calculated using λrc, reinforcement confinement factor, equal to 0.4.

Table 1.5.5.7-1 Class B Tension Lap Splice – Grade 60 – Uncoated Deformed Bars - Top bars are horizontal bars placed so that more than 12 inches of fresh concrete is cast below the reinforcement

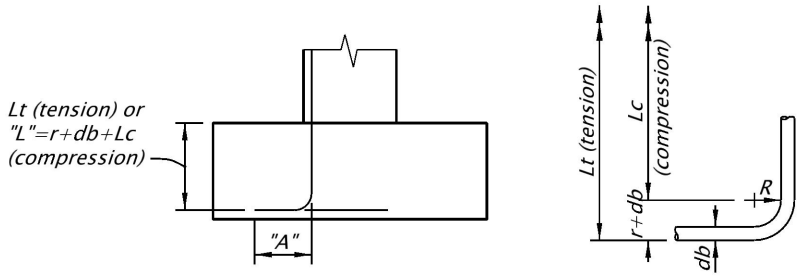
Bar #	L_d (in) f'c =3.3 ksi	L_d (in) f'c =4.0 ksi	L_d (in) f'c =4.5 ksi	L_d (in) f'c =5.0 ksi	L_d (in) f'c =6.0 ksi
3	20.09	18.25	17.21	16.33	15.60
4	26.79	24.34	22.94	21.77	19.87
5	33.49	30.42	28.68	27.21	24.84
6	40.19	36.50	34.42	32.65	29.81
7	46.89	42.59	40.15	38.09	34.77
8	53.59	48.67	45.89	43.53	39.74
9	60.45	54.90	51.76	49.11	44.83
10	68.05	61.81	58.28	55.29	50.47
11	75.56	68.63	64.70	61.38	56.03

Table 1.5.5.7-2 Class B Tension Lap Splice – Grade 60 – Uncoated Deformed Bars – Other Horizontal Bars

Bar #	L_d (in) f'c =3.3 ksi	L_d (in) f'c =4.0 ksi	L_d (in) f'c =4.5 ksi	L_d (in) f'c =5.0 ksi	L_d (in) f'c =6.0 ksi
3	15.46	15.60	15.60	15.60	15.60
4	20.61	18.72	17.65	16.74	15.60
5	25.76	23.40	22.06	20.93	19.11
6	30.92	28.08	26.47	25.12	22.93
7	36.07	32.76	30.89	29.30	26.75
8	41.22	37.44	35.30	33.49	30.57
9	46.50	42.23	39.82	37.77	34.48
10	52.35	47.55	44.83	42.53	38.82
11	58.12	52.79	49.77	47.22	43.10

1.5.5.8 Min. Column Bar Lengths in Footings – GRADE 60 & F'C =3.3 KSI

Figure 1.5.5.8-1 Minimum Column Bar Lengths in Footings



NOTE:
Increase Lt or Lc 20 percent or epoxy coated bars "A" and r+db are standard 90° hook dimensions Lc is the compression development length Lt is the tension development length for standard hooks.

Table 1.5.5.8-1 Compression Development Length – Hooked Bars

Basic Compression Development (Length for hooked bars) ldh (inches)	**Bar Size**	**"A" (inches)**	**Compression "L" * (single bar) (inches)**	**Compression "L" * (two bar bundle) (inches)**	**Compression "L" * (three bar bundle) (inches)**
16	6	12	3	16	17
19	7	14	3.5	18	20
21	8	16	4	21	22
24	9	19	6	25	27
27	10	22	7	31	33
30	11	24	7.5	37	40
36	14	31	11	53	58
47	18	41	15	90	98
* Note: Lc + (r + Db) and including 0.75 modification factor for reinforcement enclosed within a spiral per *LRFD 5.11.2.2.2*.					

Table 1.5.5.8-2 Tension Development Length – Hooked Bars

Bar Size	"A" (inches)	Basic Tension Development (length for hooked bars) ldh (inches)	Modified Tension "L" ** 0.7 ldh (inches)
6	12	10	7
7	14	13	9
8	16	17	12
9	19	21	15
10	22	27	19
11	24	33	23
14	31	47	33
18	41	84	59
** Note: #11 bars and smaller, adequate side and hook extension cover per *LRFD 5.11.2.4.2*.			

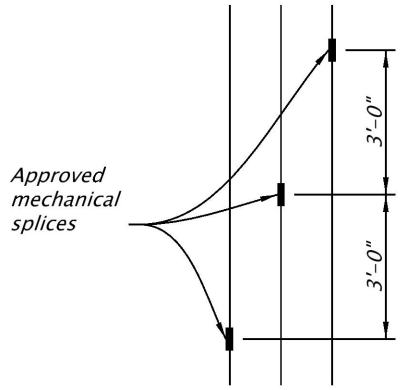
1.5.5.9 Welded Splices and Mechanical Connections

When field welding of reinforcing steel is anticipated, use *ASTM A706* reinforcing steel. Welding of *ASTM A615*, grade 60 reinforcing steel is not permitted without prior approval from the ODOT welding specialist.

Welding of *ASTM A706* for splices for column spiral reinforcing is permitted.

Use approved mechanical splices for #14 and #18 vertical column bars. Stagger splices as shown in *Figure 1.5.5.9-1*, to avoid adjacent bars being spliced in the same plane.

Figure 1.5.5.9-1 Mechanical Splice Staggering



Show lap splices on structure plans with the option of approved mechanical splices available to the contractor.

Special cases such as steel in back walls of abutments of post-tensioned concrete bridges and splicing reinforcement in existing structures may require the use of mechanical splices.

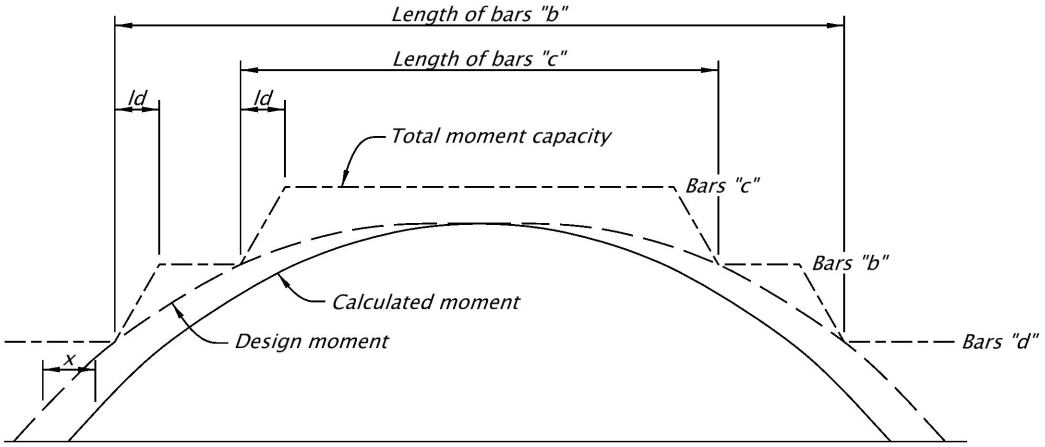
1.5.5.10 *Lap Splices – GRADE 60

(Reserved)

1.5.5.11 Development of Flexural Reinforcement

The added length, **X**, is to provide for unanticipated loading conditions or shifting of the moment diagram due to shear cracking.

Figure 1.5.5.11-1 Development of Flexural Reinforcement



x = effective girder depth,15 bar diameters or span/20
ld = bar development length

1.5.5.12 Distribution of Flexural Reinforcement

In mild areas, as defined in *BDM 1.2.4*, use $\gamma e = 1.0$. In marine, coastal and snow and ice areas, as defined in *BDM 1.2.4*, use $\gamma e = 0.75$. In decks, use $\gamma e = 1.0$.

1.5.5.13 Bundled Bars

Tie bundled bars with #9, or heavier, wire at 4 foot maximum centers. Use of bundled #14 or #18 bars requires the approval of the supervisor.

When bundled bars are used in columns, the minimum clear distance between bundles is 2.5 times the diameter of the largest bar in a bundle.

It is preferred bundled bars not be used in bridge decks. When they are used, increase the thickness of the deck by the diameter of the bar throughout the length where bundling is used.

1.5.5.14 Headed Reinforcement

Headed reinforcement can be used to reduce congestion or reduce development length over a standard hook. Headed reinforcement will always require less development length compared to a standard hook.

Headed rebar is only available for *ASTM A706* and *ASTM A615* applications. It is not available for stainless steel applications. The cost of headed rebar will generally exceed that of a standard hook. Therefore, only use them where the benefit of reduced congestion or shorter development length is significant.

Do not use headed reinforcement where their use will reduce concrete cover below the minimum required. For this reason, it may be necessary to use standard hooked bars in the corners of a rebar cage that otherwise contains headed bars.

Designate bars which require headed reinforcement on the plans. The *SP 00530* boiler plate special provision requires headed reinforcement to meet *ASTM A970*. It also requires headed reinforcement products be selected from the *ODOT QPL*. Therefore, there is no reason to say anything other than **headed bar** on the plans.

Heads may be square, rectangular, round or oval. Minimum head size for square and round heads are provided in *Table 1.5.5.14-1*. Rectangular and oval head area must exceed 10 times the bar area.

Table 1.5.5.14-1 Minimum Head Size for Headed Reinforcement

Bar Size	Min. Width for Square Heads (inch)	Min. Diameter for Round Heads (inch)
4	1-1/2	1.6
5	1-3/4	2.0
6	2-1/8	2.3
7	2-1/2	2.7
8	2-3/4	3.1
9	3-1/8	3.5
10	3-1/2	4.0
11	4	4.4
14	4-3/4	5.3

Headed reinforcement will not require project testing. Testing is required as part the QPL approval process. QC testing by the manufacturer is also required by *ASTM A970*.

When proposed by a contractor, headed reinforcement meeting the minimum head size requirement will generally be acceptable as a direct replacement for standard hooks, except where the head will not allow the required minimum concrete cover.

Use the minimum development lengths for headed reinforcement shown in *Table 1.5.5.14-2*.

Table 1.5.5.14-2 Development Length for Headed Reinforcement, Fy = 60 ksi

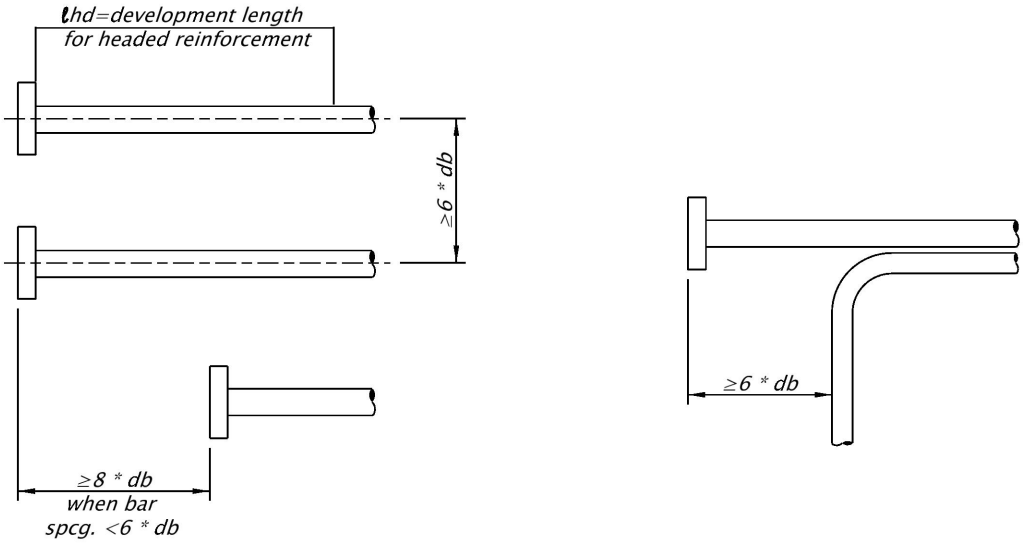
Bar #	**f'c =3.3 ksi (inch)**	**f'c =4.0 ksi (inch)**	**f'c =5.0 ksi (inch)**
4	6	5	5
5	7	6	6
6	8	8	7
7	10	9	8
8	12	10	9
9	18	14	10
10	22	18	11
11	25	20	12
14	36	28	15
Note: Increase lengths for epoxy coated bars per *LRFD 5.11.2.1.2*.			

Apply the modification factors and tie requirements in *LRFD 5.11.2.4.2* and *LRFD 5.11.2.4.3* to headed reinforcement also.

Place adjacent headed bars at a minimum spacing of $6 * db$. Spacing less than $6 * db$ can be used when heads from adjacent bars are spaced longitudinally (along the length of the bar) a minimum of $8 * db$ as shown in *Figure 1.5.5.14-1*.

When bundled bars are used, one bar in the bundle may be terminated using headed rebar. Terminate other bars in the bundle using standard hooks as shown in *Figure 1.5.5.14-1*.

Figure 1.5.5.14-1 Terminating Bundled Headed Reinforcement



Use of headed reinforcement can result in high concrete compressive stresses under the bar head. Consider the load path for head compression loads and provide distribution steel perpendicular to a headed bar to ensure satisfactory distribution of compressive stresses. The following articles may be useful to understand the load distribution of headed bars:

- *Strut-and-Tie Models for Headed Bar Development in C-C-T Nodes*, Chun and Hong, ACI Structural Journal, Vol. 106, No. 2, March-April 2009, pg. 123-130.
- *Investigation of Dispersion of Compression in Bottle-Shaped Struts*, Sahoo, Singh and Bhargava, ACI Structural Journal, Vol. 106, No. 2, March-April 2009, pg. 178-186.

LRFD 5.11.3 allows for mechanical devices as anchorage. Headed rebar meeting or exceeding the size required by *ASTM A970* has been extensively tested. A summary of testing can be found in *Texas Research Report 1855-1*, Anchorage Behavior of Headed Reinforcement Literature Review, May 2002.

The minimum development lengths for headed reinforcement are based on the greater of:

- 50 percent of the equivalent hooked bar development length or
- Calculations using a combination of head bearing capacity and bar development.

Development length calculations were based on concrete bearing capacity under the head plus additional straight bar development length as required to fully develop the yield strength of the bar. The concrete bearing capacity was taken from *LRFD equation 5.7.5-2* and was adjusted using a resistance factor of 0.7 for bearing on concrete per *LRFD 5.5.4.2.1*. Some of the proposed development lengths were increased to provide reasonable transitions between different bar sizes.

ACI 318 allows headed reinforcement, but requires a development length equal to 75 percent of the equivalent hooked bar development length. ODOT view this as overly conservative for bridge applications.

Table 1.5.5.14-3 illustrates the difference between ODOT and ACI development length requirements.

Table 1.5.5.14-3 Comparison of Headed Reinforcement Development Length

f'c	3.3 ksi			4.0 ksi			5.0 ksi		
	Calc (inch)	ODOT (inch)	ACI (inch)	Calc (inch)	ODOT (inch)	ACI (inch)	Calc (inch)	ODOT (inch)	ACI (inch)
#4		6	7.8		5	7.1		5	6.4
#5	4	7	9.8	2	6	8.9	0	6	8.0
#6	8	8	11.8	6	8	10.7	3	7	9.6
#7	10	10	13.7	8	9	12.5	4	8	11.2
#8	12	12	15.7	9	10	14.3	3	9	12.7
#9	18	18	17.7	14	14	16.1	7	10	14.4
#10	22	22	19.9	18	18	18.1	9	11	16.2
#11	25	25	22.1	19	20	20.1	7	12	18.0
#14	36	36	26.6	28	28	24.1	12	15	21.6

Development length controlled by 50 percent of equivalent hooked bar development length

Development length based on ODOT calculations, but less than ACI development length

Development length based on ODOT calculations and exceeds ACI development length

ACI development length = 75 percent equivalent hooked bar development length

Calc = Calculated development length from combination of head capacity and bar development

For concrete strengths above 5.0 ksi, the required minimum development length for headed reinforcement can be calculated using 50 percent of the equivalent hooked bar development length.

1.5.5.15 High Strength Reinforcement

1.5.5.15.1 ASTM A706 Grade 80

ASTM A706 grade 80 reinforcement is available on the market. The cost premium for A706 grade 80 reinforcement is approximately 6 to 10 percent above grade 60. Local steel mills (Cascade Steel Rolling Mills) are producing #5, #8, and #11 bars on a regular cycle and stock these bars. Avoid specifying other bar sizes, otherwise a minimum (combined size and length) of 50 tons is required. However, there can be some flexibility for smaller quantities. Contact Cascade Steel Rolling Mill for requirements when high strength rebar in non-standard rebar sizes is considered for a project with less than 50 tons.

When using A706 grade 80 reinforcement, the design yield strength is 80 ksi. ASTM A706 reinforcement is weldable. Welding would be needed when A706 grade 80 reinforcement is used for confinement hoops. The contractor needs to submit a PQR and WPS for approval as is typical for any rebar welding.

1.5.5.15.2 ASTM A1035 Grade 100

ASTM A1035 grade 100 reinforcement has a design yield strength of 100 ksi. *ASTM A1035* is described as Deformed Plain Low-Carbon, Chromium Steel Bars for Concrete Reinforcement. Proprietary products that meet the requirements of *ASTM A1035* specifications are sold under the brand names of ChromX 9100 (*ASTM A1035 CS*), ChromX 4100 (*ASTM A1035 CM*), and ChromX 2100 (*ASTM A1035 CL*). The main difference between the three products is the chromium content; the higher the number, the greater chromium content.

The reduced chromium content results in lower cost, when high corrosion resistance is not required. The products are not weldable. Currently, Cascade Steel produces ChromX 9100, ChromX 4100, and ChromX 2100 with a cost premium of approximately 192, 70 and 45 percent respectively. Cascade Steel carries some inventory of #5, #8 and #11 bars. For non-stock items, a minimum (combined size and length) of 50 tons is required. However, there can be some flexibility for smaller quantities. Contact Cascade Steel for requirements when high strength rebar in non-stock rebar sizes is considered for a project with less than 50 tons.

When *ASTM A1035* grade 100 bars are specified for a project, ensure that designation letters for the chromium content, i.e. CS, CM, or CL, are included with the ASTM material specifications shown in plan sheets and the project special provisions.

1.5.5.15.3 ASTM A615 Grade 100

Grade 100 reinforcement according to *ASTM A615* requirements is available. The cost premium for *A615* grade 100 reinforcement is approximately 35 percent above grade 60. Similar to other high strength reinforcement products, even though there is a required minimum order of 50 tons for combined size and length, there can be some flexibility for smaller quantities. Contact Cascade Steel for requirements when high strength rebar is considered for a project with less than 50 tons.

The cost premiums shown in this article are preliminary and for rebar production only.

1.5.5.15.4 Application of High Strength Reinforcement

Do not use high strength reinforcement in bridge members where plastic hinging is expected under seismic loads, such as bridge columns.. Although A706 grade 80 reinforcement has similar ductile properties compared to A706 grade 60, testing of full-scale seismic models sufficient to satisfy AASHTO concerns has not yet been completed.

The stress-strain property of A1035 grade 100 is very different from A706. There is not a well defined yield plateau. More experimental testing is necessary before its full implementation in members designed to form plastic hinges. The overstrength magnifier as defined for A706 in

the *Guide Specifications for LRFD Seismic Bridge Design* may not be appropriate. At this time, an overstrength magnifier of 1.4 is recommended when high strength reinforcement is used in capacity-protected members.

Use of high strength reinforcement is recommended in the following areas:

- Bridge decks –Refer to *Figure 1.9.1-6* and *Figure 1.9.1-7* for deck reinforcement design charts. See *BDM 1.9.2.1.2* for determining the appropriate use of high strength reinforcement in bridge decks.
- Drilled shafts – Use of high strength reinforcement reduces cost and congestion in drilled shafts thereby making them more constructible. Drilled shafts are designed for elastic seismic performance therefore plastic hinging is not expected.
- Crossbeams & End beams – Use of high strength reinforcement can reduce cost and congestion in negative and positive moment areas of crossbeams and end beams. Normally these members are capacity-protected; therefore they are designed to remain elastic during a seismic event. High strength reinforcement can be used for temperature steel and stirrups as well.

Grade 80 bars are a better option for a replacement of grade 60 bars due to lower cost premium and shorter development length compared to grade 100 bars. In addition, grade 80 has stress-strain behavior similar to grade 60 with greater yield stress and ultimate strength.

Within the same member, do not mix different rebar grades of the same bar size. This policy is to avoid any confusion that may occur during construction. It is acceptable to specify different rebar grades in the same member, when the different grades of bar are also significantly different in bar size (at least two bar sizes apart). For instance, longitudinal #8 bars in a crossbeam can be grade 80 bars, whereas #5 stirrups and temperature bars can be grade 60.

Table 1.5.5.15.4-1 illustrates rebar quantities in the previously mentioned members that are allowed to be reinforced with high strength reinforcement. The quantities shown in *Table 1.5.5.15.4-1* will be different when high strength reinforcement is used. A reduction of 10 to 30 percent in quantities can be anticipated when grade 60 rebar is replaced by grade 80 reinforcement.

All bridges in *Table 1.5.5.15.4-1* consist of precast prestressed concrete girders with a CIP deck. Most spans are simple for dead load and made continuous for live load. The bridges include drilled shafts with different lengths depending on the soil condition at the sites. Several bar sizes are grouped together since these bars can be alternately used in the design to reduce a number of different bar sizes. Specify only a few and commonly available bar sizes in each member.

Table 1.5.5.15.4-1 Comparison of High Strength Rebar Quantities

Bridge No.	Length (feet)	Deck Area (ft^2)	No. Drilled Shafts	Span Description	Deck (tons)		Crossbeams* (tons)		Drilled Shafts (tons)	
					#4 - #6	#7 - #9	#4 - #7	#8 - #11	#5 - #6	#9 - #11
22008	968	44015	8 – 8 foot	6 – 150 foot 1 – 50 foot deck girder spans	106	45	54	51	43**	218**
21576	249	31665	15 – 6 foot	2 – 122 foot deck girder spans	87	20	36	55	48	87
21343	524	25152	4 – 8 foot	3 – 180 foot deck girder spans	75	57	29	18	6	37
22248	84	11344	16 – 3 foot	1 - 80 foot deck girder span	33	1	12	8	4	10
* Includes intermediate diaphragms, end beams and cap beams.										
** A706 grade 80 rebar.										

As shown in *Table 1.5.5.15.4-1*, when non-stock rebar is specified, the amount of deck reinforcement in one bar group can meet the required minimum quantity for the first three bridges. The rebar quantities in crossbeams and end beams are not sufficient when a 20 percent reduction due to the use of high strength rebar is applied. For drilled shafts, the amount of reinforcement in one bar group can meet the required minimum quantity only for the first two bridges. However, when high strength rebar of the same size is also used in other members of the bridges, it is possible that the quantities of each bar size will reach the minimum order requirement.

Showing two options of rebar grades on bridge plans is encouraged to accommodate a contractor that may not be able to obtain high strength bars during a construction project. When this approach is taken, all dimensions need to be prepared to work with both options, especially details related to splice lengths and development lengths. Splice lengths and development lengths for high strength rebar are longer compared to grade 60.

Couplers are available on the market for high strength reinforcement. These couplers are capable of meeting 125 percent of yield strength. The ODOT Materials Lab has the capability to test rebar couplers up to #14 bars in grade 100.

1.5.5.16 Glass Fiber Reinforced Polymer (GFRP) Reinforcement

Glass fibers have an advantage over other fibers for composite materials because of an economical balance of cost and specific strength properties. Glass fibers are commercially available and exhibit good electrical insulation properties. When glass fibers are encapsulated in suitable resin as a system, the composite material is less sensitive to alkaline environment, freezing and thawing condition, and extremely elevated temperature. These properties make GFRP suitable for use as reinforcement for concrete structures. Through a number of research projects, it has been found that GFRP bars exhibit minimal loss in strength when subjected to sustained tension and have good resistance to fatigue. When GFRP bars are embedded inside concrete, there is no UV exposure concern.

ODOT has used GFRP bars on a number of projects particularly in bridge decks and sound walls. Using GFRP bars in the bridge decks was due to good corrosion-resistant and non-conductive properties. The non-conductive property of GFRP bars does not have detrimental effect to cathodic protection system often used for preserving marine and coastal area bridges as long as electrical continuity of adjacent steel reinforcement is maintained. For sound walls, wall weight needed to be minimized. Use of GFRP bars resulted in thickness reduction, since smaller concrete cover could be specified.

Since the release of the 1st Edition of the *AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete* in 2009, there have been advancements in material specifications along with new knowledge and field experiences. *AASHTO* published the 2nd edition of the guide specifications in 2018, which incorporated the new comprehension and extended design provisions for other structural members in addition to bridge decks and traffic railings.

For material specifications, *ASTM D7957* published in 2017 includes requirements for material standards of GFRP reinforcing bars. GFRP bars are commercially available from multiple manufacturers and can be produced for construction with a reasonable lead time. A specification for GFRP reinforcement has been prepared and is available when requested from the BDM technical resource.

GFRP rebar is a good corrosion-resistant reinforcement alternative for reinforced concrete bridges in corrosive environment compared to ferrous reinforcement due to cost and material strength. It is generally less expensive than other alternative reinforcement (stainless steel). Use of GFRP bars does not require electrical isolation to other ferrous reinforcement since it is not conductive. Tensile strength of GFRP bars is slightly higher than mild steel reinforcing bars, however GFRP bars have linear elastic behavior, much lower modulus of elasticity, and smaller ultimate tensile strain. Deformation or surface texture, or both, are required to ensure adequate bond capacity. Nominal material properties used for design are shown in *Table 1.5.5.16-1.*

Table 1.5.5.16-1 Material Properties for Design of GFRP Reinforced Concrete

Material Property	Material Value
Ultimate tensile strength (ksi)	90 ksi (#4 - #6) 75 ksi (#7 - #10)
Tensile modulus of elasticity (ksi)	6,500
Ultimate tensile strain (inch/inch)	0.011

GFRP reinforcement is recommended for structural members that are located in corrosive environment and do not require high ductility. Service limit states often control design over strength limit states. Use GFRP reinforcement only in the following structural members:

- Bridge deck.
- Sound walls.
- Seawalls.
- Bridge Approach Systems.

GFRP bar weight is lighter than steel reinforcement, therefore construction workers are able to handle the reinforcing bars with ease, but more rebar ties are required to maintain GFRP bar position in the rebar cage during concrete pour. GFRP bars are susceptible to abrasion and impact from studded tires and removal tools, therefore use in bridge concrete deck is limited in specific area as shown in *Table 1.26.3-1*.

1.5.5.17 FRP Reinforcement

(Reserved for Future Use)

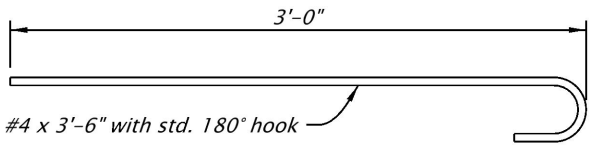
1.5.5.18 Bar Lengths

Use stock bar lengths whenever possible without sacrificing economy. Unless absolutely necessary, do not call for bars longer than 60 feet because they are difficult to handle and transport. Bar lengths specified include hook lengths unless detailed otherwise.

Table 1.5.5.18-1 Bar Length Stock Lengths

Bar Size	Stock Length *
#3	20 feet and 40 feet
#4 and #5	20, 30 and 40 feet
#6 thru #18	60 feet
*Only small quantities of #14 and #18 bars are stockpiled by the supplier because of size and weight and may require special mill orders.	

Figure 1.5.5.18-1 Specified Length of Bar with Hook



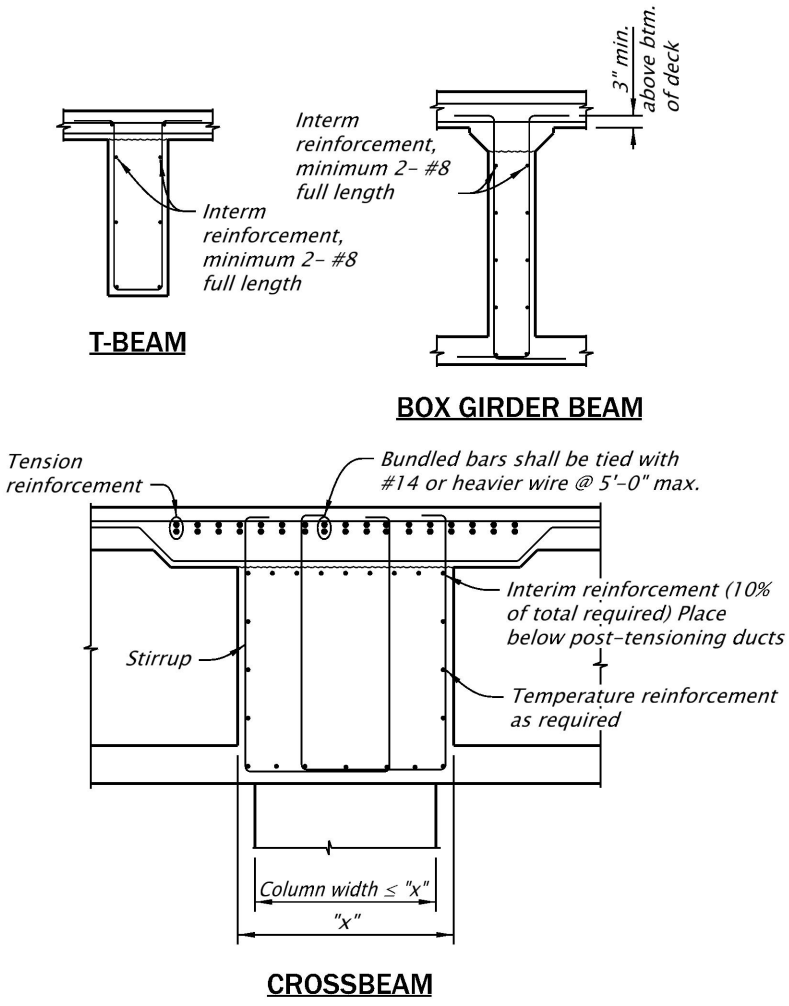
1.5.5.19 Interim Reinforcement for T-Beams and Box Girders

When the deck slab of a continuous T-beam or box girder is placed after the concrete in the stem has taken its set, place at least 10 percent of the negative moment reinforcing steel full length of the longitudinal beam to prevent cracks from falsework settlement or deflection. In lieu of the above requirement, two #8 bars full length of longitudinal girders may be used.

Locate a portion of the negative reinforcement in the stem of the crossbeam below the level of the deck slab construction joint in concrete crossbeams whose principal negative reinforcement lies in the deck slab. Provide sufficient ultimate reinforcement capacity to support 150 percent of the dead load of the crossbeam and superstructure 5 feet along the centerline of the structure either side of the center of bent. Use no less than 10 percent of the total negative reinforcement.

In cases where the bent crossbeams are skewed to the deck steel, place the top crossbeam steel in the top of the stem below the deck (dropped panel). See *Figure 1.5.5.19-1* for typical details.

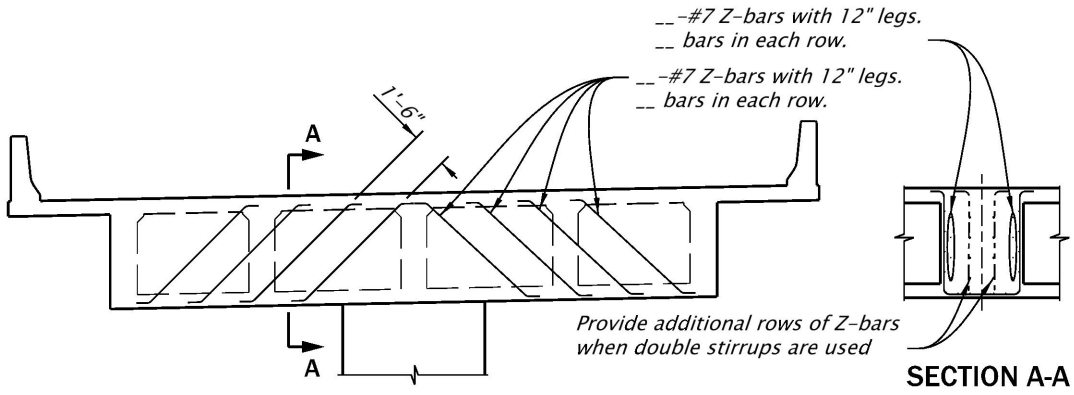
Figure 1.5.5.19-1 Interim Reinforcement



1.5.5.20 Additional Shear Reinforcement

As shown below, provide additional reinforcement to the calculated shear reinforcement in cantilevered portions of crossbeams. Pay careful attention to clearances and possible conflicts with post-tensioning ducts and other reinforcement. Detail the size and number of bars to provide at least 20 percent of the factored Strength I Limit State shear demand at the face of the column. Apply this provision to cantilevered sections of crossbeams when the crossbeam cantilever from the face of the column exceeds the crossbeam depth at the face of the column. This additional reinforcement may be omitted when the shear reinforcement provided from the critical shear section to the face of column provides 20 percent additional capacity above the controlling strength limit state.

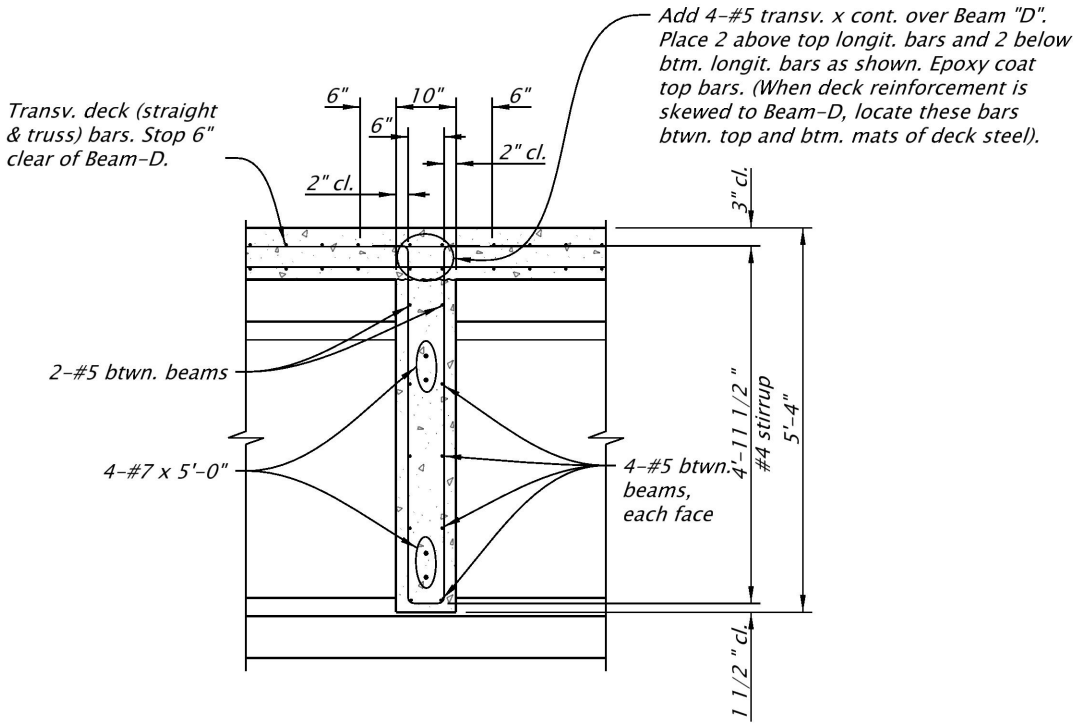
Figure 1.5.5.20-1 Additional Shear Reinforcement in Cantilevered Sections



1.5.5.21 Diaphragm Beam Reinforcement

Figure 1.5.5.21-1 below assumes the deck reinforcement is stopped 6 inches clear of the transverse beams. The added bars provide reinforcement for Beam-D and the deck overhang. When straight bars are used, the spacing of the deck steel will be continuous over the transverse beams and no additional bars will be required.

Figure 1.5.5.21-1 Diaphragm Beam Reinforcement



SECTION AT BEAM-D

1.5.6 Precast Prestressed Concrete Elements

1.5.6.1 Design of Precast Prestressed Elements

The nature of precast prestressed elements requires special handling in several areas.

DESIGN – GENERAL

- Each precast prestressed element is to be designed job specific.
- Deck requirements:
    - **Side-by-side slabs and box beams**: 5 inch minimum HPC thickness with a single mat of reinforcement (8 inch maximum centers each way). 7 inch minimum thickness for any portions overhanging the exterior slab or box beam.
    - **Side-by-side Bulb-T and deck Bulb-T girders**: 7-1/4 inch minimum HPC thickness with two mats of reinforcement (8 inch maximum centers in each mat and each direction).
    - **Spread slabs and box beams**: 8 inch minimum HPC thickness with two mats of reinforcement (8 inch maximum centers in each mat and each direction).
    - **Bulb-T (not side-by-side) and Bulb-I girders**: 8 inch minimum HPC thickness (see *BDM 1.9.1*).
    - **Deck Bulb-T girders with UHPC connection**: Precast concrete Deck Bulb-T girders are connected using UHPC at flange ends to form bridge deck. Air entrained concrete is required for girder top flange portion (8 inch minimum thickness). The deck girder system and connection details are shown in *Figure 1.5.6.1-1* and *1.5.6.1-2* respectively. See *BDM 1.9.2.3.1* for more information on UHPC. ¾ inch thick minimum PPC overlay is applied on top of the deck girder system.
    - Asphalt concrete pavement is not allowed on concrete bridge decks. See *BDM 1.9.3.1.6* for additional information.

Figure 1.5.6.1-1 Section View of Deck Bulb-T Girder System with UHPC Connections

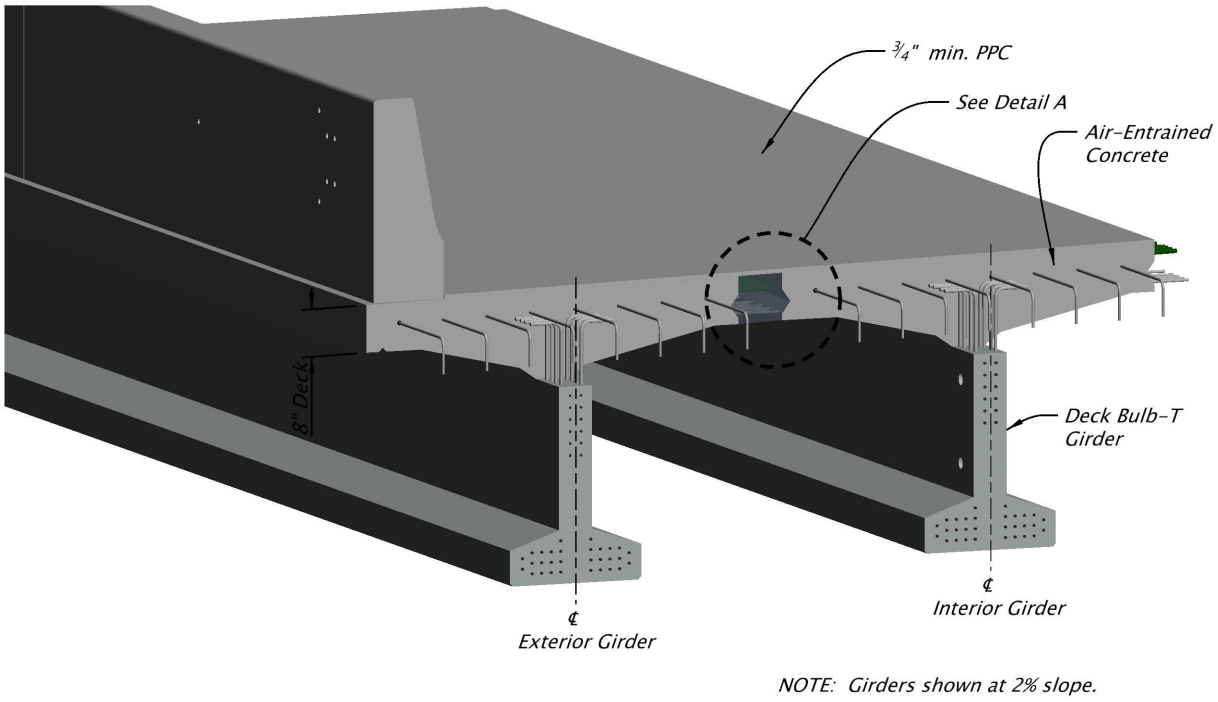
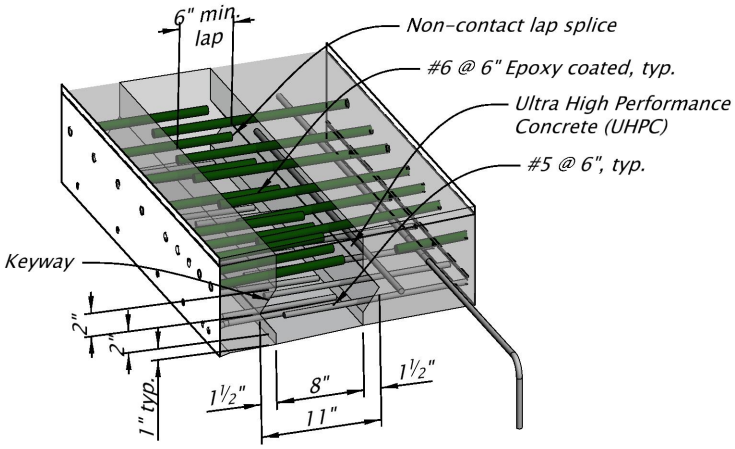


Figure 1.5.6.1-2 Connection Details Between Girder Flanges



- Concrete Strength – Ensure design compressive strengths of concrete are not significantly higher than design requirements and are within the following allowable ranges of design compressive strengths. List the required concrete strengths in the General Notes.

- The allowable range of design compressive strengths of concrete at 28 days ($f'c$) to be used are shown in *Table 1.5.6.1-1*.

Table 1.5.6.1-1 Allowable Design Compressive Strength Ranges

Precast Element	Minimum	Maximum
for precast, prestressed slabs and box beams	4000 psi	7000 psi
for precast, prestressed girders, and integral deck girders	5000 psi	9000 psi

When precast, prestressed members are used without a cast-in-place deck, the 28 day compressive strength is limited to 6000 psi. This limitation is required to ensure adequate air entrainment and to ensure adequate workability. Higher strength concretes generally are less workable and therefore are more difficult to achieve an acceptable finish suitable for a riding surface. When a separate concrete mix (6000 psi or less) is used for the top flange, then higher strengths (up to 9000 psi) may be used for the remainder of the member.

- The allowable range of design compressive strengths of concrete at release of all prestress ($f'c$) to be used are 4000 psi minimum and 7000 psi maximum.
- Do not exceed the compressive strengths listed above.
- Concrete Tensile Stress Limits:
  - Modify *LRFD Table 5.9.2.3.1b-1* as follows:
    - Modify the 9th bullet to $0.0948 * sqrt(f'c)$, where $f'c$ is in ksi.
    - **No Tension** criteria in 6th and 8th bullets still apply.
  - Modify *LRFD Table 5.9.2.3.2b-1* as follows:
    - Modify the 1st and 8th bullets to $0.0948 * sqrt(f'c)$, where $f'c$ is in ksi.
    - **No tension** criteria in 3rd, 5th and 7th bullets still apply.
  - Simple Span Girders Made Continuous for Live Load – When precast girders are made continuous for live load, design the positive moment area as if the girder was simply-supported. A maximum concrete tensile stress up to $0.19 * sqrt(f'c)$ in the positive moment area will be allowed for this condition. Also ensure that the maximum concrete tensile stress in the positive moment area does not exceed $0.0948 * sqrt(f'c)$ when the girder is considered continuous for live load.
- Use a load factor of 0.80 for live loads in Service III load combination to check tensile stresses in prestressed concrete members with prestressing strands and reinforcing bars.

PRESTRESS LOSSES

Calculate prestress losses in precast members according to *LRFD 5.9.3.4* – Refined Estimates of Time-Dependent Losses. This method of calculating losses is the **Detailed** method presented in NCHRP Project No. 18-07.

An Excel spreadsheet for calculating prestress losses using the *NCHRP 18-07* methods is available from the Bridge Engineering Section. This spreadsheet includes multiple methods for calculating prestress losses. Use the **Detailed** method.

Prestress loss estimates have generally been in the 35 to 45 ksi range. The *LRFD 5.9.3.4* loss calculations appear to be consistent with earlier loss predictions. And these loss levels have resulted in relatively accurate predictions of camber at the time of deck placement. There has also been no record of service cracking in bridges designed using these prestress loss levels.

Using prestress gain from loads permanently applied to girders, such as selfweight, bridge deck, superimposed dead loads, is allowed.

Do not include the prestress gain due to application of live load in the total long-term loss calculation.

The amount of prestress gain due to application of live load can be more than 20 percent of the total prestress loss. ODOT's policy of not including this gain results in a conservative estimate of final girder stresses. Note that prestress loss affects girder stress, but does not change the ultimate strength or capacity to carry permit loads.

Transforming the prestressing strand to increase section properties is not recommended. As stated in NCHRP 18-07, prestress losses should be calculated differently (no elastic losses or gains) when transformed properties are used for the prestressing strand. When so, the final girder stresses will be approximately the same whether gross or transformed section properties are used. Therefore, there is no significant advantage in using transformed section properties.

GIRDER SHAPE SELECTION

General – The Oregon Bulb-T girder shape is preferred for most Oregon bridge applications. This shape has a 4 foot wide top flange. This top flange provides safety for workers who must form bridge decks and ensures stability of the girder during shipping. Use Bulb-T girder shapes whenever it is appropriate to do so.

Bulb-I girders are a standard variation of the Bulb-T. To make a Bulb-I, the fabricator will start with the Bulb-T form and add blockouts to portions of the top flange to make the Bulb-I shape. Use the Bulb-I shape only when it has benefits over a Bulb-T. Since Bulb-I girders have a narrow top flange, it requires less concrete build-up over the girder compared to a Bulb-T. Therefore, bridges with high superelevation (generally, greater than 7 percent) may be candidates for the Bulb-I shape.

Since the Bulb-I section is 3 inches taller than the equivalent Bulb-T, it may be preferred for span lengths slightly longer than the equivalent Bulb-T capability. A Bulb-I section may provide benefits over a deeper Bulb-T section. However, due to shipping stability and worker safety concerns, a deeper Bulb-T might still be preferred when the deeper section can be accommodated within the available vertical clearance.

Modified Bulb-T girders include those having a non-standard top flange width and those having a wider web. Fabricators are generally able to adjust the top flange width anywhere

from 24 inches to 48 inches. At least 3 inches can also be added to the top flange. Discuss any modifications to the top flange with fabricators before placing modified details on plan sheets. Design deviations are not required for top flange modifications.

Only adjusted web thickness when necessary to accommodate post-tensioning (such as for spliced girders). For such cases, increase the web thickness from 6 inches to 7.5 inches. When doing so, increase the top and bottom flange widths by the same amount.

BT90 & BT96 girder sections are the largest in the Oregon inventory. These sections have a 5 foot wide top flange which is necessary to ensure shipping stability of very long girders. Do not consider changes to the top flange width without concurrence from fabricators. The longest girder available from fabricators is around 185 feet total length. Verify availability for any girder length exceeding 180 feet.

BT96 girders have not yet been used in Oregon. Verify availability of this section before specifying it on a project.

Roadway surface is directly provided by Deck-Bulb Tee girders with UHPC connection. Because girders are erected plumb, the top flanges are required to be cast on a slope equal to a specified superelevation. To ensure constructability of flange concrete, a superelevation is limited to 5 percent maximum. Specify thicker PPC overlay (3.75 inches maximum) and thickened flange near beam ends as required to match roadway vertical profile. See *DET3385* and *DET3386* for details.

Due to its function as the roadway surface, construction requires minimal differential camber between adjacent girders in the span. The construction specification requires a tight camber difference, however there will be some residual camber variation due to construction tolerances. Also, overfilling of the connection is a common placement method for casting UHPC. As a result, grinding is required to smooth out the uneven surface. Subtract ½ inch flange thickness in the design to account for top surface grinding. 15 percent over designed capacities are recommended as reserve capacities for locked-in force effects from the girder camber adjustment during construction.

AASHTO Type II, Type III, Type IV, and Type V shapes do not have the same efficiency as Bulb-T shapes. Therefore, use Bulb-T shapes in most cases. Use of AASHTO shapes is generally limited to bridge widenings where the existing bridge has AASHTO shapes. There may also be rare cases when an AASHTO shape may provide slightly less vertical clearance compared to the available Bulb-T shape.

WSDOT Shapes – WSDOT girder section may be specified, when there is no Oregon section that can meet an application or provide efficient design for certain situations. WSDOT girder sections may be an acceptable alternate based on cost and availability. The *Standard Specifications* allow contractors to propose an alternate shape provided it is similar to the specified shape and meets all project requirements (see *SP 00550.03*). Design WSDOT shape girders according to ODOT design requirements. Consult with fabricators for availability of WSDOT girder section forms. Two WSDOT shapes are allowed:

- WF girders: WF36G to WF83G girders. The maximum form length for the WF shapes varies depending on the girder section ranging 80ft – 170ft.
- WF Deck girders (UHPC): WF39DG to WF53DG girders. WF Deck girders are designed to be erected side-by-side and the top flanges are connected using UHPC. WF Deck girder forms will need modification because of the required thicker top flange due to the ODOT requirements, as a result there will be a cost premium to modify the forms.

Spliced girders with post-tensioning can be used to extend span capabilities of precast concrete girders. Consult with fabricators regarding the appropriate section and segment lengths for spliced girder applications.

Consult with fabricators before considering using haunched girders. Although haunches may provide an aesthetic benefit, any structural benefit from haunching a prestressed girder is minimal.

Trapezoidal Box Girders are available for applications that require special aesthetic considerations. Trapezoidal box girders can either have a uniform depth or parabolic haunches. Horizontally curved trapezoidal boxes have been used in Colorado.

STRAND TYPE

- Bulb-T and AASHTO girders were developed for use with 0.5 inch diameter prestressing strand. Do not consider use of 0.6 inch diameter strand for these sections without first consulting with fabricators. Modification of the girder section may be needed to accommodate 0.6 inch strand. BT90/96 sections were developed for use with 0.6 inch strand.

SHIPPING

When selecting the appropriate girder type, review potential shipping routes to make sure the proposed girder type can be shipped to the bridge site. Narrow roads and sharp curves may restrict the length of girder that can be used. Fabricators can generally provide assistance in this analysis.

OREGON FABRICATORS

The following northwest precast concrete fabricators can provide precast concrete members to Oregon bridge projects:

- Concrete Tech, Tacoma, WA.
- Knife River, Harrisburg, OR and Spokane, WA.

DETAILING – GENERAL

Camber - See *BDM 1.5.9* for special requirements pertaining to ACP, sidewalk, and rail requirements.

Deck Drainage - See *BDM 1.9.5* for details specific to slab and box beam elements.

Girder Storage and Shipment - *SP 00550.49* prohibits transportation before seven days and only after the 28 day compressive strength has been achieved. There may be special construction circumstances when a member needs to be transported and placed before the seven days, but it is not recommended before the 28 day compressive strength has been achieved.

When design requires a delay for placing the girder on bearing devices to decrease the bearing thickness or encasing the beam ends for fixed connection to reduce restraint moments due to long-term shrinkage and creep effect, specify a required wait time on the plan sheet.

Skew - Limit skew to 45 degrees for precast slabs and 30 degrees for precast boxes. Excessively skewed slabs and boxes tend to warp more, making fit and obtaining uniform bearing on the bearing pads more difficult. Stair stepping the bearing pads may be necessary to obtain uniform bearing.

Stage Construction of slabs and boxes with cast-in-place HPC decks - Do not use side-by-side slabs or boxes with HPC decks when precast elements must be placed in stages. Such stage construction does not allow tie rods to be placed as detailed in *BR445*. Spread slabs or boxes with a 7 ¼ inch minimum deck thickness (two mats of deck steel) would be an acceptable option for bridges constructed in stages.

Transverse Connection for Side-by-Side Slabs and Boxes – Connect side-by-side slab and box elements with transverse tie rods as detailed on *BR445*. Alternate connection details, such as intermittent weldments, are not allowed.

Surface Finish for Precast Members - The *Standard Specifications* requires a rake finish on the tops of members that are not used as a roadway surface. A rake finish combined with extending stirrup legs up into the deck is considered sufficient to provide adequate capacity to ensure composite action between the girder and deck. It is not necessary to require additional roughening.

Interface Shear – For all members with a cast-in-place deck, provide interface shear reinforcement full length of the member regardless of whether or not it is required by design. This requirement is satisfied by extending stirrups from the precast member up into the deck slab and will result in minimum reinforcement across the interface shear plane equal to two #4 bars at 18 inch centers.

Joint and Keyway details - see standard drawings for recommended details.

1.5.6.2 Design and Detailing of Precast Prestressed Girders

1.5.6.2.1 Stay-in-Place Forms

Where the spacing between edges of precast concrete girder flanges is no greater than 2 feet, steel stay-in-place deck forms may be used. However, do not use stay-in-place forms in exterior bays.

Steel stay-in-place deck forms may also be used behind end beams where the deck is continuous over interior bents. Hot-dip galvanize all steel stay-in-place forms.

When stay-in-place deck forms are used, provide a minimum section modulus of 0.15 in^3/ft and a maximum form height of 1 ½ inches. Install stay-in-place forms such that the top of the form is at the design bottom of deck thickness. The weight of a form meeting these requirements is likely to be less than 2 psf. This weight is not significant and need not be included in the design. However, add additional non-composite dead load per *Table 1.3.2-1* in the girder design to account for extra concrete weight.

Do not use stay-in-place forms at deck overhang areas or where the edges of girder flanges are greater than 2 feet apart. In such cases, access for inspection and future maintenance of the deck precludes the use of stay-in-place deck forms.

Do not use stay-in-place forms in coastal areas, as defined in *BDM 1.2.4.2*.

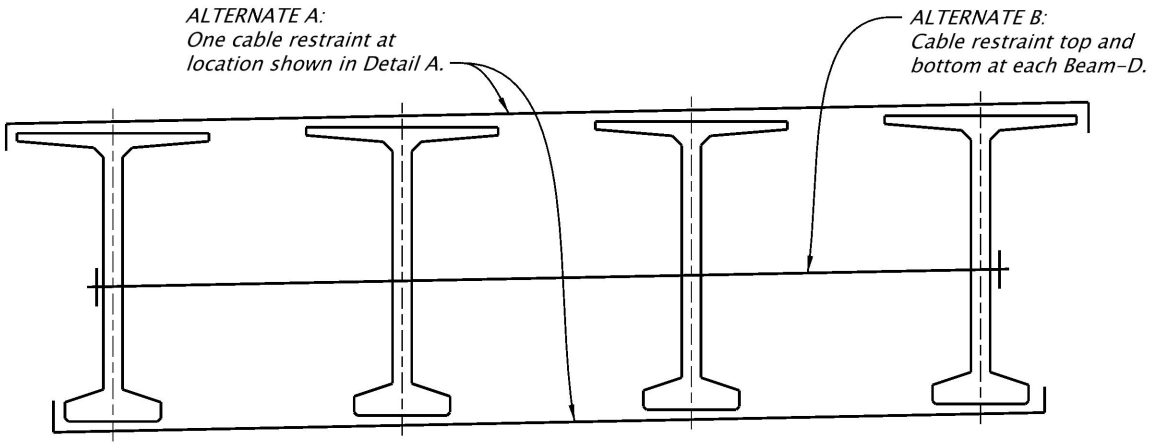
These provisions apply to precast girders, slabs and boxes.

Where stay-in-place forms are considered, add the following statement with the loading section of the general notes:

> Stay-in-place deck forms may be used except for exterior overhangs and between the exterior girder and the first interior girder on each side of the structure. XX psf additional non-composite dead load has been included in the girder design to account for extra concrete and form weight associated with stay-in-place forms.

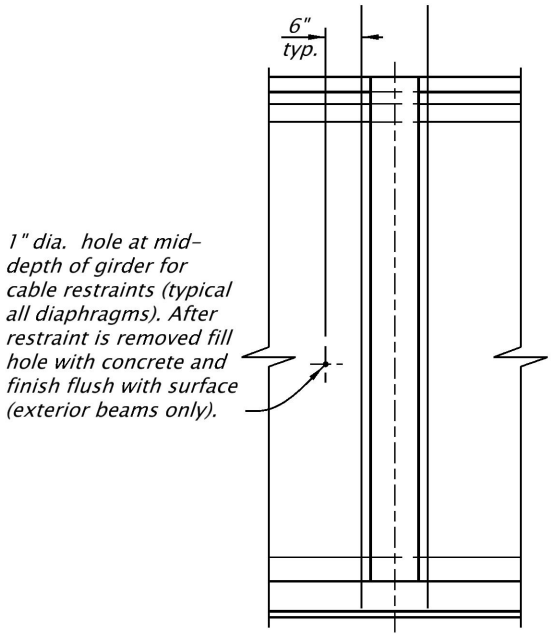
1.5.6.2.2 Diaphragm Beam Restraint

Figure 1.5.6.2.2-1 Diaphragm Beam Restraint



CABLE RESTRAINT DIAGRAM

Snug fit prestressed girders against forms prior to diaphragm pour. Restraint to remain in place a minimum of two days aftercompletion of diaphragm pour.



1.5.6.2.3 Beam Seat or Top of Crossbeam Elevation

Provide a note on the plans indicating when the beam seat (or top of crossbeam) elevations shown are for deck buildups based on three months camber. Adjust the beam seat (or top of crossbeam) elevations during construction to correct for the revised deck buildups.

1.5.6.2.4 Continuous Deck Reinforcement

Simply supported precast prestressed girders with continuous deck over interior piers can be used to create deck continuity for the entire structure. Design the girders as simply supported for superimposed dead loads and live loads, only the deck and its reinforcement are continuous over the pier.

Provide a saw-cut control joint to control cracking. Do not use the continuity detail for bridge spans greater than 160 ft. Use the bar size, length and spacing as shown in *Figure 1.5.6.2.4-1*. Specify 20 feet long reinforcing bars for bridge spans with 60 inch tall girders or smaller. Specify 30 feet long reinforcing bars for bridge spans with girders taller than 60 inches.

This detail is not allowed on bridges with SDC C and D due to unknown performance during a large earthquake.

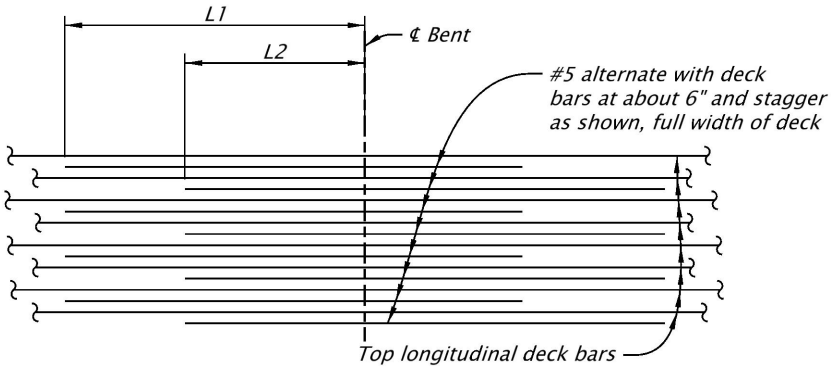
C 1.5.6.2.4

The continuity detail allows simplification of design and detailing and elimination of the expansion joint over the pier. Flexural cracks in the deck are expected at the pier due to live loads. The cracks should close when the live loads disappear.

This detail does not apply to bridges designed simple for dead loads and made continuous for live loads. The girder system requires deck reinforcement to resist negative bending moments generated from the live loads. This results in more deck steel than the detail shown in *Figure 1.5.6.2.4-1*.

NCHRP Report 519 provides design examples for girders made continuous.

Figure 1.5.6.2.4-1 Interior Bent with Continuous Deck



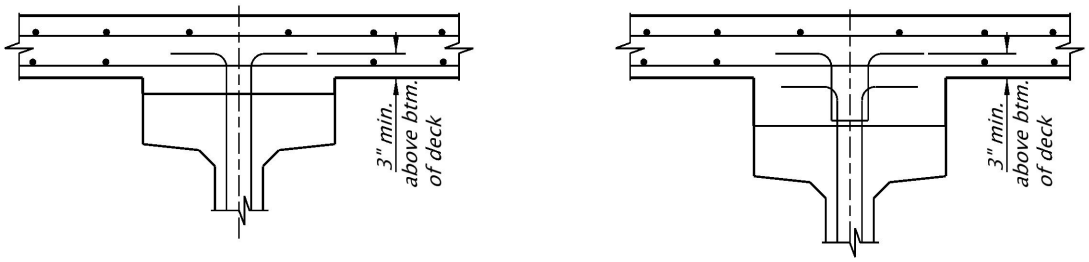
#5 Bar	L1	L2
20' long	12'-6"	7'-6"
30' long	17'-6"	12'-6"

1.5.6.2.5 Beam Stirrups

Bulb-T and Bulb-I standard drawings show stirrups with 90 degree shop bent hooks at the top of the girder. These hooks must protrude at least 3 inches above the bottom of the deck. When they do not, because of excessive build-up, the standard drawing requires the use of **U** bars to fill the gap.

There is no need for the stirrup hooks or **U** bars to extend to the top mat of deck reinforcement, as has been shown in the past. Detail plans to reflect these requirements.

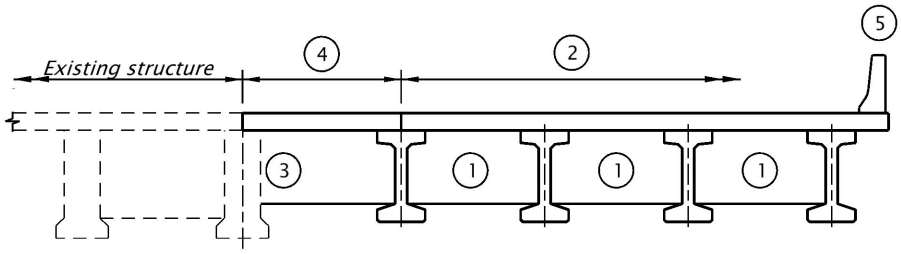
Figure 1.5.6.2.5-1 Beam Stirrup Details



1.5.6.2.6 Structure Widenings, Precast Beam Bridges

Detail connections between superstructures to prevent widening dead loads from being transferred to the existing beams. This may be accomplished by delaying the connection pour (diaphragm and deck) until most of the dead load is applied to the widening. The designer chooses the appropriate placement method. See *BDM 1.9.4.2* for additional closure pour information.

Figure 1.5.6.2.6-1 Closure Pour Method



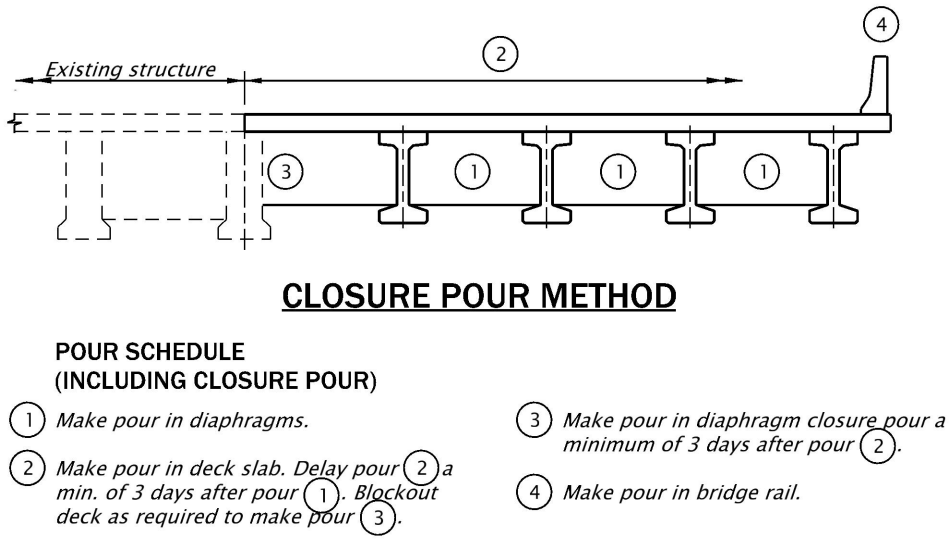
CLOSURE POUR METHOD

POUR SCHEDULE
(INCLUDING CLOSURE POUR)

1. *Make pour in diaphragms.*
2. *Make pour in deck slab. Delay pour (2) a min. of 3 days after pour (1).*
3. *Make pour in diaphragm of closure pour section.*
4. *Make pour in deck slab of closure pour. Delay a minimum of 3 days after pour (3).*
5. *Make pour in bridge rail.*

In the closure pour method shown in *Figure 1.5.6.2.6-1*, the deck screed machine would normally be placed or supported on the widening beams. As the concrete is placed, the beams tend to deflect equally. This equal deflection normally gives better control of deck thickness and deck steel cover.

Figure 1.5.6.2.6-2 Delayed Diaphragm Pour Method



In *Figure 1.5.6.2.6-2* showing the delayed diaphragm pour method, the deck screed machine rails would normally be placed or supported with one rail on the existing structure and one rail on the widening beams. As the concrete is placed, the new beams would tend to deflect more than the existing composite beams. This unequal deflection makes it more difficult to control deck thickness and deck steel cover, especially at the new beam adjacent to the existing structure.

1.5.6.2.7 Deck Pour Sequence

See *BDM 1.9.4.3*.

1.5.6.2.8 Diaphragm Beams

Intermediate diaphragms distribute loads from over-height vehicle, vessel and large debris collision. Use CIP diaphragm beams at ends of each span. Use full depth CIP diaphragms. Full depth diaphragms are more effective in distributing the impact loads.

Use the span length criteria in *Table 1.5.6.2.8-1* to determine the number of intermediate diaphragms for bridges crossing over major truck routes including Interstate 5, Interstate 84 and routes with 20 year projected ADTT greater than 5000; or waterways where there is a high probability of large debris or vessel collision.

Table 1.5.6.2.8-1 Criteria to Determine Number of Intermediate Diaphragms

Span Length (feet)	Location of Diaphragm
40 – 80	Midspan
81 – 120	1/3 points
Greater than 120	1/4 points

For other bridges, one set of intermediate diaphragms at midspan is recommended. Stagger and place intermediate diaphragms perpendicular to girder centerline, when the skew is greater than 25 degrees.

Install temporary diaphragms midway between the end and midspan diaphragm beams before pouring the end and midspan diaphragm beams (*see BR350*). Temporary beams may be removed after removing the deck overhang brackets.

1.5.6.2.9 Earthquake Restraint Details

See *Cost Data Books* for sample plans and details.

1.5.6.2.10 Fixed Girder Connections

Where girder ends are designed with a fixed connection to an end beam or bent cap, embed the girder into the end beam (or bent cap) a minimum of 8 inches. Provide transverse bars/rods through the girder ends as shown on the standard drawings (*BR300* & *BR310*). In addition to the above requirements, provide strand extensions or dowels at the end of the girder as needed to ensure adequate transfer of loads to the substructure.

To minimize restraint moments due to girder creep and shrinkage, establish continuity when the age of the girders is at least 30 days after casting. When a certain concrete age is required by design to minimize the time-dependent effect, include the following note in the plan sheet with continuity details:

> Place continuity diaphragm at least XX days after the girders are manufactured.

1.5.6.2.11 Girder Spacing

Limit girder spacing to 9 feet for girder sections up to BT72 and 1.5 times girder depth for larger girders.

PRECAST MEMBERS TOPPED WITH ACP

Side-by-side elements have been historically topped with ACP over a waterproofing membrane. See *BDM 1.9.3.1.6.* This type of construction works well in a stage construction scenario as long as the elements are placed consecutively from one side to the other.

When using this type of construction, the previous stage precast element at the stage construction joint must carry some of the wearing surface dead load from the subsequent stage since adjacent slabs must have their tie rods connected before the wearing surface is placed for the subsequent stage. This additional load is generally ignored (i.e., the members are designed as if they were all placed in one stage). Long term creep is thought to mitigate this condition. To date, annual inspections have found no distress in precast elements due to this practice.

For cases where elements cannot be placed consecutively from one side to the other, it becomes impossible to place standard transverse tie rods. For this reason, select a different structure type (ex., spread slabs or girders with CIP deck) when elements cannot be placed consecutively. Any side-by-side precast slab or box element must be connected to adjacent elements with transverse tie rods as detailed in *BR445*. Alternate details, such as intermittent weldments, are not allowed.

The use of asphalt concrete pavement is no longer allowed due to long term maintenance concerns. See *BDM 1.9.3.1.6*.

PRECAST MEMBERS TOPPED WITH CIP CONCRETE

Side-by-side elements may be topped with an HPC deck. See *BDM 1.5.6.1* for minimum deck thickness and reinforcing requirements.

For this type of construction, the deck dead load is substantially larger than the PPC or ACP case. For this reason, this type of construction must be detailed to prevent the deck dead load from later stages from being transferred to previous stages.

One solution to this problem is to provide a space (12 to 18 inches) between the stages that is filled with a CIP closure girder which is placed after full deck dead load is applied to both adjacent sections. For this case, design the precast members adjacent to the construction joint as exterior girders. Design the CIP closure girder to carry a contributory portion of live load under the strength limit states.

Use of spread slabs or boxes is another possible solution for stage construction. When so, use 7 ¼ inch minimum deck thickness with two mats of steel as required by *BDM 1.5.6.1*.

Any side-by-side precast slab or box element must be connected to adjacent elements with transverse tie rods as detailed in *BR445*. Alternate details, such as intermittent weldments, are not allowed.

1.5.7 Cast-In-Place Superstructure

1.5.7.1 General Design

STRUCTURE DEPTHS

See *BDM 1.2.7.2* for minimum depth and live load deflection requirements.

COMPUTATIONS OF DEFLECTIONS

Base computed deflections on the effective moment of inertia of the section.

Estimate long term deflections as instantaneous deflection times a factor of three for reinforced concrete elements.

1.5.7.2 Interim Reinforcement for T-Beams

See *BDM 1.5.5.19*.

1.5.7.3 Diaphragm Beam Steel

See *BDM 1.5.5.21*.

1.5.7.4 Box Girder Stem Flare

Taper changes in girder stem thickness for a minimum distance of 12 times the difference in stem thickness. See Standard Detail *DET3125* for details.

1.5.7.5 Shear Keys and Construction Joints

At construction joints between the stem and slab of concrete girder bridges, use the following note:

> Roughened surface finish. See *SP 00540.43(a)*.

See *BDM 1.14.2.2.1*.

1.5.7.6 Standard Access and Ventilation in Concrete Box Girders

Provide permanent access to all cells of concrete box girders. Access may require using manholes or access holes through bottom slabs, diaphragm beams, crossbeams and longitudinal beams. Standard Drawings *BR135* and *BR136* show standard access and ventilation details. See *BDM 1.2.10* and *BDM 1.15.5* for additional accessibility guidance.

In addition to the standard drawing for Access Holes, draw a section on the plans normal to the girder through the access hole showing the relationship of the longitudinal stems, utility lines,

and crawl holes to the access hole and ladder. When the drawing is to scale, dimensions need not be shown.

Use the following guidelines tempered with engineering judgment.

- Deck Access Holes – Avoid placing access holes through the deck of a structure. There is a potential for the access hole cover to leak. Also avoid disruption of traffic and the need for traffic protection and direction.
- Bottom Slab Access Holes - Single span bridges will normally require one access hole per cell. Multiple span bridges will normally require one access hole per cell at each end of the bridge. Locate access holes in accordance with the guidelines shown on the standard drawings. An 8 feet minimum height to the access hole is recommended to discourage unauthorized access into the structure. Keep the inspector in mind when choosing the access locations. Do not place access holes over railroad tracks.
- Girder Stem Access Holes - Girder stem access holes are to be provided through the interior stems at the midpoint of all spans. These lateral access points will allow the inspector to complete their inspection of span or spans without having to exit and reenter the structure.
- Crossbeam Access Holes - These are not detailed on the standard drawing since their design will vary widely because of structural requirements. However, only one access hole will be required per crossbeam when the girder stem access holes are provided.
- Bottom Slab Ventilation Holes - These ventilation holes, similar to the bottom slab access holes in design except top opening, are intended to be used in all cells of each span not having access holes. Generally, the ventilation holes would be located near the opposite end of the span from an adjacent span having access holes. The holes provide additional ports for removing forms, serves as an exhaust hole when forced ventilation is required and provides additional natural ventilation.
- Stem Ventilation Holes - These holes provide for the escape of lighter-than-air gases and are located near the high point of each span as detailed on the standard drawings.
- Ladder Support - The ladder support provides a safe support for the ladder while the inspector unlocks the access hole cover. After the cover is unlocked, reposition the ladder through the access hole so the inspector can grab onto the ladder while entering or leaving the box girder cell.
- Access Cover Prop - The access cover prop is designed to facilitate the opening or closing of the cover when the ladder is supported by the ladder support. Once the ladder is through the access hole, release the prop so the cover will lie flat. The prop would be reengaged upon exiting the box.

1.5.7.7 Form Removal

All forms are to be removed from cells where access is provided.

Deck forms to be removed may be supported off the bottom slab when the bottom slab is fully supported, designed to support the added load and has no detrimental effect on the structure.

Deck forms for non-accessible cells may be left in place. Deck forms left in place are not to be supported off the bottom slab. Web supported deck forms are acceptable. Include an allowance for deck form dead load in the design loads, see *BDM 1.3.2*.

1.5.7.8 Bottom Slab Details

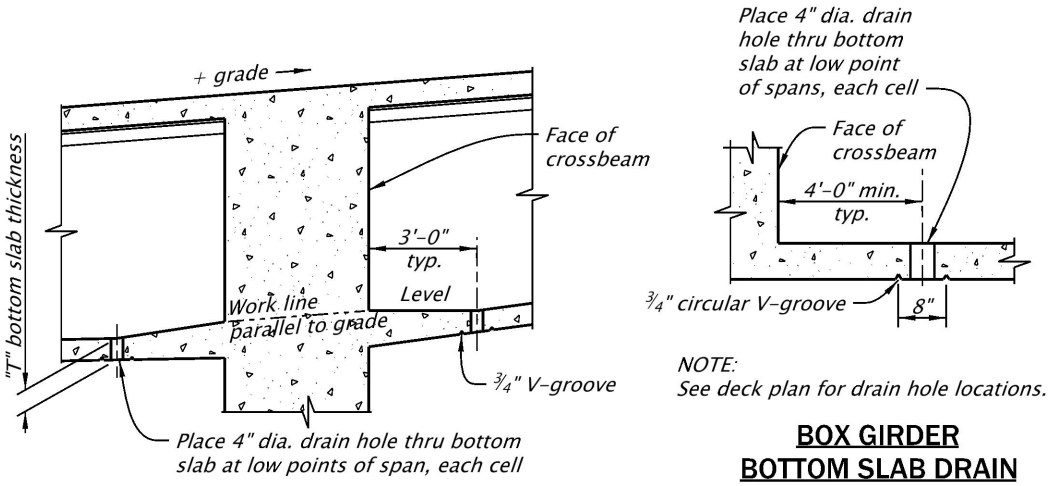
Generally, show the bottom slab of box girders to be parallel to the top slab in transverse section so that all girder stems will be the same depth.

Provide a bottom slab thickness of no less than 6 inches.

For skewed box girders, orient bottom slab transverse bars the same as the deck transverse bars. See *BDM 1.9.1* and *LRFD 9.7.1.3* for requirements.

Place a 4 by 4 inch drain hole through each diaphragm beam at the low point of each cell. Place a 4 inch diameter drain hole through the bottom slab at the low point of each series of cells in a span. For cells that carry water lines, increase 4 inch diameter to 6 inch diameter.

Figure 1.5.7.8-1 Bottom Slab Drain Details



1.5.7.9 CrossBeams

See *BDM 1.5.5.19* and *BDM 1.5.5.20*.

1.5.7.10 Fillets

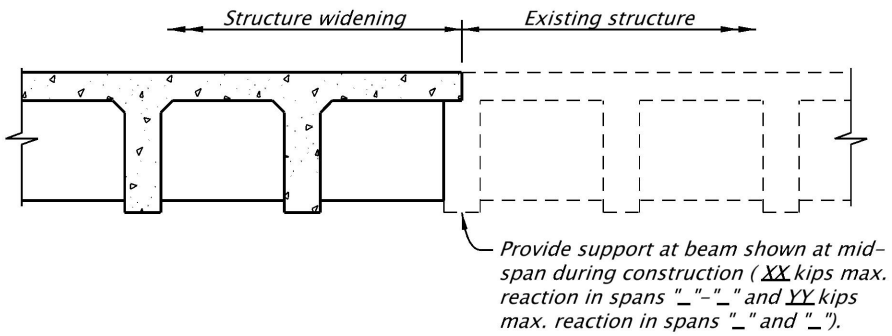
Provide adequate fillets at the intersections of all surfaces within the cell of a box girder, except at the junction of web and bottom flange where none are required.

Provide a 4 by 4 inch fillet at the intersection of the crossbeam and the deck; and the end diaphragm beam and the deck. For additional detailing requirements see *BDM 1.9.2.*

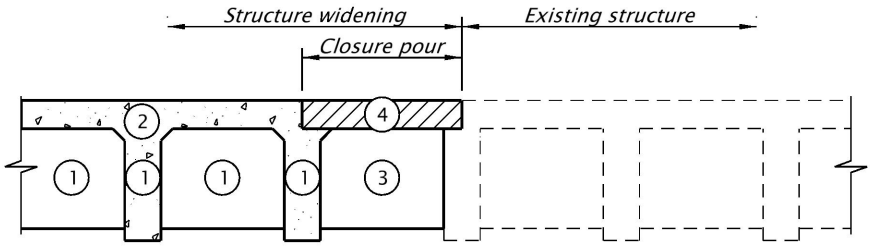
1.5.7.11 Structure Widenings, Cast-in-Place Superstructures

Detail connections between superstructures to prevent widening dead loads from being transferred to the existing beams. One method is to temporarily support the beam adjacent to the widening during construction. Designate locations where supports are required and expected maximum reactions. An alternate method requires closure pours for the diaphragm and deck slab. See *BDM 1.9.4.2* for additional closure pour information.

Figure 1.5.7.11-1 Closure Pour Method for CIP Superstructures



TEMPORARY SUPPORT METHOD



CLOSURE POUR METHOD

POUR SCHEDULE

(1) *Pour longitudinal beams and diaphragm beams shown to the bottom of the top fillets.*

(2) *Pour (2) includes top fillets and deck slab. Pour (2) to be delayed a minimum of 3 days after pour (1). A deck construction joint may be made over any diaphragm beam. Delay pouring adjacent section of deck a minimum of 36 hours.*

(3) *After falsework removal, pour diaphragm closure section.*

(4) *Make closure pour in deck slab. Delay a minimum of 3 days after pour (3).*

1.5.7.12 Stay-in-Place Forms for Deck

For deck construction, stay-in-place forms will not be allowed. Loss of access for inspection and future maintenance of the deck preclude the use of stay-in-place deck forms.

1.5.8 Post-Tensioned Structures

1.5.8.1 Design Considerations

1.5.8.1.1 Structure Depths

See *BDM 1.2.7.2* for minimum depth and live load deflection requirements.

1.5.8.1.2 Shrinkage and Creep Stresses

The stresses in the superstructure and substructure of post tensioned concrete bridges which result from elastic shortening may be assumed to remain in the structure indefinitely. The stresses which might be assumed to develop as the result of shrinkage and creep may be assumed to be relieved by creep.

1.5.8.1.3 Shortening of Post Tensioned Bridges

Values in *Table 1.5.8.1.3-1* for shortening of post tensioned, cast in place concrete bridges are based on field measurements by the ODOT Bridge Section. Compare the design values with the field measured values and use the more conservative value.

Table 1.5.8.1.3-1 Shrinkage Prior To Tensioning (Theoretical)

Shrinkage Time Period	**Shortening Length (inch/foot)**
0.4 x .0002 ft/ft x 12 in/ft x 100 ft =	0.10/100
Elastic shortening	0.44/100
Shrinkage and creep after tensioning to 1 year	0.29/100
Shrinkage and creep 1 year to 20 years (anticipated)	0.10/100

These structures were stressed to an average concrete stress of 1200 psi (1000 to 1300 psi). For other values, the elastic shortening and creep is roughly proportional. ODOT data indicates that variation of these values by 50 percent would not be unusual.

1.5.8.1.4 Deflections

Estimate long term deflections as the net instantaneous deflection ($DL + Prestress$) times a factor of two for cast-in-place post-tensioned elements.

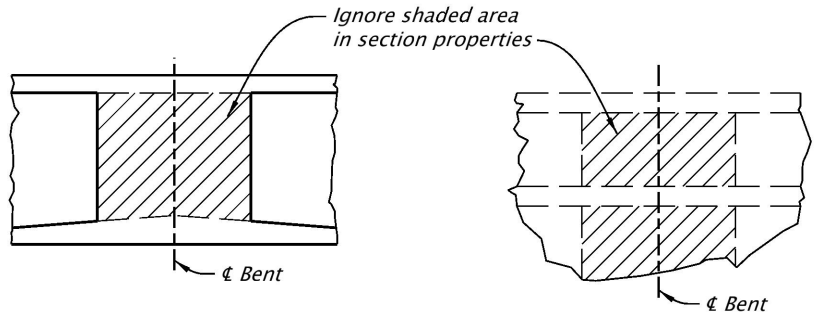
1.5.8.1.5 Curved Post-Tensioned Ducts

Design for the radial prestress forces resulting from curved tendons in post-tensioned structures. Additional shear/flexural reinforcement may be required to resist the lateral web forces and ties to resist the web bursting forces.

1.5.8.1.6 Design Moments at Interior Bents of Post-Tensioned Bridges

For crossbeams with widths less than the distance between the top and bottom slab, do not include the crossbeam in the superstructure section properties. Project the stem and slab dimensions to the centerline of the bent and use those dimensions to calculate section properties. Use the negative moment at the bent centerline for design.

Figure 1.5.8.1.6-1 Crossbeam Section Properties



For greater crossbeam widths, use the above section properties and consider adding supplementary reinforcing steel across the top of the crossbeam to control any theoretical cracking that may occur from live loading.

1.5.8.1.7 Skewed Box Girders

Box girder bridges with skews of over 20 degrees cannot be safely designed without taking into account the effects of skew. These effects generally increase as any of the following increase: skew angle, span length, torsional rigidity of the superstructure. The principal effect of skew is to increase the reactions at the obtuse corner of the structure and to reduce those at the acute corners (sometimes even causing uplift). This increases shear in the beams adjacent to the obtuse corners and produces transverse shear in the deck and bottom slab. These effects can be reduced by reducing the skew, which generally means lengthening the structure or by placing crossbeams at interior bents normal to the centerline of the structure.

When torsion due to skew is a problem, consider reducing the torsional stiffness of the structure. RCDG bridges, either cast in place or with precast girders, are torsionally limber. For additional detailing of skewed decks see *BDM 1.9.2.1.1*.

Do not design box girder bridges with bents skewed more than 45 degrees from the normal to the structure centerline.

Careful design of post tensioning with regard to the deflection and slope of the girder at a skewed end can nullify or reverse the tendency of the obtuse corner of the skewed structure to take a disproportionate part of the dead load. Theoretically, this could be done so that under

full $DL + LL + I$, the reactions would be equal at all bearings. Even an approximation of this condition will benefit the design.

1.5.8.1.8 Concrete Tensile Stress Limits

The concrete tensile stress limits given in *BDM 1.5.6.1* also apply to post-tensioned members.

1.5.8.2 General Details

BDM 1.5.7 generally apply to post-tensioned box girders as well as conventional box girders.

CONVENTIONAL BOX GIRDERS

See Standard Details *DET 3125* and *DET 3130* for general details.

PRECAST TRAPEZOIDAL BOX GIRDERS

See *BR133* and Standard Details *DET 3131*, *DET 3132* and *DET 3134* for general details.

ACCESS AND VENTILATION

See Standard Drawings *BR135* and *BR136* for general details.

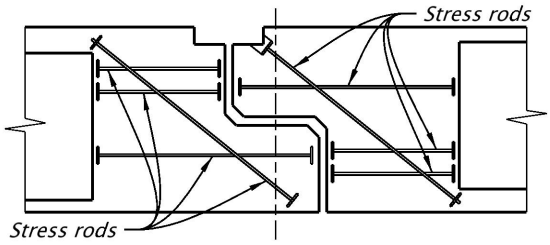
OVERHANG DESIGN AND REINFORCING

See *BDM 1.9.2.2.*

1.5.8.3 Stress Rod Reinforcement of Bearing Seats

An example of a stress-rod reinforced bearing seat is shown below. In order to retain a significant amount of prestressing force, provide a rod with stressed length of not less than 10 feet.

Figure 1.5.8.3-1 Stress Rod Reinforcement of Bearing Seats



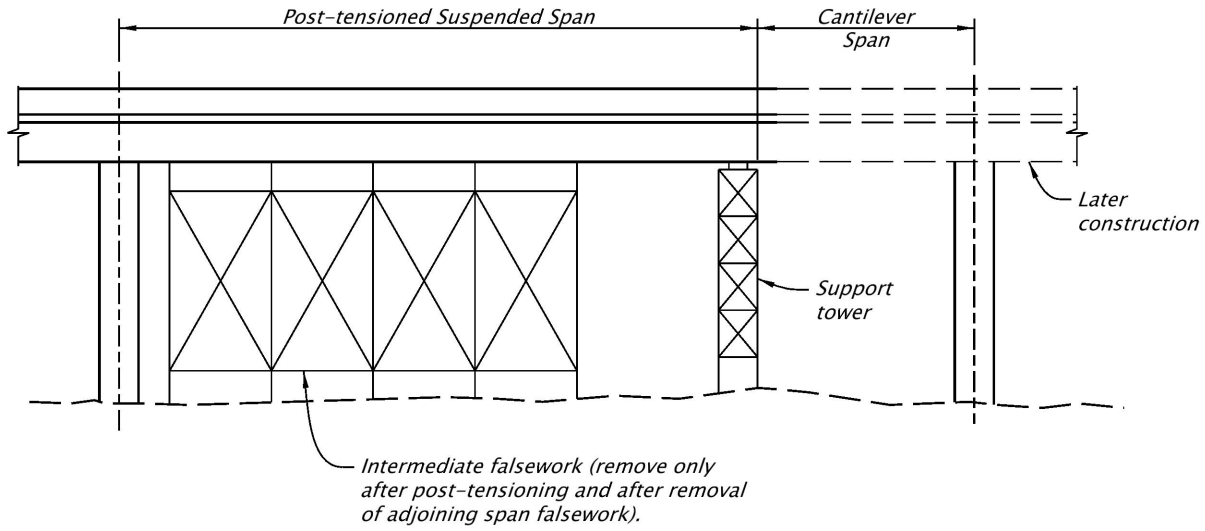
1.5.8.4 Segmental Construction

Where precast and cast-in-place elements are joined in a continuous, segmental structure, chamfer exterior corners of the cast-in-place portion to match precast elements. Standard practice is to chamfer precast elements, even when the chamfer is not shown on our drawings or the shop drawings.

1.5.8.5 Support Tower Details and Notes

Design the support tower at the end of the suspended span to support the reaction from the suspended span including the additional reaction due to post tensioning. Show on the plans the approximate total reaction in kips. Design the tower to accommodate the elastic shortening of the superstructure due to post tensioning. Make provisions so that the superstructure may be returned to the plan elevation (raised or lowered) in the event that actual settlement at the top of the tower differs from the anticipated settlement. Keep the support tower in place until the suspended span is fully supported by the cantilever and adjoining span.

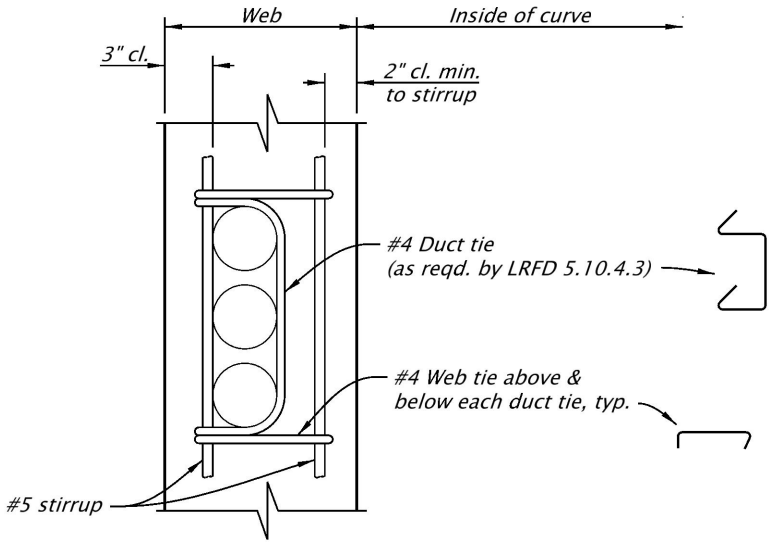
Figure 1.5.8.5-1 Support Tower Details



1.5.8.6 Post-Tension Strand Duct Placement

Place ducts for post-tensioned bridges using the detail provided on *DET3130*. The most common type of duct arrangement has been the bundled duct detail. This detail can be used when the duct size does not exceed 4 ½ inches and when the horizontal curvature of the bridge does not require the use of supplemental ties (see *LRFD 5.10.4.3*). When the horizontal curvature does result in the need for supplemental ties, do not use bundled ducts. When supplemental ties are required due to horizontal curvature, use the following detail:

Figure 1.5.8.6-1 Typical Web and Duct Tie Detail



Detail post tensioned box girders to allow pouring the bottom slab and stems as separate pours. Design the prestressed tendon path to ensure that the ducts do not fall in the area of the bottom slab. See Standard Details *DET 3125* and *DET 3130* for general details. To ensure the ducts are fully encased in concrete, do not place ducts in the bottom slab and keep ducts at least 1 inch below the fillet construction joints near the top slabs. Show *Figures 1.5.8.6-2 and 1.5.8.6-3* on the project plans when needed.

Figure 1.5.8.6-2 Low Point Detail

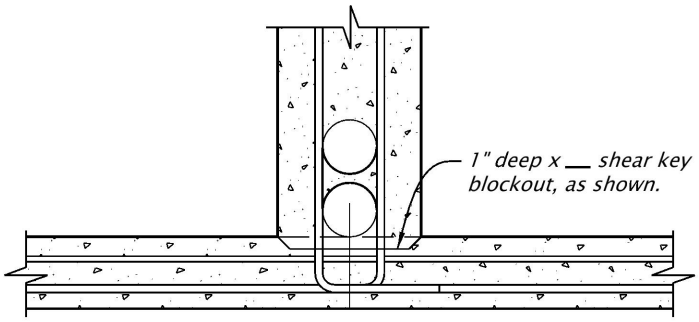
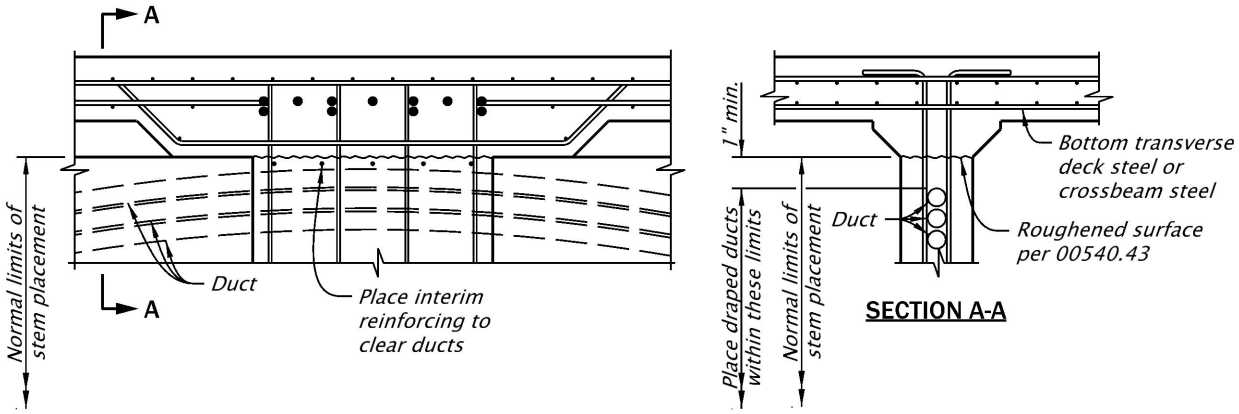


Figure 1.5.8.6-3 High Point Detail



In some cases it may be necessary to place ducts outside the limits shown above. When so, special concrete placement details will normally be needed to ensure the ducts are fully encased in properly consolidated concrete for the entire length of the bridge. For these cases, submit a design deviation request which shows the proposed duct placement detail. Include with the request the details and specification language intended to ensure the concrete will be fully consolidated in areas where the ducts penetrate either into the bottom slab or above the stem fillet construction joint.

1.5.9 Camber Diagrams

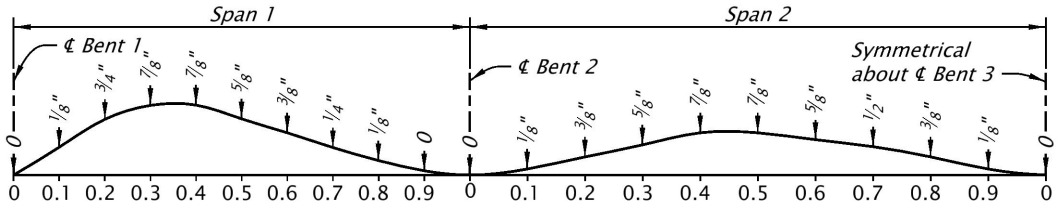
1.5.9.1 Camber Diagrams, General

Show camber diagrams on the plans for all types of cast-in-place concrete structures. The camber diagram shall be titled, **Camber Diagram** and be accompanied by the applicable portions of the following note:

Camber is designed to compensate for deflection due to prestressing, the dead load of all concrete, stay-in-place forms and wearing surface and the long-term effects of shrinkage and creep.

An example of a camber diagram for a cast-in-place structure is shown below in *Figure 1.5.9.1-1*.

Figure 1.5.9.1-1 Camber Diagram Example

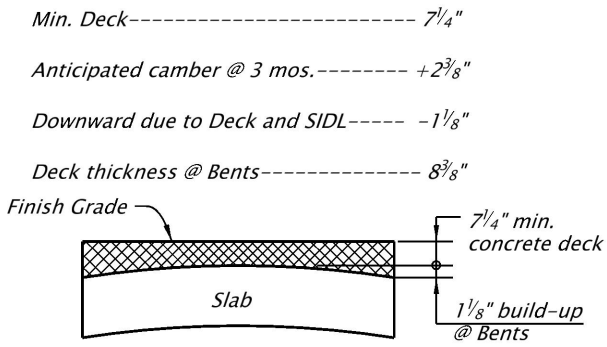


NOTE:
Camber is designed to compensate for deflection due to prestressing, the dead load of all concrete, stay-in-place forms and wearing surface and the long term effects of shinkage and creep.

1.5.9.2 Precast Prestressed Slabs and Box Beams

Camber of precast prestressed elements occurs due to prestressing strands generally located below the center of gravity of slabs or box beams. Reflect allowances for camber and grade correction in top of slab elevations. Increase rail posts lengths and curb heights accordingly near the ends to obtain the proper finish rail height and curb exposure. Note on the typical deck section that post lengths may vary due to camber or superelevation. Include information on the contract plans as shown in *Figure 1.5.9.2-1*. The figure shows slabs with 7 ¼ inch thick concrete deck as an example.

Figure 1.5.9.2-1 Concrete Deck Build-Up Detail



Note to Designer:
When a wearing surface is specified instead of deck concrete, replace "Deck" and "concrete deck" with the wearing surface name, e.g. PPC, MPCO, etc.

1.5.10 Pour Sequences

1.5.10.1 Pour Sequences, General

In order to avoid misunderstanding and claims by the contractor, ensure that construction sequences and pour sequences are clearly described. Particular care is needed when symmetrical structures are covered by sketches showing half of the structure.

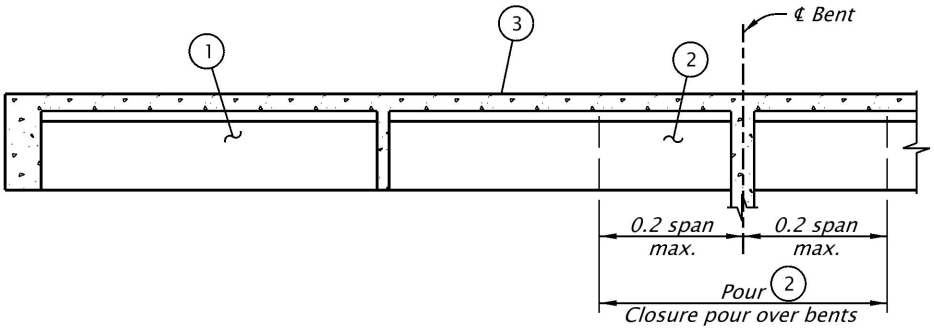
In general, longitudinal pours in continuous spans are stopped near the bents to allow concrete shrinkage to occur in the majority of the span. Closure pours over the bent are generally shorter to minimize shrinkage cracking that could occur between fixed supports or placements.

It is recommended to place bottom slab or beam construction joints at a falsework bent rather than a permanent bent. Cracking may develop at a permanent bent, when the adjacent falsework settles or deflects during the concrete placement. For deck closure pours and deck pour sequence see *BDM 1.9.4.2* and *BDM 1.9.4.3*.

1.5.10.2 T-Beams Supported on Falsework

A typical sketch and pour sequence is shown below in *Figure 1.5.10.2-1*.

Figure 1.5.10.2-1 Pour Sequence Example Detail



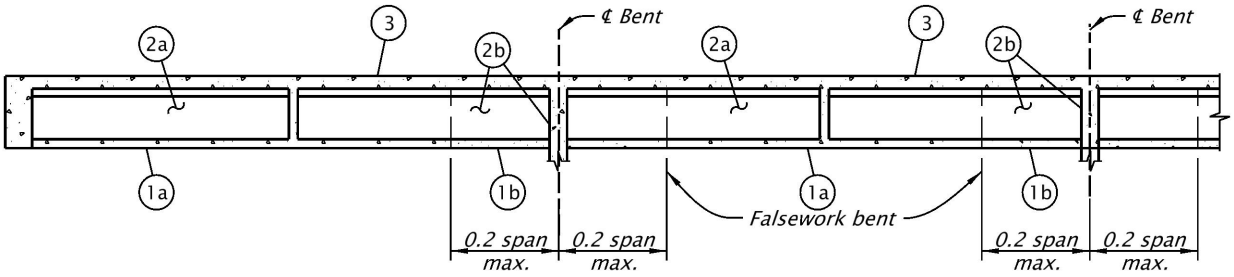
POUR SEQUENCE

1. Pours (1) and (2) are the longitudinal and transverse beams to the bottom of deck (or fillets). Make all pours (1) prior to pours (2). Beam construction joints shall not be near a permanent bent but shall be made at a falsework bent. Delay adjacent beam pours by a minimum of three days.

2. Pour (3) is the (fillets and) deck. Pour (3) to be delayed a minimum of three days after completion of all pours (2). A deck construction joint may be made over any transverse beam. Delay pouring adjacent sections of deck a minimum of five days. Do not remove bulkheads for deck pours until at least three days after completion of pour. Deck pours may extend over any part of a span or spans so long as they meet these requirements.

1.5.10.3 Box Girders on Falsework

Figure 1.5.10.3-1 Pour Sequence Example Detail



POUR SEQUENCE:

1. Pours (1a) and (1b) are the bottom slab. Stop pours (1) at a falsework bent and not at a permanent bent. Delay a minimum of three days between adjacent pours (1). Complete all pours (1a) prior to starting pours (1b). Complete all pours (1) prior to starting pours (2).
2. Pours (2a) and (2b) are the longitudinal and transverse beams to the bottom of the fillets. Stop pours (2) over a falsework bent. Delay the start of pours (2) a minimum of five days after bottom slab pours (1) are complete. Delay a minimum of three days between adjacent pours (2).
3. Pour (3) includes the fillets and deck slab. Pour (3) to be delayed a minimum of three days after completion of all pours (2). Pours (3) may be stopped over any transverse beam, with the use of a deck construction joint. Delay a minimum of five days between adjacent pours (3). Do not remove bulkheads for deck pours until at least three days after completion of the pour. Deck pours may extend over any part of a span or spans as long as they meet these requirements.

Generally, it is preferred that the bottom slab be completely poured first and separately from the longitudinal beams. This ensures a more uniform bottom slab thickness, the slab provides a good base for stem forms, and the continuous bottom slab helps stabilize the falsework system. It also allows the falsework to take its initial settlement without affecting other superstructure components.

1.5.10.4 Drop-In Precast Prestressed Elements

Complicated types of construction require detailed construction sequence notes, such as the following:

Figure 1.5.10.4-1 Pour Sequence Example Detail



POUR SEQUENCE:

1. Make pour (1).

2. Make pour (2), includes Bent 2 column.

3. Make pour (3a), includes bottom slab and webs to bottom of top fillet, Beam **C** to bottom of deck.

4. Make pour (3b), includes deck and top fillets for cast in place section. Delay pour (3b) a minimum of three days after completion of pour (3a).

5. Apply Stage I post tensioning to cast in place section. Stressing to begin a minimum of 14 days after completion of pour (3), but not until concrete in pour (3) has reached its design strength.

6. Place prestressed beams. Beams to be placed so that the number of beams in one span does not exceed by more than four the number in the opposite span.

7. Make pour (4), includes diaphragm beams **D** and end beams **E**.

8. Make pour (5), (no less than 60 days after transfer of stress in precast, prestressed beams), includes deck on prestressed beams to diaphragm beam nearest Bent 2.

9. After pour (5) has been made in Spans 1 and 2, make pour (6a), includes remainder of Beam **C**. Let concrete take initial set, and make Pour (6)b, includes remainder of deck.

10. Apply Stage II post tensioning to assembled Spans 1 and 2. Stressing to begin a minimum of 14 days after completion of Pour (6), but not until concrete in pour (6) has reached its design strength.

11. Pour curbs.

NOTES:

- 1. Bents 1 and 3 footings and walls may be poured any time up to seven days prior to placing of prestressed beams, but concrete must have reached its design strength prior to beam placement. No part shall interfere with post tensioning operations.
- 2. Paving slab and sidewalls may be poured at any time except that no part shall interfere with post tensioning operations.
- 3. Screed deck concrete parallel to bents.
- 4. Composite decks and closure pours shall not be made until at least 60 days have elapsed from the time of transfer of prestressing force in the precast elements.

1.5.10.5 Continuous Cast-in-place Slabs on Falsework

For pours over 600 cy, allow a transverse deck construction joint at $0.2 * (span)$ from the next interior bent.

1.5.10.6 End Bents

When the fit of superstructure elements is critical, be sure to consider the end bent construction sequencing. Normally the end wall construction is delayed until the superstructure elements are in place. Delaying the end wall construction also allows the contractor to compensate for errors in superstructure element lengths and end bent locations. Show a construction sequence diagram, with notes, as needed.

1.5.10.7 Steel Girders

See *BDM 1.6.2.14* for examples.

1.5.11 Concrete Anchors

1.5.11.1 Anchor Bolts / Rods

1.5.11.1.1 Materials

Anchor bolts / rods, including those for bridges, signs, traffic signals, and illumination structures, should normally be specified according to one of the following specifications:

- *ASTM F1554* is the preferred specification.
- *ASTM F1554*, grade 36 for low-strength.
- *ASTM F1554*, grade 55 for medium-strength.
- *ASTM F1554*, grade 105 for high-strength.

Equivalent ASTM designations for anchor bolts / rods are:

- *ASTM A307*, Low-strength carbon steel bolts for general use (non-headed rods conform to *ASTM A36*).
- *ASTM A449*, Medium carbon steel bolts and rods to 3 inch diameter. Proof load requirements are similar to *ASTM F3125* GR A325.

Galvanize anchor bolts or rods full length, when galvanizing is desired.

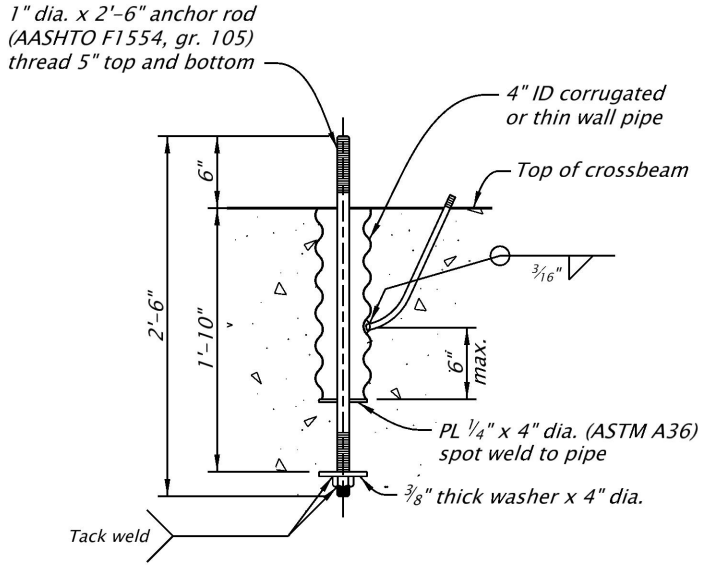
Anchorage of anchor bolts and rods may be accomplished by hooks for *ASTM A307* and grade 36 materials. For higher strength materials, a bearing plate tack welded to a nut or a plate between two nuts should be used.

When tensioning of anchor rods or bolts is desired, load indicator washers may be used up to 1¼ inch diameter (the largest available). Specify load indicator washers on the plans or in the Special Provisions, when required. Recognize that concrete creep and shrinkage may significantly reduce anchor rod stress over time.

1.5.11.1.2 Anchor Bolt Sleeves

To allow for some flexibility in placement and small corrections in bearing locations, an anchor bolt sleeve is often used. The anchor bolt can be field bent slightly to fit the required bearing location. The bearing plate can be temporarily shimmed and then the pad constructed or the pad can be constructed with a blockout around the bolt. The sleeve is grouted at a later time. There are commercially produced anchor bolt sleeves or a fabrication detail can be added to the drawings.

Figure 1.5.11.1.2-1 Anchor Rod Detail



1.5.11.2 Post-Installed Anchors

1.5.11.2.1 Materials

RESIN BONDED ANCHORS

Normally specify *ASTM F1554*, as the anchor rod material. ASTM specifications may be substituted as shown in *Table 1.5.11.2.1-1.*

Table 1.5.11.2.1-1 ASTM Material Substitutions

Anchor Rod Specifications	ASTM Specifications
ASTM F1554 GR 36	A307
ASTM F1554 GR 105	A193 (grade B7) or A449
M31 Rebar, grade 60	A706 or A615

Galvanizing is only required when portions of the anchor are exposed.

Anchor rods do not necessarily need to be fully threaded. Specify the thread length to best fit the particular application.

BONDING MATERIAL

Use a resin bonding system from the Division's QPL for anchor bolts 1 inch diameter or less. For larger anchors, use other types of anchorage such as epoxy grout or cementitious grouts with traditional development lengths.

MECHANICAL ANCHORS

A mechanical anchor system consists of multiple materials and differs from one manufacturer to another. Corrosion-resistant materials are required for mechanical anchors installed in bridge elements. As a minimum, specify hot-dip galvanized coating. Type 316 stainless steel is preferred. All mechanical anchor products on the QPL have an option for providing corrosion-resistant materials. Use the same type of materials for attachments connected to mechanical anchors. When this is unavoidable, provide electrical isolation for all dissimilar metals to avoid galvanic corrosion.

1.5.11.2.2 Design

Ensure that post-installed anchors are embedded in good concrete without active cracks. Avoid using anchors in sections of the bridge with high tensile stresses perpendicular to anchor holes. Drilled holes in concrete attract or even induce cracks at the hole location. Cracks in the concrete will then tend to break down the bond between concrete and epoxy resin for resin bonded anchors or compromise the mechanical anchor system. Do not use post-installed anchors to resist earthquake forces within plastic hinge regions of bridge elements (i.e. columns). Use of post-installed anchors for shear lugs and beam seat extensions is acceptable.

RESIN BONDED ANCHORS

Do not specify anchors larger than 1 inch in diameter using a resin bonded anchor system.

Table 1.5.11.2.2-1 Anchor Diameters and Stress Areas

Diameter (inch)	Stress Area (in^2)	Bar Size	Stress Area (in^2)
0.5	0.142	4	0.20
0.625	0.226	5	0.31
0.75	0.334	6	0.44
0.875	0.462	7	0.60
1.00	0.606	8	0.79

FHWA Technical Advisory T5140.34 regarding use of adhesive anchor under sustained tension loads was issued in January 2018. According to the Technical Advisory, FHWA recommends that post-installed adhesive anchors can be used for resisting sustained tension loads only when

specific requirements are met. The recommendations were based on *NCHRP Reports 639* and *757, ACI 318, and ACI 355.4*. Anchoring to concrete design guidance was added to *Section 5 of the AASHTO LRFD design specifications*, which refers to the *ACI design code*. The new requirements apply to all new Federal-aid projects.

Design resin bonded anchors with loads, load factors, and load combinations specified in *LRFD Section 3* and with resistance factors according to *ACI 318 Chapter 17* with the following modifications:

- Uncracked concrete is assumed for design of resin bonded anchor system.
- Use a resin bonded anchor system in concrete with a compressive strength of 2,500 – 8,000 psi.
- Use resistance factors as shown in *Table 1.5.11.2.2-2* for applicable failure modes.

Table 1.5.11.2.2-2 Resistance Factors for Design of Resin Bonded Anchors

Failure Mode	**Strength Reduction Factor ϕ**		
	Strength Limit State		**Extreme Event II Limit State**
	Others	**Sustained Tension**	
Reinforcement in tension	0.75	0.75	1.00
Concrete breakout in tension	0.65	0.65	0.90
Adhesive bond in tension	0.65	0.33	0.90
Reinforcement in shear	0.65	0.65	1.00
Concrete breakout in shear	0.70	0.70	0.90
Concrete pryout in shear	0.70	0.70	0.90

- For concrete breakout strength in tension, $kc = 24$. Use $\psi c, N = 1.0$.
- Use characteristic bond stresses from *Table 1.5.11.2.2-3* for bond strength calculation. The bond stresses shown in the table are obtained from the Evaluation Service Report (ESR) of epoxy resin products on the QPL. The reports are approved by the *International Code Council Evaluation Service, Inc. (ICC-ES)*. The recommended bond stresses are based on threaded rod or reinforcing bars installed in holes drilled with a hammer drill and carbide bit.

Table 1.5.11.2.2-3 Characteristic Bond Stress for Bond Strength in Tension Check

Short-Term Peak Temperature (24 hrs.)	Load Application	$\tau uncr$ (psi)
Equal or Less than 130 F	Others	1200
	Sustained Tension	480
Greater than 130 F but less than 176 F	Others	800
	Sustained Tension	320

Before 2019, ODOT used a set of equations developed in-house using historical test data for calculating anchor capacities. The equations are located in *Appendix A1.5.11.2.2*.

MECHANICAL ANCHORS

Each manufacturer establishes its own material strength and it differs for different anchor sizes. Do not specify anchor sizes larger than ¾ inch (nominal) in diameter. Use the nominal material strengths for anchor design found in *Table 1.5.11.2.2-4*.

Table 1.5.11.2.2-4 Material Properties for Design of Mechanical Anchors

Undercut Anchor		
Material (psi)	**Stainless Steel**	**Hot-Dip Galvanized**
Yield strength	87,000	Not on the QPL
Tensile strength	105,000	Not on the QPL
Expansion and Screw Anchors		
Material (psi)	**Stainless Steel**	**Hot-Dip Galvanized**
Yield strength	75,000	55,000
Tensile strength	90,000	75,000

Design mechanical anchors with loads, load factors, and load combinations specified in *LRFD Section 3* and with resistance factors according to *ACI 318 Chapter 17* with the following modifications:

- Assume uncracked concrete for design of mechanical anchor system.
- Use a mechanical anchor system in concrete with a compressive strength of 2,500 – 8,000 psi.
- Use resistance factors as shown in *Table 1.5.11.2.2-5* for applicable failure modes.

Table 1.5.11.2.2-5 Resistance Factors for Design of Mechanical Anchors

Failure Mode	Strength Reduction Factor ϕ	
	Strength Limit State	**Extreme Event II Limit State**
Reinforcement in tension	0.75	1.00
Concrete breakout in tension	0.65	0.90
Pullout strength in tension	0.65	0.90
Reinforcement in shear	0.65	1.00
Concrete breakout in shear	0.70	0.90
Concrete pryout in shear	0.70	0.90

- For concrete breakout strength in tension, $kc = 24$. Use $\psi c, N = 1.0$.

Mechanical anchors from each manufacturer have different details and specifications. The recommended design parameters listed in *Table 1.5.11.2.2-6* ensure that an anchor product on the QPL can meet the design requirements.

Table 1.5.11.2.2-6 Design Parameters for Mechanical Anchor Design

Undercut Anchor					
Nominal Diameter (inch)	0.375	0.500	0.625	0.750	
Anchor O.D., d_a (inch)	0.625	0.750	1.000	1.125	
Effective embedment depth, h_{ef} (inch)	3.75	4.75	7.25	9.75	
Effective cross-sectional area, A_{se} (in^2)	0.078	0.131	0.226	0.334	
Pullout strength, $N_{p,\,uncr}$ (lbs) Use $\psi c, P = 1.0$.	12600	16000	21000	31000	
Hole depth (inch)	4.75	5.75	8.25	10.75	
Minimum member thickness (inch)	7.25	8.0	10.75	14.0	
Expansion Anchor					
Nominal Diameter (inch)	0.250	0.375	0.500	0.625	0.750
Anchor O.D., d_a (inch)	0.250	0.375	0.500	0.625	0.750
Effective embedment depth, h_{ef} (inch)	1.5	2.0	3.25	4.0	5.0
Effective cross-sectional area, A_{se} (in^2)	0.020	0.051	0.101	0.162	0.237
Pullout strength, $N_{p,\,uncr}$ (lbs) Use $\psi c, P = 1.0$	1600	3200	5400	7100	11600
Hole depth (inch)	2.75	3.25	4.50	5.25	6.25
Minimum member thickness (inch)	4.0	4.25	5.5	6.25	7.5
Screw Anchor					
Nominal Diameter (inch)	0.250	0.375	0.500	0.625	0.750
Anchor O.D., d_a (inch)	0.250	0.375	0.500	0.625	0.750
Effective embedment depth, h_{ef} (inch)	1.5	2.0	2.5	3.5	4.0
Effective cross-sectional area, A_{se} (in^2)	0.024	0.099	0.183	0.276	0.414
Pullout strength, $N_{p,\,uncr}$ (lbs) Use $\psi c, P = 1.0$	900	3000	3600	4800	9500
Hole depth (inch)	2.75	3.25	4.0	5.25	6.0
Minimum member thickness (inch)	4.0	4.25	5.0	6.25	7.0

Undercut anchors are good alternative to resin bonded anchors for overhead situation with sustained tension loading.

Overall design calculations for mechanical anchors are similar to resin bonded anchors without bond strength check. Adequate member thickness is important for mechanical anchors to avoid splitting failures. When the member thickness is limited and there is not much space for a longer anchor than the design, add a note on the plan.

For mechanical anchors, the difference between effective embedment depth of anchors (h_{ef}) and total drilled hole depth can vary from ½ to 2 inches depending on each manufacturer. Use the

hole depths shown in *Table 1.5.11.2.2-6* as a guide to ensure that anchors will fit inside the member and around existing rebar during the design.

When any of the above design requirements cannot be met, contact the BDM technical resource for guidance.

1.5.11.2.3 Drilling Holes in Concrete

When existing reinforcing steel is required by design, require bars to be located prior to drilling.

Drill holes according to manufacturer's recommendations. Spalling of adjacent concrete is the main concern when determining the hole location and type of drill to be used. When recommendations to prevent spalling do not exist, use the following drilling methods:

RESIN BONDED ANCHORS

When center of hole is 6 inches or less from the edge of concrete use either a diamond bit core drill or a carbide bit rotary hammer with four cutting edges on the diameter.

When center of hole is more than 6 inches from the edge of concrete use either an air hammer, maximum 9 pound class, or a carbide bit rotary hammer with two cutting edges on the diameter.

MECHANICAL ANCHORS

Use either a diamond bit core drill or a carbide bit rotary hammer with four cutting edges on the diameter.

GROUTED ANCHORS

Any type of drill will normally be acceptable. Grouted anchors should always be placed more than 6 inches from the nearest concrete edge.

1.5.11.2.4 Plan Details

Post-installed anchors are considered critical when failure of the anchors can compromise public safety. Anchors installed in the following members are considered critical.

- Items attached from tunnel ceiling or under bridge deck.
- Bridge rail anchor.
- Shear lug.
- Beam seat extension.
- Sign structure support.
- Structural connection between existing and new concrete or different materials.
- Fencing support.

List all anchors specified for a construction project in *SP 00535.45(c)*. Indicate which tests are required for the specified anchors, especially for critical anchors. For anchors that are not critical and will not see significant loads, tests during construction may be omitted.

RESIN BONDED ANCHORS

For horizontal applications, show drilled holes angled down a minimum of 15 degrees on plan sheets. For thin members, such as bridge decks, a smaller angle of drilled holes may be specified to avoid protrusion of the anchors. Specify a minimum drilled angle of 5 degrees. When any down angle of drilled holes will not work with the design, specify horizontal drilled holes. *SP 00535* will require a certified anchor installer to ensure good quality of installed resin bonded anchors.

When critical resin bonded anchors are used, include the following note on the plans:

> Provide and install (___" diameter F1554 grade (36) (55) (105)) or (#__ AASHTO M 31, grade 60 rebar) resin bonded anchors with epoxy resin from the QPL. The characteristic bond stress used in the design is ____ psi. The minimum pullout strength is ___ lbs with a minimum embedment (h_{ef}) of _____ in. Install anchors according to the manufacturer's recommendations.

The minimum pullout strength is the smaller unfactored strength of steel reinforcement and adhesive bond in tension obtained from calculation for a single anchor including applicable eccentricity, edge effect, and splitting factors. For anchors subjected to sustained tension, the characteristic bond stress is reduced by a factor of 0.40 in design as shown in *Table 1.5.11.2.2-3*, however use the characteristic bond stress of 1,200 psi in calculation for the minimum pullout strength and show it on the plans for field testing purposes.

MECHANICAL ANCHORS

When critical mechanical anchors are specified, include the following note on the plans:

> Provide and install ___" nominal diameter Type (A Undercut) (B Expansion) (C Screw) mechanical anchors using a product from the QPL. The minimum pullout strength is lbs with a minimum effective embedment depth (h_{ef}) of _____ in. (The maximum depth of the drilled hole is _____ in.) For the design, the steel anchor yield strength is _____ psi and the tensile strength is _____ psi. Provide anchor materials with (Type 316 stainless steel) (hot-dip galvanized coating). Install anchors according to the manufacturer's recommendations.

The minimum pullout strength is the controlling factored strength for a single anchor including applicable eccentricity, edge effect, and splitting factors in tension using resistance factors for Extreme Event II Limit State.

1.5.11.2.5 Testing Requirements

Two types of field tests are required during construction to ensure proper installation and to achieve as-specified capacity of post-installed anchors. Demonstration Test includes installation of anchors using the same material and methods as shown on the plan and testing the anchors to a load at minimum pull out strength. Production Test is performed during construction on actual anchors used in final position up to a load level specified in the Special Provisions.

1.5.11.2.6 Construction

DRILL TYPES

See *BDM 1.5.11.2.3* or *SP 00535* for the drill type to be used.

HOLES

Holes for resin bonded anchors are normally 1/8 inch diameter larger than the nominal bolt diameter. Holes should be cleaned with compressed air, a non-metallic brush and water. Concrete dust is one of the most destructive elements to a resin bonded system and water is the best method to remove the dust. Holes for mechanical anchors are dependent on the type and manufacturer. Holes for grouted anchors are normally 1/4 inch diameter larger than the anchor diameter.

TEMPERATURE

Epoxy resin is not allowed for low temperature applications. The set times become quite long at low temperatures. It will normally be better to use a deeper embedment with a non-epoxy product at low temperatures.

TIGHTENING

SP 00535 requires tightening to only 1/6 turn past snug tight. Consider what tightening is appropriate for the application and show on the plans, when different than the specifications. Check when distribution plates are needed to transfer the bearing loads (from the tensioned bolt) to the concrete.

1.5.11.3 Cast-In Anchors

1.5.11.3.1 Generic Anchor Types

Design cast-in anchors, i.e. hex head bolt with washer, L-bolt, J-bolt, and welded headed stud, according to *Section 5 of the AASHTO LRFD design specifications*, which refers to the *ACI design code*. Use resistance factors as shown in *Table 1.5.11.3.1-1* for applicable failure modes.

Table 1.5.11.3.1-1 Resistance Factors for Design of Cast-in Anchors

Failure Mode	Strength Reduction Factor ϕ	
	Strength Limit State	Extreme Event II Limit State
Reinforcement in tension	0.75	1.00
Concrete breakout in tension	0.70	0.90
Reinforcement in shear	0.65	1.00
Concrete breakout in shear	0.70	0.90
Concrete pryout in shear	0.70	0.90

1.5.11.3.2 Coil Inserts

The smallest coil inserts commercially available are for ¾ inch diameter rods or bolts. Specify inserts for rods or bolts with a minimum size of ¾ inch diameter. Detail inserts to have a ½ inch setback under the concrete surface. Use design parameters from *Table 1.5.11.3.2-1* for strength calculation and details for coil inserts.

Table 1.5.11.3.2-1 Design Requirements for Single Concrete Inserts

Bolt or Rod Diameter (inches)	Insert Length (inches)	Min. Member Thickness (inches)	Nominal Capacity in Tension or Shear (lbs)	Strength Reduction Factor		Min. Edge Distance (inches)	Min. Spacing to Adjacent Inserts (inches)*
				Strength	Extreme Event		
¾	4 5/8	8	16900	0.70	0.90	12	8
1	5 5/8	8 ¾	24300	0.70	0.90	16	10
1 ¼	7 5/8	10 ¾	38900	0.70	0.90	24	12
1 ½	9 5/8	12 ¾	55500	0.70	0.90	26	16
*To avoid group effect.							

The minimum member thickness accounts for the insert length, setback, and concrete clear cover. The nominal capacity assumes concrete breakout strength controls. Consider locating concrete inserts at a distance from the edge of deck according to the minimum edge distance shown in the table, otherwise an additional strength reduction factor will apply. When the inserts are not to be used immediately, install short galvanized bolts in the inserts to prevent rusting of the threads.

1.6 STEEL STRUCTURE DESIGN AND DETAILING

1.6.1 Structural Steel, General

This chapter covers primarily the design and construction of steel plate and box girder bridge superstructures. It provides guidance for bridge designers working on ODOT projects to achieve optimal quality and value in steel bridges.

1.6.1.1 Structural Steel, Design Considerations

Take into consideration, fabrication requirements, construction techniques and future maintenance needs in the design, including the following items:

- Use the most recent version of the design specifications.
- Become familiar with construction and fabrication specifications and standard drawings applicable to steel structures.
- Evaluate how construction and fabrication specifications influence the design and what modifications or special provisions may be required.
- During the initial evaluation of design options, consider consulting with the Steel Bridge Standards Engineer, fabricators, steel erectors or contractors for ideas on achieving an economical, easy to build and robust design.
- Ensure that all individual bridge components fit well together by accounting for how rotation, deflection (especially differential deflection), twist, stiffness (vertical bending, lateral bending, and torsion), and skew affect interaction between different elements.

C1.6.1.1

Familiarity with design and construction specifications are key to steel bridge design. Designs that merely satisfy the design specifications are often problematic.

- Provide clear and distinct load paths that mitigate or preferably, eliminate out-of-plane bending.
- Provide adequate access for bolting, welding, and painting.
- Keep design simple by maximizing the use of common details and minimizing the number of plate sizes and rolled shapes fabricators are required to purchase. Complicated details are always difficult to fabricate and build.
- Do not use details that permit water and debris to collect on girders.

1.6.1.2 Codes and Standards

Design according to *AASHTO LRFD Bridge Design Specifications* unless specified otherwise in this document. See *NSBA Guide to Navigating Routine Steel Bridge Design* for a guide to designing routine steel I-girder bridges.

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

- *G13.1-2019, Guidelines for Steel Girder Bridge Analysis.*
- *G12.1-2020, Guidelines to Design for Constructability and Fabrication.*
- *G1.4-2006, Guidelines for Design Details.*
- *S10.1-2019, Steel Bridge Erection Guide Specification.*
- *G9.1-2022, Steel Bridge Bearing Guidelines.*
- *G1.2-2003, Design Drawing Presentation Guidelines.*
- *S2.1-2018, Steel Bridge Fabrication Guide Specification.*
- *G1.3-2002, Shop Detail Drawing Presentation Guidelines.*
- *G1.1-2020, Shop Drawing Approval Review/Approval Guidelines.*
- *G4.1-2019, Steel Bridge Fabrication QC QA Guidelines.*
- *G2.2-2016, Guidelines for Resolution of Steel Bridge Fabrication Errors.*
- *G4.2-2021, Guidelines for the Qualification of Structural Bolting Inspectors.*
- *G4.4-2006, Sample Owners Quality Assurance Manual.*
- *S8.1-2014, Guide Spec. for Application of Coating Systems w/ Zinc-Rich Primers to Steel Bridges.*
- *S8.2-2017, Specification for Application of Thermal Spray Coatings for Steel Bridges.*

The Steel Bridge Design Handbook publication includes 19 chapters of steel bridge design topics with six design examples to be used as design aids when deciding the design, fabrication and construction of steel bridges.

1.6.1.3 ODOT Steel Bridge Practice

ODOT does not require Certified Erector Qualification for erection of steel bridges. For complex projects in which a contractor with such qualification is deemed necessary, obtain Bridge Engineering Section approval prior to including such requirement in the contract documents.

Specify caulking according to *Standard Specifications 00594.43(f)* to fill and seal crevices and gaps between structural shapes and plates, around bolt heads and nuts, weld terminations and similar areas that would retain moisture.

Refer to *AASHTO/NSBA G1.4-2006, Guidelines for Design Details* for design details that allow for the economical fabrication and erection.

FATIGUE DESIGN REQUIREMENTS

Design all welded and bolted connections for infinite fatigue design life using ADTT from *LRFD Table 6.6.1.2.3-2*. Do not use details category E or E′ in any connections for primary load-carrying members.

C1.6.1.3

C FATIGUE DESIGN REQUIREMENTS

The fatigue live load specified in *AASHTO LRFD Article 3.6.1.4* should be used for checking girder details in accordance with *Article 6.6*. A single fatigue truck, without lane loading or variable axle spacing, is placed for maximum and minimum effect to a detail under investigation. The impact is 15 percent, regardless of span length. As specified in *Article 3.6.1.1.2*, multiple presence factors are also not to be applied to the fatigue limit state check for which one design truck is used. The load factor is 1.75. It is generally possible to meet the constant amplitude fatigue limit (CAFL) requirement for details with good fatigue performance. Limiting the calculated fatigue range to the CAFL ensures infinite fatigue life.

Use G13.1-2019, Guidelines for Steel Girder Bridge Analysis *Section 4 and Appendix B* on selecting the appropriate level of analysis.

Curved and skewed deck girder bridges have the potential for three-dimensional deflection and rotation. Longer spans magnify the rotation of the girders and cause unaccounted stresses on the diaphragm connections.

1.6.1.4 Estimating Structural Steel Weights

For preliminary quantities or preliminary girder design, obtain an estimate of steel weights for built-up plate composite I-girders from the Span to Weight Curves from the AISC website.

For a more detailed estimate of continuous span bridges, refer to the Continuous Span Standards on the AISC website.

For a more detailed estimate of short span bridges (40 to 140 feet), refer to Short Span Steel Bridge Alliance website.

C1.6.1.4

The *Steel Span to Weight Curves* are the quickest way to determine the weight of steel per square foot of bridge deck for straight, low skew, plate girder bridges. The curves are organized by span arrangement (1, 2 or 3 or more span bridges) and girder spacing. The curves also provide a double check for final quantities.

The *Continuous Span Standards* include 88 unique solutions for three span bridges with center spans between 150 and 300 feet, girder spacing between 7 feet-6 inches and 12 feet, and plate girder designs utilizing both homogenous and hybrid steel options. Each conceptual solution presented in these tables is organized based on the following:

- Girder plate sizes.
- Diaphragm spacing.
- Intermediate stiffener sizes and locations.
- Shear connector spacing.
- Camber.
- Girder weights.

BID ITEMS

Use the following bid items for structural steel. Use horizontally curved steel (plate or box) girder bid item when the radius of horizontal curve on the structure is less than 1,000 feet.

- Steel Rolled Beam.
- Steel Plate Girder.
- Steel Box Girder.
- Steel Plate Girder with haunch.
- Trapezoidal Steel Box Girder with haunch.
- Horizontally Curved Steel Plate Girder.
- Horizontally Curved Steel Box Girder.
- Specialty Bridges (tied arches, Cable Stayed).
- Structural Steel Maintenance.

For seismic retrofit project, break down the retrofit item under Structural Steel Maintenance:

- Structural Steel Maintenance Cross Frame Strengthening.
- Structural Steel Maintenance Bumper Blocks.
- Structural Steel Maintenance Restrainers.
- Structural Steel Maintenance Shear Key.

1.6.2 Structural Steel, Design

1.6.2.1 Steel Grade Selection

C1.6.2.1

Identify all steel by grade on the contract plans.

Provide structural steel for bridges conforming to *ASTM A709* (*AASHTO M270*).

These specifications include grades 36, 50, 50W, HPS 50W and HPS 70W. *ASTM A709* steel specifications are written exclusively for bridges wherein supplementary requirements for Charpy V-Notch Impact tests are mandatory.

ASTM A36, A572 or *A588* may be used for structural steel for bridges provided the supplementary Charpy V-Notch Impact test requirements are included in the *Special Provisions*. When Charpy V-Notch Impact tests are required for *ASTM A36, ASTM A572* or *ASTM A588* structural steel, use the supplementary requirements of *ASTM A709*.

Specify *ASTM A709* Grade 50 steel for all structures that require yield strengths between 36 ksi and 50 ksi and are to be painted or galvanized.

Do not use *A709* (Grades 36, 50, 50W) steels for plates thicker than 3-inches, or butt welds in tension members over 3 inches.

Specify Grade HPS (high-performance steel) 50W and HPS 70W to be **Quenched and Tempered** in the contract document. For thermo-mechanical control processed steel, require the contractor to provide test samples at both ends of each rolled plate. Plates that pass the required test are acceptable for fabrication.

Grade HPS 70W steel has recently been developed and provides high strength, enhanced durability and improved weldability. Depending on the availability, grade HPS 70W may be economical only in hybrid girders. With grade 50W webs, use a hybrid configuration with HPS 70W tension and compression flanges in high moment regions. See Steel Bridge Design Handbook Chapter 1 Article 2.1.6 Grades HPS 50W, HPS 70W and HPS 100W for more background.

Limit plate thickness for HPS 50W and HPS 70W to 2 inches. Consult with the Steel Bridge Standards Engineer for specific project needs.

Provide structural steel for steel piling, metal sign structures and other incidental structures conforming to *ASTM A36, ASTM A572 or ASTM A588*. Incidental structures include luminaire and traffic signal supports, bridge metal rails and metal rail posts, guardrail connections, earthquake restraints, bridge deck expansion joints and fencing post connections, etc.

Use merchant quality steel (non-spec) in items such as catch basin frame, catch basin, deck drain grate, manhole rungs and steps, access hole cover, guardrail spacer blocks, shims, anchor bolt plate embedded in concrete, etc. and where a high degree of internal soundness, chemical uniformity or freedom of surface defects are not required. Acceptance of such items is based on visual inspection.

1.6.2.2 Weathering Steel

Avoid requiring weathering steel in painted applications.

To achieve the performance expected of uncoated weathering steel (UWS), prevent prolonged exposure to wetness or high levels of chlorides, without the opportunity to dry, by avoiding use in the environments stated below.

C1.6.2.2

Weathering steel performs as well but not better than non-weathering steel in painted applications.

Review the following references for appropriate application of weathering steel:

- NSBA Uncoated Weathering Steel Reference Guide.
- Steel Bridge Design Handbook: Chapter 19 – Corrosion Protection of Steel Bridges.
- Computer Modeling and Analysis of Truck Generated Salt Spray Transport Near Bridges.

Similar to regular construction steel, weathering steels also rust under a wide range of exposure conditions. But during this process it forms oxides that remain tightly adherent to the steel substrate and

develops a much more stable oxide layer than non-weathering steel. Shortly after blast cleaning to remove mill scale, weathering steel turns rusty in appearance. Through several cycles of wetting and drying (usually between 6 and 24 months, depending on the environment), the surface of the steel develops a tight oxide coating (patina) that provides its own corrosion resistant surface finish, eliminating the need for painting and resulting in minimal future maintenance and lower life cycle costs.

Do not use weathering steel in the environments and locations listed below:

Design deviations will be approved for environments and locations of concern when there is a reasonable case for weathering steel at the specific site. The intent is not to discourage reasonable use of weathering steel, but to provide an orderly, documented process for the judgment necessary to properly apply weathering steel in environments and locations of concern. See NSBA Uncoated Weathering Steel Reference Guide Article 2.2 and 2.3 for detailed consideration related to the macro- and micro- environment.

ENVIRONMENT

- When the bridge site is within two aerial miles of the coastline.
- At coastal areas (*BDM 1.2.4.2*) with eight or more months of the year experiencing average relative humidity between 76 to 80 percent or higher.
- Industrial areas where concentrated chemical fumes may drift onto the structure.

Average relative humidity by month can be obtained through *NOAA*:

- Archived data on from stations in Oregon in NOAA's Annual Local Climatological Data publications.
- Morning and afternoon averages by a station's period of record from NOAA's Comparative Climatic Data.

LOCATION

- Grade separations in tunnel like conditions.
- Low level water crossings.
    - Ten feet or less over stagnant, sheltered water.
    - Eight feet or less over moving water.
- Conditions that do not allow for the drying of the steel necessary to develop a good patina.

With the proper details, good performance from weathering steel can be achieved by reduce or eliminate unsightly concrete staining. Comply with the following design detailing these requirements:

- Provide adequate drainage beneath overpass structures to prevent ponding and continual traffic spray from below. Communicate the importance of adequate drainage to roadway designers.
- Do not detail deck drains that can discharge water onto the steel, especially in regions that use de-icing chemicals.
- Avoid any type of open joint that allows runoff to reach the steel.
- Provide details that take advantage of natural drainage.
- Provide drip plates (also called drip tabs) to divert runoff water and protect abutments and columns from staining.

Provide flange thickness transitions instead of flange width transitions for bottom flanges (with the thickness added to the underside of the flange).

- Eliminate details that retain water, dirt, and other debris. Provide stiffener clips for proper ventilation and drainage

Use haunched girders with generous allowances for drainage through stiffener copes, snipes, and drain holes since the use of haunched girders complicates the efficient removal of run-off and can result in the collection of debris at the haunch.

1.6.2.3 Simple for Dead Load Continuous for Live Load

Consider a simple steel bridge system (SDCL) for dead load and continuous for live load in the design of multi-span structures when simple spans or full continuous spans do not provide efficient span configuration and design.

Critical to the functionality of SDCL structures is the continuity connection at the interior bents. Consider use of concrete diaphragms over the bridge piers. See *DET3620*.

For designs intending to eliminate the deck joint at interior bents while allowing simple-span bridge behavior to be retained, consider using ultra-high performance concrete (UHPC) link slabs as a continuity connection. Do not use link slab for bridges with SDC C and D due to unknown performance during a large earthquake.

C1.6.2.3

Span configuration plays an important role in using steel efficiently. Two span continuous girders for dead and live loads are not always efficient because of high negative moments. Three span units with interior spans about 20 to 30 percent longer than end spans are preferable, but not always possible.

Relative to full continuous, SDCL bridges reduce uplift in unbalanced spans, reduce negative moments at the bents, simplify fabrication, and eliminate the need for bolted field splices.

Concrete diaphragms are a feasible connection detail for providing live load continuity over the pier in a SDCL steel girder system when conventional construction methods are employed.

Link slabs are proven to be an economical detail for eliminating deck joints on bridges. Elimination of the deck joint increases a bridge's service life, reduces the need for frequent maintenance, and provides a smooth riding surface. See *BDM 1.9.2.3.1* for more information on UHPC.

Research in steel industry practices is continuously improving. Designers are encouraged to discuss appropriate connection details with the Steel Bridge Standards Engineer.

1.6.2.4 Uplift

Avoid span layouts with uplift at end bents for DL case.

Always consider the presence of uplift at the ends of continuous girders, particularly with light, rolled beam units or short end spans. Uplift restraint, when needed, must satisfy the Strength limit state and the Fatigue limit state.

C1.6.2.4

When the end spans are much shorter than the interior spans, uplift at the girder ends can occur and create design and construction problems. When end spans are too long in relation to interior spans, a disproportionate amount of steel will be required for the end spans.

Commentary to *AASHTO LRFD Bridge Design Specifications, Article C3.4.1*, indicates that uplift needs be checked under a strength load combination. It also provides guidance in the appropriate use of minimum and maximum load factors.

1.6.2.5 Girder Spacing

Use wider girder spacing to reduce the number of lines of girders, which will reduce shop and field labor. Consider the following to optimize girder spacing:

- ODOT prefers a minimum of four I-shaped beams/girder span for vehicular bridges.
- Stability and redundancy of the structure during future re-decking.
- Thicker concrete deck results in more concrete and reinforcing steel, and possibly in more superstructure weight.
- Wide girder spacing can create challenges for deck formwork, and slabs (or floor systems) cannot adequately support certain overloads.
- On straight bridges, interior and exterior girders must be detailed identically. Spacing must be such that the distribution of wheel loads to the exterior girder is close to that of the interior girder.

C1.6.2.5

Many studies show that the weight of structural steel per square foot of deck area decreases as girder spacing increases. Refer to *G12.1-2020, Guidelines to Design for Constructability Article 1.2* for more information. Generally, for a bridge with an average span length less than 175 feet, there is not an appreciable difference in the structural steel unit weight for the various girder spacing summarized in the graph in *G12.1 Article 1.2*. For a bridge with an average span length more than 175 feet, the designer may want to consider a wider girder spacing, perhaps between 11 and 13 feet, as this wider girder spacing trends to a lighter steel superstructure.

With wider girder spacing and *AASHTO* simplified live load distribution factor, the fatigue limit-state check at the cross-frame connection-plate weld to the bottom flange could control the design of the bottom flange. It is prudent for the designer to consider moving the cross-frame away from high flexural location or use refined analysis

to determine the stress at the cross-frame connection-plate weld to the bottom flange to achieve economical girder design, rather than increase the bottom flange thickness for the entire girder segment.

Bolted tab plate detail is not recommended (Refer to *G12.1-2020, Guidelines to Design for Constructability Article 2.1.2.2*).

1.6.2.6 Shop Lengths of Welded Girders

Optimize the girder segments by reducing the number of field splices.

All field splices must be bolted. Locate field splices in welded steel girders by not exceeding the following shop lengths and mass:

- 150 feet when bridge site is readily accessible.
- 130 feet when bridge site is not readily accessible.
- Fabricators are limited to their shop crane sizes. Contact fabricators in the state of Oregon for project specific needs and requirements.

For curved girders, limit the girder sweep plus the flange width to 6 feet for ease of shipping. The current legal vehicle width is 8 feet 6 inches without a permit. Limiting the overall shipping width of curved girders to 6 feet permits fabricators to offset the girder on the trailer, as is frequently done, while not exceeding an overall width of 8 feet 6 inches. Add optional field splices if required.

C1.6.2.6

Maximum girder or girder field segment lengths without a field splice ranges from 130 feet to 150 feet depending upon cross section. There may be locations where girder lengths will be controlled by weight or access to the bridge site. Long and deep girders may also require auxiliary lateral support during transportation.

It is typical to show an optional bolted field splice to allow the fabricator and contractor some flexibility in fabrication and transportation.

Longer girders have been fabricated and hauled to project sites, however contact fabricators and the Steel Bridge Standards Engineer when project requires girder segments longer than 150 feet.

1.6.2.7 Rolled Beam Sections

C1.6.2.7

Rolled beams can be more economical than plate girders for their applicable span lengths (up to 80 feet) because of decreased fabrication costs. Do not use rolled beam sections smaller than W18.

Select beams that have a top flange that is sufficiently wide to provide adequate spacing for three stud connectors per row.

Beams must be large enough that the elastic neutral axis of the composite section is within the steel beam (not within the slab or haunch).

Rolled beams are not exempt from camber requirements of *BDM 1.6.2.13*.

Verify the need for bearing stiffeners for rolled beams by using the provisions in *LRFD Appendix D6.5*.

Rolled beams usually do not need bearing stiffeners.

The diaphragms between beams usually consist of rolled shapes with channels being the most common choice.

1.6.2.8 Plate Girders

1.6.2.8.1 Flange Width and Thickness

C1.6.2.8.1

The minimum size flange is ¾ inch thick by 12 inch wide.

The minimum ¾ inch flange thickness is to minimize the distortion of the flange due to welding of the flange to the web.

It may not be prudent to minimize the top flange. Flange width affects girder stability during handling, erection, and deck placement. Keep the girder length (length of the unspliced individual girder field section) to flange width ratio below 85.

The girder needs significant lateral load capacity to resist lateral transportation loads and lateral loads from deck overhang brackets and deck placements. Another side benefit of providing generous top flange is that the non-composite deflections are reduced. See *LRFD Appendix C6.10.2.2* for commentary.

Minimize the number of changes in flange size, as the cost of a butt weld will offset a considerable length of excessive flange area. When locating flange thickness transitions (shop flange splices), include no more than two butt splices or three different flange thicknesses for an individual flange between field splices, except for unusual cases such as very long or heavy girders or mill length availability limits.

Limit the maximum change between adjacent plates to 6 inches in width, at both welded and bolted connection section changes.

The rule of thumb is to limit flange transitions such that the smaller flange at a welded transition is no less than 50 percent of the area of the larger flange.

Make top and bottom flanges a constant width where possible. When a change in bottom flange width is needed, make it at a bolted splice location.

Limit the maximum flange thickness to 3 inches. At welded flange splices, the thinner plate must not be less than one-half the thickness of the thicker plate.

Constant width flanges enable the fabricator to order the flanges in multiple width plates which are more economical than universal ill plates. The shop flange splices can be made while the plates are in wide slabs and cut to widths simultaneously with multiple cutting torches. Also, the recommended practice for general constructability is to favor flange thickness transitions instead of flange width transitions.

Efficiently locating thickness transitions in plate girder flanges is a matter of plate length availability and the economics of welding and inspecting a splice compared to the cost of extending a thicker plate. Refer to NSBA G12.1 Guidelines to Design for Constructability Article 1.5.1 for optimizing flange thickness transitions.

The rule of thumb accomplishes two things. First, the bending stress gradient in the girder web due to the change in section properties does not become overly steep when this criterion is met. Second, it has been demonstrated in past designs that, when the flange transition results in greater than a 50 percent reduction in flange area, either the transition is not in the optimum location or an additional transition may prove to be economical.

Minimizing the number of changes in the top flange will also facilitate easier deck forming.

Generally, use a minimum flange width that is equal to the width of the flange resisting the maximum positive moment. Widen the flange as necessary in negative moment areas so the flange thickness will not exceed 3 inches at the bent.

1.6.2.8.2 Web Depth and Thickness

Girder depths, particularly for haunched girders, may be limited because of transportation constraints.

Use web depths in whole-inch increments. Minimum web thickness is ½ inch. Thinner plates are subject to excessive distortion from welding.

Use web plate of sufficient thickness to eliminate the need for transverse stiffeners either entirely or partially. In high shear regions, when transverse stiffeners are spaced at 8 to 10 feet prevent the need for a thicker web, the use of a stiffened web can be justified.

C1.6.2.8.2

Commonly used web plates range from 48 to 96 inches deep. The recommendations in *LRFD 2.5.2.6.3* provide a good estimate of a minimum web depth for straight girders. If vertical clearance is not a problem, adding depth can result in lighter girders. For curved girder web depth, use either the *AASHTO* recommended minimum depth for straight girders, increased by 10 to 20 percent, or use *LRFD Equation 2.5.2.6.31* as a starting point. A rule of thumb for a well-proportioned superstructure is to have total section depth (slab plus girder) in the range of 0.033L to 0.04L (L = c–c bearing length). For web depths up to 96 inches, provide sufficient thickness to preclude the need for longitudinal stiffeners.

The labor to place and weld one foot of stiffener is equal to about 25 pounds of steel. Unstiffened webs reduce fabrication, painting costs (for non-weathering steel) and flange sizes. Thicker webs are also helpful in reducing web distortion due to welding and in supporting deck overhang brackets for the deck placement.

Design web plates in 1/16 inch increments with a note that the contractor may increase the web thickness shown by 1/16 inch at no additional cost to the State. Minimize web transitions as the cost of a butt weld web splice often exceeds the cost of the added material between sections.

The cost of a square butt joint web splice is equal to about 800 pounds of steel per foot of splice. When web plates are over 80 feet long and constant thickness, provide the fabricator an optional shop splice on the design plans. The most economical bid can then be prepared according to the mill length extras, market areas available, and transportation and handling costs.

1.6.2.8.3 Girder Splices

C1.6.2.8.3

Locate splices to avoid conflicts with wind bracing, diaphragms and intermediate stiffeners. Layout locations of all intermediate stiffeners, diaphragms and wind bracing to avoid conflicts with the flange cutoff points (and possible splice locations).

Splices are a natural location to make changes in the flange size to eliminate flange welds. Maintain the same web thickness on each side of the splice.

For flexural members, it is recommended that the smaller section at the point of splice be taken as the side of the splice that has the smaller calculated moment of inertia for the non-composite steel section.

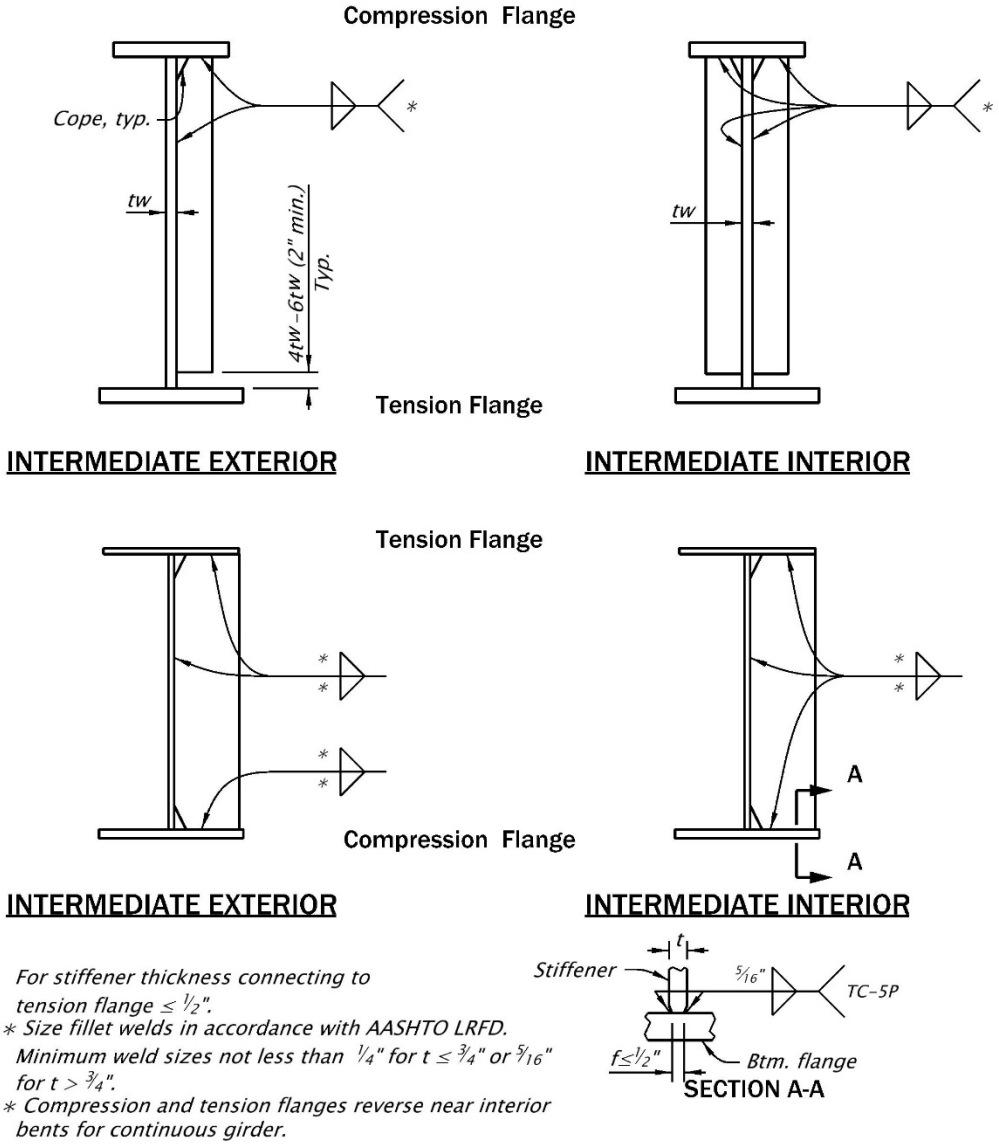
1.6.2.8.4 Intermediate Web Stiffeners

C1.6.2.8.4

When used, provide transverse intermediate stiffeners on both faces of the webs of interior girders and on the interior faces, only, of exterior girders. Specify stiffener widths in ½ inch increments. Specify thickness in 1/8 inch increments using 3/8 inch as an absolute minimum.

Rigidly connect the stiffeners to the compression portions of the flanges. Stiffeners may be welded to compression flanges. Ends welds ¼ inch away from the edge (snipe, clip, etc.) to avoid a poor quality weld termination. See *Figure 1.6.2.8.4-1* for details of intermediate web stiffeners.

Figure 1.6.2.8.4-1 Intermediate Web Stiffeners



For stiffener thickness connecting to tension flange ≤ ½".

∗ *Size fillet welds in accordance with AASHTO LRFD. Minimum weld sizes not less than ¼" for t ≤ ¾" or 5/16" for t > ¾".*

∗ *Compression and tension flanges reverse near interior bents for continuous girder.*

1.6.2.8.5 Bearing Stiffeners

Select bearing stiffener widths in increments of ½ inch. Specify thickness in 1/8 inch increments. Bearing stiffeners should extend to ½ to ¾ inch from the flange edge. Minimum size of fillet weld is specified in *BDM 1.6.3.2*. See *Figure 1.6.2.8.5-1* for details of bearing stiffeners.

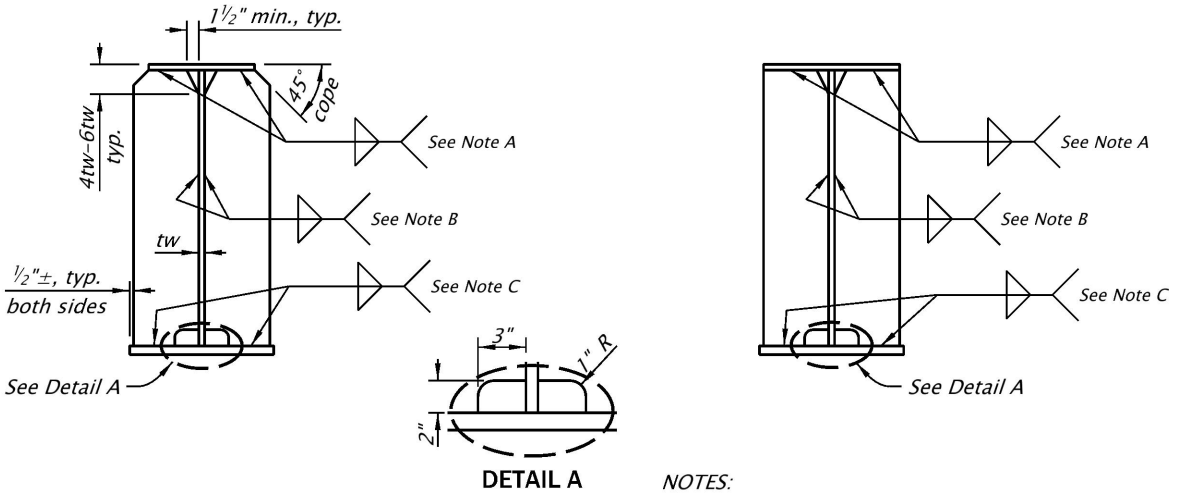
Fabricators strongly discourage full-penetration welding of bearing stiffeners to flanges.

C1.6.2.8.5

Bearing stiffeners and the web act as a column section, transferring loads from the superstructure to the substructure. In combination with the end frames, they also transfer lateral loads from the superstructure to the substructure.

Full-penetration welds distort the bearing area of the bottom flange.

Figure 1.6.2.8.5-1 Bearing Stiffener Detail



NOTES:

A. Use fillet weld in lieu of tight fit when a diaphragm or crossframe attaches to stiffener. Tight fit only without a diaphragm or crossframe attached.

B. Fillet weld sizes shall be shown on the design drawings or a note stating that minimum AASHTO/AWS welds shall be used. Where ever possible avoid fillet welds over 5/16".

C. Finish to bear (typ.) and fillet weld if a diaphragm or crossframe attaches to stiffener. Insure clearance to bearing attachment bolts. Investigate load for requirement of CJP welds.

Limit bearing stiffeners skew angle at end bents or interior bents to the values shown in *Figure 2.3* of the *AWS D1.5* for bearing stiffeners to web connection. Discard the footnotes of the figure which permits angles

less than 60 degrees. When the skew angle exceeds limit shown on *Figure 2.3*, use bent plates or half-round bearing stiffeners.

1.6.2.9 Check Samples and Nonredundant Steel Tension Members

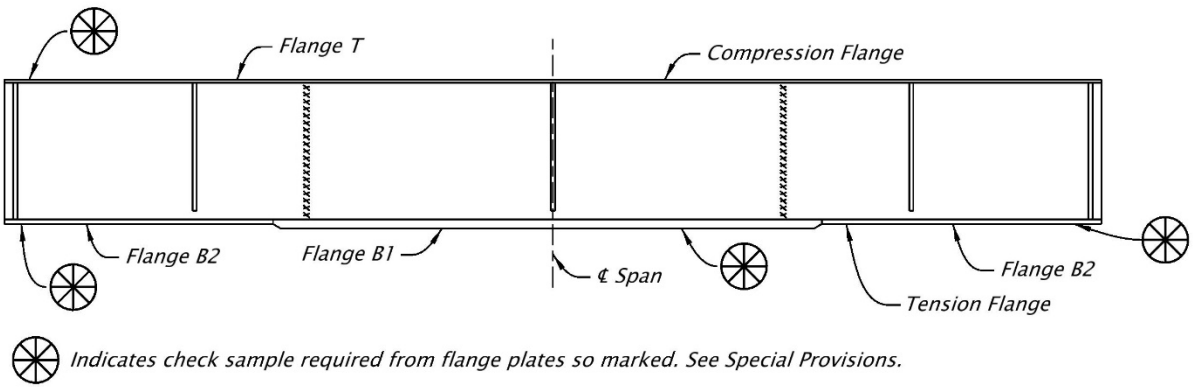
1.6.2.9.1 Check Samples

C1.6.2.9.1

Identify tension members and elements that require notch toughness check samples on the plans.

Check samples are required for cross-frame members on curved steel girders.

Figure 1.6.2.9.1-1 Check Sample Identification Detail



1.6.2.9.2 Nonredundant Steel Tension Members

C1.6.2.9.2

Clearly identify nonredundant steel tension members (NRST) on the plans. See FHWA-HIF-19-088 Bridge Welding Manual *Article 7.2.4 Design Detailing Practice* for identifying nonredundant steel tension members.

1.6.2.10 Fit-up and Intermediate Cross-Frames

Fit-up of steel bridge members is a critical component in the overall success of a project. Clearly convey to the fabricator and erector the intent of the design as it relates to bolting and pinning, camber, bolted field splices, and differential deflections. State in the contract documents the fit condition for which the cross-frames or diaphragms are to be detailed according to *LRFD 6.7.2*.

Include a note in the contract drawings stating that the girder webs are plumb in the final condition.

This requires the erector to force fit the diaphragms with the girders out-of-plumb prior to deck placement. Rotation of girders resulting from the deck placement plumbs the girders web and releases stresses caused from force fitting the diaphragms.

C1.6.2.10

Steel bridges, including straight and skewed bridges, should be detailed so they are plumb in the final condition. For steel girder bridges this means that the girder webs should be plumb after deck and barrier placement. This is accomplished by detailing the cross-frames to the final position. The girders are then installed to fit the cross-frames, requiring that for skewed bridges they be **rolled** during fit-up so that they are out of plumb under steel dead loads. Tolerances for web plumbness/girder layover are specified in the *AASHTO/NSBA Guide Specification S10.1-2014, Steel Bridge Erection Guide Specification Section 9*.

Steel Dead Load Fit (SDLF, also known as Erected Fit): For bridges which are detailed for SDLF the girder webs should be plumb (within reasonable construction tolerance) at the end of steel erection, prior to deck placement. When they are not plumb at the end of steel erection (prior to deck placement), the engineer should be consulted and remedial action should be considered. Later, when the deck is placed, the webs will lay over and be out of plumb. This sequence of webs being plumb prior to deck placement and out of plumb after deck placement is normal and generally does not represent a problem.

Total Dead Load Fit (TDLF, also known as Final Fit): For bridges which are detailed for TDLF the girder webs should be plumb (within reasonable construction tolerance) at the end of deck placement. The webs will be out of plumb at the end of steel erection, prior to deck placement. When the webs are plumb at the end of steel erection (prior to

deck placement), or are out of plumb in the wrong direction or beyond reasonable construction tolerances, remedial action should be considered. When the webs are in their correct, anticipated out of plumb position prior to deck placement, then when the deck is placed the webs will rotate (twist) to a plumb position (within reasonable construction tolerance), at least at the supports. This sequence of webs being out of plumb prior to deck placement and plumb after deck placement is normal and generally does not represent a problem.

SKEWED AND CURVED I-GIRDER BRIDGE FIT AND FRAMING ARRANGEMENTS

See Skewed and Curved Steel I-Girder Bridge Fit (Standalone Summary) and Skewed and Curved Steel I-Girder Bridge Fit (Full Document) for more information.

Recommended Fit Conditions for the following I-girder bridges:

- Straight bridges where one or more support lines are skewed more than 20 degrees from normal;
- Horizontally curved bridges where one or more support lines are skewed more than 20 degrees from normal and with an L/R in all spans less than or equal to 0.03;
- Horizontally curved bridges with or without skewed supports and with a maximum L/R greater than 0.03;

where:

L = actual span length bearing to bearing along the centerline of the bridge (feet)

R = girder radius at the centerline of the bridge (feet)

Fit Condition – deflected girder geometry associated with a targeted dead load condition for which the cross-frames are detailed to connect to the girders

Table 1.6.2.10-1 Recommended Fit Conditions for I-Girder Bridges

Loading Condition Fit	Construction Stage Fit	Description
No-Load Fit (NLF)	Fully-Cambered Fit	The cross-frames are detailed to fit to the girders in their fabricated, plumb, fully-cambered position under zero dead load.
Steel Dead Load Fit (SDLF)	Erected Fit	The cross-frames are detailed to fit to the girders in their ideally plumb as-deflected positions under bridge steel dead load at the completion of the erection.
Total Dead Load Fit (TDLF)	Final Fit	The cross-frames are detailed to fit to the girders in their ideally plumb as-deflected positions under the bridge total dead load.

Table 1.6.2.10-2 Recommended Fit Conditions for Straight I-Girder Bridges (including Curved I-Girder Bridges with L/R in all spans less than or equal to 0.03)

Span Length	Recommended	Acceptable	Avoid
Square Bridges and Skewed Bridges up to 20 deg Skew			
Any span length	Any		None
Skewed Bridges with Skew greater than 20 degree and $Is \leq 0.30$ +/−			
Any span length	TDLF or SDLF	-	NLF
Skewed Bridges with Skew greater than 20 degree and $Is > 0.30$ +/−			
Span lengths up to 200 feet +/-	SDLF	TDLF	NLF
Span lengths greater than 200 feet +/-	SDLF	-	TDLF & NLF

Table 1.6.2.10-3 Recommended Fit Conditions for Horizontally Curved I-Girder Bridges ($(L/R)MAX > 0.03$)

Radial or Skewed Supports			
Span Length	**Recommended**	**Acceptable**	**Avoid**
$(L/R)MAX \geq 0.2$	NLF	SDLF	TDLF
All other cases	SDLF	NLF	TDLF
Detail for a Steel Dead Load Fit, unless the maximum L/R is greater than or equal to 0.2.			
When $(L/R)MAX \geq 0.2$, detail for No-Load Fit, unless the additive locked-in force effects from Steel Dead Load Fit detailing are considered.			

CROSS-FRAME DESIGN

Design and detail cross-frame based on rational analysis for all stages of construction and service life according to *LRFD 6.7.4*. See G13.1-2019, Guidelines for Steel Girder Bridge Analysis Section 4 and Appendix B on selecting the appropriate level of analysis. Rational analysis of cross-frames depends on accurately analyzing the structure and determining the appropriate force effects.

- When a bridge is designed using line girder analysis methods, evaluate the cross-frames for minimum stability bracing strength and stiffness requirements as well as wind loads and construction loads. Per *LRFD 6.6.1.3.1*, in the absence of better information, design the welded or bolted connection to resist a 20.0-kip lateral load for straight, nonskewed bridges. Per *LRFD C6.10.10.1.2*, in lieu of a refined analysis, cross-frame or diaphragm force for an exterior girder may be taken as 25.0 kips.

- Where cross frame fatigue forces control the design, live load force effects in cross-frames and diaphragms should be calculated by 2D grid or 3D analysis model, careful consideration for modeling of cross-frames and the distribution of loads among the girders is crucial.
    - *LRFD C6.6.1.2.1* – Calculate fatigue stress using only a single transverse lane position. Determine maximum fatigue stresses in cross frames with the truck confined to one critical transverse position per each longitudinal position throughout the length of the bridge. Standard grid design software is generally not considering this. This requires isolating loading from individual lanes in the analysis. To satisfy this requirement, an influence-surface grid of 5 feet (longitudinal) by 4 feet (transverse) has been shown to adequately capture the maximum force effects.
    - For 2D models using equivalent beam, use Timoshenko beam approach, see G13.1-2019, Guidelines for Steel Girder Bridge Analysis *Section 3.11.3*. The Timoshenko beam approach provides the most realistic estimate of the cross-frame stiffness because it considers both flexural and shear deformations. Include the transverse stiffness of the deck.
    - Using a single adjustment factor of 0.65 applied to the Fatigue I load factor of 1.75 and the Fatigue II load factor of 0.8.
    - Apply the stiffness reduction factor in the model (apply to AE). Independent stiffness modification factors are proposed for the construction stages (R_{con} = 0.65) and for in-service conditions (R_{ser} = 0.75).

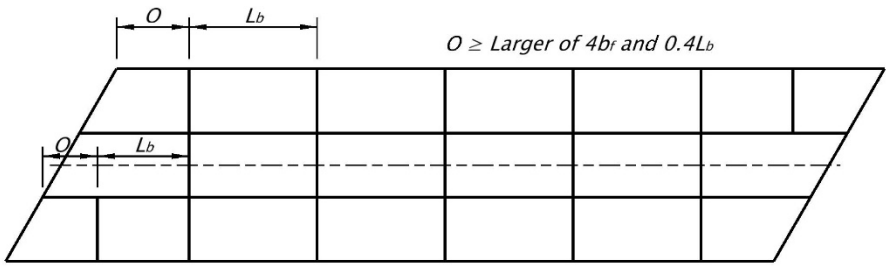
See the following references for additional guidance:

- Proposed Modification to AASHTO Cross-Frame Analysis and Design
    - Appendix B: Cross-Frame Design Example (Straight Bridge)
    - Appendix C: Cross-Frame Design Example (Curved Bridge)
- *G12.1-2020, Guidelines to Design for Constructability Section 2.2.6*
- *FHWA Steel Bridge Design Handbook: Volume 13 – Bracing System Design*

For skewed (greater than 20 degrees) and curved I-girder bridges:

- See *LRFD C6.7.4.2* for discussion about beneficial framing arrangements in skewed and curved I-girder bridges to alleviate detrimental transverse stiffness effects.
- It is recommended to offset the first intermediate cross-frame placed normal to the girders adjacent to a skewed support as shown in *Figure 1.6.2.10-1*.

Figure 1.6.2.10-1 Cross-Frame Layout for Bridge with Skewed Support



- Framing of a normal intermediate cross-frame into or near a bearing location along a skewed support line is strongly discouraged unless the cross-frame diagonals are omitted.
- At skewed interior bents and end bents, place cross-frames along the skewed bearing line, and locate intermediate cross-frames greater than or equal to the recommended minimum offset from the bearing lines.
- For curved I-girder bridges, provide contiguous intermediate cross-frame lines within the span in combination with the recommended offset at skewed bearing lines.

MEMBER SELECTION AND DETAILING

Select the most economical cross-frame members which meet the strength and detailing requirements of the design. Minimize the number of different types or sizes of members used in a given design. Consider the following for cross-frame member selection:

- Equal leg angles are often more cost-effective than unequal leg angles. Do not use back-to-back angles as cross-frame members. Common angle sizes for diaphragms range from L3 x 3 x 5/16 to L6 x 6 x 5/8.
- Only use WT sections when design forces exceed the capacity of angle sections. Do not use WT shapes in small quantities.
- When channel sections are used, C shapes are preferable to MC shapes. A bent-plate diaphragm, in the shape of a channel, is a possible option to provide to the fabricator for diaphragms on shallow plate girders (4-foot-deep web or less) or rolled beams.
- Design and detail cross-frames such that they can be erected as a single unit. Fabricators and erectors discourage diaphragms that require erection in separate pieces.
- Design cross frames as deep as practical so that the diagonals of the cross frame have large enough angles to prevent the gusset-type plates at the ends of the cross frame from becoming too large.

In choosing between intermediate cross-frames of **K** or **X** form, use the **X** form when the ratio of the beam spacing to the frame depth is less than two. Use the **K** form when the ratio is greater than two. Consider a solid plate diaphragm when the depth of the frame approaches 3 feet or less.

Consider maintenance and inspection requirements in the cross-frame design. Provide adequate clearance for sandblasting and painting. Avoid inaccessible areas. It may be necessary to provide for maintenance walkways or utilities through the cross-frames. See *BDM 1.15.5* for accessibility requirements and *BDM 1.12.6* for utility requirements.

Rigidly connect cross-frames to the top and bottom flanges to prevent web distortions and cracking. Weld stiffeners to compression and tension flanges as shown on *Figures 1.6.2.10-2 and 1.6.2.10-3*. Stop ends of welds about 1/4 inch away from the edge (snipe, clip, etc.) to avoid a poor quality weld termination.

Where two adjacent plate girders have significant differential deflection, such as the first row of cross-frame from the end bents, do not use the **K** or **X** type of cross-frames. Use details shown on *Figure 1.6.2.18-2*. Check fatigue requirements of all welded connections.

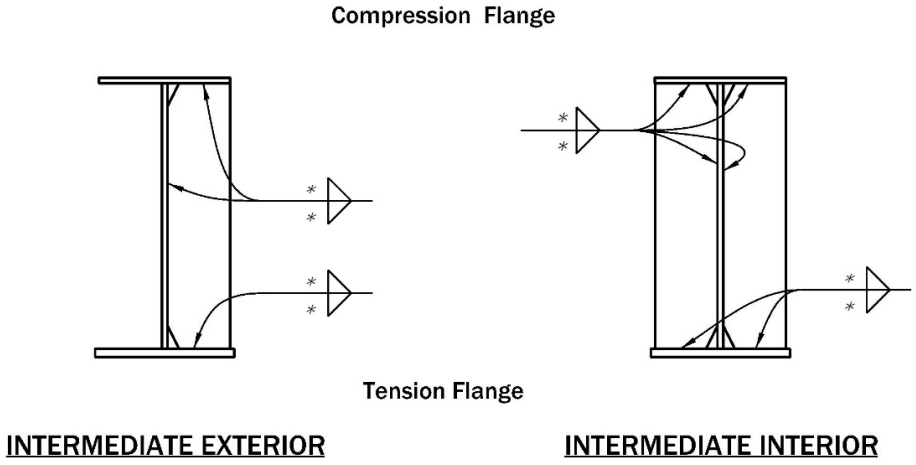
Provide intermediate cross-frames between the box girders and steel tub girders.

CONNECTION PLATES FOR BRACING MEMBERS

Cope diaphragm connection plates, which are welded to both the web and flange of a plate girder, a minimum of 1 ½ inches to prevent intersection of the two welds. Avoid lateral connection plates for lateral bracing which will be connected to the web of the plate girder or box girders. Bolt lateral connection plates to the flange of the steel girder. Cope lateral connection plates to be clear of any transverse web stiffener or diaphragm connection plate.

It is desirable to have all cross-frame member center of gravities intersecting at a common point (identified as work points, WP on design drawings). Locate work points either at the centerline of the girder web or at the corner bolt within the connection. It is difficult to develop details such that the member force vectors intersect the bolt group center of gravity. Therefore, design for the eccentric loads in the connection.

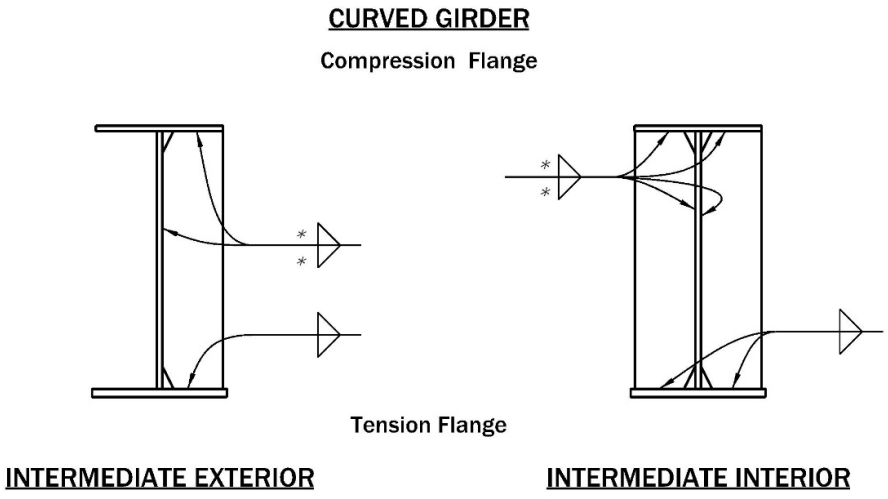
Figure 1.6.2.10-2 Transverse Connection Plates



∗ *Size fillet welds in accordance with AASHTO LRFD minimum welds sizes shall not be less than 1/4" for t ≤ 3/4" or 5/16" for t > 3/4".*

Compression and Tension flanges reverse near interior bent of continuous girder.

Figure 1.6.2.10-3 Transverse Connection Plates on Curved Girders



Compression and Tension flanges reverse near interior bent of continuous girder.

1.6.2.11 Cross-Frames at Bents

Design cross-frames at bent for seismic loading obtained from a linear elastic seismic analysis or the force V_{po}, whichever is less for bridges with SDC B and C.

Cross-frames at bents are more critical to transfer seismic forces from the superstructure to the substructure. One solution is to use detail *Figure 1.6.2.11-1* with welded studs added to the top flange of top chord members to provide the lateral resistance and transfer seismic loads from the concrete deck to substructure.

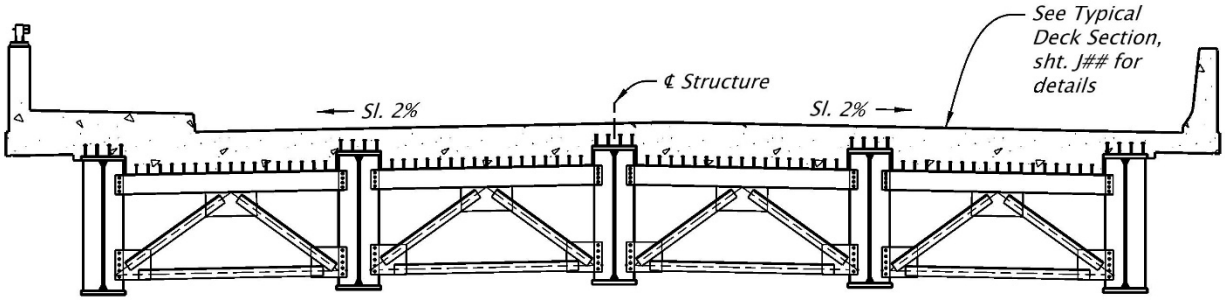
When a joint system is required for a cross-frame at end bents, it may be necessary to use details similar to cross-frames at continuous beam interior bents.
Diaphragms or cross-frames are required along skewed interior bents and end bents. See *BDM 1.6.2.10* for more information.

C1.6.2.11

The assumed load path for the flow of forces is accommodated through deck slab to-girder interfaces to main beams or girders and cross-frames, and to all components of the bearing assembly to substructure (see *LRFD 4.6.2.8*).

The shear studs at the ends are critical in transferring the transverse seismic forces from the deck slab into the steel superstructure and down to substructure. A top chord connecting the deck slab to the end cross frames was found to be effective in transferring these forces and resist rotation at the top of the girders in the absence of sufficient studs on top of the girders.

Figure 1.6.2.11-1 Cross-Frame Details at Bent



BENT DETAILS

1.6.2.12 Composite Action and Flange Shear Connectors

Provide shear connectors in all portions of continuous spans, positive or negative moment.

For deflection and moment calculations include longitudinal reinforcing steel in the composite section properties of the girder in the negative moment areas.

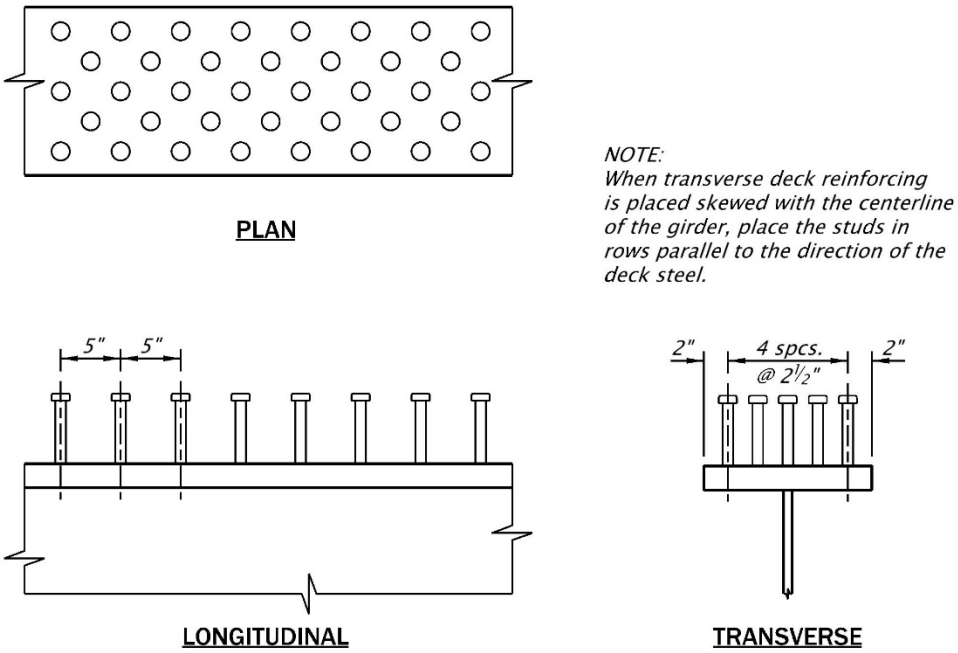
Extend shear connectors at least one inch above the mid depth of the deck and no less than one inch of clearance below top mat deck reinforcing.

C1.6.2.12

Previous practice was to not use concrete reinforcement to increase the moment capacity of composite girders in the negative moment areas.

Generally, the deck build up on steel girders is constant except for bridges with variable cross-slopes (super elevation) along the bridge. However the top flange plate thicknesses may vary. Consider the effect of top flange thickness variation and bridge deck super elevations when checking the shop drawings or specifying the shear connector's length. The advantages of longer shear connectors are in distributing load to larger area of the bottom mat reinforcing steel when a girder fails in fatigue. The concrete deck will distribute a portion of the unsupported load of the failed girder to adjacent girder(s).

Figure 1.6.2.12-1 Shear Connector Layout



1.6.2.13 Beam Camber

C1.6.2.13

Steel beams are cambered to compensate for dead load, shrinkage deflections and gradelines. The final position of the bottom flange is either flat or follows the grade, except in a sag vertical curve. Do not place a final negative camber in a beam. Profile grades can be incorporated into the camber by either added camber in the beam or by varying the deck flange build-ups along the beam. Sag vertical curves always require flange build-ups.

Consider the superelevation of the deck in the design of minimum flange build-ups.

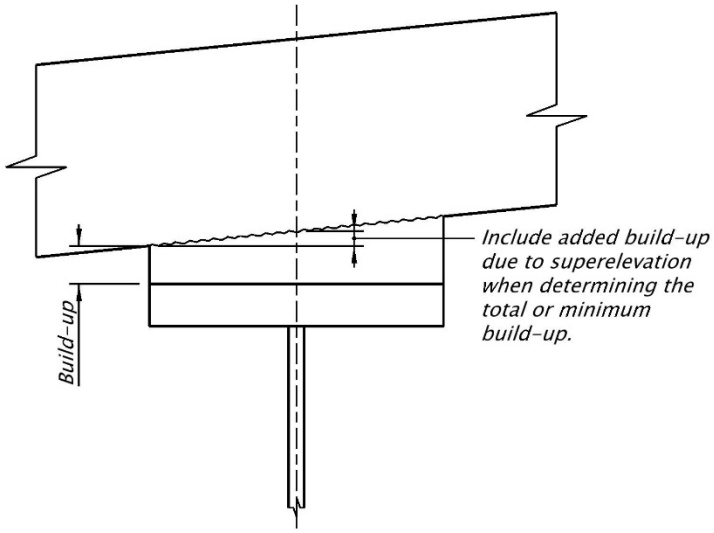
Slope adjustment or build-up for straight girders on curved roadways must also be

considered. Deck grades are based on the roadway centerline and straight girders are offset at midspan from the centerline. As a result, the adjustment is the superelevation times the midspan offset. Additional beam camber at midspan or additional build-up at the ends will be required. See *Figure 1.6.2.13-1*.

In addition to girder deflections, show girder rotations at bearing stiffeners.

Showing girder rotations will allow shop plan detailers to compensate for the rotations so that bearing stiffeners will be vertical in their final position.

Figure 1.6.2.13-1 Superelevation Deck Build-Up



Sketches of camber options for simple spans are shown in *Figures 1.6.2.13-2* through *1.6.2.13-4*.

Figure 1.6.2.13-2 Case 1: Crest Vertical Curve with Beam Grade Camber

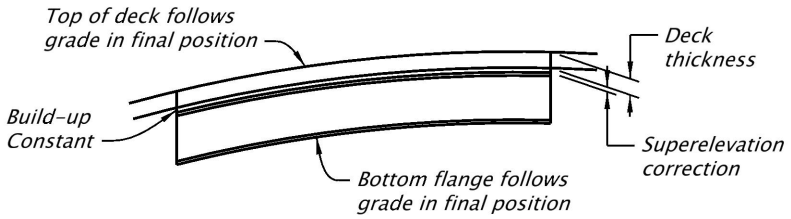


Figure 1.6.2.13-3 Case 2: Crest Vertical Curve with Build-Up for Grade Camber

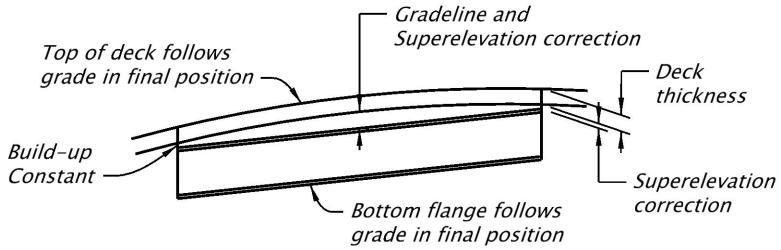
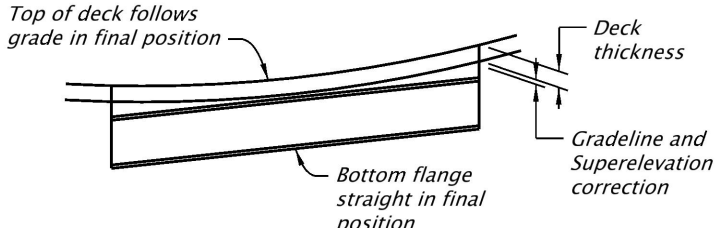


Figure 1.6.2.13-4 Case 3: Sag Vertical Curve with Build-Up for Grade Camber



SHRINKAGE CAMBER

Bridge deck shrinkage has a varying degree of effect on superstructure deflections. Use some judgment in evaluating this effect on camber.

A shrinkage strain of 0.0002 applied to the long-term composite section using 3n may be used to estimate the long-term effects of shrinkage using an approximate method.

For structures requiring close tolerances on girder cambers, perform refined analysis of shrinkage effects using structural analysis software such as MIDAS Civil.

C SHRINKAGE CAMBER

CAMBER DIAGRAMS

Show the data in *Table 1.6.2.13-1* for steel beam camber on the contract drawings.

C CAMBER DIAGRAMS

Bridge deck shrinkage should be the smallest portion of the total camber. It has greater influence on shallower girder sections, e.g. rolled beams. Simple spans will see more effect than continuous spans.

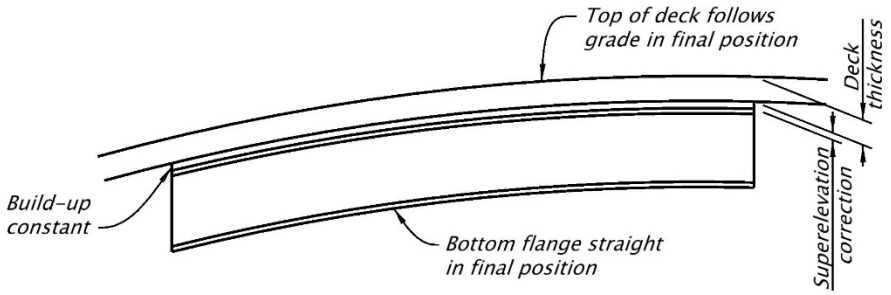
Tests have indicated that the unit shrinkage of the slab in composite beams (i.e., the shrinkage strain adjusted for long-term relaxation effects) may be taken equal to 0.0002.

Table 1.6.2.13-1 Steel Beam Camber Template

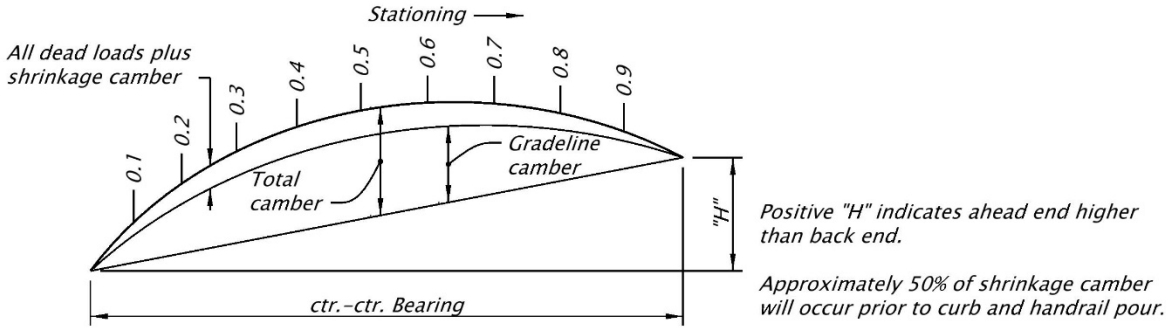
Camber Item	Value
Grade line camber	
Dead load camber	
Superimposed Dead load camber	
Shrinkage camber	
Total Camber	
Camber due to weight of steel beam and diaphragm	

Sketches of camber diagram examples are shown in *Figures 1.6.2.13-6 and 1.6.2.13-7.*

Figure 1.6.2.13-6 Camber Diagram Example 1



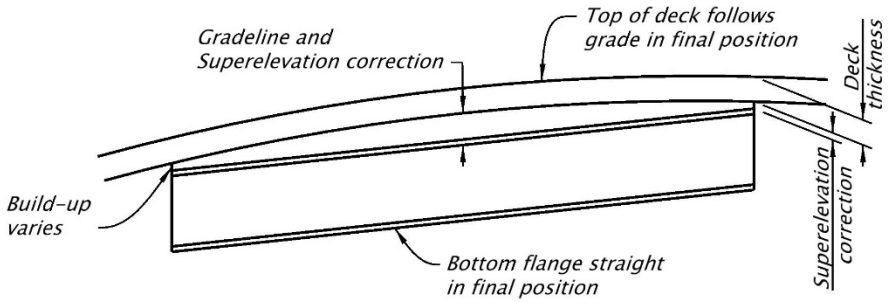
CREST VERTICAL CURVE WITH BUILD-UP FOR GRADE CAMBER



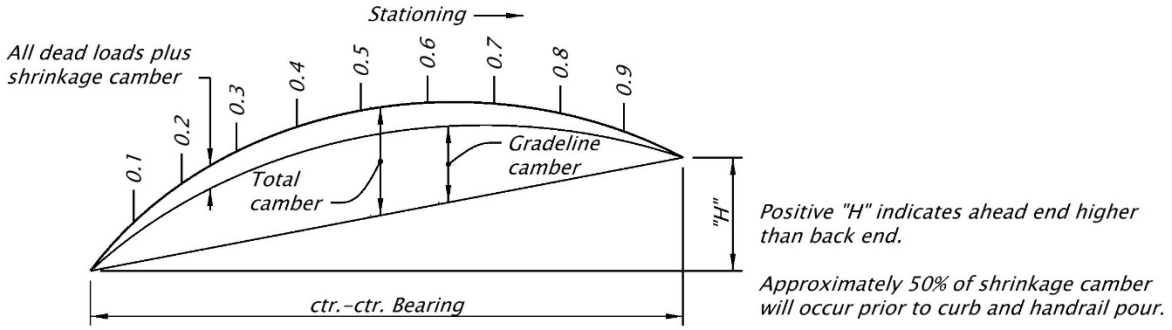
GIRDER CAMBER

SPAN	ITEM	CAMBER									H Inches
		0.1 Pt.	0.2 Pt.	0.3 Pt.	0.4 Pt.	0.5 Pt.	0.6 Pt.	0.7 Pt.	0.8 Pt.	0.9 Pt.	
1	Beam Dead Load + Diaphragms	5/8"	1"	1 3/8"	1 1/2"	1 3/8"	1 1/8"	3/4"	3/4"	1/16"	+16 3/8"
	Deck Dead Load + Form	1 3/8"	2 1/2"	3 3/16"	3 1/4"	3 3/16"	2 3/16"	1 3/8"	1 1/16"	1/4"	
	Sidewalk, Rail & WS Dead Load	3/8"	5/8"	1 3/16"	7/8"	1 3/16"	1 5/16"	5/8"	5/16"	1/16"	
	Shrinkage	5/8"	1"	1 1/4"	1 1/4"	1 1/8"	1"	3/4"	1/2"	1/4"	
	Sub Total	3"	5 1/8"	6 5/8"	6 7/8"	6 3/8"	5 1/8"	3 1/2"	1 7/8"	5/8"	
	Gradeline	3/8"	3/4"	1"	1 1/8"	1 1/4"	1 1/8"	1"	3/4"	3/8"	
	Total	3 3/8"	5 7/8"	7 5/8"	8"	7 5/8"	6 1/4"	4 1/2"	2 5/8"	1"	
2	Beam Dead Load + Diaphragms	1/16"	1/4"	5/8"	7/8"	1 1/16"	1/16"	5/8"	5/16"	1/16"	+7 5/8"
	Deck Dead Load + Form	1/8"	3/4"	1 5/8"	2 7/16"	3 1 1/16"	3 1/4"	1 5/8"	1 3/16"	1/8"	
	Sidewalk, Rail & WS Dead Load	1/16"	1/4"	1/2"	1 3/16"	7/8"	1 3/16"	1/2"	1/4"	1/16"	
	Shrinkage	-1/4"	-3/8"	-3/8"	-3/8"	-3/8"	-1/4"	-1/8"	-1/8"	0	
	Sub Total	0	7/8"	2 3/8"	3 3/4"	4 1/4"	3 7/8"	2 3/4"	1 1/4"	1/4"	
	Gradeline	5/8"	1 1/4"	1 5/8"	1 3/4"	1 7/8"	1 3/4"	1 5/8"	1 1/4"	5/8"	
	Total	3/8"	2 1/8"	4"	5 1/2"	5 1/8"	5 5/8"	4 3/8"	2 1/2"	7/8"	

Figure 1.6.2.13-7 Camber Diagram Example 2



CREST VERTICAL CURVE WITH BUILD-UP FOR GRADE CAMBER



GIRDER CAMBER											
SPAN	ITEM	CAMBER									H Inches
		0.1 Pt.	0.2 Pt.	0.3 Pt.	0.4 Pt.	0.5 Pt.	0.6 Pt.	0.7 Pt.	0.8 Pt.	0.9 Pt.	
1	Beam Dead Load + Diaphragms	5/8"	1"	1 3/8"	1 1/2"	1 3/8"	1 1/8"	3/4"	3/4"	1/16"	+16 3/8"
	Deck Dead Load + Form	1 3/8"	2 1/2"	3 3/16"	3 1/4"	3 3/16"	2 3/16"	1 3/8"	1 1/16"	1/4"	
	Sidewalk, Rail & WS	3/8"	5/8"	13/16"	7/8"	1 3/16"	15/16"	5/8"	5/16"	1/16"	
	Shrinkage	5/8"	1"	1 1/4"	1 1/4"	1 1/8"	1"	3/4"	1/2"	1/4"	
	Sub Total	3"	5 1/8"	6 5/8"	6 7/8"	6 3/8"	5 1/8"	3 1/2"	1 7/8"	5/8"	
	Gradeline	None	None	None	None	None	None	None	None	None	
	Total	3"	5 1/8"	6 5/8"	6 7/8"	6 3/8"	5 1/8"	3 1/2"	1 7/8"	5/8"	
2	Beam Dead Load + Diaphragms	1/16"	1/4"	5/8"	7/8"	1 1/16"	1/16"	5/8"	5/16"	1/16"	+7 3/8"
	Deck Dead Load + Form	1/8"	3/4"	1 5/8"	2 7/16"	3 1 1/16"	3 1/4"	1 5/8"	1 3/16"	1/8"	
	Sidewalk, Rail & WS	1/16"	1/4"	1/2"	13/16"	7/8"	13/16"	1/2"	1/4"	1/16"	
	Shrinkage	-1/4"	-3/8"	-3/8"	-3/8"	-3/8"	-1/4"	-1/8"	-1/8"	0	
	Sub Total	0	7/8"	2 3/8"	3 3/4"	4 1/4"	3 7/8"	2 3/4"	1 1/4"	1/4"	
	Gradeline	None	None	None	None	None	None	None	None	None	
	Total	0	7/8"	2 3/8"	3 3/4"	4 1/4"	3 7/8"	2 3/4"	1 1/4"	1/4"	

1.6.2.14 Deck Pour Sequence

See *BDM 1.9.4.3*.

1.6.2.15 End Bents Detailing

It is desirable to eliminate end bent joints or make construction jointless to protect the girder steel from leaking joints.

Use the extended deck detail or semi integral abutments similar to *Figures 1.6.2.15-1* or *1.6.2.15-2*.

Use the integral abutments when geometry and span length allow.
Show a coated or painted section at the ends of plate girders. On jointless bridges coat or paint the end of the girder with three coat paint system for a length of 1 foot outside the concrete interface and 4 inches inside the concrete interface. See *Figure 1.6.2.15-3*.

Where joints cannot be avoided, show a coating or painting detail at the end of plate girders. Coat or paint the end of the girder for a length at least 1.5 times the depth of the girder and all attachments within this limit. See *Figure 1.6.2.15-4*. Match the paint color developed by the weathering steel patina 2.5 years after completion of the bridge construction. See *Figure 1.6.2.15-4*.

C1.6.2.15

See *BDM 1.11.2.5* and *1.11.2.6* for additional guidance.

Figure 1.6.2.15-1 Extended Deck Details at End Bents

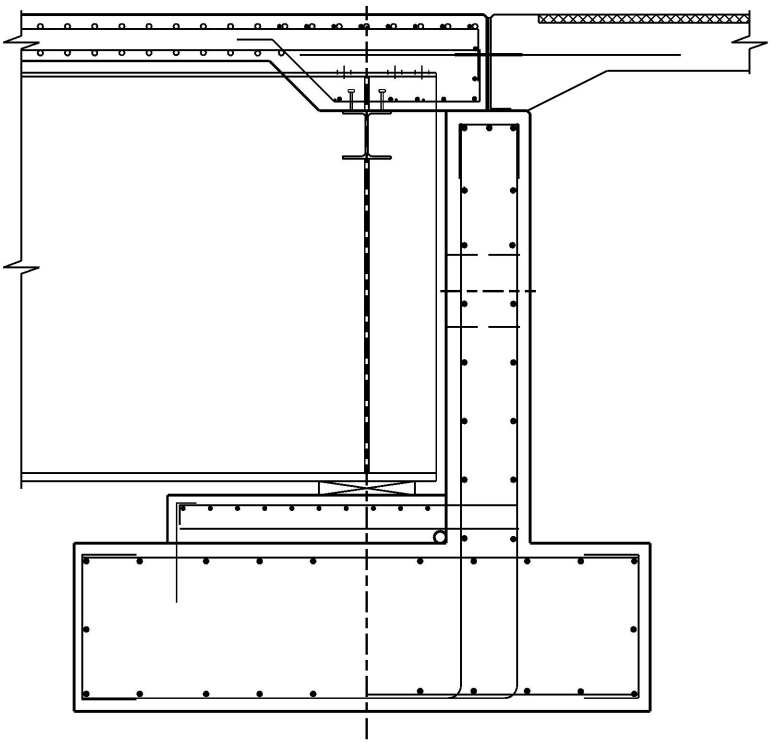


Figure 1.6.2.15-2 Semi Integral Bent Detail

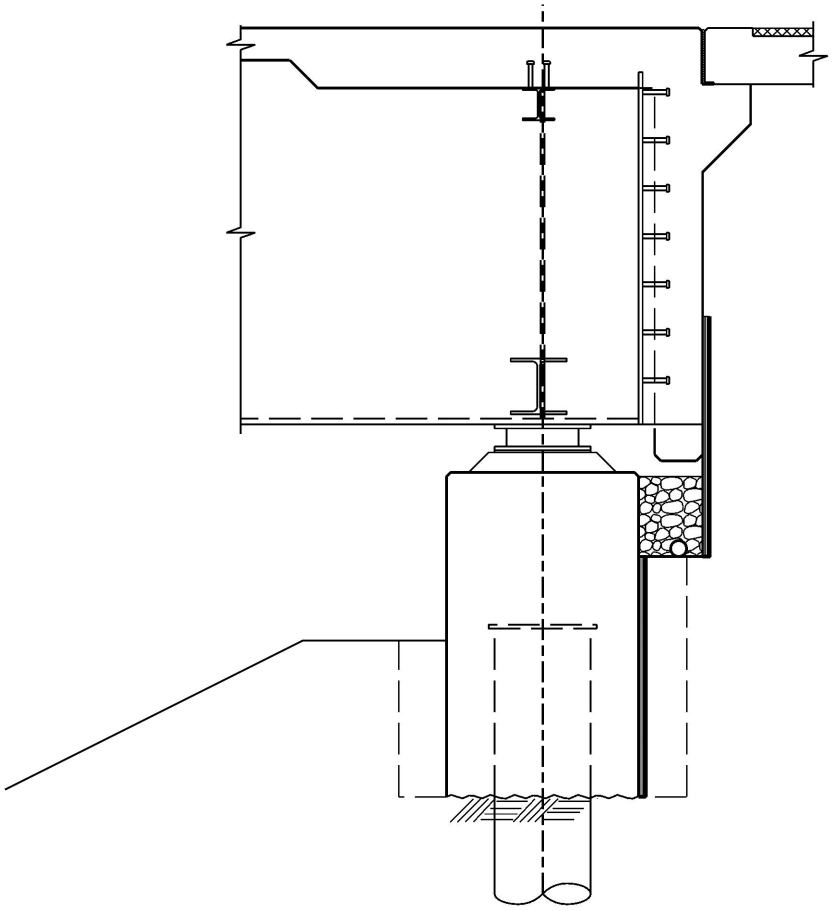


Figure 1.6.5.15-3 Integral Bent Detail

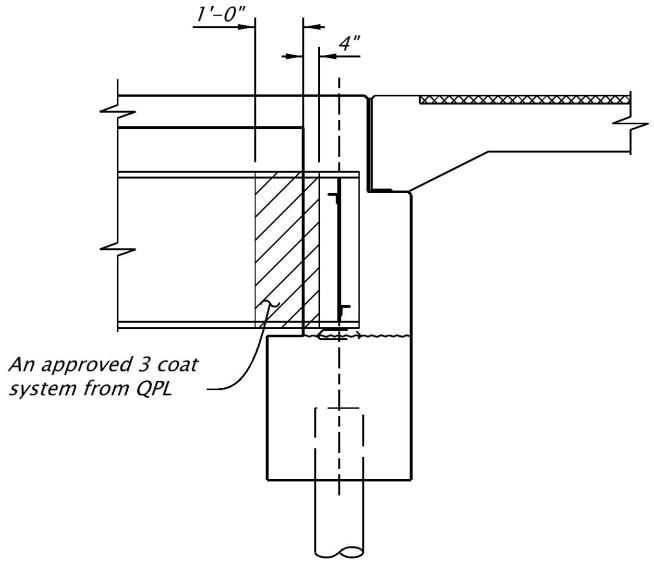
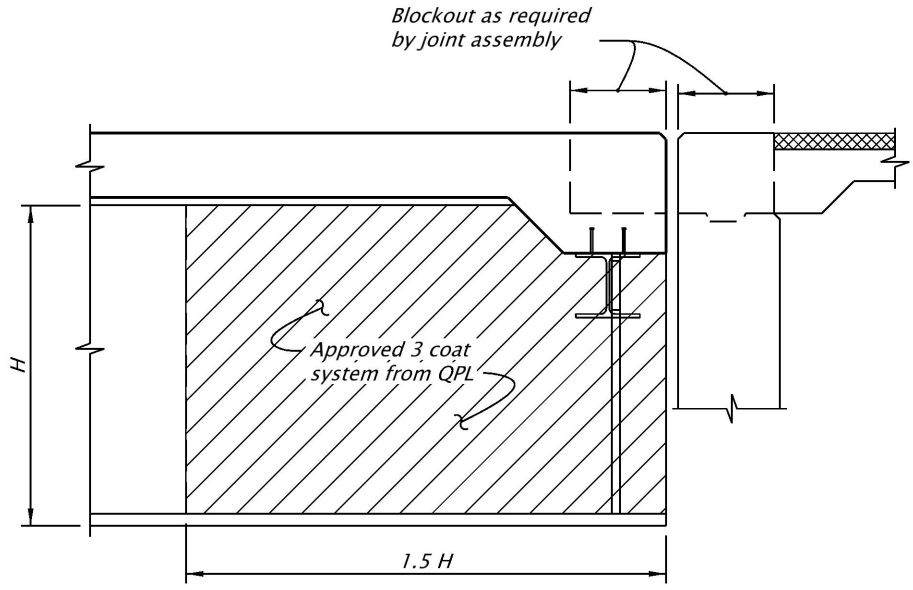


Figure 1.6.2.15-4 Girder End Coating Limit



1.6.2.16 Expansion Joint Blockouts

Show a blockout detail (see *Figure 1.14.2.4-1*) on the plans to allow the expansion joint assembly to be placed after the final deck pour.

C1.6.2.16

Providing a blockout makes the adjacent deck pour easier, provides smoother deck transition to joint, and allows the majority of the superstructure shrinkage to occur prior to joint assembly placement.

1.6.2.17 Bearings and Anchor Rods

Due to high cost, avoid using built up steel bearings, pot bearings, and spherical bearings.

Use elastomeric bearings wherever possible.

Use circular elastomeric bearings on curved steel girders.

See also *BDM 1.14.1* and G9.1-2004, Steel Bridge Bearing Design and Detailing Guidelines for additional guidance. See *Drawing E2.3* for preferred bearing anchor rods connection detail.

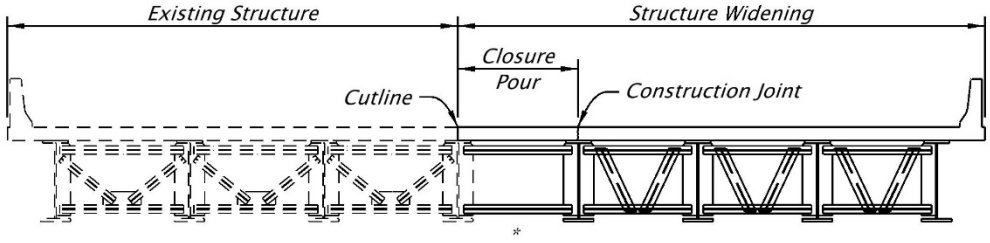
C1.6.2.17

1.6.2.18 Structure Widening

Avoid transferring dead loads from the widening to existing beams. Specify diaphragms between the widening and existing girders with a closure pour at the deck level as shown in *Figure 1.6.2.18-1 and Figure 1.6.2.18-2*. The diaphragms are for lateral load resistance.

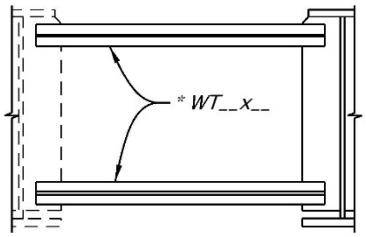
C1.6.2.18

Figure 1.6.2.18-1 Widening Closure Pour Detail



NOTE:
Place closure pour a minimum of 3 days after adjacent deck widening concrete placement.

Figure 1.6.2.18-2 Diaphragm Connection Detail



** Install temporary 4– __ dia. H.S. bolts (A 325) (snug tight) in each end of each horizontal WT__x__ and omit the diagonal WT__x__'s until the entire deck and closure pours have been completed.*

1.6.3 Welding

1.6.3.1 Welding, Design Considerations

C1.6.3.1

The following four categories loosely describe the most common types of welding needed for design work in roadway and bridge sections.

1. **INCIDENTAL STRUCTURES** (*AWS D1.1*): Welding under this category consists of light structural joining such as handrails, fencing, and sheet metal products.

In general the weld is not required to fully develop the strength of the joining parts. Visual inspection of the final product is all that is expected.

2. **GENERAL STRUCTURAL WELDING** (*AWS D1.1*): Welding under this category consists of partially or fully developing the strength of the joining parts such as pile splices and attachments, guard rails, signing and lighting support, expansion joints (unless prefabricated by an approved supplier), seismic restraint fixtures and bearings (unless directly welding to main structural elements of a bridge).

In general the weld will develop the ultimate strength of the joining parts but is not expected to provide maximum fatigue life unless nondestructive testing is specified for acceptance.

3. **STRUCTURAL WELDING OF REINFORCING STEEL** (*AWS D1.4*): Welding under this category consists of splicing and anchoring either new construction or existing reinforcing steel in concrete columns and girders. Note that *LRFD 9.7.2.5* does not allow welded splices of bridge deck reinforcement due to fatigue considerations. The particular weld joint design usually consists of either flare-bevel welds or butt joints with back up bars see *Figure 1.11.3.6-2* for examples. Ensure the contractor provides a CWI during field welding.

In general it is desired to develop the full strength of the reinforcing steel to be joined. Almost any type of reinforcing steel can be successfully welded provided the chemistry of the steel is known (from either mill certifications or field testing) and an appropriate welding procedure is developed and followed. Unknown steels need to have a sample extracted (approximately 2 to 4 grams) and testing for chemistry.

The welding procedure is developed from *AWS D1.4* using the carbon equivalent method. This type of welding is almost always performed in the field and thus needs

4. **BRIDGE WELDING** (*AWS D1.5*): Welding under this category consists of fabricating or modifying any main load path carrying members of a bridge that have some or all portions that experience tensile stresses under normal loads. This includes girders, floor beams, stringers, trusses, and hanger assemblies. The member does not necessarily have to be fracture critical.

Certification of Steel Fabricators: *SP 00560.30* requires the American Institute of Steel Construction (AISC) Certified Bridge Fabricator - Intermediate (IBR) certification. For fracture critical Structures, the fabricator is also required to have an AISC Fracture Critical Endorsement (FC). All fabricators of earthquake restraints are required to have either a current AISC IBR certification or a Certified Bridge Fabricator - Simple (SBR) certification.

1.6.3.2.1 Typical Pathways for Successful Welding in Your Design

INCIDENTAL WELDING:

- Specify the welds needed on the drawings (type, size, and length).
- In general, welding procedure specifications and welder certification are not required to be submitted.
- Quality assurance will be based on general appearance (visual testing) only. When you want a trained to be monitored by a certified welding inspector (CWI). Acceptance is usually based on visual examination, but other methods can be used when concerned about fatigue.

In general the welding is expected to develop both full ultimate strength of the joining parts and maximum fatigue performance. Joint toughness and nondestructive testing are typically required for acceptance.

C 1.6.3.2.1

person to inspect the workmanship send a copy of the plans to the ODOT Portland Materials Inspection Crew. The same inspectors will also check for quality of painting and galvanizing. When the workmanship is poor then the parts can be rejected.

GENERAL STRUCTURAL WELDING:

- Specify the welds needed on the drawings (type, size, and length).

Even though the *Standard Specifications* invoke *AWS D1.1* welding code for all incidentals structures, it is recommended that the following statement be included on the drawings (usually the plan and elevations):

  - All welding shall conform to the *AWS D1.1 Structural Welding Code*.

- Generally welding procedure specifications (WPS) and welder certification are required to be submitted and approved. Any shop drawings that have welding shown are not legally approved until the WPS are approved under *AWS D1.1*.

  Quality assurance is typically based on visual inspection by a certified welding inspector (CWI) and may also incorporate nondestructive testing such as ultrasonic (UT), radiographic (RT), and magnetic particle (MT) testing when specified on the design drawings. Various stages of the fabrication process may also be monitored if necessary. It is recommended that a copy of all plans and specifications that require this category of welding be sent to the ODOT Portland Materials Inspection Crew.

REINFORCING STEEL WELDING:

- Specify the welds needed on the drawings (type, size, and length).
- In the general notes, put the following:
    - All reinforcing steel welding shall conform to *AWS D1.4* Structural Reinforcing Steel.
- When the steel is not *ASTM A706* extract a field chemistry sample and analyze for the carbon equivalent. Base the welding procedure on this information. When the steel is *A706* the *D1.4* welding code has recommended heat inputs.
- Inform the ODOT Portland Materials Office of the work and have a CWI review the welding procedure, welder certification and observe the welding.

BRIDGE WELDING:

- Specify the welds needed on the drawings (type, size, and length).

Calling out the specific weld ID number (i.e. TC-U4a is an example) is preferable but not required. Typically this category of welding requires a significant Quality Assurance (QA) effort so please include this in your construction cost estimate.

- Even though the *Standard Specifications* invoke *AWS D1.5* welding code for all bridge welding it is recommended that the following statement be included on the drawings (usually the plan and elevations):

    All welding shall conform to the *AWS D1.5 Bridge Welding Code*.

- Welding procedure specifications (WPS) and welder certification are required to be submitted and approved by the Engineer of Record. Any shop drawings that have welding shown are not legally approved until the WPS are approved under *AWS D1.5*.

Quality assurance is based on a more complicated Owner/Fabricator relationship that involves frequent inspections during the entire fabrication and erection process. Most individuals involved have stringent requirements for their duties including certified welders, inspectors, fabricators, and testing personnel. Most welding in this category requires some form of nondestructive testing for acceptance. Theoretically all materials and processes are traceable with archived documentation. Send a copy of all plans and specifications that require this category of welding to the ODOT Portland Materials Inspection Crew.

1.6.3.2 Fillet Welds

When adequate structural performance from fillet welds in **T** and corner joints can be obtained, use fillet weld in preference to groove welds.

The minimum fillet weld size for prequalified joints is shown in *Table 1.6.3.2-1*.

C1.6.3.2

Fillet welds can be non-destructively inspected with greater certainty of result and at lower cost.

Table 1.6.3.2-1 Minimum Fillet Weld Size for Joints

Material Thickness of Thicker Part Joined (T) (inch)	**Minimum Size* of Fillet Weld (inch)**
To ¾ inclusive	¼**
Over ¾	5/16**
* Except that the weld size need not exceed the thickness of the thinner part joined. For this exception, take particular care to provide sufficient preheat to ensure weld soundness.	
** Welds of this size must be made in a single pass.	

Size fillet welds in accordance with *AASHTO LRFD Design Specifications*.

WEB TO FLANGE CONNECTION

Use the minimum fillet weld necessary to join the flange to the web.

Calculate shear stress capacity of fillet welds (equal legs) using equation *LRFD 6.13.3.2.2b*.

C WEB TO FLANGE CONNECTION

The weld size will vary along the length of the girder depending on the size of the plates being joined.

1.6.3.3 Flange Welds

C1.6.3.3

The design tensile stress in butt welded joints may equal the allowable stress in the base metal.

Show flange butt weld splices as in *Figures 1.6.3.3-1 or 1.6.3.3-2*. Include this detail on all steel structure plans. Indicate the type of butt weld splice for each splice on the plans. This may be accomplished by:

- Adding the word **tension** or **compression**, whichever is the case, to the tail of the weld symbol.
- Indicating which flanges or which portions of the flanges are in compression (C) and which are in tension (T).

Figure 1.6.3.3-1 Unequal Thickness Flange Splice

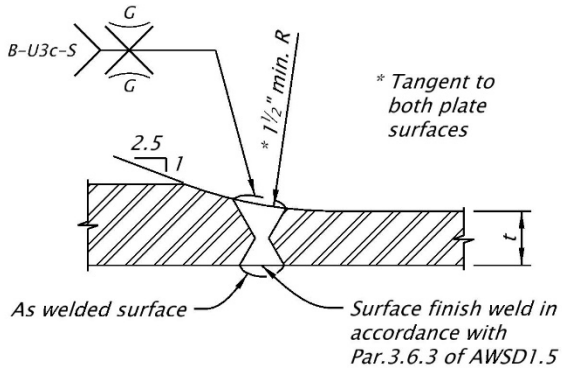
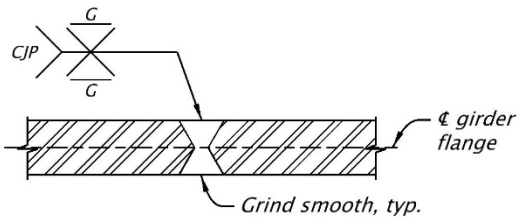


Figure 1.6.3.3-2 Equal Thickness Flange Splice



1.6.3.4 Welded Web Splices in Steel Bridge Girders

C1.6.3.4

Use complete joint penetration butt weld in web splices. Grind off 100 percent of the weldments reinforcing of all web splices. To facilitate NDE during fabrication, specify on the design drawings which portion of the girder webs are tension and compression. See *Figures 1.6.3.4-1* and *1.6.3.4-2*.

Figure 1.6.3.4-1 Unequal Thickness Web Splice

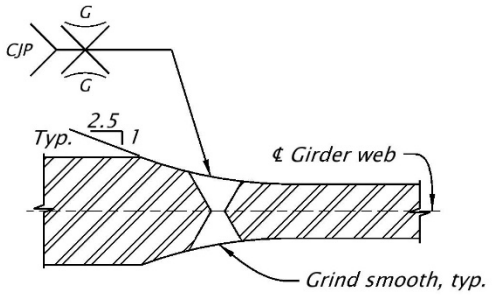
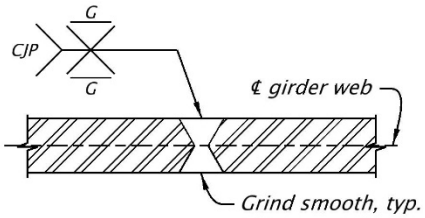


Figure 1.6.3.4-2 Equal Thickness Web Splice



1.6.4 Galvanizing and Painting

1.6.4.1 Processes

Hot-dip galvanizing has proven to provide better long term corrosion protection and should be required for all galvanized items.

C1.6.4.1

Galvanizing is a process of applying a sacrificial metal (zinc) to a base metal. The zinc will corrode, or sacrifice itself, to protect the base metal. Hot-dip galvanizing involves cleaning the items with a combination of caustic and acidic solutions and the dipping them into a tank of molten zinc for a specified period of time. After removal, small items are spun to remove excess zinc.

Mechanical galvanizing involves cleaning as mentioned above and then loading the items in a multi-sided rotating barrel. The barrel contains a mixture of various sized beads and water. As the barrel turns, chemicals and powdered zinc are added. The collision between the items, the glass beads and zinc causes the zinc to cold weld to the part. Powdered zinc is added until the required thickness is obtained.

1.6.4.2 Detailing

To ensure proper hot-dip galvanizing, provide venting and drain holes in details.

Provide a minimum vent opening of 25 to 30 percent of the cross-sectional area of a tubular section when full open venting is not possible.

Provide drains holes at closed corners or clip all corners at gusset plates to allow complete drainage.

C1.6.4.2

These insure proper circulation and removal of cleaning solutions and the molten zinc. They may also prevent potential explosions during dipping caused by trapped air.

1.6.4.3 Silicon Control

The silicon content of the steel must be held within either of the range of 0 to 0.06 percent, or 0.153 to 0.25 percent to obtain and maintain a pleasing appearance.

Call out all members that will have visual impact on the drawings with Galvanize - Control Silicon.

For economic reasons, silicon need not be controlled in galvanized structural members that are hidden from motorist view or are too small to have significant visual impact. Generally, these members that are too small to have significant visual impact are steel shapes whose least dimension does not exceed 3 inches.
The general notes on each contract drawing that includes members are to be called out as Galvanize-Control Silicon.

C1.6.4.3

The silicon content of the steel influences the corrosion resistance and strength of the galvanized coating and the thickness of the zinc layer.

Examples of these members are the chords, posts and diagonals of sign bridges; arms and shafts of luminaire, sign and signal support structures; steel traffic rail posts and railing members and pedestrian railings.

An example of an exception is pedestrian rail members that should have silicon control. Examples of hidden members and others which for practical reasons do not require silicon control are base plates and guard rail connection plates, flex-beam rails and their posts and single-post, breakaway sign posts.

The specification for control of silicon in steels to be galvanized is included in the *Standard Specifications for Construction*.

1.6.4.4 Painting or Coating of New or Existing Metal

Coating of metal structures is discouraged in most circumstances due to maintenance costs of recoating. Weathering steel (see *BDM 1.6.2.2*) and galvanizing are preferred options.

Sacrificial thickness is another option that may be appropriate in some circumstances.

C1.6.4.4

Coating work consists of preparing and coating new metal structures and features in the shop and in the field, and preparing and coating existing metal structures. This includes all:

- Interior and exterior steel surfaces.
- Steel railings, bridge bearings, and bridge expansion joint assemblies.
- Other miscellaneous steel.

Perform a life cycle cost comparison when considering sacrificial thickness vs coated steel. Include the cost comparison in the TS&L narrative.

Coating steel may be appropriate in the following situations:

- Marine areas (as defined in *BDM 1.2.4.1*).
- When use of weathering steel is improper per *BDM 1.6.2.2.*
- Structure is easily accessible and has minimally restricted lane closures (i.e. does not cross a roadway).

Specify coating of metal structures in conformance with *SP 00594* and the special provisions. Powder coating is discussed in *SP 00593*.

In the construction drawings, require inside surfaces of boxes or tubs (bottom flange, top flange, web and diaphragm) to be painted with a silver gray prime coat.

One of the main concerns in steel tubs or box girders in the State of Oregon is corrosion inside the girders.

Painting inside the tub (box) girders will prevent corrosion resulting from leakage thru the deck and condensation. Light color paint also increases illumination inside the tub (box) and eases detection of corrosion or cracks in steel members.

1.6.4.4.1 Design Features of Coated Steel

Provide the following design features for bridges fabricated from coated structural steel:

C1.6.4.4.1

1. Where structure access and lane closures are expected to be improbable, consider providing additional vertical clearance beyond the required minimum (per *BDM 2.14.4.2*) according to the following criteria. Allow for future ACP overlay when applicable.
   - For box girders: half of the box girder bottom flange width plus 1 foot, but not exceeding 3 feet.
   - For plate girders: 1.5 foot minimum, but not exceeding 3 feet.
   - All other situations: 1.5 feet.

This is to allow a minimal amount of access for work platforms and performing the work on the lowest members over traffic, without affecting freight movement.

Submit a design deviation when providing additional clearance beyond the required minimum.

Additional vertical clearance only needs to be provided at girders over traffic lanes or where low water clearance is expected.

When only the minimum required clearance is provided (no additional), it will be more difficult and costly to recoat the bridge in the future. Reduced clearances typically require lane closures, detours, or night work to gain access to the work. In high traffic areas where lane closure is prohibited the efficiency of the work can be further degraded when allowable work shifts are too short.

Evaluate providing additional clearance with the project team based on the following criteria:

- Projected AADT – projected AADT high enough to impact future lane closures.
- Railroad project involvement – railroad involvement will restrict structure access.
- Freeway projects - either on or above.
- Urban locations – surroundings (buildings, structures, utilities, etc.) may confine and limit access.

Evaluate the lifecycle cost of increasing vertical clearances against future recoating maintenance.

- When evaluating increased project construction cost due to a raised profile, consider the following items that may be affected:
    - Roadway construction due to a grade profile change.
    - Environmental impact and mitigation.
    - Additional right-of-way needs.
    - Retaining wall(s) needs.

  - Seismic requirements (i.e. increased column heights, foundation stabilization).
  - Over-height warning system - for protecting workers on platforms over live traffic.

Evaluate the following items, with assistance from the Senior Cost Engineer and the Senior Structure Coatings Engineer, when considering lifecycle costs of future painting when only the minimum required clearance (no additional) is provided:

- Over-height warning system.
- Additional traffic control.
- Detours.
- Effect of night work.
- Mobilizing and demobilizing equipment each work shift; effect of work shifts that are not long enough for reasonable efficiency.
  - Increase in access and containment cost due to complexity and additional setups.
  - Increase in surface preparation cost due to reduced efficiency.
  - Increase in coating application cost due to reduced efficiency.
  - Increase in traffic control or detour costs due to reduced efficiency.
- Increased overhead costs due to reduced efficiency (longer calendar duration of work).
  - Delay costs borne by the public.

Document the various clearance alternatives, including cost comparisons (based on the items outlined above) and justifications in the TS&L narrative.

2. Provide a minimum of 3 feet horizontal separation between the front face of traffic rail and the nearest steel surface (e.g. tied arch and stacked deck structures).

This is to allow a minimal amount of access for scaffolding, worker and traffic protection shielding, and performing the work on the members closest to traffic, without affecting traffic or freight movement.

3. Provide stainless steel padeyes, stainless steel eyebolts or deck inserts located at appropriate intervals (approximately 15 by 15 foot to 20 by 20 foot grid typical) for support of future work platforms from upper structure members above roadway, main structure members below roadway, and the deck.

This is to allow surface preparation and coating of members without having to move hangers, beam clamps, chains, cables or chokers. These temporary attachment points require extra work and often receive poor surface preparation and coating.

List the allowable loading for the attachment points in the structural notes on the bridge plans. This requirement may be coordinated with the inspection requirements of *BDM 1.15.5*.

4. Include a temporary load representing the dead loads, live loads and wind loads acting on work platforms, scaffolding and containment needed to recoat the bridge. Add this Temporary Load to each applicable Load Combination as shown in *LRFD Table 3.4.1*, using a load factor of:

- 1.50 for Strength and Extreme Event limit states.
- 1.00 for Service I and Fatigue limit states.
- 1.25 for Service II limit state.

Design work platforms that access the structure with little need for scaffolding for a

When the member is too small for a worker to turn around in, but larger than 2 by 3 feet, provide access openings 18 by 30 inches minimum with semicircular ends, spaced from 42 to 60 inches on centers.

For members up to 2 by 3 feet, provide hand hole access 6 by 12 inches with semicircular ends, spaced 30 inches on center. For members large enough for workers to enter see *BDM 1.2.10* for additional accessibility guidance.

$DL + LL$ of 25 pounds per square foot of platform area. Design work platforms that require significant scaffolding to reach the structure for a $DL + LL$ of at least 50 pounds per square foot. See *BDM 1.3.4* for temporary loading.

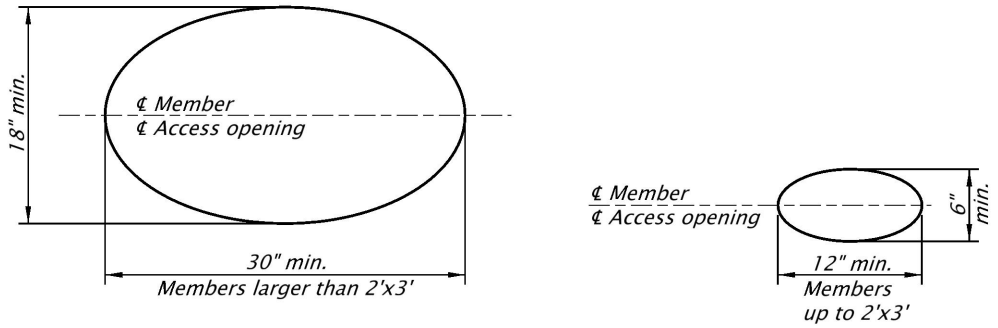
Assume work platforms and containment extend 5 feet beyond the sides of the structure and full length to 5 feet beyond the ends of the entire structure or each span.

List the permissible work platform sizes and loading, and the permissible containment sizes and wind speeds in the structural notes on the bridge plans. It may be necessary to separate the work platforms and containment into zones for structural reasons, in which case carefully size the zones to allow efficient work by the recoating contractor.

5. Do not create spaces where blind sides of members cannot be reached for surface preparation and coating work.

6. Closed members or areas which are too small for workers to enter to perform surface preparation and coating work are highly discouraged. See *Figure 1.6.4.4.1-1*.

Figure 1.6.4.4.1-1 Access Openings for Closed Members



Be aware of an existing structure's condition prior to completing TS&L of a recoating project. Recoating of an existing bridge is very costly and requires a careful examination of the structure's condition.

Collect all necessary information for such projects. The Steel Bridge Recoating Checklist provides a list of required information and guidance on specifications. Complete the checklist and include in the TS&L Report.

Older structures are typically painted and have potential deficiencies that may need to be addressed during a recoating project.

1.6.5 Bolts and Connections

Design all high-strength bolted connections as slip-critical connections. Assume Class B faying surfaces where inorganic zinc primer is used. When steel will be given a full paint system in the shop, mask the primed faying surfaces to maintain the Class B surface.

Include additional costs for access, paint removal and recoating rivet or bolt replacements, when rivets or bolts are outside normal paint area limits.

1.6.5.1 High Strength Bolts

HIGH-STRENGTH BOLT USE GUIDELINES:

- *ASTM F3125* GR A325 & GR F1852 - Headed structural bolt for use in structural connections. Do not specify for anchor bolts.

C1.6.5.1

These may be hot-dip galvanized. Limit the maximum bolt diameter for girder bolted connection to 7/8 inch. Bolts with diameters greater than 7/8 inch are difficult to tighten. Checking the inspection torque in the field becomes problematic from an equipment standpoint when bolts with diameters greater than 7/8 inch are used.

- Use Type 3 bolts conforming to *ASTM F3125* when specifying weathering steel.
- *ASTM A449* - Steel bolts and studs for general applications including anchor bolts.
  When using bolts of larger diameter, accounted for a reduction in strength as indicated in the *Table 1.9.5.2-1*.
  Do not use these as anchor bolts for seismic applications due to low CVN impact toughness.

  Recommended for use where strengths equivalent to *ASTM F 3125* GR A325 bolts are desired but custom geometry or lengths are required. Strengths for *ASTM A449* bolts are equivalent to GR A325 up to 1 inch diameter. These bolts may be hot-dip galvanized.

- *ASTM F3125* GR A490 & GR F2280 -
  Do not use *ASTM F3125* GR A490 bolts in bridge applications. When a design deviation is approved for use of these bolts, do not galvanize these bolts because of high susceptibility to hydrogen embrittlement. Instead of galvanizing, require two or three coats of approved zinc rich paint.
  Do not specify for anchor bolts.

  Alloy steel headed structural bolt for use in structural connections.

- *ASTM F1554* grade 105 - Higher strength anchor bolts used for larger sizes (1½ to 4 inch). When used in seismic applications, such as bridge bearings that resist lateral loads, specify supplemental CVN requirement S4 with a test temperature of -20 degree Fahrenheit.

  Lower grades may also be suitable for sign structure foundations. This specification should also be considered for seismic restrainer rods, and may be galvanized.

1.6.5.2 Properties of High-Strength Bolts

Table 1.6.5.2-1 Properties of High Strength Bolts

Material	Bolt Diameter (inch)	Tensile Strength (ksi)	Yield Strength (ksi)
ASTM F3125 GR A325 & GR F1852	½ – 1½	120	92
	Over 1½	--	Not Available
ASTM A449	¼ – 1	120	92
	1⅛ – 1½	105	81
	1¾ – 3	90	58
	Over 3	--	Not Available
ASTM F1554 GR 105	¼ – 3	125 – 150	105
ASTM F1554 GR 55	¼ – 4	75 – 95	55
ASTM F1554 GR 36	¼ – 4	58 – 80	36
ASTM F3125 GR A490 & GR F2280	½ – 1½	150 – 173 (max)	130
	Over 1½	--	Not Available

1.7 RESERVED

(Reserved)

1.8 TIMBER BRIDGE DESIGN AND DETAILING

1.8.1 Timber Bridge Locations

1.8.2 Timber Design and Details

1.8.3 Timber Connections

1.8.4 Timber Rails

1.8.5 Preservative Treatments

1.8.6 Field Installation

1.8.1 Timber Bridge Locations

Timber structures may be considered as an alternate to concrete structures on low volume highways or roads with an AADT of less than 500, especially for sites located away from possible concrete sources. Timber bridges are generally best suited to the drier climate east of the Cascade Mountains. Consult the region's project development team in the early stages of a project to determine whether a timber bridge is desired.

1.8.2 Timber Design and Details

Before specifying structural grades for timber members, check with the fabrication industry for actual availability.

Consider timber dry for design unless submerged.

ODOT does not design composite wood-concrete structures and has no corresponding construction specifications.

For structures carrying only pedestrian or bicycle traffic,

- The maximum allowable live load deflection is:
  - For simple or continuous spans, $Deflection = span/360$
  - For cantilever arms, $Deflection = arm\ length/135$
- Provide adequate slip resistance to meet ADA requirements.

GLUED LAMINATED TIMBER BRIDGES

Glued laminated timber bridge single spans are generally feasible up to 50 feet. To achieve longer spans, consider cantilever techniques. The width of glued laminated beams is generally limited to 10 ¾ inch or less, but 12 ¾ inch, 14 ¾ inch, and 16 ¾ inch widths are available for extra cost.

Consider performance specification for glued laminated timber members. Identifying actual stresses for bending, horizontal shear, etc., is preferred by the fabrication industry instead of specifying an actual glued laminated timber grades.

It is preferred to use a non-interconnected glued laminated timber deck as opposed to an interconnected glued laminated timber deck. A longitudinal timber stiffener under the deck between longitudinal beams for transverse deck bridges may be beneficial for differential deflection control.

- A glued laminated longitudinal deck bridge is a possible solution for short spans (under 25 feet) with a tight freeboard clearance requirement. These deck members could be used in a continuous span arrangement to increase member efficiency.

Include a waterproofing membrane according to *BDM 1.9.3.1.6* when using asphalt concrete pavement.

For smaller timber members, such as posts, rails, etc., specifying solid sawn timber as an option to glued laminated timber may be more cost effective.

Timber substructures are not recommended.

1.8.3 Timber Connections

Use of the **Weyerhaeuser clip** to connect timber decking to timber beams allow for easy fabrication and installation of the timber members.

Steel diaphragm beams, as opposed to timber diaphragm beams, between longitudinal glued laminated timber beams are recommended.

Use slotted holes whenever possible in the steel connectors to allow for shrinkage and expansion of the wood, and for construction tolerances.

1.8.4 Timber Rails

A crash-tested rail has been completed for a longitudinal glued laminated timber deck bridge. Several other glued laminated timber bridge configurations will be crash-tested in the near future. Thrie beam railing can be used as an alternate in lieu of timber.

1.8.5 Preservative Treatments

Pentachlorophenol Type A (heavy solvent) or Pentachlorophenol Type C (light solvent) is recommended for most locations as a preservative treatment.

Eliminate all field cuts and bores when possible. Treat any field modifications with copper napthanate.

1.8.6 Field Installation

Shop assembly of the timber bridge components immediately after fabrication is recommended to eliminate any possible future field installation problems, especially on more complicated projects.

Field staking of the structure before fabrication is recommended to eliminate any future installation problems.

1.9 DECKS AND SLABS

1.9.1 Design Considerations

Consider the site-specific conditions when designing both new and existing decks. These can include but are not limited to:

- Rutting.
- Mobility.
- Truck traffic volumes.
- Weather conditions.
- Future and adjacent projects.
- Chloride contamination levels, etc.

Actions to treat a deck could range from district maintenance tasks to MBM funded tasks associated with large STIP projects.

C1.9.1

ODOT spends most of our maintenance dollars on bridge decks and we should take measures to protect them. Research is continually being performed on ways to construct, maintain and replace bridge decks. As a result this section is dynamic.

In an effort to get to longer service life of decks, a silane treatment is applied to all decks and structural overlays 12 months after casting. Refer to the *Bridge Maintenance Design Manual* for information on silane and epoxy treatments.

1.9.2 Deck Systems

1.9.2.1 Deck Slabs

Ensure project specific deck design conforms to the following minimum requirements:

- *AASHTO LRFD Bridge Design Specifications Section 4.6.2.1*.
- Concrete Class per *BDM Table 1.5.1-1*.
- Provide wearing surface specified in *BDM 1.3.2.1*.

For bare concrete decks, discount ½ inch deck thickness when calculating composite properties for girder/slab systems, but include as a non-composite dead load.

HPC decks must be cast-in-place, unless full-depth precast panels are used with either longitudinal post-tensioning or ultra-high performance concrete closures.

C1.9.2.1

For additional information on concrete type for CIP bridge decks see *BDM 1.5.1*. For additional deck requirements on precast prestressed elements, see *BDM 1.5.6.1*.

For a typical 8 inch deck, consider 7 ½ inches structural and ½ inch a sacrificial wearing surface and included as non-composite dead load.

See *BDM 1.9.3.1* for structural capacity of deck overlays and *BDM 1.3.2.1* for wearing surface thickness and loading.

Do not use FRP deck panels in traffic lanes of bridges. FRP deck panels are allowed on pedestrian structures.

When the opening between parallel bridges is 10 feet wide or less, provide protection between the opening between structures. Protection can be achieved with protective fencing or pedestrian rail. Protection must be a minimum 54 inches in height.

Do not use ACP on state-owned structures with bare concrete decks. ACP is allowed on new decks on local agency projects. When an ACP is used see *BDM 1.9.3.1.6* for requirements.

Installations of thermal ice-melting equipment on bridge decks or problematic sections of highway require a design deviation. Contact the Bridge Preservation and Design Engineer for further information.

It is not recommended to install thermal equipment unless there is a natural source of warm water at the bridge, as exists in the city of Klamath Falls. All other installations in Oregon have been turned off due to overly expensive power bills or early failure of key components.

Limit the use of stay-in-place forms for decks as required in *BDM 1.5.6.2.1* and *1.5.7.12*.

1.9.2.1.1 Skewed Decks

For skewed decks, orient transverse bars according to *LRFD 9.7.1.3*.

LRFD 9.7.1.3 provides no guidance when skew angle exceeds 25 degrees.

C1.9.2.1.1

Per *LRFD 9.7.1.3*, the primary reinforcement may be orientated along the skew for skew angles that do not exceed 25 degrees, where the skew angle is measured from a line that is perpendicular to the centerline of the bridge to the centerline of the support.

The acute corners of a skewed (greater than 25 degrees) concrete deck slab are often difficult to adequately reinforce. As the angle of skew increases, large portions of the deck can be unreinforced and therefore subject to spalling and chipping, as shown in *Figure 1.9.2.1.1-1*. Because the orthogonal bars are too short to develop, it is typically necessary

to detail diagonal bars that extend into the deck over the girders, to carry the deck overhang loads. Similarly, acute corners in concrete barriers are also difficult to reinforce, and require special consideration.

When skews exceed 25 degrees, use breakback detailing where the ends of the skewed deck are turned so that the end is normal to the longitudinal edge of the deck, as shown in *Figure 1.9.2.1.1-2*.

This breakback detailing shown in *Figure 1.9.2.1.1-2* effectively eliminates the acute and obtuse corners of the concrete deck and barriers.

Use a minimum breakback width of 3 feet and increase width with increased skew angle.

Figure 1.9.2.1.1-1 Inadequate Reinforcement Provided in Acute Corner of Concrete Deck Slab

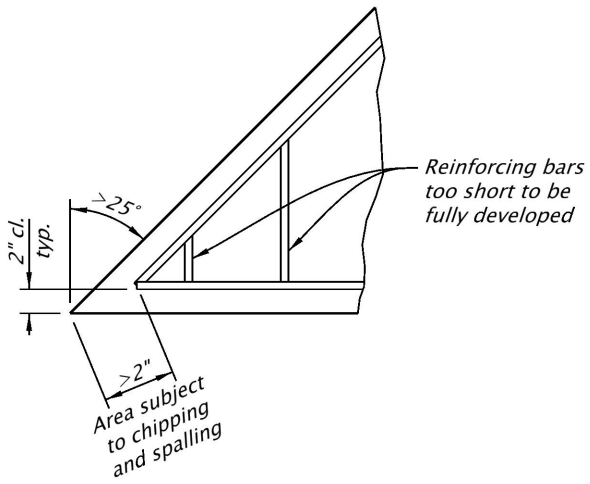
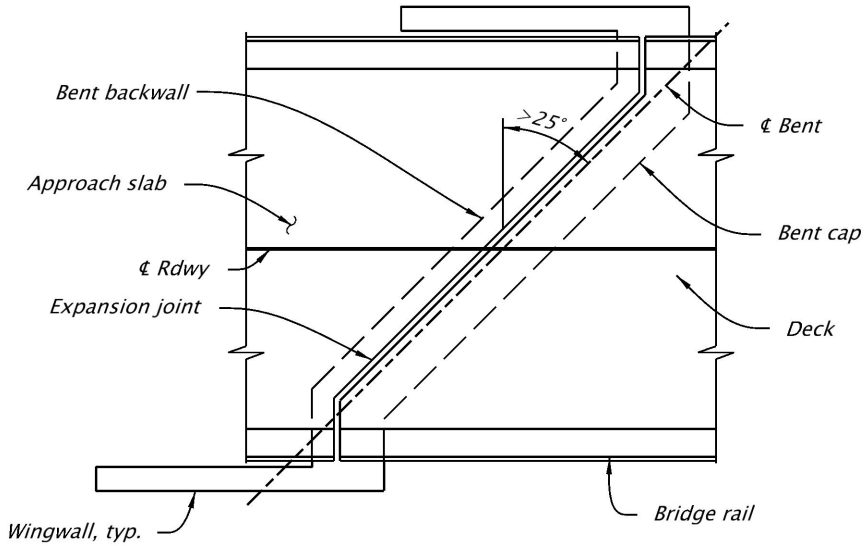


Figure 1.9.2.1.1-2 Breakback Detailing to Eliminate Skew Effects Associated with Detailing Acute Corners at Ends of Skewed Deck Slab



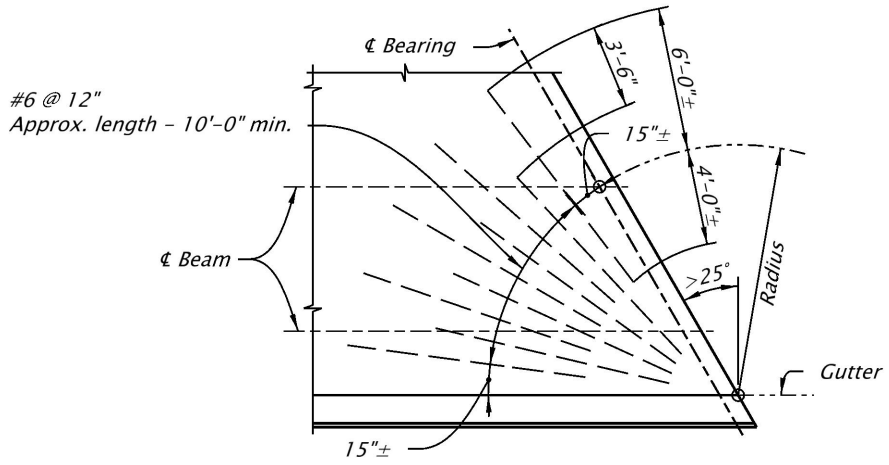
On skewed bridges with concrete end diaphragms and when a breakback detail is not used, place additional reinforcement in a radial manner to eliminate diagonal cracks which form in the acute corners of concrete deck. See *Figure 1.9.2.1.1-3* for the reinforcement pattern.

A portion of the bars must extend back into the corner sufficiently to terminate above the junction of exterior beam and end diaphragm. Place a note on the plans that states:

Place the corner reinforcement beneath the longitudinal and transverse reinforcement in the top of the slab.

The objective of the reinforcement fan is to offset buildup of shrinkage across the long diagonal dimension of the slab which would pull a shrinkage crack across the weak corner of the slab.

Figure 1.9.2.1.1-3 Corner Reinforcing Detailing



In skewed box girders, orient bottom slab transverse bars the same as the deck transverse bars. Note the intended bar placement on the bridge contract plans.

See *BDM 1.5.7.8* for additional bottom slab requirements.

See *BDM 1.9.4.1* for detailing requirements due to screed machines on skewed decks.

1.9.2.1.2 Reinforcing

DESIGN

Do not use the empirical design method for deck reinforcing steel.

Use the *Concrete Deck Reinforcement (LRFD Design)* and *DET3170* for design and detailing decks on steel girders and CIP concrete box girders.

Use *Concrete Deck Reinforcement (LRFD Design)* and *DET3171* for design and detailing decks on precast prestressed concrete girders.

Develop a project specific bridge deck reinforcement design only when the design criteria does not apply. Ensure project specific deck design conforms to the following minimum requirements:

C1.9.2.1.2

Excessive deck cracking, apparently due to under reinforcement, precludes the use of the empirical method.

- *BDM 1.9.2.1.*Reinforcement: grade 60 or grade 80.
- Reinforcement no larger than #6 bar (except in distinct areas).
- Reinforcement spacing ≥ 5 inches and ≤ 8 inches (applies to top mat only).
- Limit top of concrete compressive service stress due to positive moment in the deck (between girders) to $0.4f'c$.

Local agency projects only need to meet minimum *LRFD AASHTO 4.6.2.1* design criteria.

Submit a design deviation request for any concrete bridge deck reinforcement design not meeting any one of the minimum requirements listed above and in *DET3170* or *DET3171*.

Include the following with the design deviation request:

- Design loading assumptions (dead, live, and future wearing surface).
- Documentation of which minimum requirements were met and which were not met.
- Orientation of the top mat (longitudinal on top or transverse on top).
- Deck thickness.
- Maximum service stress in the top of the deck due to positive moment in the deck (between girders).
- Maximum service stress in the bottom of the deck due to negative moment in the deck (over a girder).

LRFD 5.7.3.4 (*Control of Cracking by Distribution of Reinforcement*) is applicable for negative moment steel for bridges made continuous for live load but is not applicable to bridge deck slab reinforcement.

The 8 inch maximum bar spacing is adequate to control cracking in bridge decks.

PLACEMENT

Orient the top mat of deck steel with the transverse bars on top when the direction of primary loading is transverse. For typical deck steel placed in two mats, place bottom mat bars such that each bottom mat bar is directly below and in line with a top mat bar. At deck expansion joints and at deck construction joints, it is not necessary for all bottom bars to be directly below a top bar.

Reinforcing shall conform to *BDM 1.5.5*.

When necessary, you may use larger than a #6 bar for deck reinforcement in distinct sections such as joints, post tensioned anchorage areas and to resist negative moment for continuous-span girders. When the top mat has longitudinal bars on top, place any longitudinal reinforcement larger than a #6 bar in the bottom mat.

TYPE

When high strength reinforcement (higher than grade 60) is used in a bridge deck, use it for both longitudinal and transverse bars. See *BDM 1.5.5.15.4* for application of high strength reinforcement.

Verify the quantity of deck steel to determine if grade 80 rebar is appropriate. When the quantity of deck reinforcement using grade 80 rebar is less than 30 tons, verify rebar availability with the steel mill.

Use of grade 80 rebar is expected to reduce construction cost and potentially reduce rebar congestion.

When the quantity of deck reinforcement using grade 80 rebar is less than 30 tons, consider providing primarily details with grade 80, showing grade 60 rebar as an alternate.

See *BDM 1.5.5.15.1* for additional information on grade 80 reinforcing.

Grade 100 rebar can be considered as well, however the rebar does not provide substantial benefit due to the compressive concrete stress and the 8-inch maximum bar spacing limits.

See *BDM 1.5.5.15.2 and BDM 1.5.5.15.3* for additional information on grade 100 reinforcing.

See *BDM 1.5.5.17* for additional information on FRP reinforcing.

Design GFRP-reinforced concrete according to the latest edition of the *AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete Bridge Decks and Traffic Railings*.

See *BDM 1.5.5.16* for additional information on GFRP reinforcing.

Using GFRP for deck and slab temperature reinforcement is a good example of the complimentary use of this non-ductile material.

It is generally advised that GFRP reinforcement be used in a straight bar configuration. Consider accommodating necessary bar bends (such as bridge rail stirrups embedded in the deck) with stainless steel reinforcement. Do not use GFRP reinforcement in locations where ductility is a desirable material property (negative moment reinforcement, etc.).

To meet bridge performance and service life expectations, you are encouraged to use and design alternative materials for concrete reinforcement applications. When proposing to use alternative reinforcements in environments not already required by *Table 1.9.2.1.3*, obtain concurrence from the BDM technical resource before the draft DAP milestone via the DCSA.

There are a variety of reinforced concrete applications that may benefit from the use of alternative reinforcing materials. Common benefits include corrosion resistance, high strength, ductility, increased service life and reduced concrete cover for reduced dead load. These benefits are usually compromised by factors that may include material availability, cost, unique material testing and non-ductile material properties.

When multiple metals are specified for reinforcing bars in the same elements, provide electrical isolation between the different rebar metals to avoid galvanic corrosion.

Consider interaction between dissimilar metals and the increased complexity of design and construction when deciding if it is appropriate to use alternative reinforcements. Consider bar support and wire tie materials. Obtain technical input from the Bridge Corrosion Engineer.

1.9.2.1.3 Reinforcement Protection

For protection of existing deck reinforcement see *BDM 1.9.3.2*.

The protection system for deck and approach slab reinforcement is shown in *Table 1.9.2.1.3-1*. Environments are defined in *BDM 1.2.4*.

See *Table 1.5.5-2* for required reinforcement cover for decks.

C1.9.2.1.3

The level of effort required to prevent reinforcing steel corrosion depends mainly on the potential for exposure to a corrosive environment.

Table 1.9.2.1.3-1 Deck and Approach System Reinforcement Protective Practices

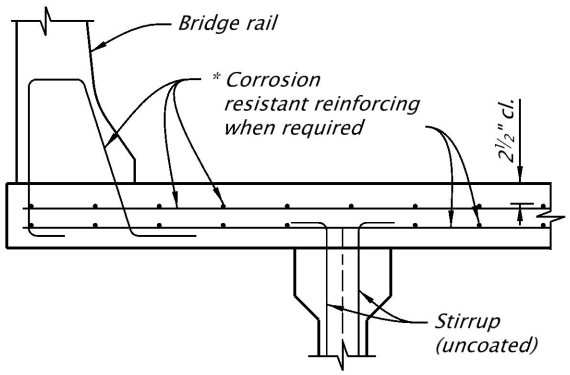
Environment (Defined in BDM 1.2.4)	**Marine Areas**	**Coastal Areas**	**Snow and Ice Areas**	**Mild Areas**
Wearing Surface Type	HPC	HPC	HPC or PPC overlay	HPC or PPC overlay
Reinforcement Type	Deck - stainless steel or GFRP top and bottom mats. Precast Deck Panels - black steel (no epoxy coating) or uncoated prestressing strands (to allow for future cathodic protection if needed). Paved Approach System - black steel (no epoxy coating)	Deck – stainless steel, GFRP or ChromX9100 top and bottom mats. Approach System – black (uncoated) top and bottom mats.	Deck and Approach System - epoxy coated or ChromX4100 top and bottom mats.	Deck and Approach System - black (uncoated) top and bottom mats.

Use the same type of reinforcement as used in the deck or approach slab for reinforcing steel extending out of the deck or approach slab into bridge rails, curbs or sidewalks. This applies to both CIP and precast deck elements. Use black (uncoated) steel for all other bridge rail, curb or sidewalk reinforcement. Stirrups having a portion extending above the top surface of precast concrete members do not need to be corrosion resistant. The portion of the stirrups is covered by the deck concrete.

Examples are shown in *Figures 1.9.2.1.3-1* and *1.9.2.1.3-2*.

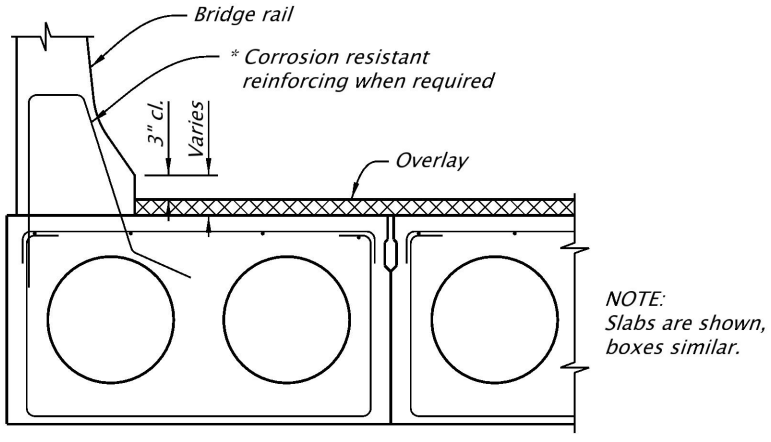
See the standard drawings for other corrosion resistant bars in the precast slabs and box beams.

Figure 1.9.2.1.3-1 Reinforcement Protection for Reinforced Concrete Decks



** See Table 1.9.2.1.3–1 for locations where corrosion resistant reinforcing is required.*

Figure 1.9.2.1.3-2 Reinforcement Protection for side-by-side Precast Slabs and Boxes



** See Table 1.9.2.1.3–1 for locations where Corrosion resistant reinforcing is required.*

MARINE AREAS

Review additional protection measures including concrete sealers, cathodic protection or others with the Bridge Corrosion Engineer on a project-by-project basis.

C MARINE AREAS

There is no evidence of approach slabs failing due to corrosion, hence the use of black steel per *Table 1.9.2.1.3-1*. Consider site specific considerations, such as higher AADT locations, that would make approach slab replacement more difficult. Consider using more corrosion resistant reinforcing at those locations.

The two inch cover required in marine areas, per *Table 1.5.5-2*, is because of the corrosion resistant reinforcing used. Chloride content has to be significantly higher to corrode a stainless, thus the concrete cover can be reduced. GFRP bars do not corrode, and thus concrete cover can be reduced.

1.9.2.1.4 Inlaid Durable Striping on Bridge Decks

Concrete deck surface removal of up to ¼ inch is acceptable for placing longitudinal inlaid striping on new concrete bridge decks, except on decks with an MPCO. Use only sprayed striping on decks with an MPCO.

Placement of such striping will likely reduce wear at stripe locations. In nearly all cases, the majority of wear for concrete bridge decks occurs within the travel lane. Therefore, it is unlikely ¼ inch maximum removal will significantly impact bridge load capacity.

For existing concrete bridge decks, allow deck surface removal of up to ¼ inch for placing inlaid striping only in the longitudinal direction and only in locations where there is no significant rutting or other deck wear.

Do not allow inlaid striping on concrete decks where the striping would be placed in the transverse direction.

Concrete removal for such striping would reduce the load capacity of the bridge.

Allow concrete removal using a diamond grinder according to *SP 00503*.

SP 00503 also permits removal by micro-milling and by hydrodemolition. However, only allow diamond grinding for striping applications. *SP 00850* also requires diamond grinding equipment for

Do not allow rumble strips on concrete bridge decks.

Allow raised pavement markers on concrete bridge decks only when installed without removal of any deck concrete (i.e. no grooving). Do not allow recessed pavement markers on concrete bridge decks.

Allow grooving up to 5/8 inch depth for striping (longitudinal or transverse) or rumble strips on existing asphalt concrete pavement.

Do not install thermally applied pavement markings (i.e. thermal plastic) on an MPCO or PPC overlay.

installation of inlaid/grooved pavement markings.

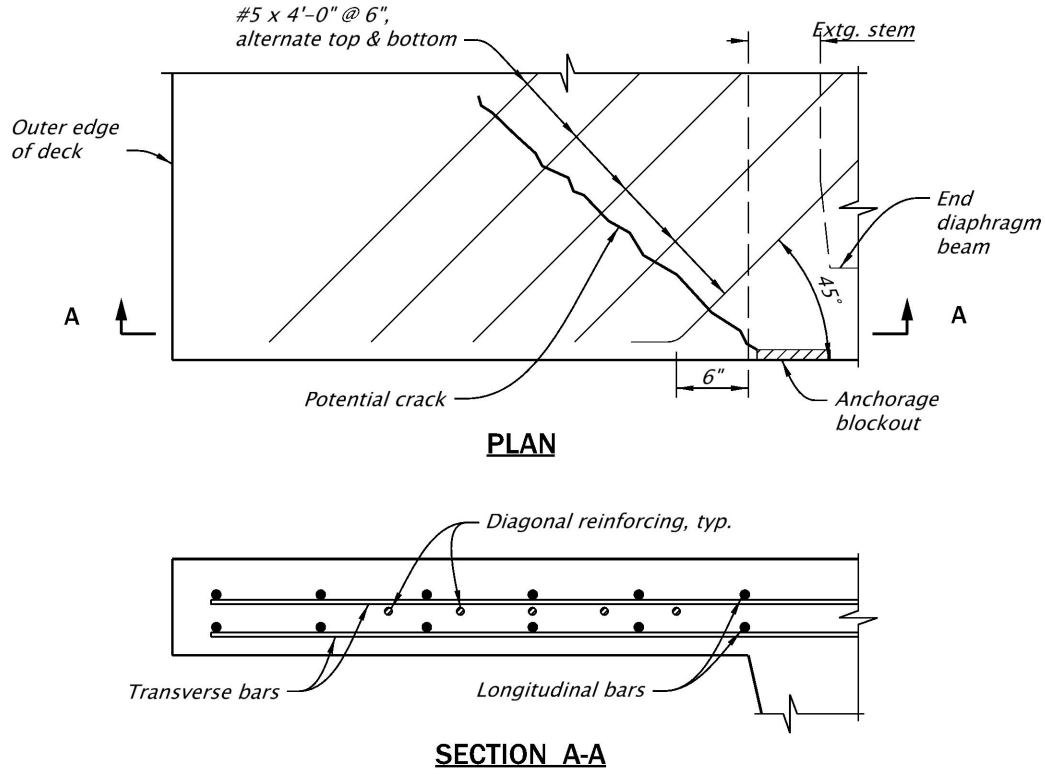
1.9.2.2 Deck Overhangs

1.9.2.2.1 Reinforcement of Deck Overhangs

To prevent cracking at the end of post-tensioned spans, extend the end diaphragm beam to the edge of the deck or provide additional diagonal deck reinforcement similar to shown in *Figure 1.9.2.2.1-1.*

C1.9.2.2.1

Figure 1.9.2.2.1-1 Deck Overhang Reinforcement Detail



1.9.2.2.2 Deck Overhang Capacity

Do not consider bridge railings to be structurally continuous for the purposes of distributing the deck loads per *LRFD 3.6.1.3.4* as this limits options for bridge rail retrofits in the future.

Place deck overhang reinforcement according to *DET3170* and *DET3171*.

C1.9.2.2.2

See *BDM 1.13.5* and *1.13.6.2* for rail and collision loading requirements.

There is crash test report data available upon request to facilitate overhang reinforcing design for *BR208*. Contact the BDM Rail Specialist.

TL5 barrier requires a custom overhang design and is not listed on *DET3170* and *DET3171*. Consider using a diaphragm in the end section instead of additional overhang reinforcement as a more efficient solution when using TL5 barrier.

It is only necessary to strengthen a bridge deck overhang as a result of a rail retrofit or rail replacement when any resulting damage would be within a traffic lane. To determine this, identify the critical sections in the deck. When a critical section is within a traffic lane, evaluate the capacity to demand ratio and design strengthening when less than 60 percent.

1.9.2.2.3 Post-Tensioned Deck Overhangs

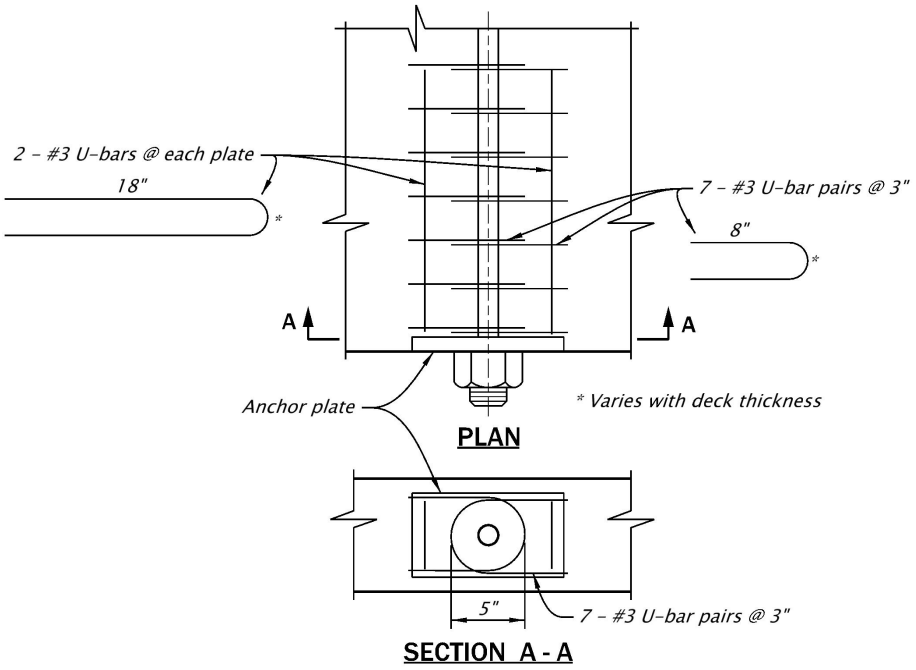
Place post-tensioning ducts and deck reinforcement normal to the centerline of the structure, as shown in *Figure 1.9.2.2.3-1*.

See *BDM 1.13.4* for guidance on when to use wall segment vs end segment in regard to joint placement.

The intent of this policy is to avoid unnecessary deck strengthening for situations that will not cause loss of life or irreparable damage to the bridge. Capacities below 60 percent indicate the possibility of concrete deck cracking. Cracks under live traffic are harder to repair. All other cracks that may occur after a collision can be repaired as part of rail repair.

C1.9.2.2.3

Figure 1.9.2.2.3-1 Transverse Prestress Anchorage



1.9.2.3 Precast Concrete Deck Panels

When determined to be the preferred option, use full depth precast deck panels with high-strength abrasion-resistant concrete for bridge decks. Partial depth precast deck panels are not permitted, except on local agency projects.

Apply a PPC overlay to precast deck panels.

To make deck panels fit on a bridge span, provide either CIP end segments or precast exterior deck panels (end pieces).

Deck panels can accommodate skew, superelevation, slight horizontal curve, and vertical roadway profile. For a mild vertical roadway profile, a flat layout of deck panels constructed on bridge girders is adequate and makes the construction of joint connections easier. When the vertical roadway profile is significant, chorded deck

C1.9.2.3

Bridge deck replacement can be accelerated by using precast deck panels.

Standard details are available for precast concrete deck panels to be used with precast concrete girders and steel girders. The current standard details for precast deck panels limit the maximum deck panel length to 50 feet and the effective width to 10 feet. These dimensions are limited by shipping weight and maximum shipping width (assuming panels are placed flat).

CIP end segments can accommodate construction tolerances, variations, and are a better option for most cases.

For a precast end option, a different panel width may be required. When deck panels are post-tensioned, end pieces may need to be thickened to accommodate post-tensioning anchorages.

Prestressed reinforcement is typically used on the long side of deck panels that span between the bridge girders. This is the main reinforcement that provides flexural strength for resisting applied loads during shipment, erection, superimposed dead load, and vehicular live load. Panel thickness may be increased to accommodate final surface grinding and reinforcement detailing.

panels are recommended to fit the profile with CIP reinforced concrete joints connecting the deck panels. Reinforcement and anchor bolts for bridge railing can be cast into the deck panels.

1.9.2.3.1 Deck Panel Connections

Two types of transverse joint connections along the length of the bridge are:

- CIP reinforced concrete.
- Longitudinal post-tensioning.

CAST-IN-PLACE CONNECTIONS

For CIP joint connections use ultra-high performance concrete (UHPC).

Specify splice length according to FHWA Research, Development, and Technology published *FHWA-HRT-19-011*.

Show exposed aggregate finish at the connection surface on plans.

POST-TENSIONING CONNECTIONS

Use grouted keyway or match cast joints for post-tensioning deck panel connections.

Identify and detail a point of no movement and the direction of the movement due to post-tensioning on the plans. Account for compressive stresses from post-tensioning in positive bending zones in the composite section.

C1.9.2.3.1

Each connection type has its own advantages and disadvantages. A survey of connection details used by various states was published in 2006 as *NCHRP 12-65*.

C CAST-IN-PLACE CONNECTIONS

UHPC has superior bond properties, durability, compressive strength, and tensile strength.

This document provides substantial information regarding design and construction of UHPC. The excellent bond behavior provided by UHPC creates a significantly shorter non-contact splice length for rebar extending out from deck panels than required in conventional concrete.

To ensure good bonding against precast deck panels, pre-wet the interface and an exposed aggregate finish.

C POST-TENSIONING CONNECTIONS

To ensure that camber or deflection of deck panels does not occur after post-tension, place P/T ducts at the CG of the deck panel section.

When there are geometric constraints that prevent locating P/T ducts at the CG of the section, consider the movement after post-tensioning in the design or provide hold-down devices to maintain the deck panel position.

POST-TENSIONED GROUTED KEYWAY CONNECTIONS

Post-tensioned grouted keyway joints include deck panels constructed with shear key edges, erected on girders having a 1 ¾ inch wide space, filled with keyway grout, and then post-tensioned together.

Post-tensioning ducts are spliced in pockets with an air and water-tight seal. Fill all splicing pockets and keyways with grout. Gain specified strength before the post-tensioning operation to ensure continuous flow of the compressive force.

The grouted keyways joints are similar to the joints between precast prestressed slab and box girders.

POST-TENSIONED MATCH CAST CONNECTIONS

Deck panels are match cast at the precaster site. Each deck panel is identified, delivered to the site, and erected in sequence as an erection plan.

Specify duct couplers used in precast segmental construction. With a proper installation, this type of duct couplers provides a continuous, air, and watertight seal. Detail match cast deck panels to fit all possible duct couplers.

In the U.S. a number of P/T suppliers can provide segmental-type duct couplers. Historically only a few suppliers have furnished post-tensioning for bridge construction projects in Oregon. Duct coupler dimensions vary from different suppliers.

Specify epoxy be applied at match cast deck panel interfaces before post-tensioning.

As each match cast deck panel is added, temporary post-tensioning is used to secure the new panel to previously installed panels until the epoxy begins to set. The epoxy serves as a lubricant during placement of the match cast deck panels, prevents water intrusion, and provides some tensile strength across the joint. This construction technique reduces the number of pockets in the finished deck panels.

1.9.2.3.2 Leveling Bolts

Specify location of leveling bolts for erection. Ensure leveling bolts are located on all supporting girders for proper load distribution. Place steel plates on top of precast concrete girders under the leveling bolts to accommodate deck panel erection. Remove leveling bolts or cut down the top 1 ½ inch minimum below the finish surface and leave in place. Fill leveling bolt holes with non-shrink grout.

C1.9.2.3.2

Leveling bolts are used to place deck panels to the appropriate elevation before duct splice openings and joints are filled with concrete or grout.

1.9.2.4 Bridge Approach Systems

A bridge approach system provides a smooth transition between a highway pavement and a bridge. The key components of a bridge approach systems are the following:

Approach slabs and their wearing surfaces. See *BDM 1.9.2.4.1*.

- Abutments and their foundations. See *BDM 1.11.2*.
- Connections between the deck, abutment, and approach slab. See *BDM 1.14*.
- Expansion joints. See *BDM 1.14.2*.
- Approach slab supports (such as ledges and sleeper slabs). See *BDM 1.9.2.4.1 and 1.9.2.4.2*.
- Backfill materials and reinforcement.
- Subsurface and surface drainage systems.

C1.9.2.4

Refer to *NCHRP Synthesis 566* for additional information on bridge approach systems.

Moment slabs are not considered part of the bridge approach system. Refer to the *GDM* for guidance.

1.9.2.4.1 Approach Slabs and Supports

Provide reinforced concrete bridge approach slabs for bridges.

C1.9.2.4.1

Approach slabs were formerly known as bridge end panels in Oregon.

Approach slabs on new bridges may be excluded under unique conditions, which must be documented in a design deviation. Include a geotechnical and structural evaluation as supporting documents to the design deviation.

Refer to *BDM 1.9.2.1.3* for approach slabs corrosion protective practices, such as reinforcement type. Refer to *BDM 1.5.5* for cover requirements.

Show the general outline of approach slabs on the bridge plans with reference to the slab details shown on *Bridge Standard Drawings* or detail plans.

Detail ledges, corbels or other methods of approach slab supports on all bridges (even when approach slabs are not called for when the bridge is built). A design loading for approach slab ledges detailed in *BR165* may be used as follows:

- DC = 1.60 klf.
- DW = 0.35 klf.
- LL = 6.50 klf per lane distributed over 10 feet (includes impact and multiple presence factor).

When the site geometry is non-standard, determine the design loading for approach slab ledges using site specific geometry.

Evaluate existing ledges, corbels and other methods of approach slab supports for required capacities when replacing or adding approach slabs to existing bridges.

For integral and semi-integral end bents where the approach slab movement is used to accommodate thermal expansion:

- Design and provide additional confinement and dowel reinforcement at bearing seat connection.
- Use a sleeper slab per *BDM 1.9.2.4.2* and replace the standard 180-degree

Traffic restrictions may require staging of the approach slab installation or the use of Type III cement (high-early strength concrete) to accelerate construction times.

For bridges with sidewalks and no approach slabs, provide a method of supporting approaching sidewalks at the bridge ends (present or future).

Bridges constructed after 1960 generally have paving ledges at the bridge ends, even though approach slabs were not installed at the time of construction. When adding approach slabs to bridges built prior to 1960, assume addition of approach slab support will be needed.

hook bars with horizontal connection bars.

In most cases, extend the bridge rail to the end of the approach slab.

In cases where the bridge rail is not extended to the end of the approach slab, the required width of the approach slab depends on the approach rail type as follows:

- When the approach rail is a flex-beam rail, provide an approach slab width of inside face to inside face of the flex-beam rails at the end of the bridge.
- Where the approach rail is concrete, support the barrier on the approach slab and provide a slab width equal to the out-to-out dimension of the barriers at the end of the bridge.

Avoid supporting barriers on wingwalls (rail cast with wingwall) because water leaks into the subgrade along the wall. Rather, place barriers on approach slabs or moment slabs.

Use a nominal approach slab length of 20 feet, unless otherwise required due to site-specific settlement concerns.

The measured length of approach slabs is 20 feet 4 inches to simplify rebar fabrication while accommodating two inch cover.

Following the introduction of *DET3160* and the use of granular structural backfill behind the abutment, it is no longer required to use 30 foot long approach slabs for typical bridges.

Where the approach roadway is flexible pavement Use asphalt concrete pavement (ACP) on the approach slab. See *BDM 1.9.3.1.6* for additional information on using ACP.

With flexible pavement, if the approach slab settles, compensating overlays can be easily feathered onto the existing ACP.

Where the approach roadway is rigid pavement, do not use ACP on the approach slab. See *BDM 1.9.2.4.2* for sleeper slab requirements. Work with the pavement designer to modify *DET1604* to the site.

The intent of adding a sleeper slab and joint for rigid pavement is to reduce roadway impact on the bridge.

Evaluate the roadway end of the approach slab for movement. Consider if a larger bridge joint is appropriate to address any anticipated large movement.

When widening a bridge with existing approach slabs, use the same approach slab length for the new portion as the existing. Connect the new approach slab segment to the existing with dowels.

Do not add approach slabs to existing bridges unless there is ongoing settlement or the project impacts the full width of the approach embankment.

On-going settlement is noted in maintenance records or when inspection element 999 (roadway impact) is in condition state three or higher. A site visit may be necessary to determine whether or not adding approach slabs to the structure is the best choice to minimize pavement cracks and settlement at the bridge ends. The largest costs associated with adding approach slabs is often traffic control. When traffic control is already in place, it is often beneficial to add approach slabs.

PRECAST APPROACH SLABS

C PRECAST APPROACH SLABS

Oregon has limited experience with precast approach slabs.

Consider issues regarding subgrade compaction and the contractors' ability to construct the surface of the subgrade to a smooth level condition prior to placement.

Consider the ability of precast slabs to accommodate differential settlement (especially if subgrade is not level)

Consider the design of the connection detail to pile cap/abutment wall and any joint construction.

SKEWED APPROACH SLABS

C SKEWED APPROACH SLABS

See *BDM 1.9.2.1* for reinforcing details in acute corners of approach slabs.

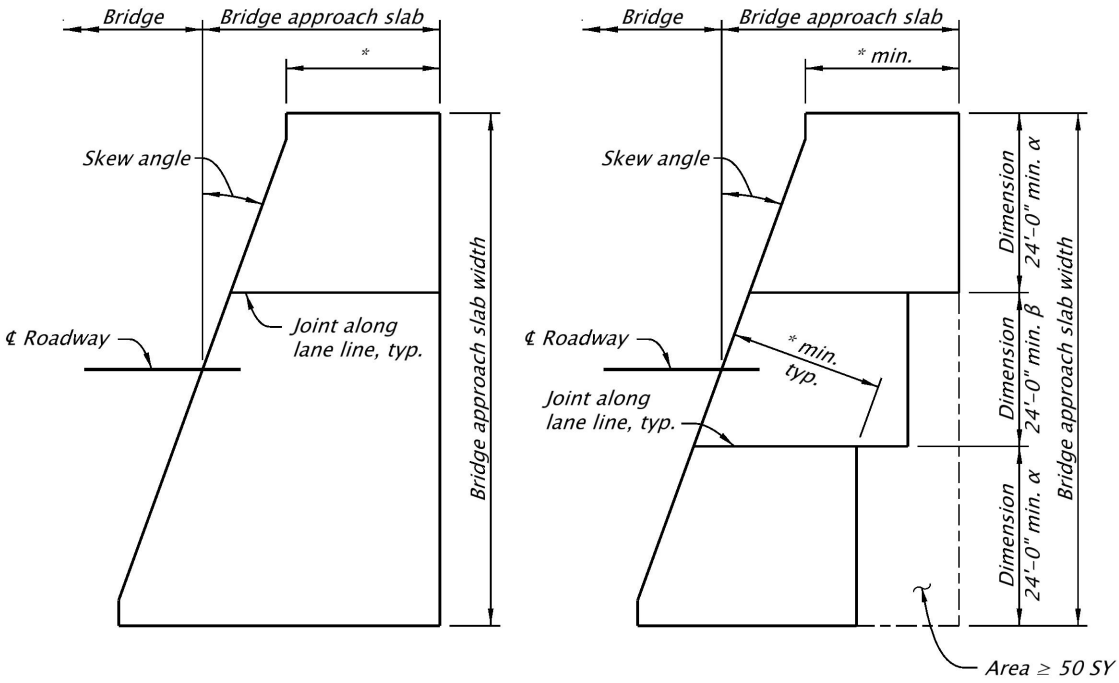
Where the approach slab has flexible pavement, detail the roadway end of the bridge approach slab to parallel the bridge end.

Skews greater than 25 degrees may require modification of the expansion joint. Skews greater than 45 degrees require a unique design for the approach slab reinforcement, see *BDM 1.9.2.1*.

Where the approach slab is rigid pavement, detail the roadway end of the bridge approach slab normal to the roadway centerline.

The roadway end of bridge approach slabs may be stepped to reduce size. A general rule of thumb is that when approach slab area can be reduced by 50 SY or more, consider methods for reducing approach slab area. Provide a minimum 20-foot approach slab length at any point. When stepped, provide the absolute minimum number of steps and locate the longitudinal construction joint(s) on lane lines. See *Figure 1.9.2.4.1-1* for clarification.

Figure 1.9.2.4.1-1 Stepped Approach Slab



SKEWED APPROACH - TYPICAL **SKEWED APPROACH - STEPPED**

α – Dimension may be one lane width plus the shoulder width if the shoulder ≥ 8'-0"

β – Dimension may be two lane widths

** – Minimum Approach Slab Length per 1.9.2.5.1*

1.9.2.4.2 Sleeper Slabs

Provide a sleeper slab at the roadway end of the approach slab when any of the following conditions are met:

- Adjacent roadway pavement is rigid.
- Abutment is integral or semi-integral, and the approach slab slides ½ inch or more.
- Adjacent roadway pavement is flexible and there is no AC on the approach slab.
- When recommended by the Geotechnical Engineer.

Show the general outline of sleeper slabs on the bridge plans with reference to the slab details shown on *Bridge Standard Drawings*.

Regardless of the approach slab width, use a sleeper slab length that matches the roadway width between inside faces of the bridge rails.

C1.9.2.4.2

Sleeper slabs help maintain good ride quality by mitigating rotation of the approach slab and by supporting the roadway end of the approach slab, as well as the joint between the approach slab and the roadway approach.

The approach slab movement is to accommodate for thermal expansion and contraction of the superstructure. Considerations for creep and shrinkage movement depends upon design assumptions.

When an expansion joint is needed at the roadway end of the approach slab and the adjacent roadway pavement is flexible, the stem of the sleeper slab provides the rigid surface needed for the joint.

In combination with proper backfill installation behind the abutment, sleeper slabs provide secondary support for the roadway end of the approach slab to further reduce the risk of slab rotation and provide smooth transition between the roadway pavement and the approach slab.

See *DET 3161*.

This is to accommodate for the first post space of the guardrail.

1.9.3 Existing Decks

Evaluate the structure for the possibility of:

- Bridge rail and bridge rail transition retrofit or replacement.
- Deck joint repair or replacement.
- The addition of reinforced concrete approach slabs.
- The addition of protective fencing (see *BDM 1.13.11*).
- The need for scour protection (see *BDM 1.10.5.8*).
- Seismic retrofit and bearing repair.

For load restricted bridges, confirm that the weight of the construction equipment will not overstress the bridge. Restrictions may be required on the spacing of a paving train or the size of the milling equipment.

1.9.3.1 Deck Overlays

There are three overlay categories available for use on bridge decks:

- Structural Concrete Overlays, *BDM 1.9.3.1.4*.
- Non-Structural Concrete Overlays – Including Multi-Layer Polymer Concrete Overlay (MPCO) and Premixed Polymer Concrete (PPC), *BDM 1.9.3.1.5*.
- Asphalt Concrete Pavement (ACP), *BDM 1.9.3.1.6*.

C1.9.3.1

The purpose of an overlay on a bridge deck can be to:

- Restore the structural integrity of the deck.
- Improve the load capacity.
- Improve or restore rideability.
- Improve skid resistance.
- Improve deck drainage.
- Improve deck cross-section by modifying roadway profile or cross-slope.
- Seal deck cracking.
- Increase reinforcement cover and provide sacrificial wearing surface.

See *BDM 1.15* for more information on maintenance and preservation activities.

Provide a line on the DCSA for all deck treatment selections. Technical concurrence from Structural Materials Engineer is required.

See the *Bridge Maintenance Design Manual* or consult with the Structural Materials Engineer for additional overlay and membrane options not listed in the *BDM*.

1.9.3.1.1 Field Investigation

C1.9.3.1.1

At project kickoff, review the latest Bridge Inspection Report, noting the ratings for the deck, superstructure, bridge rails, deck joints and deck drains.

A site visit may be needed to gather additional information.

Thorough investigation means a delamination survey of the entire deck, chloride profiles taken from areas of highest exposure to drainage and requires concrete cores. Chloride content at the surface is not adequate as levels can vary greatly.

These results are used to determine the remaining concrete deck integrity before determining the appropriate deck treatment or if deck replacement is warranted (see *BDM 1.9.3.4*)

When performing a structural concrete inlay, a reinforcement depth survey is required to determine accurate depth of reinforcement.

A reinforcement depth survey may be good practice for other overlay types as well depending on the chloride depth and deck preparation methods. Contact the Structural Materials Engineer.

A typical reinforcement depth survey may involve a GRP scan correlated with physical measurements.

Use a rebar detector to locate existing deck reinforcement. Avoid coring through existing rebar. Repair core locations with a rapid setting repair mortar from *Section 02015.20* of the *QPL*.

Obtain guidance from the Bridge Corrosion Engineer to verify the coring and testing requirements stated below are appropriate for the structure. Coordinate pavement cores on bridge decks and bridge approach slabs with the Pavement designer at project initiation.

Coordinate pavement cores with Pavement Section and roadway coring request to avoid separate efforts.

Generally the bridge deck program performs concrete cores and pavement section can perform ACP cores.

For chloride testing take minimum 4 inch diameter cores with minimum 4.5 inches long. Take a minimum of four cores at a frequency of a pair of two cores for every 10,000 square feet of bridge deck, not to exceed 12 cores total. For each pair include one core within a wheel track/rut and one core between wheel track/ruts. Avoid locations with existing patches. Take photos of the cores and core holes to assist designer. Log photos and core holes on a pdf of the existing plan sheet. Provide information in Chloride Report or PW as appropriate.

Sampling a single lane of a multilane structure is often sufficient. In rare cases, additional cores may be required for compressive strength testing, other chemicals of interest or to perform petrographic analysis. Cores may not be needed when there is information from previous cores on the same structure, an adjacent structure, or an applicable structure within the same corridor. Contact the Bridge Corrosion Engineer for this information.

Test all cores for chloride analysis according to *ASTM C1152* or *AASHTO T260.* Discard the top ¼ inches and slice cores in ½ inch increments to a depth of 2.75 inches below the surface of the concrete or the bond line between overlay and substrate materials.

Typically, an additional 2 inches below the deepest test is necessary to cut and pulverize the core for testing. For thin decks or thick overlays where 2.5 inch depth testing is not practical, test as many sample depths as possible without taking a full depth core.

For structures with ACP, take core samples to determine ACP thickness. Obtain one core at each approach slab and a minimum of two bridge deck cores. Take one core at the bridge centerline and one core near the curb line when possible. Consider additional cores at joints and mid span to determine existing ACP depths that vary due to camber. Identify any membranes present during coring operations and collect hazmat samples.

It is common for ACP to be used to modify or build the bridge crown so consider taking cores at different locations in the deck profile.

One bridge deck core may be sufficient for bridges less than 20 feet in length. Methods other than deck cores are acceptable to determine ACP thickness only (e.g. drilling, GPR).

1.9.3.1.2 Overlays Warrants

Use the overlay criteria matrix in *Table 1.9.3.1.2-1* to determine whether an overlay is warranted.

Bridge decks that do not meet the matrix criteria are not recommended for an overlay. Consider an alternative action (e.g. maintenance activity, deck replacement, no action).

C1.9.3.1.2

See *BDM 1.9.3.1.1* for investigation requirements and definition of thorough investigation.

See *BDM 1.9.3.2* for corrosion considerations.

See *BDM 1.9.3.4* for deck replacement warrants.

Table 1.9.3.1.2-1 Deck Treatment Warrants Matrix

Deck Preservation Action Guidance			MPCO	PPC Overlay	Structural Concrete Overlay (HPC, HESC)	Structural Concrete Inlay (HPC, HESC)	Install New ACP Overlay With Membrane	Remove ACP & Reinstall ACP with Membrane
Primary Factors	Warrants	Inspection Report Item 58 "Deck Condition"	≥ 4	4-6	4-6	-	≥ 4	≥ 4
		Elements 12, 13, 15, 16, 38, or 39 defect 1080 "spall/delam/patch" in CS2 or worse	1% ≤ deck area ≤ 5%	1% ≤ deck area ≤ 5%	1% ≤ deck area ≤ 5%	*requires thorough investigation per *BDM 1.9.3.1.1*	-	1% ≤ deck area ≤ 5%
		Area identified by delamination survey for class 2 prep (must be less than 2 years old)	deck area ≤ 10% - YES	deck area ≤ 10% - YES	deck area ≤ 15% - YES	*requires thorough investigation per *BDM 1.9.3.1.1*	-	-
		Chloride threshold (* chloride data required) (ǂ chloride data suggested)	< 0.04 % by mass of sample at shallowest rebar *	< 0.04 % by mass of sample at shallowest rebar *	< 0.04 % by mass of sample at shallowest rebar *	Remove concrete contaminated beyond 0.04% by mass of sample.*	< 0.04 % by mass of sample at shallowest rebar ǂ	< 0.04 % by mass of sample at shallowest rebar ǂ
		Roadway classification	Limited use on Interstates	Any	Any	Any	Acceptable in Region 4	Any

Table 1.9.3.1.2-2 Deck Treatment Installation Criteria Matrix

			MPCO	PPC Overlay	Structural Concrete Overlay (HPC, HESC)	Structural Concrete Inlay (HPC, HESC)	Install New ACP Overlay with Membrane	Remove ACP & Reinstall ACP with Membrane
Primary Factors	**Can be installed on:**	CIP deck	YES	YES	HPC - YES HESC - with design deviation	HPC - YES HESC - with design deviation	Requires design deviation	YES
		CIP box/tub/slab	YES	YES	HPC - YES HESC - with design deviation	HPC - YES HESC - with Design Deviation *Removal depth limited by structure	Requires design deviation	YES
		Precast slabs & boxes without a cast in place deck	NO	NO	HPC - YES HESC - with design deviation	NO	Requires design deviation	YES
		Deck bulb-T girders without a cast in place deck	NO	YES with UHPC connections	HPC - YES HESC - with design deviation	NO	Requires design deviation	YES
		Precast deck panels	YES	YES	HPC - YES HESC - with design deviation	HPC - YES HESC - with design deviation *Removal depth may be limited by structure	Requires design deviation	YES

Table 1.9.3.1.2-3 Deck Treatment Material Characteristics Matrix

			MPCO	PPC Overlay	Structural Concrete Overlay (HPC, HESC)	Structural Concrete Inlay (HPC, HESC)	Install New ACP Overlay with Membrane	Remove ACP & Reinstall ACP with Membrane
Secondary Factors	**Material Characteristics**	Minimum thickness (inch)	3/8	¾	≥ 2 minimum	≥ 2 minimum	≥ 2 Varies by membrane type	≥ 2 Varies by membrane type
		Cure time	2-6 hours temperature dependent	2-4 hours temperature dependent	HPC- 7 days HESC- 3 hours	HPC - 7 days HESC - 3 hours	2-6 hours (polymer membrane)	2-6 hours (polymer membrane)
		Minimum closure width	Can be done in single lane phases, placing longitudinal joints on the lane lines.	14 feet for slipform 6 feet for roller screed	16 feet for Deck Finishing Machine 6 feet for roller screed	16 feet for Deck Finishing Machine 6 feet for roller screed	16 feet *can be reduced with extra labor	16 feet *can be reduced with extra labor
		Atmospheric restrictions (degrees F when applicable) Amb. – Ambient temperature Surf. – Surface temperature	visibly dry for ≥ 72 hours 50 ≤ temp ≤ 90 deck temp ≥ dew point+5 No rain	Visibly dry for ≥ 5 days 50 ≤ temp ≤ 90 deck temp ≥ dew point+5 No rain forecasted for 12 hours	40 ≤ Surf. rising ≤ 45 40 ≤ temp ≤ 80	40 ≤ Surf. rising ≤ 45 40 ≤ temp ≤ 80	Varies by membrane type as specified in the specification.	Varies by membrane type as specified in the specification.

				forecasted for placement and cure time	from the start of placement.				
		Provides structural strength		NO	NO	YES	YES	NO	NO
	Cost	2020-2021 $/unit H: High L:Low W: Weighted Average	Materials	SqYd H $33.50 L $15.26 W $19.69	CuYd H $4,015.71 L $2,173.52 W $2,911.16	CuYd H $871.40 L $342.50 W $534.87		CuYd H $695.00 L $258.00 W $390.89 (Converted from Tons to CuYd by assuming 0.5 Cy/Ton)	
			Installation	SqYd H $152.75 L $13.90 W $20.29	SqYd H $155.00 L $26.88 W $42.86	SqYd H $154.97 L $97.00 W $152.69			
Information Only		Common unit weight		Negligible	Refer to *BDM 1.3.2*				
		Surface preparation		According to *00504*					
		Specification number		*00556*	*00557*	*00559*	*00559 & 00540*	*00590 – 00592* *00744 & 00745*	*00590- 00592* *00744 & 00745*

Table 1.9.3.1.2-4 Deck Preparation Matrix

		Micromill	Shot blast	Hydrodemolition	Rotomill
Removal Method Characteristics	Description	Rotating tool that grinds away small thickness of concrete	Steel shot used to impact surface removing concrete	High pressure water jet used to remove unsound concrete	Grinder or cold planer used to remove variable thickness asphalt
	Preparation rate	16,000 sy/day	3,500 sf/hr	Refer to *BDM 1.9.3.3*	72,000 sf/day
	Set up time per shift (minutes) (plus traffic control)	30	30	90	30
	Best used for:	Inlays where removal ≤ 1 inch	Overlays where minimal removal is expected	Inlays where removal > 1 inch Shallow rebar	Asphalt removal
	Can also be used for:	Inlays where removal >1 inch, membrane removal	Secondary process to remove micro fractures	Selective concrete repairs	N/A
	Requires	Shot blast to remove micro fractures	N/A	Water handling	N/A
	Pros and Cons	Efficient removal method		Hydrodemolition is not required for HPC and HESC overlays.	

1.9.3.1.3 Overlay Selection

After determining whether a bridge deck overlay is warranted, use the overlay criteria matrix in *Table 1.9.3.1.2-1* to select an overlay type based on design criteria.

Consider the structure type and secondary factors listed in *Tables 1.9.3.1.2-2, 1.9.3.1.2-3* and *1.9.3.1.2-4.*

During the overlay selection process, review the structure's **As Constructed** plans, paying special attention to the following items:

- Existing bridge rail – Review the existing bridge rail for functional adequacy and replace if unacceptable (see *BDM 1.13*). Verify the dimension from the top of the rail to the overlay finish grade meets the minimum rail height.
- Effect of additional dead load

- Deck joints – When necessary, clean and repair deck joints as part of the overlay project. Review the Bridge Inspection Report or field notes for information to determine if any deck joint work is needed.
  Adjust joints when placing a new overlay exceeding 3/8 inch thickness. Revise joint types when thickness deviates by more than one inch.

C1.9.3.1.3

Check for the presence of an existing overlay or wearing surface. When one is present, note what material type it is. Also, check for the presence of an existing waterproof membrane. Some bridges may have asbestos containing membranes, which require additional testing and care during removal. This information is used in estimating unit costs for deck preparation.

Typically the dead load from a 2 inch concrete overlay has little effect on the capacity or operation of the structure. Exceptions to this are load posted bridges or movable bridges, where a structural concrete overlay's dead load may have a significant impact. A thinner or non-structural concrete overlay may be required.

A site visit may be necessary to determine the best type of joint repair or replacement. See Standard Joint Drawings for typical deck joint reconstruction details. See *Standard Specifications* and *SP 00585* for expansion joints.

- Deck drains – Note existing deck drains on the overlay plan view. Adjust deck drain grates to match the new deck surface when necessary. Deck drains do not require adjustment for MPCOs. Verify if deck drain grates need to be upgraded for bicycle safety.

  For a non-structural concrete overlay, the existing deck drain taper may be adequate. See *BDM 1.9.5* for additional information about bridge drainage.

- Approach slabs – Confirm the need for approach slabs per *BDM 1.9.2.4.1* and by reviewing the current Bridge Inspection Report and the maintenance file records.

  See *BDM 1.9.2.4* and *SP 00545* for additional information about bridge approach slab design.

- Protective fencing - When a structure crosses over a roadway and does not have existing protective screening, consult with the Bridge Standards and Program Engineer during scoping to determine whether screening is appropriate to include with the overlay project.

  This is required since the 1993 law *ORS 366.462*.

  There is a statewide priority list for protective fencing. See *BDM 1.13.11*.

1.9.3.1.4 Structural Concrete Overlays

The term **structural** is used to describe an overlay that is rigid enough and thick enough to increase the stiffness of the deck and decrease live load deflections. Structural concrete inlays are considered **structural** because they restore the stiffness of the deck but they typically do not increase deck strength. Overlay and inlay can be used interchangeable in this section.

See *BDM 1.3.2.1* for wearing surface thickness and loading. Include the structural concrete overlay in the stiffness and capacity calculation.

C1.9.3.1.4

A deck overlay is when a deck treatment is applied on top of the existing deck material. A deck inlay is when parent deck material is removed and replaced with deck treatment material.

Structural overlays typically have a compressive strength and elastic modulus similar to conventional concrete, but it may vary depending on the specific product used.

Do not include the top ½ inch of overlay in the structural deck thickness, since it is considered a sacrificial wearing surface per *BDM 1.9.2.1*.

Structural concrete overlays require elastomeric concrete nosings or armored corners at the bridge ends and joints.

This is because of the high incidence of debonding at expansion joints or at bridge ends. See *SP 00584* for specifications developed for concrete nosing.

It may be possible to place a non-structural concrete overlay and not do any work to the joints.

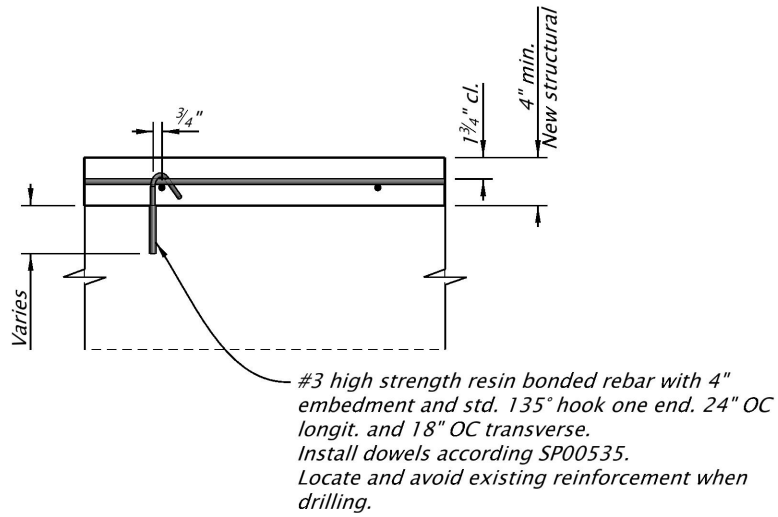
STRUCTURAL CONCRETE OVERLAY DEPTH

Adjust nominal overlay depths according to the following guidelines:

- For depths of 2 to 3 inches use a full depth structural concrete overlay with no added reinforcing.
- For depths between 3 to 4 inches contact the BDM Technical Resource for options to decrease shrinkage.
- For depths 4 inches and greater, provide shear dowels from existing concrete to improve bond according to *Figure 1.9.3.1.4-1*.
- For depths greater than 5 inches, include both shear dowels and temperature reinforcing steel. Dowels are designed for shear loading only.

The required minimum thickness of a structural overlay on a bridge deck is 2 inches per *00559*. Thickness of 1 ½ inches is allowed in certain circumstances (e.g. not able to apply an inlay, cannot increase grade due to bridge rail, vertical clearance, etc.). A 1 ½ inch thick overlay comes with added risk of delamination and shorter service life. Load is better distributed with increased thickness, decreasing delamination and cracking risk. Do not base overlay thickness solely or primarily on cost or volume of materials.

Figure 1.9.3.1.4-1 Shear Dowel Detail



STRUCTURAL CONCRETE OVERLAY TYPES

High performance concrete (HPC) overlays are the preferred material over SFC, often referred to as microsilica concrete (MC). HPC is a similar mix to SFC, but uses larger aggregate, among other changes, to reduce cracking.

Hydrodemolition is generally the preferred method of surface preparation because it creates a better bond. It is not always the appropriate surface preparation method to use. The consistency of hydrodemolition decreases with increased depth. When removing more than one inch, mill first (to with ½ inch of reinforcement) then hydrodemo. Verify the rebar is located low enough to allow this type of removal. When doing hydrodemolition of existing overlays, consider the risk of punch through if the overlay does not debond. Consider the cost of containment needed. It is often not economical to use hydrodemolition for overlays. See MPCO and PCC for additional information. See *BDM 1.9.3.3* for additional information.

SFC is a specialized concrete mix with a silica fume modifier. Batching is normally done at a batch plant. SFC placement is accomplished with more conventional construction methods.

Other structural concrete overlay material include high-early strength concrete (HESC). Specification of HESC as an overlay type requires a design deviation.

Review *SP 00559* for structural concrete overlay requirements and restrictions.

Latex modified concrete (LMC) overlays are no longer used.

LMC is a concrete mix with a latex emulsion modifier. The latex emulsion has a milky color and texture. LMC overlay technology was a common type of structural concrete overlay used since 1958, but has not been used in Oregon since the early 2000s.

1.9.3.1.5 Non-Structural Concrete Overlays

Use non-structural concrete overlays in situations where structural integrity is not an issue and does not add to the deck stiffness.

C1.9.3.1.5

Polymer is a very general term used to classify a wide variety of compounds that chemically combine in a reaction (polymerization).

Polymer binder resins are formulated in hundreds of different combinations, depending upon the properties desired. The most common categories of polymer binder resins in use as bridge deck overlays or patching material include epoxy and polyester.

Polymer concrete is a composite material in which coarse aggregate is bound together with the polymer binder resin.

Polymer concrete can be placed as an overlay in generally two different ways – as a MPCO (also known as broom and seed) or as PPC, which is screed finished to grade.

The most common polymer used for MPCOs is epoxy.

The most common polymer used for PPC overlays is polyester.

Construction advantages of non-structural concrete overlays include:

- Non-structural concrete overlay flexibility reduces the potential for cracking due to thermal or design load movement.
- Non-structural concrete overlays are very light as compared to structural concrete overlays. This reduction in dead load can be significant on load posted bridges or movable bridges.
- Non-structural concrete overlay construction time is much less compared to structural concrete overlay applications. The short construction time provides a great advantage in time critical urban areas.
- Non-structural concrete overlay bond strength is typically double that of a structural concrete overlay.

Construction disadvantages of non-structural concrete overlays include:

- Atmospheric conditions: The prepared deck surface must be dry prior to placement. This provision could influence construction schedules. For off-season applications, *SP 00556* and *SP 00557* provide guidance for inclement weather plans, which can involve heating the bridge deck to force it to dry. This will impact traffic control requirements and costs. Avoid scheduling non-structural concrete overlays in wet weather.

MULTI-LAYER POLYMER CONCRETE OVERLAYS

MPCO's cannot be considered a **structural** concrete overlay.

Place MPCOs to a nominal 3/8 inch thickness. Do not use MPCOs for grade corrections.

MPCOs may be used as a preservation measure on decks in good condition (outside the range shown in *Table 1.9.3.1.2-1*) at the request of maintenance.

MPCO's are a composite material formed by combining polymer binder resin and coarse aggregates. MPCOs have a significantly lower modulus compared to PCC.

MPCO's are constructed using any of the commonly available polymer resins. Each resin has its own advantages and disadvantages. Use each in accordance with manufacturer's recommendations.

Avoid placement of MPCOs on interstates and high traffic volume locations. Consult with the Bridge Maintenance Engineer and Structural Materials Engineer for additional guidance.

MPCO's have been used on the interstate in some locations with reduced lifespan. MPCO aggregates have a tendency to polish in the wheel lines, potentially reducing skid resistance as compared to other overlay types.

The finished MPCO surface is not tined or screeded.

MPCO applications don't require specialized equipment and are well suited for maintenance crews and smaller contractors. There have, however, been advances in application methods which have increased application efficiency vs. traditional manual application methods.

A typical MPCO is constructed by first removing all dirt, debris and laitance on the deck surface. This is best accomplished with the use of a shot-blast system. Since the deck surface must be clean and dry prior to the application of the MPCO, the industry recommends the use of the shot-blasting method. Shot-blasting leaves the surface dry and vacuumed.

A layer of polymer is next applied to the prepared deck using a squeegee, broom,

spray bar or other methods at a rate specified by the manufacturer. The aggregate is then broadcast, at a specified rate, over the surface. The excess aggregate is swept off the surface. Apply lifts according to manufacturer's directions to achieve a nominal 3/8 inch thickness. Place additional MPCO material in ruts to provide a finished MPCO surface that is free of ruts, depressions, and irregularities.

Refer to the *Qualified Products List* for MPCO products that have been approved for use. MPCOs and MPCO aggregates are listed separately on the *QPL*. The MPCO binder manufacture is required to select the appropriate MPCO aggregate from the *QPL*. *SP 00556* covers the use of MPCOs.

PREMIXED POLYMER CONCRETE OVERLAYS

PPC has a significantly lower modulus compared to PCC and therefore cannot be considered a **structural** concrete overlay.

PPC is a composite material formed by combining polymer binder resin and coarse aggregates in a mobile mixer, then applied to the deck and finished to grade. See *BDM 1.9.4.1* for screed information.

PPC may be used as a preservation measure on decks in good condition (outside the range shown in *Table 1.9.3.1.2-1*) at the request of maintenance.

Preliminary numbers indicate a slight advantage over MPCO's in skid resistance.

PPC overlays are typically placed to a nominal ¾ inch thickness.

PPC is more appropriate for minor grade improvements than MPCO's, due to the grade controls of the screed finish machine. Due to the increased material thickness, PPC overlays are more expensive than MPCO's.

See *Table 1.9.3.1.2-1* for estimated application rates. See *BDM 1.3.2* for unit weight of PPC.

PPC is rapid setting and is best placed with a paving machine whenever practical. There are circumstances where a machine cannot

be used, such as along the gutter lines which will require manual finishing.

PPC is constructed by first removing all dirt, debris and laitance on the deck surface using a shot-blast system. Since the deck surface must be clean and dry prior to the application of the PPC mixture, the industry recommends the use of the shot-blasting method over the others. Shot-blasting leaves the surface dry and vacuumed.

A layer of primer is next applied to the prepared deck surface using a squeegee or brooms. Next the polymer resin binder is mixed with the other components into a premixed condition. The premixed material is then placed onto the primed surface and finished to grade with specialized equipment designed for PPC applications. Silica sand is broadcast in areas of high resin content to maintain skid resistance.

The final product looks similar to PCC but with longitudinal tining.

Do not include PPC in projects that do not have a viable construction window based on the weather requirements in *SP 00557*.

Placement of PCC is dry weather work, in line with the paving season. Do not bid a job with anticipated placement in wet weather.

PPC is currently not listed on the *QPL* and needs to be specified accordingly. *SP 00557* covers the use of PPC.

1.9.3.1.6 Asphalt Concrete Overlays

C1.9.3.1.6

ACP is not permitted on new decks or existing bare concrete decks per *BDM 1.9.2.1*, except on local agency projects.

ACP does not add to the deck stiffness and is not considered to be a **structural** concrete overlay.

On bridges with existing ACP and where roadway ACP depths allow, consider replacing the ACP with another overlay type.

When ACP is the only feasible option for overlay, install a waterproofing membrane per *BDM 1.9.3.1.6.1*.

Do not use emulsified asphalt concrete (EAC), cold mix, on bridge decks or slabs. Specify ACP materials per *00744* and *00745* for use on bridge decks and slabs.

Use of commercial grade ACP and vibratory rollers are not permitted on bridge decks or slabs.

Use the same ACP on the bridge except for layer thickness and vibratory compaction as discussed below:

- 2 inch lift or thicker: ½ inch ACP.

The pavement designer assigns an appropriate pavement type, design level, layer thickness, and aggregate size for ACP on the highway and bridge approaches. Consider impacts to the approach roadway when adjusting ACP thickness on bridge decks. Reducing pavement thickness can cause rebuilding of roadway approaches.

- 1 inch lift but thinner than 2 inch lift: 3/8 inch ACP.

3/8 inch ACP is only cost effective when used in substantial quantity and when multiple changes in paving operations are not required by the change in the mix.

- Lift thinner than 1 inch: not allowed.

When a Class **F** mix (free draining) is used, give special attention to drainage details at joints and deck drains to prevent trapping water adjacent to these areas.

Matching the wearing course (top lift) thickness used beyond the bridge ends may improve asphalt durability on bridge decks, if the project is staged such that the wearing course on the pavement and bridge can be paved in the same operation.

Discuss pavement mix design recommendations with the Pavement designer to address lateral shoving on bridges with superelevations of five percent or greater.

Less than four inches of ACP has demonstrated lateral shoving on structures with superelevations of five percent or greater. The issue is exacerbated on high speed roadways (over 40 mph) and where chains and studded tires are used. Consider higher quality asphalt or a different overlay material in these situations.

Provide details on plans to pave out cambers in the bridge deck. Provide ACP quantities to the roadway designer, including additional ACP to pave out cambers.

ACP thickness can vary by several inches due to camber.

See *BDM 1.9.2.1.4* for striping information on ACP.

See *BDM 1.9.3.1.1* for field investigation requirements.

See *BDM 1.14.2.2* for control joints and asphalt paving on bridge joints.

1.9.3.1.6.1 WATERPROOFING MEMBRANES

Waterproofing membranes are required when paving a CIP bridge deck or a timber deck with ACP. For bridge decks with adjacent slabs and minimum ACP thickness less than four inches, waterproofing membranes are encouraged but not required.

Do not use spray membranes.

Polymer waterproofing membranes are specified in *SP 00590* and are selected from the *ODOT Qualified Products List*. They are primarily applied to bare concrete decks.

When performing ACP removal, maintain 1 inch minimum ACP over an existing polymer membrane with approval of pavement designer. Otherwise remove ACP to bare deck.

Rolled membranes are specified by *SP 00592* and accepted according to *SP 00592.10*. There is not currently a rolled membrane section in the *QPL*. Place rolled membranes between layers of ACP according to *SP 00592*.

Waterproofing membrane types change over time based on industry research. Consult with the Structural Materials Engineer to select the correct membrane type for each structure.

C1.9.3.1.6.1

Prior to 2002, FHWA requirements for State owned bridges stated that If deicing salts may be used in the future, some type of deck protection shall be used.

Waterproofing membranes are used as part of an overall deck protection concept to increase service life.

Membranes serve the following purposes:

- Protect reinforcing steel in concrete members from corrosion by preventing moisture from roadway runoff (which potentially contains chlorides and other contaminates) from penetrating the concrete surface.
- Protect galvanized tie rods in precast prestressed concrete members placed side-by-side from roadway runoff.
- Protect timber bridge decks from moisture damage.
- Prevent untreated roadway runoff from passing through bridge elements to a roadway, waterway, bikeway or pedestrian way underneath the bridge.

EXISTING STATE BRIDGES

All areas of Oregon potentially use deicing chemicals. Actual use of deicing chemicals in the project area can be verified by contacting the Region District Maintenance Manager.

Perform chloride testing on the existing bridge deck when warranted per *BDM 1.9.3.1.2* to verify existing chloride thresholds are acceptable.

Do not skip bridges during roadway paving operations. Remove all ACP and existing membranes during paving projects, except when the existing membrane is a spray or polymer membrane that is performing well. When possible, protect these types of membranes in place during paving operations.

LOCAL AGENCY PROJECTS

A deck protection system is required for projects with Federal Funds, otherwise it is desirable and should be investigated on each project, whether NHS or non-NHS.

When a local agency chooses not to use a cast-in-place deck or ACP with a waterproofing membrane for side-by-side construction, obtain written confirmation from the local agency. Include a copy of the local agency confirmation in the calculation book. Also confirm with the project environmental coordinator whether elimination of a membrane is acceptable when there is potential for roadway runoff to enter a waterway by leakage through adjacent bridge members.

Local agencies can use spray membranes, but it is not recommended due to difficulty meeting the warranty requirements in *SP 00591*.

C EXISTING STATE BRIDGES

Mobilization is a significant cost to bridge paving projects.

1.9.3.2 Corrosion Considerations

Discuss the proposed deck treatment project with the Bridge Corrosion Engineer when any of the following conditions are met. Otherwise follow reinforcement protection design provided in *BDM 1.9.2.1.3*.

- The structure is in a marine area, per *BDM 1.2.4.1*.
- Deck rebar corrosion is visible.
- There is reason to suspect the structure may be occasionally salted during winter months.

C1.9.3.2

When the maximum acceptable chloride level in the deck has been exceeded, deterioration of the deck rebar will continue regardless of the presence of a new overlay.

See *BDM 1.9.3.4* for deck replacements.

1.9.3.3 Partial Depth Deck Removal

Use partial depth deck removal only in conjunction with a structural overlay or inlay (see *BDM 1.9.3.1.4*).

HYDRODEMOLITION METHODS

Use one of the two types of hydrodemolition as specified in *SP 00505*:

- Selective Hydrodemolition
- Deep Cut Hydrodemolition

C1.9.3.3

See *SP 00505* for concrete deck hydrodemolition. Refer to *ICRI Guideline No. 310.3R Guide for the Preparation of Concrete Surfaces for Repair Using Hydrodemolition Methods* for additional information not contained within this manual.

The primary function of this process is to remove chloride contaminated concrete and unsound concrete using hydrodemolition equipment. Hydrodemolition processes also remove microfractures caused by impact-based deck removal equipment and prepares the deck for a structural inlay or overlay.

Selective Hydrodemolition - This process selectively removes all unsound concrete and achieves a roughened profile of approximately ½ inch in areas of sound concrete. When using high early strength concrete overlay for a "fast-track" hydrodemolition project, only selective hydrodemolition can be used.

When operating at pressures ranging from 14,000-20,000 psi, robotic hydrodemolition units selectively remove unsound concrete from the parent deck. It is often more economical to remove unsound concrete with hydrodemolition than it is with chipping hammers.

Selective hydrodemolition limits the amount of removal and debris generated which allows for increased production rates. Deep cut hydrodemolition produces too much debris and cannot be used in conjunction with high early strength concrete overlays for "fast-track" hydrodemolition projects.

Deep Cut Hydrodemolition – This process removes the parent deck material to a specified depth, typically beyond the top mat of reinforcement. This is specified to remove chloride contaminated concrete. The plans and specification should clarify depth, whether it is measured from existing top of deck, or to a depth below the top mat of reinforcement. To facilitate consolidation of concrete, removal to ¾ inch below the top mat should be specified.

Typically, chloride contaminated concrete that exceeds the corrosion threshold of *BDM Table 1.9.3.1.2-1*, should be removed. Reinforcement depth plays a critical role in the final removal depth. In traditionally reinforced decks, removing ¾ inch beyond the top mat will help unsure consolidation and bond of the new overlay material.

This process requires longer duration closure windows than scarification due to the amount of wastewater, debris, and slurry produced.

Consider consequences of undermining the top mat of reinforcement in negative moment areas.

The results of the field investigation (see *BDM 1.9.3.1.1*) will help determine which type of hydrodemolition to specify.

In most situations, selective hydrodemolition is sufficient and comes with significant cost and schedule advantages compared to deep cut hydrodemolition.

MICROMILLING

Micromilling prior to hydrodemolition should be specified on all projects; exceptions are listed in the commentary. The micromilling depth should be specified to minimize the amount of hydrodemolition, while maintaining a minimum of ½ inch clearance from reinforcement to prevent damage to the reinforcement and the milling equipment during milling operations.

Micromilling is important prior to hydrodemolition as it:

- Removes deck surface treatments.
- Results in a more uniform removal depth, helping ensure minimum overlay thickness is achieved.
- Decreases required hydrodemolition pressure by opening pores of the concrete, which helps prevent full depth blow-throughs.
- Reduces amount of slurry and wastewater produced.
- Often more economical than hydrodemolition alone.

When there are existing bridge deck treatments (PPC, MPCO, structural overlay, etc.), the payment for removal of these deck treatments is included in the payment of the micromilling bid item. Account for the thickness of the PPC or structural overlay in the depth of removal. Asphalt Concrete Pavement overlays are removed according to *Section 00503*.

ESTIMATING

For deep cut hydrodemolition, the following removal rates can be used as part of schedule estimates:

- 3,000psi concrete – 50 cuft/hr
- 5,000psi concrete – 30 cuft/hr
- 7,000psi concrete – 20 cuft/hr

Removal rates with hydrodemolition equipment is affected by the depth of removal and the compressive strength of the concrete. These removal rates are approximate, and do not account for the percentage on unsound concrete, which is removed at a faster rate. These values are also per robotic unit, and most systems have the capability to run multiple robotic units in parallel.

Selective hydrodemolition production rates can be estimated to be approximately 1,000 sqft/hour.

These rates do not include cleanup, which can add a significant amount of time depending on removal depth.

Account for ¼ inch overlay material thickness when specifying selective hydrodemolition. The specified depth of deep cut hydrodemolition can be used as the average overlay thickness.

Account for delaminated areas when estimating overlay volume quantities.

ACCOMODATIONS FOR TRAFFIC

Provide protection for traffic adjacent to the hydrodemoliton removal process as follows, in order of preference:

1. Remove traffic from work zone via detour, holding of traffic with flaggers, or similar.
2. Maintain traffic as far from work as feasible and shield traffic from debris with positive protection.

Concrete removal via hydrodemolition results in difficult to contain high pressure water and concrete debris being flown from the work area. It is also possible to unintentionally remove the entire deck thickness in an isolated area, or blow-through, resulting in uncontained water and debris falling below the structure. Therefore, protection from these hazards for the traveling public is required.

Positive containment for protection of traffic typically consists of items like temporary traffic barriers, plywood shielding installed adjacent to the work activity, and under deck work containment systems for blow-through protection.

EXISTING STRUCTURE CONSIDERATIONS

Do not specify hydrodemolition on prestressed members without cast-in-place concrete decks.

Unintentional removal of concrete around critical reinforcing, of prestressed elements, and where the concrete is critical to the stability of the structure can result in damage or failure of the structure or element.

When hydrodemolition is required on cast-in-place post tensioned structures, evaluate the capacity of the structure, using LRFR techniques, assuming the specified concrete removal plus 1 inch. Employ mitigation when the resultant critical rating factor is <1.0.

Concrete removal by hydrodemolition is not easily controlled in regards to removal limits and material types removed. Be aware of the structure type and functionality when detailing the hydrodemolition removal.

Hydrodemolition can damage GFRP reinforcing. Do not perform hydrodemolition on structures that contain GFRP in the top mat of deck reinforcing.

Detail hydrodemolition stopping 1-foot from bridge joints that are to remain in place.

Evaluate overhangs for stability assuming the specified concrete removal plus 1 inch. Employ shoring when required to maintain stability of the overhang.

Critical reinforcement includes negative moment reinforcement located in the deck, prestressing tendons near the top of the element, and similar reinforcing where their continued effectiveness is critical to the stability of the structure.

SPECIAL REPAIR ZONES

Identify in the Special Repair Zones in the contract documents. Add clarity, as needed, to identify what is contributing to the area being designated a Special Repair Zone. Detail the allowable concrete removal limits and removal methods withing the Special Repair Zones.

Special Repair Zones are areas that contain critical reinforcement or concrete whose removal could lead to damage or failure of the element or structure.

When concrete removal is required in Special Repair Zones, ensure stability of the structure during construction.

Ensure stability of the structure during construction by employing techniques such as:

- Staged concrete removal.
- Staged concrete replacement.
- Strengthening prior to concrete removal.
- Temporary shoring of structure or element.

1.9.3.4 Deck Replacements

BDM 1.9.3.4 applies to composite concrete deck replacements.

Consider a deck replacement when any of the following warrants are met:

- The deck condition is rated as a four or less in Item 58 of the Bridge Inspection Report.
- Delaminated and patched areas are greater than 15 percent of the deck area.
- 10 percent or more of the bridge deck area will require full depth repairs during hydrodemolition.

- Cracking represented in Condition State 3 is greater than 50 percent of the deck area.
- The chloride content exceeds 0.040 percent by mass of sample at the depth of shallowest rebar. See *BDM 1.9.4.6*.
- Additional investigation confirms that the deck deterioration has become too severe to repair and exceeds values in *Table 1.9.3.1.2-1*.

When superstructure strengthening is required, include a superstructure or bridge replacement option in the alternatives analysis, as well as a deck replacement.

Perform a pre-design load rating per *BDM 2.10.8.1* to verify existing structure meets strength requirements of *BDM 1.3.3.4* when there is not a current load rating.

C1.9.3.4

See *BDM 1.9.3.1.2* and *Table 1.9.3.1.2-1* for deck treatment warrants.

For estimating purposes, the combination of soffit map cracking and heavy efflorescence is a strong indication of locations that may blow through during hydrodemolition and will require full depth repairs.

When extensive strengthening or seismic retrofit is necessary, consider whether a bridge replacement is more feasible. Coordination with bridge program is needed to determine the appropriate course of action.

Life cycle costs analysis is an effective tool for evaluating project scope. Incorporate the design considerations from *BDM 1.9.1.*

Deck replacements fall under the Bridge Modernization design category. In the spirit of practical design, perform a holistic review of the structure from a service life perspective when a bridge deck replacement is being scoped. Opportunities often exist to minimize future maintenance costs and efficiently extend the service life of other bridge elements including, but not limited to:

- Bearing upgrades.
- Omitting joints.
- Adding approach slabs.
- Incorporating seismic retrofit components (e.g., installing diaphragms or shear lugs).
- Repairing existing paint systems.
- Washing structures.

Coordination with other disciplines is necessary to set adequate project limits and consider impacts to roadway drainage.

Design a deck replacement according to *BDM 1.9.2.* Ensure superstructure elements meet the strength requirements of *BDM 1.3.3.4*.

Consider that the bridge deck elevation will likely rise to accommodate various features of a new deck (i.e., additional deck thickness, build-up for constructability, etc.) and new geometry will increase dead load.

Install approach slabs per *BDM 1.9.2.4.1*.

During project initiation, obtain cores from concrete girders for compressive strength tests. Use this information (f'c and calculated modulus) to inform estimated deflection and profile grade calculations.

Consider remaining service life of bridge when selecting reinforcement materials. This may not align with the requirements of *BDM 1.9.2.1.3.* See *BDM 1.9.3.2* for additional guidance.

Design for all stages of construction loading.

Verify the system is stiff enough and will remain stable without a deck in place. See *BDM 1.3.4* for construction loading. See *NCHRP Report 536* for information on bridge demolition.

Check interface shear between the girder and deck for composite action. Add interface shear reinforcement or welded studs as needed.

Deck removal work is likely to damage existing stirrups/studs. Provide details to address damaged stirrups/studs.

PLANS AND SPECIFICATIONS

C PLANS AND SPECIFICATIONS

Incorporate into *SP 00501* a requirement for contractors to submit a girder repair plan when there is damage.

Incorporate into *SP 00501* a girder damage repair plan for unknown conditions.

Consider during design whether details are required for repair of known girder damage and corrosion.

Require a stamped removal plan in *SP 00501*.

Require a survey of the top of existing girder elevations after removal of the bridge deck. Review survey information prior to final placement of formwork to verify final screed elevations.

Coordinate with Roadway to include a profile gradeline or provide top of deck elevations at discrete points (i.e., typically tenth points along the bridge) in the bridge plans.

Falsework will need to be attached to existing girders in a manner that does not damage reinforcement, prestressing strands or steel elements. The contract should specify that attachment details are subject to Engineer approval.

1.9.4 Construction Considerations

Cure bridge decks according to *BDM 1.5.4.*

1.9.4.1 Deck Screeding

C1.9.4.1

Consider deck constructability issues when specifying deck screeding requirements for both new decks and when installing an overlay.

When the deck width or skewed dimension causes the length of the screed equipment to be excessive (more than 100 feet), the deck may need to be placed in stages with or without a closure pour.

At a minimum, provide a 3-line profile for structural and PPC overlay projects on existing decks.

It is challenging to fit a design surface to the existing surveyed surface. The existing surface is going to vary and equipment will need to accommodate variations. Advanced grade control is typically needed. Consider the minimum constructible thickness of the chosen overlay material and the appropriate construction tolerances. The nominal thickness of the overlay may need to be increased to accommodate these tolerances. Unless other information is available, the minimum overlay thickness is typically two times the aggregate thickness.

On skewed decks, a sharp vertical curve on the structure may cause problems with screeding on the skew.

When a structure has different skews at adjacent bents, base the skew of the screed equipment on the average of the bent skews.

It may be necessary to perform some unique sequencing, such as preloading the deck with plastic concrete far enough ahead of the screed machine to preload the beams to get unison deflections and allow the screed to run normal to the beams.

There are circumstances where a machine cannot be used, such as along the gutter lines which will require manual finishing.

Where staging is shown on the plans, place a longitudinal joint along a longitudinal beam line and not in a wheel line. Consider this in the beam layout.

Consider whether the finishing machine can follow the actual slope of the roadway in one placement. Place longitudinal joints along a longitudinal beam and consider this in the beam layout.

Deck screeds can accommodate a crown section in one placement, full width, when the superelevation remains constant. When the superelevation rates vary, the deck will normally need to be placed in separate placements.

When a structure is curved with radial bents, the screed equipment and deck placement remains normal to the roadway centerline. In this case, the screed equipment must be equipped with variable speed capacity at both ends.

Perform sufficient geometric calculations to determine the best method or direction of deck screeding. When necessary, place the required sequencing and direction of screeding, skewed or normal, on the detail plans.

1.9.4.1.1 Beams not Supported by Falsework

When the structure has a skew, add a note to the plans specifying that the screed equipment shall run parallel to the bents, on a skew.

1.9.4.1.2 Falsework Supported Beams

Ideally it would be best to place and screed skewed decks on the skew, but practically it is not required.

1.9.4.2 Deck Closure Pours

The use of polypropylene fibers are required in all portions of the deck, including the deck closures. On local agency projects polypropylene fibers are recommended but not required.

C 1.9.4.1.1

The main concern of this type of placement is that the beams deflect equally in unison, so deck thickness and clearances end up as shown on the plans. To deflect equally the beams need to be loaded equally.

C1.9.4.1.2

There is less concern regarding how the concrete is placed for falsework supported beams. There will still be a small amount of falsework crush due to the added dead load of the deck.

C1.9.4.2

Where deck closures are placed under traffic, minor cracking within the closure can be expected. This cracking is typically minor and does not result in significant long-term maintenance. The amount of cracking expected will be a function of the traffic induced vibrations at the site. See *BDM 1.9.4.4* for a discussion on vibration levels.

See *BDM 1.9.4.3* for discussion of closure pour options.

See *BDM 1.5.6.2.5* for discussion of precast beam bridges.

See *BDM 1.5.7.11* for discussion of CIP superstructures.

See *BDM 1.6.2.18* for discussion of steel structure widening.

See *BDM 1.11.2.5* for discussion of integral abutments.

1.9.4.3 Deck Pour Sequence

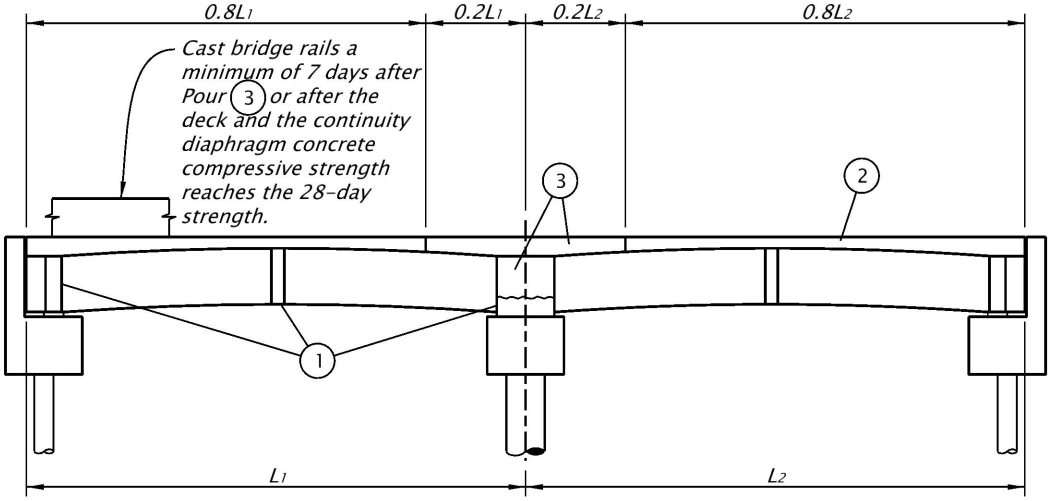
1.9.4.3.1 Prestressed Concrete Girder Bridges

Place decks on precast prestressed beams no less than 30 days after prestress transfer. For bridges designed simple for dead loads made continuous for live loads, pour the bottom portion of the continuity diaphragm 30 days minimum after prestress transfer. Cast bridge deck seven days minimum after the diaphragm concrete pour. Cast top portion of the continuity diaphragm and the remaining bridge deck a minimum of seven days after the previous bridge deck cast. The pour sequence of two spans designed simple for dead loads made continuous for live loads is shown in *Figure 1.9.4.3.1-1*.

C1.9.4.3.1

The 30 day wait is to allow a majority of the prestress camber to occur, thus enabling more accurate determination of beam build-up for the deck screeding and to minimize restrain moments due to girder creep and shrinkage.

Figure 1.9.4.3.1-1 Deck Pour Sequence – Prestressed Concrete Spans



DECK POUR SEQUENCE - PRESTRESSED CONCRETE GIRDER SPANS

POUR SEQUENCE

1. *Cast end diaphragms, intermediate diaphragms, and bottom portion of continuity diaphragm 30 days minimum after prestress transfer.*
2. *Cast bridge deck a minimum of 7 days after Pour (1).*
3. *Cast top portion of continuity diaphragm and the remaining bridge deck a minimum of 7 days after Pour (2).*

1.9.4.3.2 Steel Girder Bridges

Develop deck pour sequences for continuous steel spans according to the span and deflection characteristics of the particular bridge.

Provide minimum negative flexure slab reinforcement per *LRFD 6.10.1.7* as needed. Set retarding admixture may be required to reduce excessive induced stresses in adjacent spans placed sequentially.

Any changes to the pour sequence during construction must be analyzed by the contractor's engineer to determine any effects on stresses and camber. Complete this review early in the process, because it may affect the beam fabrication.

The following steps are a general rule for pour sequence of continuous steel bridges:

1. Pour (1) consists of all positive moment areas along the bridge which will not cause upward deflection on other span/s. No waiting period is required between these spans.
2. Pour (2) consists of multiple separate placements of all positive moment areas of spans that cause upward deflection on other spans. The wait period between these span placements is a minimum of three days after the last pour (1) ended and reaches 70 percent of final strength. When multiple spans are placed sequentially in the same pour, set retarding admixture may be required to reduce excessive induced stresses in adjacent spans.

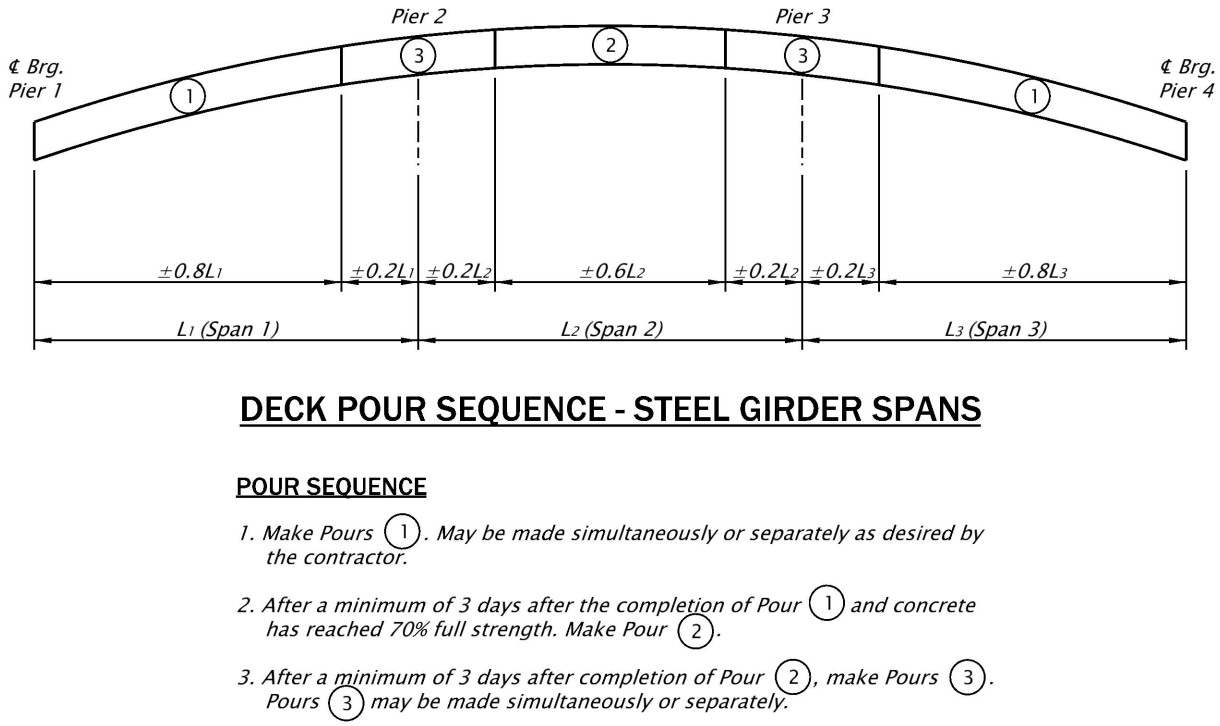
C1.9.4.3.2

The general principal is to first place the sections that are outside of the negative moment zones. Subsequent placements may produce negative flexure in the previously placed sections (See *LRFD C6.10.3.4.1* for commentary).

3. Pour (3) consists of all negative moment areas. The pour can be placed a minimum three days after the last pour (2) ended.

The pour sequence of three span continuous balanced bridges is shown in *Figure 1.9.4.3.2-1*.

Figure 1.9.4.3.2-1 Deck Pour Sequence – Steel Spans



The deck pour sequence for bridges designed continuous for live load consists of two pours. Pour (1) for all positive moment areas except for closure pours. Pour (2) consists of all closure pours at interior and end bents a minimum of three days after pour (1).

1.9.4.4 Vibrations

Although damage to concrete due to vibrations is rare, avoid unnecessary vibrations where reasonable measures can be taken. For staged construction, providing deck closure segments is preferred to minimize both vibrations and the effects of adding additional deck dead load, creep and shrinkage to the first stage.

Providing either a joint or closure segment between substructure (cap) stages will also reduce potential for traffic vibrations to be transmitted through those elements. See BDM *1.11.2.5* for more discussion.

Where there is a concern that vibrations may be excessive, consider the following practices as mitigation:

- Low-slump concrete – Although concrete damage due to vibrations is rare, use of low-slump concrete (greater than 4 inches) will minimize the risk. ODOT's HPC deck concrete mix is generally a low-slump mix that meets this requirement. Therefore, generally no change to the standard HPC deck concrete mix is necessary.
- Reinforcing details – Do not use hooked bars in closure segments. Ensure lap splices are in contact and well-tied as much as possible. Where lap splices cannot be in contact, use two rows of longitudinal bars tied to both lap splice segments to create a rebar mat that cannot be easily moved.

C1.9.4.4

Vibrations from adjacent traffic and construction activity are not likely to cause cracking in freshly placed deck concrete. One ODOT project recorded vibrations up to 0.6 in/sec during a second stage deck placement with only minor deck cracking near the closure area. Typical deck closure placements may have even higher vibrations.

Minor cracking can be expected in deck closures placed under traffic. However, this cracking rarely results in long-term maintenance concerns. See the **Deck Closure Pours** discussion in *BDM 1.9.4.2* for closure pour options.

See *BDM 1.11.2.5* for joint options at integral abutments.

Table 1.9.4.4-1 is a very rough guide to vibration levels.

- Retarder admixture – Varying amounts of set retarder admixture can be used such that the entire deck will set up at about the same time. The Structure Quality Engineer from the ODOT Construction Section can assist in determining when this admixture is needed and how to apply it.
- Reduce vehicle speed – Where vibration is due to adjacent traffic, reducing vehicle speed will generally reduce the amount of vibrations. However, vehicle speeds will generally need to be reduced down to around 15 mph before a significant reduction in vibrations can be obtained. Therefore, only consider this measure in extreme circumstances. Where possible, moving traffic laterally from an adjacent deck placement will likely be more effective than reducing vehicle speed.

Table 1.9.4.4-1 Vibration Levels

Measured Vibration (inch/second)	**Vibration Perception Level**
0.08	vibrations perceptible
0.1	continuous vibrations may begin to annoy people
0.2	short-term vibrations may begin to annoy people
0.4 to 0.6	typical max. short-term vibration, concrete bridge < 100 foot span
0.6 to 1.0	typical max. short-term vibration, concrete bridge > 100 foot span

1.9.4.5 TP&DT / Stage Construction

Discuss traffic control issues early in the project with both region project development team and the Traffic Control designer.

When stage construction is proposed, arrange the stage construction widths so that the overlay can be constructed in widths between 6 feet and 30 feet which are comfortable widths for structural concrete overlay finishing machines and placement of non-structural concrete overlays or ACP. Avoid placing longitudinal construction joints in the wheel paths.

C1.9.4.5

Temporary protection and direction of traffic (TP&DT) requirements are important design considerations and could control project cost, project scheduling and, as a result, the type of overlay. Urban projects or narrow roadway width structures may require very short overlay cure times that could limit the use of a SC overlay.

1.9.4.6 Quantity Estimates

All overlays require the use of *SP 00504*. Bridge decks with existing ACP also require *SP 00503*.

Use any necessary removal bid items, depending on the existing bridge deck surface. All existing overlay removal is measured by square yard. When the ACP is too thick to be removed in one pass (i.e. greater than 2 inches), increase the unit cost for the additional passes required.

Bonded waterproofing membranes, such as polymer membranes, have an additional bid item in *SP 00504* for removal. Fabric membranes are removed as incidental to *SP 00503*.

C1.9.4.6

Once the existing overlay or membrane is removed, the deck is prepared per *SP 00504*.

When there is a known quantity, use a separate bid item. The quantity should be estimated after consulting with Maintenance and the bridge inspector. A site visit may be required. Additional Class 3 Deck Preparation beyond the known quantity can then be paid for as extra work.

A deck survey is required to confirm the estimated quantity of both Class 2 and Class 3 Deck Preparation. Visual inspection and sounding (e.g. chain drag) are the primary methods for determining quantities. Advanced non-destructive evaluation (NDE) methods, such as infrared scanning, impact echo, or ground penetrating radar (GPR) have been used in limited, high-risk, environments. Consult with Structure Services prior to using advanced methods.

Class 1 Preparation takes place in areas where no additional concrete removal is necessary to reach sound concrete. This level of preparation is not measured and payment for it is included in the construction of the new overlay.

Class 2 Preparation is any removal of unsound concrete that does not extend the full depth of the deck. It is measure by the square yard. The repair method and pay item for Class 2 preparation varies depending on the new overlay type.

Class 3 Preparation is any removal that extends through the entire deck. Class 3 Deck Preparation is usually required due to severe deep delaminations, a severely cracked deck in all directions, a badly spalled bottom deck or poor aggregates. In most cases, the quantity of Class 3 Deck Preparation is very small. If so, no bid item is necessary. The work will normally be performed on an extra work basis.

A typical structural overlay for a bridge deck consists of the following structure bid items in addition to any removal items:

- Class 2 preparation for SC Overlay installations (per sy)- *SP 00504.*
- Furnish Concrete Overlay (per cy) – *SP 00559.*
- Construct SC Resurfacing (per sy) – *SP 00559.*
- Saw Cut Texturing (per sy) – *SP 00559.*

Class 1 Deck Preparation, which is not paid or measured separately from the overlay placement, includes roughening the surface to a surface texture depth profile of 1/8 in.

Furnish Concrete Overlay – Calculate this quantity from the Class 1 deck preparation area and a depth of ½ inch greater than the specified minimum depth. This increase accounts for field quantity overruns due to minor grade corrections and irregular Class 1 deck preparation. When Class 2 deck preparation has been identified, add that quantity into the **Furnish Concrete Overlay** total. Work with the roadway designer to confirm that the 3D model, when available, and roadway finish grade profile match the estimated overlay quantities.

Construct Structural Concrete Overlay Resurfacing – This quantity is typically measured for gutter to gutter and end joint to end joint.

A typical non-structural concrete overlay for a bridge deck consists of the following structure bid items in addition to any removal bid items:

- Class 2 Preparation (per sy) – *SP 00504.*
- Furnish MPCO Material (per sy) – *SP*

00556.

Or

- Furnish PPC Material (per cy) – *SP 00557.*
- Construct PC Concrete Overlay (per sy) – *SP 00556* or *SP 00557.*

Deck Preparation - Deck preparation for non-structural concrete overlays is constructed per *SP 00504*. Class 1 Preparation for non-structural concrete overlays requires roughening the existing deck to a 1/16 inch surface texture profile depth. Class 2 Preparation for non-structural concrete overlays includes repairing the deck with a PCC repair material at least five days prior to placing the overlay.

Furnish non-structural concrete overlay – Calculate this quantity from deck area (gutter to gutter and end joint to end joint) and a depth of 1/8 inch greater than the specified nominal depth.

Construct non-structural concrete overlay – Calculate this quantity from the deck area (gutter to gutter and end joint to end joint).

A typical ACP for an existing bridge deck consists of the following bid items:

- Bridge Deck Cold Plane Pavement Removal (per sy) – *SP 00503.*
- Class 2 Preparation (per sy when needed) – *SP 00504.*
- Membrane waterproofing (per sy).
- Asphalt concrete mixture (per ton).

The membrane waterproofing bid item includes full compensation for applying the membrane waterproofing system and the asphalt tack coat.

The asphalt concrete bid item is typically the responsibility of the roadway designer.

Communicate with the roadway designer to make sure all the bid items are covered.

In addition to these bid items, the following items may also be required:

- Deck joints (each or linear feet).
- Deck drain construction (each).
- Bridge rail retrofit or replacement (linear feet).
- Reinforced concrete approach slabs (per sy).

1.9.5 Bridge Drainage

Provide an appropriate drainage system on or adjacent to bridges that have curbs or concrete parapet rails.

Review the roadway plans drainage details. Provide the hydraulics designer the bridge length, deck grades, cross-slope, typical section, and deck surface so the deck drain layout can be determined.

1.9.5.1 Design Considerations

Incorporate features into the bridge deck drainage system to facilitate and minimize maintenance activities. In all cases, coordinate with the project team members representing Environmental and Hydraulics to determine the appropriate bridge drainage system.

C1.9.5

Normally, drainage retrofitting needs to be addressed only when the project involves a major rehabilitation of the bridge. Generally, retrofitting existing bridges from a direct discharge to a piped system is not necessary. Bridge widening normally can use the same type of drainage system as the existing bridge. Contact the Hydraulics Unit for additional guidance about when drains are required.

When drains are required, the Hydraulics Unit does the designs and determines the size and spacing. See the *Hydraulics Design Manual Chapter 12 (Storage Facilities, Chapter 13 Storm Drainage* and *Chapter 14 Water Quality* for additional information.

C1.9.5.1

The primary objective of the bridge deck drainage system is to remove runoff from the bridge deck before it collects in the gutter, shoulder or lane thus exceeding the allowable design spread of water. Drainage and stormwater management design can be

very complex and early coordination is beneficial to the project delivery process.

The benefits of a properly designed and maintained deck drainage system include the following:

- Efficient removal of water from the bridge deck which decreases the risk of hydroplaning and enhances public safety.
- Long-term maintenance and service life of the bridge is improved.
- Preservation of the structural integrity of the bridge.
- Enhanced aesthetics by elimination of superstructure and substructure staining.
- Minimized or eliminated erosion on bridge end slopes.
- Reduced pollutant discharges into streams and wetlands, and compliance with permits and laws that regulate these discharges.

Capture drainage upslope of the bridge in inlets before coming onto the bridge. When grades allow, carry drainage off the bridge to inlets. Drainage not carried to inlets at the ends of the bridge is removed from the bridge deck using drains.

Drains are not allowed to discharge directly into:

- Designated water quality limited streams.
- Streams with severe non-point source pollution problems.
- Streams with populations of listed, proposed or candidate threatened and endangered (T&E) species of fish or other aquatic life.

Special environmental considerations may be required on some projects (see Environmental *BDM 2.14.8*). Hydraulic requirements take precedence over water quality requirements (see Storm Water *BDM 2.14.9*).

In these conditions, provide a piping system that carries the drainage to a storm water control facility (see the Hydraulics Design Manual for additional information). Roadway surface and sidewalk runoff needs to be actively contained and treated because they are considered pollutant sources. When there is a hard barrier separating the sidewalk from the roadway surface, collecting drainage is more complex. In these situations, coordinate with the hydraulic designer to find a solution to treat or mitigate storm water to the maximum extent possible. Coordinate with the hydraulic designer to include sidewalk runoff in drainage calculations. When new sidewalks are added to an existing bridge where storm water previously sheet flowed off the side, contain and treat the runoff from the roadway surface for the new condition.

When the above conditions are not present, direct discharge to the ground below may be allowed. Drainage directly discharged to the ground below is not to cause erosion or be a hazard to the public. To prevent exposure of the superstructure to the drainage, carry it by drainpipes to 3 inches below the bottom of the superstructure.

1.9.5.2 Detailing

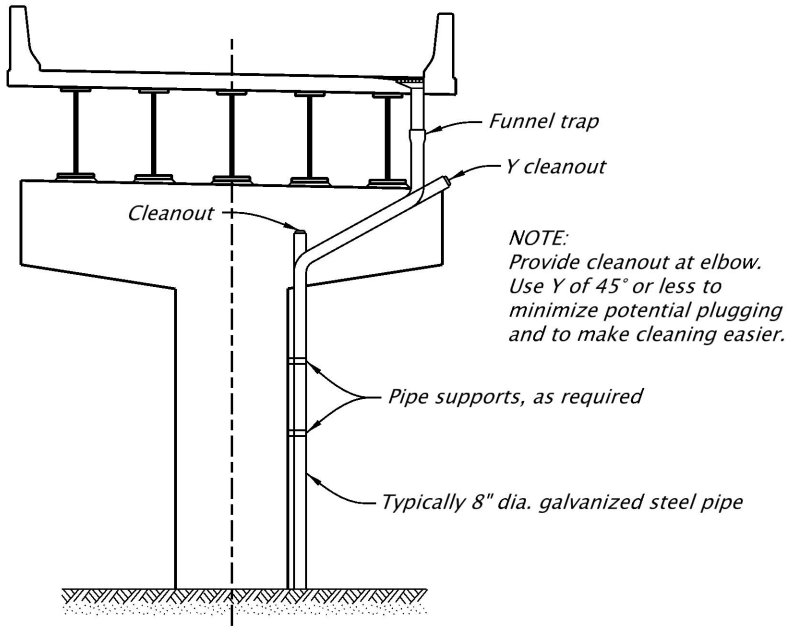
C1.9.5.2

Indicate drain locations on the General Plan and Girder Layout sheets, and appropriate Detail sheets (see *DET 3120* for general details) that are inserted in plans. Show drainpipes and details on abutment, interior bent, deck and girder detail sheets. On viaducts or structures with extensive piping, provide a separate sheet showing entire

drainage layout. The following figures are examples of different applications.

Provide minimum 8 inch diameter galvanized steel drainpipe.

Figure 1.9.5.2-1 Drainpipe Detail



Seismic design requirements for concrete containment within columns precludes placement of drainpipes within columns.

For 90 degree pipe connections, use 2-45 degree connections or a 4 foot minimum radius sweeping 90 degree connection. Add clean-out ports or junction boxes at every 90 degree connection. Clean-outs should be at a 45 degree angle to the main line.

Verify that the gutter profiles do not result in **birdbaths** or unsightly dips in the rail.

Place drains upslope from expansion joints to capture drainage before it reaches the joints.

When long pipe runs are utilized for longer crossings (greater than 100 feet) and bridges

Deck drains and drainpipes become easily clogged and are a continual maintenance problem. High pressure hoses used for cleaning cannot make 90 degree turns.

When there is a question, plot the gutter grade.

with multiple spans, match pipe grade with roadway grade and see the following references for additional guidance:

- *Bridge Crossing with Ductile Iron Pipe, DIPRA*.
- *Guidelines for Bridge Water Pipe Installations*, Raj B, Mark P and Web C.

Provide sufficiently sturdy and properly located pipeline support to accommodate the weight of the pipe, its contents and other applicable loadings such as joint movement, seismic activity, traffic vibrations, insulation, etc.

Include expansion/contraction couplings on plans when required (longer than 100 feet lengths). Locate expansion/contraction couplings in the pipeline adjacent to bridge expansion joints.

The number and location of expansion/contraction couplings, if required, are determined by the length and design of the bridge in consideration of the maximum anticipated temperature differential.

Figure 1.9.5.2-2 Bridge Drainage Plan and Elevation

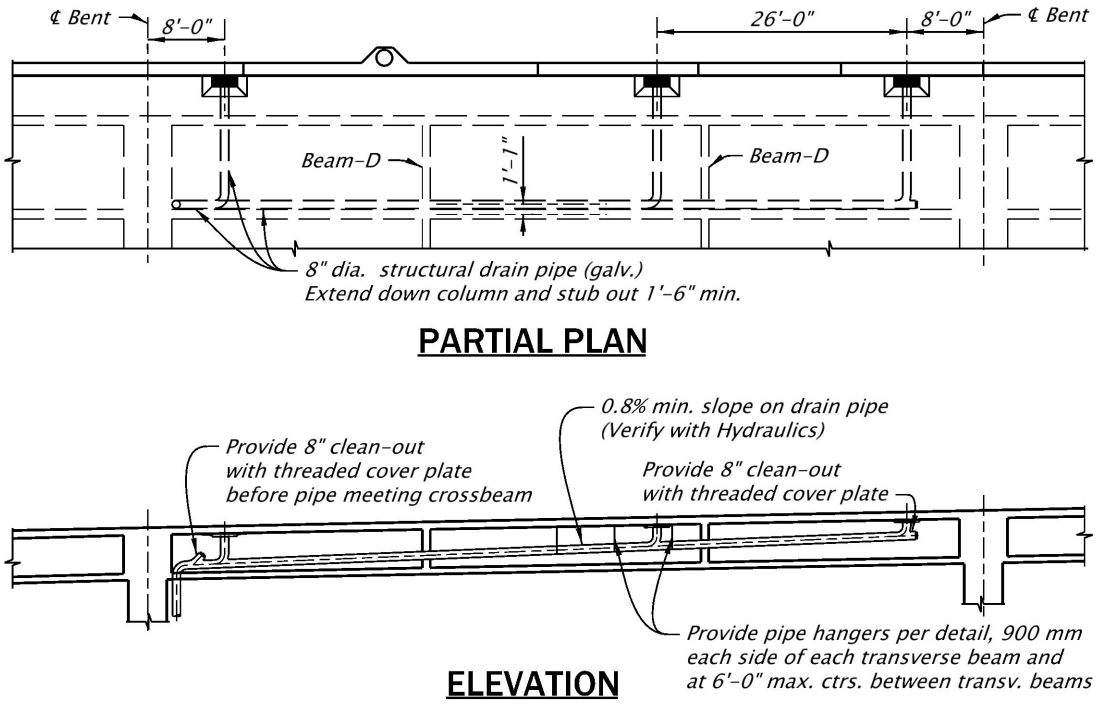
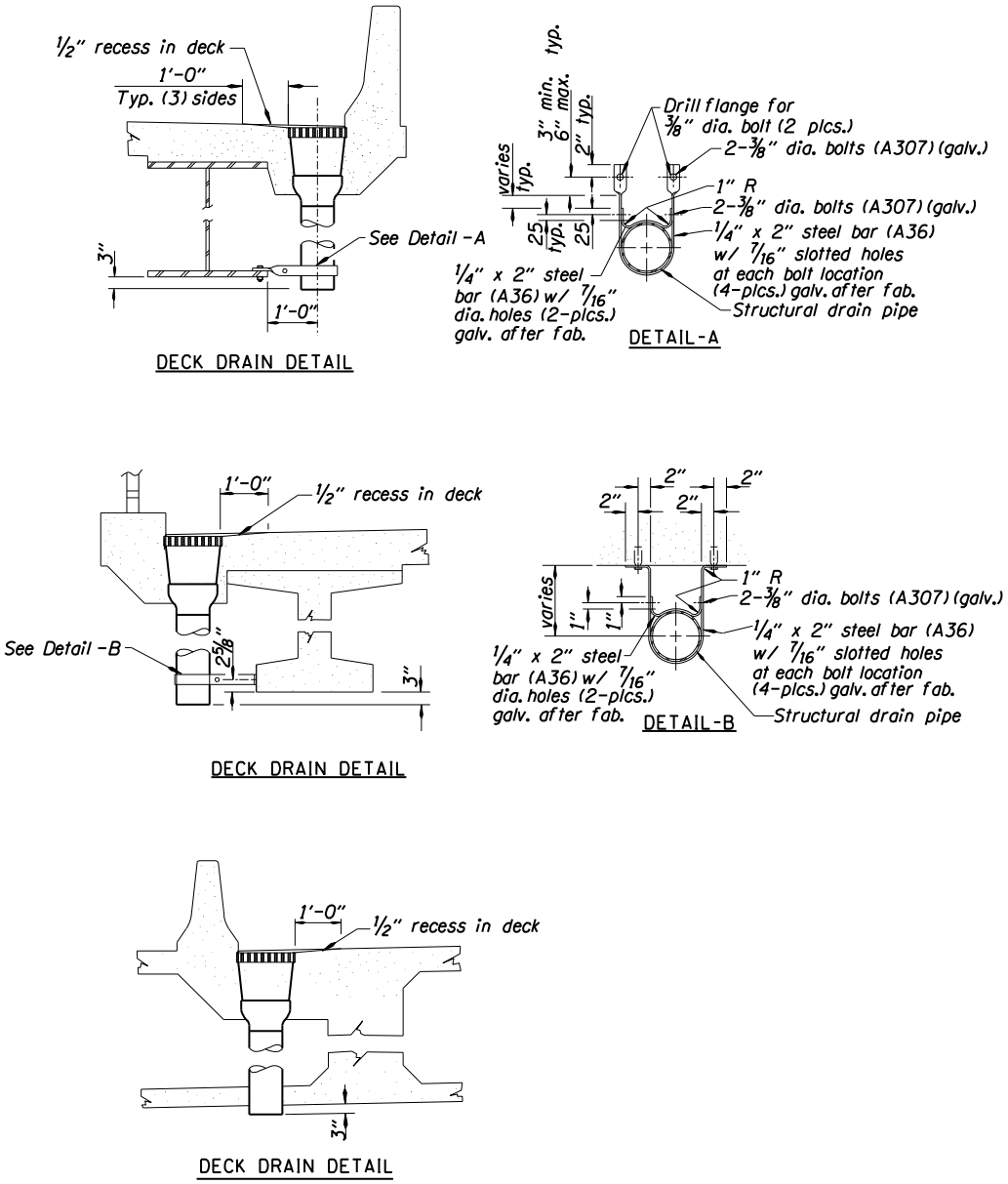


Figure 1.9.5.2-3 Bridge Deck Drain Details



1.10 FOUNDATION CONSIDERATIONS

1.10.1 Foundations, General

1.10.2 Lateral Earth Restraint

1.10.3 Underwater Construction

1.10.4 Foundation Modeling (Foundation Springs)

1.10.5 Foundation Design

1.10.1 Foundations, General

The geotechnical designer will provide data and recommendations with respect to types of footings, footing elevations, nominal and factored resistances, types of piling, pile tip reinforcing, and drilled shaft tip elevations which are to be used at each bridge site. The designer should be satisfied that the recommendations are adequate with respect to factored loads and economy. When there are questions in this matter, they should be discussed with the geotechnical designer. Special factors in the type of construction selected may cause a reconsideration of the original recommendation. Some basic guidelines include:

- When the Geotechnical Report is not available, the fact should be noted and the basis for the design of the footings should be indicated.
- Except for special cases, provide a minimum of 2 feet of cover over the top of spread footings.
- Make the top of footings within the right of way of the Union Pacific Railroad a minimum of 6 feet below the bottom of the low rail to allow for future underground utilities.

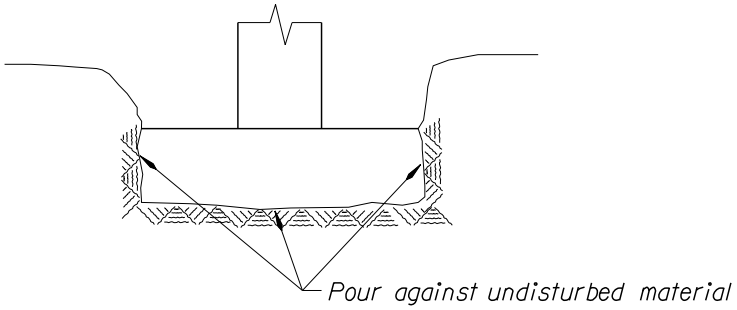
When geotechnical analysis shows that a bridge site will have excessive soil movement due to an earthquake, the bridge and geotechnical designers shall collaborate to determine the potential impact on the bridge performance and the available mitigation methods, which may lead to the need for ground improvement at the project site. See *BDM 1.17.4* for mitigation guidelines.

1.10.2 Lateral Earth Restraint

When passive earth pressures are used in design to resist seismic or other lateral loads, detail the plans to ensure assumed soil conditions exist after construction. Where possible, plans should specify placing footings against undisturbed material. The soil type may be such that it will not stand vertically after excavation. When soil is disturbed, *SP 00510.41* requires backfilling with compacted granular material. When there are questions concerning this, consult with the geotechnical designer. When the excavation will not stand vertically, add a

reference note, **See** *Standard Specifications for Construction* to the **Structure Excavation Limits** detail shown on the plans. The contractor will be allowed to excavate beyond the footing limits and backfill with compacted granular structure backfill (*SP 00510.46*). When footings, such as pile supported, etc., do not require the lateral soil resistance for stability, then do not call for pouring against undisturbed material.

Figure 1.10.2-1 Lateral Earth Restraints



1.10.3 Underwater Construction

1.10.3.1 Underwater Foundation Design Considerations

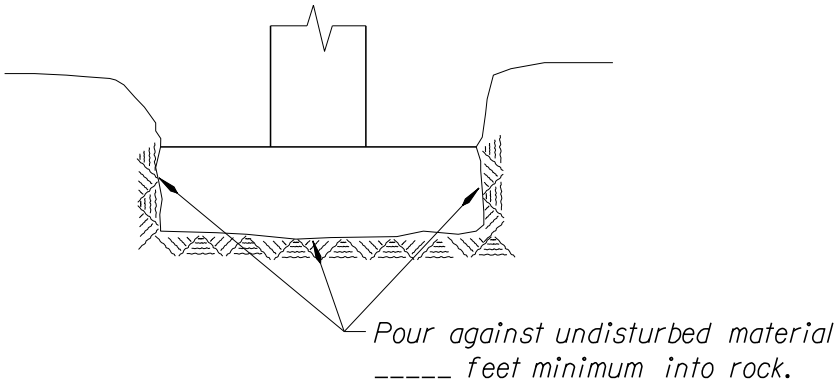
- Requirements for scour protection, potential scour depths and elevations, recommendations for riprap protection can be found in the *Hydraulic Report* and *BDM 1.10.5.8.*
- The seal size, which ultimately determines the cofferdam size should be large enough to accommodate the footing plus footing forms inside the cofferdam walers. A minimum of 2 feet on each side of the footing should be provided.
- Require the contractor to remove all underwater formwork.
- In streams where there is a potential for scour, riprap should be placed as soon as possible and before removal of the cofferdam.
- Scour calculations do not take into account debris loading. A pile of debris will cause a larger obstruction thereby increasing the scour depth.
- Streambeds are often **mobile** and the top few feet or so are moving downstream all the time. During extreme flood events the mobile streambed material cannot be counted on for protection.
- The depth component of the bearing resistance equation has the most significant contribution to the footing's ability to support the load.
- Riprap is not considered permanent protection against scour for seals.
- When placing a footing in a stream, the material around and over the footing has been reworked and doesn't have the in situ strength of the native streambed.
- Another factor that is not always taken into account during a scour calculation is that the stream may be degrading or have the possibility of degrading in the future.

For scour design requirements see *BDM 1.10.5.8*.

1.10.3.2 Footing Embedment

On stream crossings and where horizontal forces are involved, the following sketch should appear on the plans if the foundation material is suitable.

Figure 1.10.3.2-1 Footing Note Callouts at Streams



The bottom of footings in streambeds shall be a minimum of 6 feet below the normal streambed, except in solid rock. When in solid rock, the top of the footing shall be flush with the rock line.

1.10.4 Foundation Modeling (Foundation Springs)

In foundation modeling it is common practice to first assume translational and rotational fixity of the foundation supports and perform a preliminary structural frame analysis. The resulting reactions are checked against the factored resistances. This procedure underestimates global deflections but establishes an upper bound for forces. This type of foundation modeling may be sufficient in certain loading conditions, such as thermal expansion, where deflections are not a controlling factor in design provided the forces are not excessive. However, under higher lateral loading conditions, such as moderate to severe seismic loading, more accurate deflections and forces are desirable. Excessively conservative design forces can be expensive to accommodate. In these cases, foundation springs are typically used in the structural frame analysis. The computer program such as Midas Civil or SAP2000 allows the use of these springs. Foundation springs are typically equivalent linear springs representing the translational (horizontal), axial (vertical) and rotational load-deflection behavior of a nonlinear soil response. The use of foundation springs can significantly reduce the upper bound foundation reactions and more accurately models the entire soil-structure interaction system. Nominal geotechnical resistances are typically used with seismic loading conditions unless otherwise directed by the geotechnical designer. Factored resistances are typically used for all other load combinations. Factored resistance is the nominal resistance multiplied by the appropriate resistance factor.

1.10.4.1 General Modeling Techniques

There are three options for foundation modeling:

- (Option 1) Fixed foundations.
- (Option 2) Fully coupled foundation spring model.
- (Option 3) Uncoupled translation and rotational springs.

Option 1 fixes all foundation supports in the computer model. The resulting forces are simply compared to the resistances stated either in the Geotechnical Report or as determined in this section of the design manual. When the resulting forces exceed the resistances, foundation modeling using springs is recommended.

Option 2 allows stiffness coupling for both shear and moment and also cross-coupling (off diagonal). This option is not required for most problems. This option should be used for drilled shafts, trestle piles and for some pile foundations where the piles are connected to the substructure or superstructure such that a fixed condition exists. A massive footing with deeply embedded piles is an example. The method is applicable to all types of foundations.

Option 3 is the most commonly used method to represent footing and piling flexibility. It is a simplified version of the fully coupled spring model (Option 2) and is used in cases where there is no significant moment transfer between superstructure and foundation elements. This option is appropriate for most problems except as noted in Option 2 above. Use this option with vertical piling only. Battered piles result in larger lateral stiffness, which this option does not presently address.

1.10.4.2 General Procedures and Typical Values

The following guidelines are provided for Option 3 as general information, and are intended to be supplemented with engineering judgment. Methods are presented for developing foundation springs, including factored and nominal resistances, for the following foundation types:

- Abutments and wingwalls.
- Spread footings.
- Piles and pile caps.

Foundation springs are typically nonlinear in form although some are represented in bilinear form. The curve typically consists of an initial (straight line) stiffness followed by a nonlinear relationship leading up to a nominal resistance. Various methods are used, depending on the type of spring, to develop the entire nonlinear load-deflection curve (spring).

The procedures described in this section, and typical values, come from the following sources:

- *Seismic Design of Highway Bridges*, Workshop Manual by Imbsen & Associates, Inc., prepared for the FHWA, October, 1989.

- *Design and Construction of Driven Pile Foundations*, FHWA Workshop Manual, Volume 1, December, 1996.
- Design Manual 7.2, *Foundations and Earth Structures*, Dept. of the Navy, May, 1982.
- *Foundation Analysis and Design*, (4th ed.) by Joseph E. Bowles.
- *Design Manual for the Foundation Stiffnesses Under Seismic Loading*, prepared for Washington DOT by Geospectra, April, 1996.
- *Design Guidance: Geotechnical Earthquake Engineering For Highways*, Volumes I & II; FHWA Report No. *FHWA-SA-97-076-77*, May, 1997.
- Pile capacity and stiffness work done by Bridge Engineering and Geotechnical Group personnel in 1996 and 1997.

Standard Penetration Test (SPT) numbers presented in the Design Manual (**Nc** values) refer to **N** values for granular soils corrected to an effective overburden pressure of 1 tsf. Uncorrected **Nc** values should be used for cohesive soils. The geotechnical designer should be consulted for representative values to use in these methods.

1.10.4.2.1 Abutments and Wingwalls

Use translational springs in both the longitudinal and transverse directions.

TRANSLATIONAL STIFFNESS

The abutment and wingwall translational stiffness should account for displacements resulting from expansion joints associated with seat abutments.

Initial backfill stiffness, is determined by the backfill of the abutment and wingwalls. Wingwalls should be modeled similar to the abutment. Direction of the wingwall contribution is into the approach fill. The initial stiffness should be adjusted proportional to the backwall width and height according to the following equation where the height of the backwall is normalized (Maroney 1995).

Use this value for unknown backfill or when the backfill does not meet the requirements shown on *DET3160*.

$$k_i = \frac{20\frac{kips}{in}}{ft}$$

When the approach fill is constructed/reconstructed using granular structure backfill and meeting the limits shown on *DET3160* the following value for initial stiffness can be used.

$$k_i = \frac{50\frac{kips}{in}}{ft}$$

The stiffness of the abutment is calculated with the following:

$$K_{abut} = k_i * W * \frac{H}{5.5ft}$$

Where:

$k_i = the\ initial\ stiffness\ with\ units\ of\ \frac{\frac{kips}{in}}{ft}$

$W = the\ width\ of\ the\ abutment\ with\ units\ of\ ft$

$H = the\ height\ of\ the\ abutment\ with\ units\ of\ ft$

Piles: Refer to *Pile Supported Footings and Abutments* (see *BDM 1.10.4.2.4*). Use pile translational stiffnesses in tables below for loading conditions other than seismic. For seismic loading conditions, perform an analysis using soil response program such as LPILE. Consult with the geotechnical designer to verify LPILE soil properties.

TRANSLATIONAL CAPACITIES

The passive force resisting the movement at the abutment is modeled using a bi-linear curve with respect to displacement. The ultimate static passive force should be calculated using the following equation. Maximum passive force can only be applied once the soil has been mobilized. Acceptable passive mobilization values are found in *AASHTO Table C.3.11.1-1*. When the wall is backfilled with a medium dense sand or compacted silt a value of $0.02H$ $(where\ H\ is\ the\ height\ of\ the\ wall\ in\ ft)$ should be used. When granular structure backfill is used to fill the active and passive wedge a value of $0.01H$ should be used to determine if mobilization occurs. Only when this deformation is equal to or exceeds this value, then the ultimate passive force can be employed.

$F_{ult} = WH^2 \frac{5ksf}{5.5ft}$

Where:

$F_{ult} = the\ maximum\ passive\ force\ with\ units\ of\ kips$

$W_{bw} = the\ width\ of\ the\ backwall\ in\ feet$

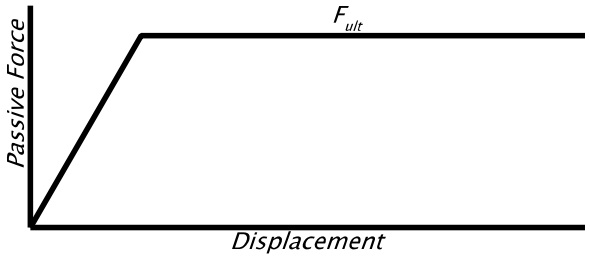
$H_{bw} = the\ height\ of\ the\ backwall\ and\ cap\ in\ feet$

Piles: For seismic loading, use ultimate values derived from LPILE analysis by comparing the maximum yield moment of the pile to the maximum moment output from LPILE. Take end slope and side slope effects into account. Generally assume dense granular fill representing granular wall backfill. This material should be present in the entire passive wedge area. Consult with the geotechnical designer to verify LPILE soil properties. Use allowable pile capacities in tables below for loading conditions other than seismic.

TRANSLATIONAL LOAD-DEFLECTION CURVE

Use the initial stiffness up to the capacity limit. The curve form is:

Figure 1.10.4.2.1-1 Translational Load Deflection Curve



1.10.4.2.2 Skewed Abutments with Wingwalls

Recent large scale testing and numerical modeling of skewed abutments with wingwalls parallel with the roadway show a significant reduction in passive pressure as well as increased displacements, and increased bending moment that must be accounted for in design (Rollins and Snow, 2019). When the abutment is skewed the passive force calculated above is further reduced using the following and applied perpendicular to the skewed bent.

$F_{ult-skew} = F_{ult} * R_{skew}$

$R_{skew} = e^{-\frac{\theta}{45}^{\circ}}$

Where:

$F_{ult} = the\ maximum\ static\ passive\ resistance.$

$F_{ult-slew} = the\ ultimate\ static\ passive\ resistance\ on\ a\ skewed\ bridge\ permitted\ by\ ODOT.$

$\theta = skew\ angle\ of\ the\ bridge\ with\ units\ of\ degrees$

$R_{skew} = the\ reduction\ factor$

To allow for modeling of the bridge in the longitudinal direction the ultimate static passive resistance distribution is modified by increasing it at the obtuse corner of the bridge deck by 1.25 and decreasing at the acute corner by 0.75 as shown in *Figures 1.10.4.2.2-1* and *1.10.4.2.2-2*.

Figure 1.10.4.2.2-1 Skewed Abutment Loading, Plan View

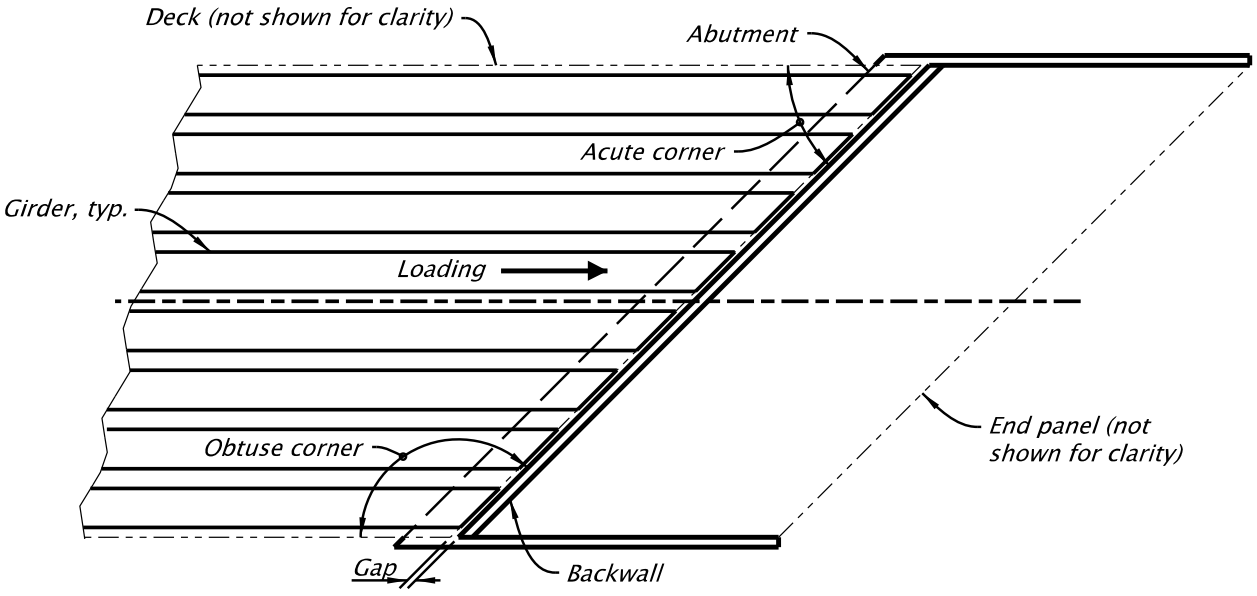
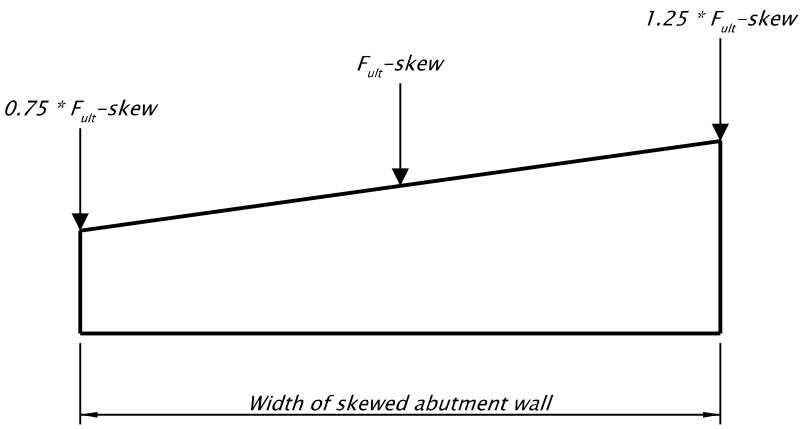


Figure 1.10.4.2.2-2 Skewed Abutment Loading, Elevation View



1.10.4.2.3 Spread Footings

Unless constructed on solid bedrock, use translational and rotational springs in both the longitudinal and transverse directions. In general, footings keyed into a rock mass that has an elastic (Young's) modulus typically greater than 14,000 ksf (Unconfined Compressive Strength = 1000 psi) can be considered **fixed** against both rotation and translation. Consult with the geotechnical designer to determine the compressibility of very soft or highly fractured bedrock materials.

TRANSLATIONAL AND ROTATIONAL STIFFNESSES

Use the equivalent circular footing formulas with information from *Table 1.10.4.2.3-1*, to develop translational and rotational spring constants. Consult with the geotechnical designer for the appropriate soil values to use in *Table 1.10.4.2.3-1*.

Table 1.10.4.2.3-1 Soil Properties

Soil Classification	**SPT "Nc"***	**E (ksf)**	**Poisson's Ratio (*v*)**	**G (ksf)**
Granular				
V. Loose	4	300	.35	110
Loose	10	1000	.35	370
Medium	30	2000	.35	750
Dense	50	3000	.35	1100
Cohesive				
Soft	4	400	.50	150
Stiff	8	1000	.50	350
Very Stiff	16	1500	.50	500
Hard	32	2000	.50	650

* **Nc** is the average of **Nc** values over a depth of 2B below the footing, ($B = footing\ width$).

STIFFNESS CALCULATIONS FOR SPREAD FOOTINGS

Spring constants for rectangular footings are obtained by modifying the solution for a circular footing bonded to the surface of an elastic half-space. The formula is as follows where:

$k = initial\ stiffness\ (spring\ constant)$

$\alpha = foundation\ shape\ correction\ factor;\ (from\ graph)$

$\beta = embedment\ factor, (from\ graph)$

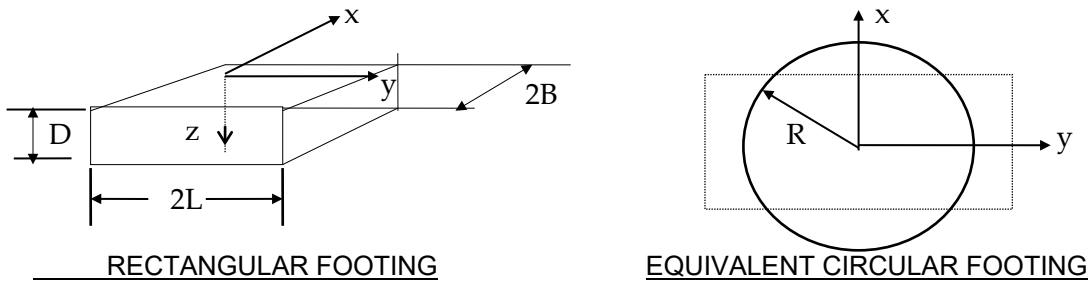
$K0 = stiffness\ coefficient\ for\ the\ equivalent\ circular\ footing\ (see\ formulas\ in$ Table 1.10.4.2.3 − 2)

The stiffness term, K_0, is calculated using the equations in *Table 1.10.4.2.3-2*.

Table 1.10.4.2.3-2 Stiffness Equations for Spread Footings

Displacement Degree-of-Freedom	**K_0**
Vertical translation	$4GR/(1-v)$
Horizontal translation	$8GR/(2-v)$
Torsional rotation	$16GR^3/3$
Rocking rotation	$8GR^3/(3(1-v))$
Note: G = Shear Modulus (low strain range) v = Poisson's ratio for elastic half-space material R = Equivalent footing radius as determined from the following equations	

Figure 1.10.4.2.3-1 Equivalent Radii, R, for Rectangular Footing Spring Constants



RECTANGULAR FOOTING EQUIVALENT CIRCULAR FOOTING

Figure 1.10.4.2.3-2 Shape Factors for Rectangular Footings

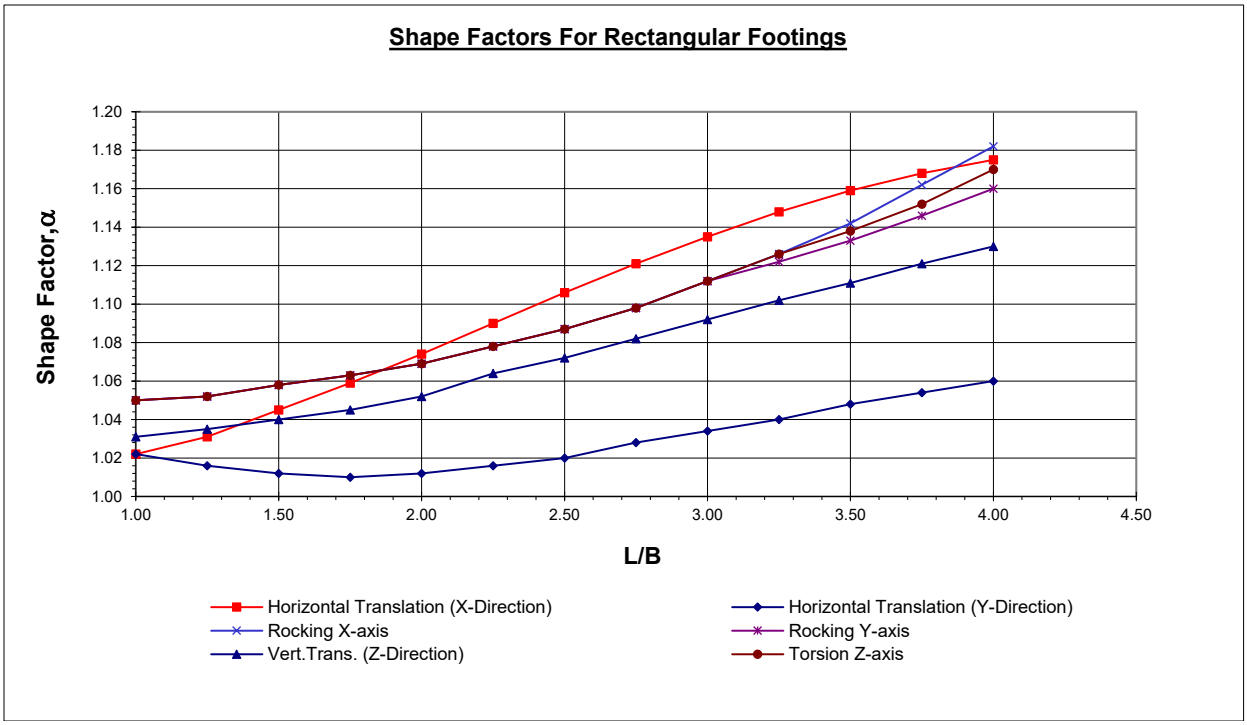
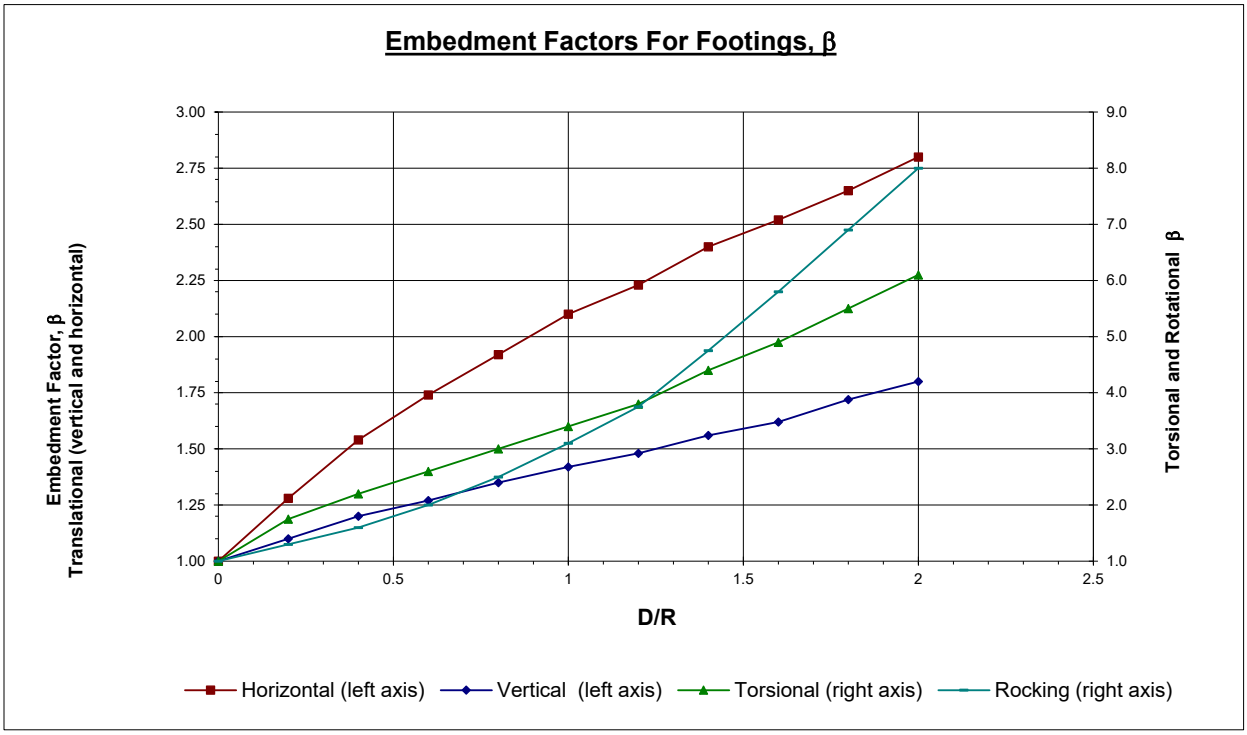


Figure 1.10.4.2.3-3 Embedment Factors for Footings, β



EQUIVALENT RADIUS:

TRANSLATIONAL:

$$R = \sqrt{\frac{4BL}{\pi}}$$

ROTATIONAL:

$$R = \left[\frac{(2B)(2L)^3}{3\pi}\right]^{1/4} \quad \text{; for x-axis rocking}$$

$$R = \left[\frac{(2B)^3(2L)}{3\pi}\right]^{1/4} \quad \text{; for y-axis rocking}$$

$$R = \left[\frac{4BL(4B^2 + 4L^2)}{6\pi}\right]^{1/4} \quad \text{; for z-axis torsion}$$

TRANSLATIONAL CAPACITIES

The use of the following values depends on the footing construction method (i.e. formed with backfill material or poured against undisturbed material). Only the passive resistance developed from the front face of the footing, combined with the shear resistance along the footing base, is considered. Column and footing side resistance is neglected. Consult with the geotechnical designer for recommended soil properties, groundwater levels and proper effective unit stress to use in the analysis. Also consider scour effects.

Use the values from *Table 1.10.4.2.3-3* in the general formula:

$$\begin{aligned} Force\ Capacity \\ &= (Kp\ x\ effective\ unit\ stress\ x\ footing\ face\ area) \\ &+ (Su\ x\ footing\ face\ area) + (\square\ x\ support\ reaction) \\ &+ (Su\ x\ footing\ base\ area) \end{aligned}$$

Use appropriate components depending upon soil type. Consult with the geotechnical designer for the appropriate soil values to use.

Note: $Effective\ Unit\ Stress = (Buoyant\ Unit\ Weight\ x\ Depth\ to\ middle\ of\ footing)$

Table 1.10.4.2.3-3 Translational Capacities

Soil Classification	SPT "Nc"	STATIC CAPACITY			Total Unit Wt. (k/ft^3)
Granular		**K_p**	**Su (ksf)**	**μ**	
V. Loose	4	2.7	0	.34	0.090
Loose	10	3.0	0	.40	0.100
Medium	30	3.7	0	.47	0.115
Dense	50	4.6	0	.56	0.120
Cohesive					
Soft	4	-	0.5	-	N.A.
Stiff	8	-	1.0	-	N.A.
Very Stiff	16	-	2.0	-	N.A.
Hard	32	-	4.0	-	N.A.

Deflection required to fully activate capacities (⌾max):

Granular:

$$Loose = .06H$$
$$Dense = .02H$$

Cohesive:

$$Soft = .04H$$
$$Stiff = .02H$$
$$H = Soil\ surface\ to\ middle\ of\ footing\ depth$$

Specific applications may require the use of less than the full capacity due to deflection restrictions.

ROTATIONAL CAPACITIES

The rotational capacity is typically determined by comparing the total footing pressure, including the overturning moment, to the factored bearing resistance provided in the Geotechnical Report, unless otherwise directed by the geotechnical designer. The bearing resistance of footings with overturning moments and eccentricity are determined using **effective** footing dimensions.

TRANSLATIONAL LOAD-DEFLECTION CURVE

The following equation may be used in conjunction with the translational stiffnesses and capacities for developing a translational load-deflection curve for spread footings and pile caps.

$$P = \frac{\Delta}{\frac{1}{k_{max}} + \left[R_f \times \frac{\Delta}{P_{ult}} \right]}$$

where: $P = Load\ at\ deflection\ \Delta$

$Pult = Ultimate\ passive\ force\ (neglect\ base\ shear\ for\ pile\ caps)$

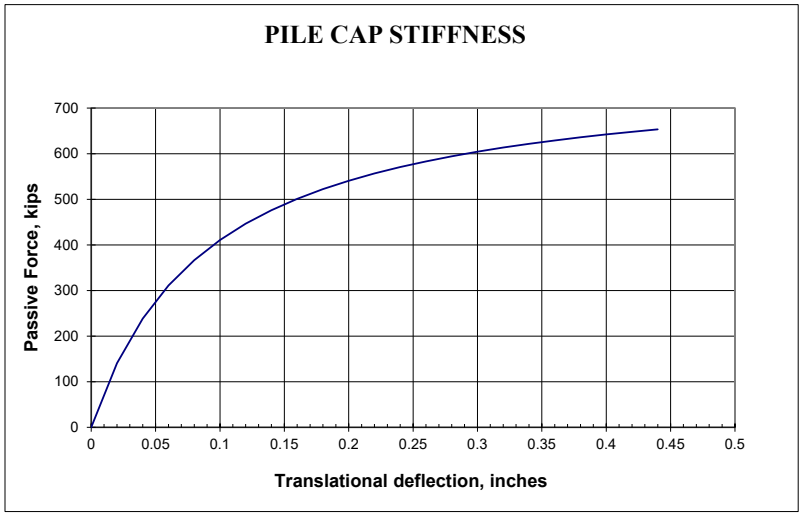
$kmax = Initial\ stiffness$

$Rf = Ratio\ between\ the\ actual\ and\ the\ theoretical\ ultimate\ force.\ Rf$ can be determined by substituting Δmax from the previous section for Δ and $Pult$ for P in the above equation and solving for Rf.

$\Delta = Translational\ deflection, inches$

An example of the use of this equation is given below. This graph represents the form of the equation only.

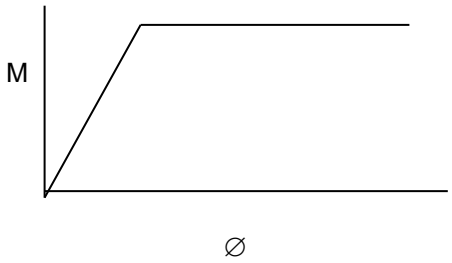
Figure 1.10.4.2.3-4 Pile Cap Stiffness



ROTATIONAL LOAD-DEFLECTION CURVE

Use the initial stiffness up to the capacity limit. The curve form is show in *Figure 1.10.4.2.3-5.*

Figure 1.10.4.2.3-5 Rotational Load-Deflection Curve Form



1.10.4.2.4 Pile Supported Footings and Abutments

Use translational and rotational springs for pile supported footings in both the longitudinal and transverse directions. This approach is recommended in cases where seismic loading is the controlling factor in the structural frame analysis. Springs may also be used to model pile supported footings in non-seismic conditions at the designer's discretion. Nominal resistances may be used for both non-seismic and seismic design conditions unless otherwise recommended by the geotechnical designer.

In cases where seismic loading is not the maximum group loading for the structure, the stiffnesses and nominal lateral resistances given in the following tables are acceptable for most design cases, provided the site conditions generally satisfy the assumptions made in developing these values. In general, for soils with Nc values less than four, the pile translational stiffness should be evaluated using a soil response program such as LPILE programs and the geotechnical designer should be consulted for further guidance.

The use of battered piles is generally discouraged due to the greatly increased stiffness contribution from the battered piles. This in turn can result in excessive battered forces and induce undesired or unrealistic uplift forces in adjacent piles. In lieu of battered piles, it is recommended to use vertical piles throughout the footing.

Refer to the seismic design example problem for further clarification.

TRANSLATIONAL STIFFNESSES

Normally the translational stiffness should include the lateral pile stiffnesses (total pile group stiffness) plus the passive soil stiffness on one side of the footing. Typically, a single lateral pile-head stiffness is estimated from either the pile-top, load-deflection curve generated by LPILE soil response program output or from pile stiffness values given in the following tables. This single pile-head stiffness is then multiplied by the number of piles in the group and the resulting group stiffness value is then multiplied by a group reduction factor depending on pile spacing. Instead of using a group reduction factor, account for pile group effects using p-y curve multipliers as described under **Pile Group Reduction Factors and p-y Multipliers**.

Pile cap, or footing, stiffnesses should be developed using the methods described under **Spread Footings**, except the soil stiffness contribution along the base of the pile cap should be neglected. This is accomplished by calculating the stiffness of the pile cap (footing) at the ground surface ($D = 0$) and subtracting this value from the stiffness calculated for the embedded pile cap footing. The resulting stiffness curve is then combined with the pile group stiffness curve as described in **Translational Load-Deflection Curve**.

SEISMIC CONTROLLED LOADING CONDITION – EXTREME EVENT LIMIT STATE

The pile-head translational stiffness curve is generated using a soil response program such as LPILE using soil input parameters supplied by the geotechnical designer. Pile head boundary conditions (fixed, free or fixed-translational) must be assigned by the designer. Refer to the LPILE computer program manuals. This method is shown in *Figure 1.10.4.2.4-1*.

NON-SEISMIC LOADING CONDITIONS

Use the following pile stiffnesses for non-seismic loading conditions provided the site conditions generally satisfy the assumptions given.

Table 1.10.4.2.4-1 Pile Translational Stiffness (kip/inch) – HP Piles

Axis - W=Weak S=Strong	**SPT "Nc"***	**HP 10x42**		**HP 12x53**		**HP 12x74**		**HP 14x89**		**HP 14x117**	
Granular		W	S	W	S	W	S	W	S	W	S
V. Loose	4	5	8	6	10	7	11	9	13	10	14
Loose	10	12	14	12	18	14	20	16	24	18	24
Medium	30	16	20	18	27	20	30	25	38	28	41
Dense	50	25	34	29	44	31	46	40	61	44	64
Cohesive											
Soft	4	2	2	2	3	2	3	3	4	3	4
Stiff	8	4	6	5	7	6	8	7	9	7	9
Very Stiff	16	8	10	9	12	10	13	12	15	12	16
Hard	32	14	19	17	22	18	24	21	27	23	30

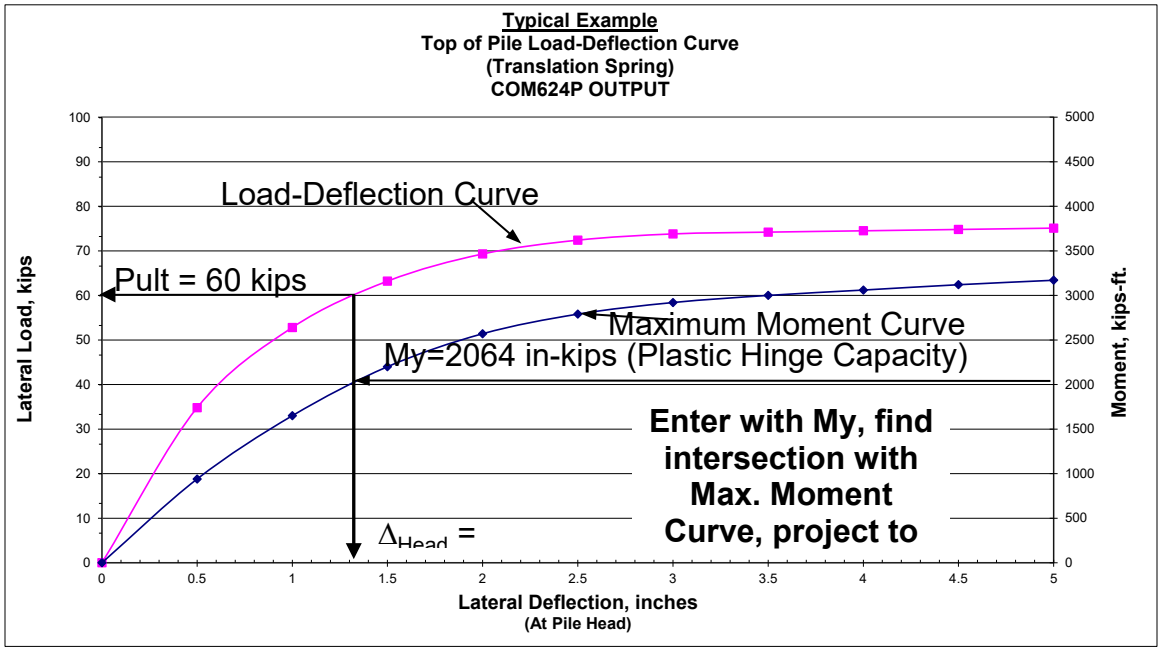
Table 1.10.4.2.4-2 Pile Translational Stiffness (kip/inch) – Pipe Piles

Pipe Piles	**SPT "Nc"***	**12x 0.25**	**12x 0.38**	**16x 0.38**	**16x 0.50**	**24x 0.38**	**24x 0.50**
Granular							
V. Loose	4	7	8	11	12	20	22
Loose	10	14	15	20	21	33	37
Medium	30	20	23	29	34	48	57
Dense	50	32	37	46	54	81	87
Cohesive							
Soft	4	2	3	3	3	4	4
Stiff	8	6	7	8	9	11	12
Very Stiff	16	10	11	13	14	18	18
Hard	32	18	20	24	26	34	36

Table 1.10.4.2.4-3 Pile Translational Stiffness (kip/inch) – Prestressed Piles

Prestressed Piles	**SPT "Nc"***	**12 inch**	**14 inch**	**16 inch**
Granular				
V. Loose	4	8	8	11
Loose	10	12	14	19
Medium	30	22	24	28
Dense	50	34	38	45
Cohesive				
Soft	4	3	3	3
Stiff	8	7	7	8
Very Stiff	16	12	12	14
Hard	32	22	23	26

Figure 1.10.4.2.4-1 Example of Top of Pile Load-Deflection Curve (Translational Spring)



TRANSLATIONAL CAPACITIES

The base shear resistance of pile supported footings, or caps, is typically not included in calculating the nominal passive resistance. The same equation used for determining the nominal translational capacity of footings should be used for pile caps, neglecting all base shear resistance. The nominal passive resistance of pile caps can be used for both seismic and non-seismic design conditions.

Use the nominal resistances in the following table for non-seismic loading conditions provided the site conditions generally satisfy the assumptions given.

Table 1.10.4.2.4-4 Nominal Pile Translational Resistances (kip/inch) – HP Piles

H-piles W=Weak S=Strong	**SPT "Nc"***	**HP 10x42**		**HP 12x53**		**HP 12x74**		**HP 14x89**		**HP 14x117**	
Granular		W	S	W	S	W	S	W	S	W	S
V. Loose	4	12	21	14	25	25	43	29	50	41	69
Loose	10	13	23	16	27	28	48	33	55	46	82
Medium	30	16	26	17	31	31	53	37	62	51	86
Dense	50	17	29	20	34	34	59	41	69	57	93
Cohesive											
Soft	4	16	25	17	28	29	47	34	53	45	69
Stiff	8	20	34	22	37	38	63	43	70	59	94
Very Stiff	16	24	43	25	47	49	83	55	90	76	122
Hard	32	30	54	29	58	58	104	63	113	92	155
*The **Nc** values to use are the averaged **Nc** values over a depth of 8 to 10 pile diameters (8D to 10D).											

Table 1.10.4.2.4-5 Nominal Pile Translational Resistances (kip/inch) – Pipe Piles

Pipe Piles	**SPT "Nc"***	**12x 0.25**	**12x 0.38**	**16x 0.38**	**16x 0.50**	**24x 0.38**	**24x 0.50**
Granular							
V. Loose	4	22	29	43	52	85	103
Loose	10	25	32	48	57	95	113
Medium	30	29	37	54	65	107	130
Dense	50	31	41	60	71	118	143
Cohesive							
Soft	4	26	34	46	55	82	98
Stiff	8	34	44	60	72	104	126
Very Stiff	16	42	56	74	91	130	158
Hard	32	50	69	91	110	151	187
*The **Nc** values to use are the averaged **Nc** values over a depth of 8 to 10 pile diameters (8D to 10D).							

Table 1.10.4.2.4-6 Nominal Pile Translational Resistances (kip/inch) – Prestressed Piles

Prestressed Piles	**SPT "Nc"***	**12 inch**	**14 inch**	**16 inch**
Granular				
V. Loose	4	12	16	23
Loose	10	15	18	25
Medium	30	18	20	29
Dense	50	20	23	31
Cohesive				
Soft	4	18	19	25
Stiff	8	22	23	31
Very Stiff	16	27	27	36
Hard	32	29	29	43
*The **Nc** values to use are the averaged **Nc** values over a depth of 8 to 10 pile diameters (8D to 10D).				

The above translational stiffnesses and allowable capacities are based on the Broms' method and the following assumptions:

- Free head condition, no applied moment.
- Pile top at the ground surface.
- Level ground surface.
- One, uniform soil layer with uniform soil properties.
- No groundwater.
- Static loading, no cyclic soil degradation.
- Constant pile properties and dimensions.
- Stiffnesses are for first ½ inch deflection (initial secant modulus).
- Values are for **long** pile conditions and minimum pile embedment depths are required. When pile lengths are less than 75 percent of the assumed penetration lengths in *Table 1.10.4.2.4-7*, a separate Broms' or LPILE analysis is required.

Table 1.10.4.2.4-7 Pile Penetration Lengths

Soil Classification	"Nc"	Assumed Length (feet)
Granular	4	55
	10	50
	30	40
	50	35
Cohesive	4	55
	8	50
	16	40
	32	35

The geotechnical designer should be consulted for piles installed in conditions outside of the above stated assumptions or a LPILE analysis should be performed.

For seismic design conditions, the maximum moment capacity of the pile (My) must be calculated separately and compared to the LPILE output to determine the nominal lateral resistance and associated deflection. An example is shown in *Figure 1.10.4.2.4-1*.

TRANSLATIONAL LOAD-DEFLECTION CURVE

Translational Load Non-seismic - Deflection estimates for piles designed under non-seismic conditions should be determined using the initial pile stiffness values given in the above tables extended up to the nominal pile resistance (bilinear curve). This curve, representing the pile

group, is then added to the load-deflection curve developed for the pile cap. A LPILE analysis may also be used as described below if so desired.

Translational Load Seismic - Deflection estimates for seismic design conditions are determined from the composite load deflection curves developed by combining the pile group stiffness from the LPILE analysis with the stiffness contribution from the pile cap. An example of this procedure is provided in the section on **Load-Deflection Curves, Stiffness Iteration Analysis and Capacity Checks**.

PILE GROUP EFFECTS AND P MULTIPLIERS

The P multiplier approach, utilizing the LPILE program, is recommended to evaluate the response of pile groups subjected to lateral loads. The P multipliers are applied to standard p-y curves to account for pile group effects. *LRFD 10* should be referenced for the P multiplier values to be used in the analysis. The P multipliers are dependent upon the center to center spacing of piles in the group in the direction of loading expressed in multiples of pile diameter. The geotechnical designer should be consulted for the procedures to use in this design approach.

ROTATIONAL STIFFNESSES

Normally the rotational stiffness should only include the moment versus rotation stiffness from the pile group. The pile cap is usually considered rigid in this analysis and no additional stiffness due to soil bearing at the base of the pile cap/footing is included. Therefore, the rotational stiffness of pile caps is simply a function of pile axial compression and the pile group layout. See the example problem in the Bridge Example Designs notebook for more details. Static formulas for pile compression are typically used. The computer program APILE may also be used for a more detailed analysis of the predicted load-deflection behavior of a single, axially loaded pile. This program takes into account unusual soil conditions and the nonlinear aspects of pile-soil interaction. The geotechnical designer should be consulted for axial pile stiffnesses using the APILE program.

The following formulas for axial pile stiffness may be used in developing rotational stiffnesses for pile supported footings. For friction piles, the APILE program may also be utilized to better model axial stiffness when axial loads are greater than about half of the nominal resistance of the pile.

End bearing pile:
$Kv = \frac{AE}{L}$
Friction Piles:
$Kv = \frac{2AE}{L}$

where: $Kv = Axial\ Pile\ Stiffness\ (kN/mm)$

$A = Area\ of\ pile\ normal\ to\ load$

$L = Length\ of\ pile$

$E = Young's\ Modulus\ of\ Pile\ Material$

Compute the rotation stiffness (M vs. ∅) for a pile group as follows:

Assume a pile head deflection for the pile farthest from the pile group centroid.

Using the appropriate relation from above, determine the pile force accompanying this assumed pile head deflection. Prorate the other pile forces by their location relative to the group centroid. Piles on one side of the centroid will have positive forces and piles on the other side will have negative forces (uplift).

Determine the pile group moment by summing the product of the pile force and the pile-to-group centroid distance for all piles. This is the moment (M) required to rotate the footing through an angle of ∅. Determine the angle ∅ as the arctan of the assumed extreme pile head deflection divided by the pile-to-centroid distance.

The relation of M to ∅ is the initial rotation stiffness.

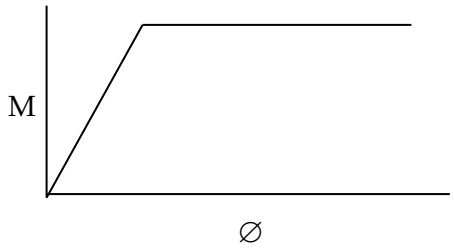
ROTATIONAL RESISTANCES

For pile supported footings, compare computed pile loads to nominal axial pile resistances for seismic cases and to factored axial pile resistances for non-seismic cases, unless otherwise recommended by the geotechnical designer.

ROTATIONAL LOAD-DEFLECTION CURVE

Use the initial stiffness up to the resistance limit. The curve form is:

Figure 1.10.4.2.4-2 Rotational Load-Deflection Curve



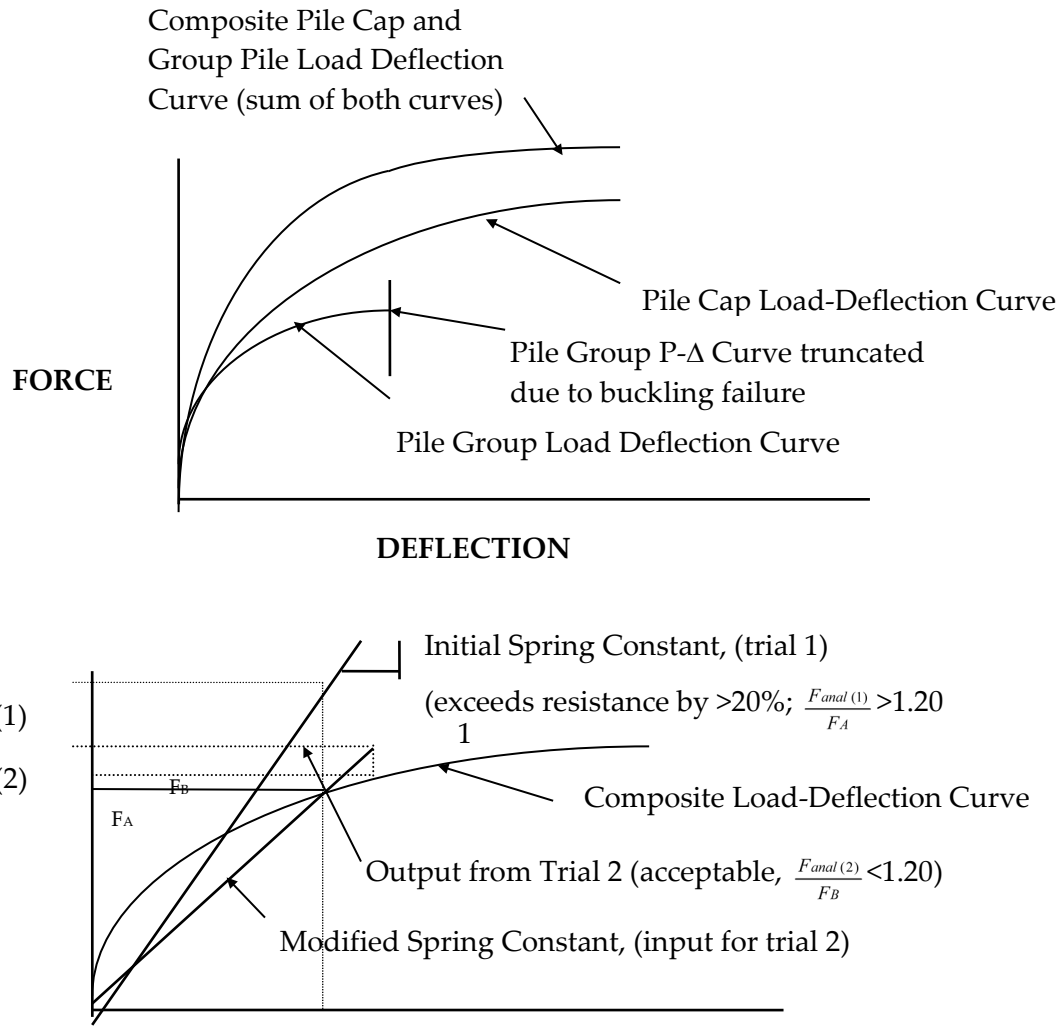
LOAD-DEFLECTION CURVES, STIFFNESS ITERATION ANALYSIS AND CAPACITY CHECKS

Using the previous information one develops a composite load-deflection relationship for each applicable support spring. Next, an initial spring constant is assumed, the structure and loading analyzed and the resulting load-deflection position compared to the initial assumption. Cycling through this process may be needed to achieve reasonable closure. See the graphical explanation below.

It is also necessary to check the required resistance against the factored or nominal resistance. Resistance factors of 1.0 are typically used in the case of seismic design, however this should be verified by the geotechnical designer. Factored resistances are used for all other cases. For the rotational capacity, this is normally done by checking the resultant forces against the maximum (nominal), effective soil bearing resistance (footings) or nominal pile resistance.

For lateral pile resistances, the nominal resistance is either the maximum determined from the LPILE analysis (based on My of the pile for seismic design), or from the tables. The nominal resistance may also be a function of maximum allowable structural deflections. When the limiting resistance is exceeded when using the initial spring coefficient then modified springs are required as shown in the graphical explanation below, *Figure 1.10.4.2.4-3*.

Figure 1.10.4.2.4-3 Modified Springs: (A) Development of Composite Load, Deflection Curve (B) Spring Iteration Process and Resistance Check



1.10.4.3 Drilled Shaft Modeling (Fully Coupled)

Programs Midas Civil or SAP200 and LPILE can be used in an iterative approach to model a drilled shaft supported structure. The approach is to determine the approximate force magnitudes for the controlling loading and then use these forces to develop a better representation of the superstructure/shaft/soil problem. This allows a good approximation of soil stiffness non-linearity as well as the non-linearity of the shaft-soil interaction.

The following steps would be typical for drilled shaft modeling for design and checking:

- Develop a full Midas Civil or SAP2000 model (superstructure with substructure) using shaft fixity at two shaft diameters below the groundline. Using the model, run the controlling load case – typically seismic loading will be the controlling case and the worst effect, either longitudinal or transverse, will be used for the next steps.
- Develop LPILE models (shaft with soil) for each bent using the full shaft from its tip to its connection to the superstructure.
- Using the top of shaft shear and moment results from the first Midas Civil or SAP2000, load the LPILE models to develop a stiffness matrix for each shaft. This represents a condensing of the substructure/soil effect to the point of connection with the superstructure. The LPILE program can develop a stiffness matrix.
- Develop a new Midas Civil or SAP2000 model using only the superstructure and supports represented by the LPILE developed substructure stiffness matrices. Run the same controlling load case.
- Use the top of shaft shear and moment results from this latest Midas Civil or SAP2000 to again load the LPILE models to develop new substructure stiffness matrices.
- Use the latest Midas Civil or SAP2000 model with the most recent substructure stiffness matrices and again run the same controlling load case.
- Compare the results of this Midas Civil or SAP2000 with the previous Midas Civil or SAP2000 run for correlation. When the results do not correlate well, cycle through steps five and six to get better convergence. Results which change no more than 15 percent per cycle are normally sufficiently close and further cycling is not required.

1.10.5 Foundation Design

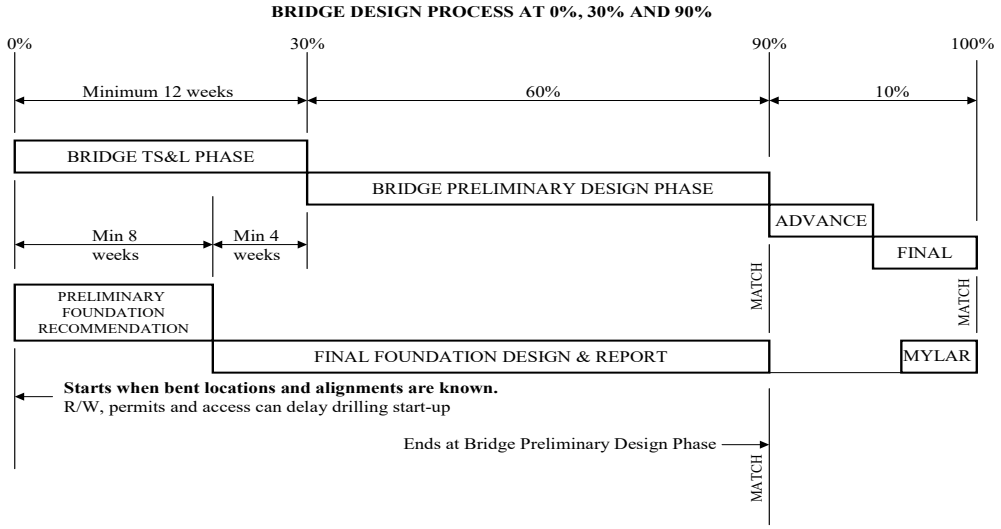
Foundation design should be performed in accordance with the most current version of the *AASHTO LRFD Bridge Design Specifications*. Foundation design should also follow the policies and guidelines described in the *ODOT Geotechnical Design Manual*, available through the ODOT Geo-Environmental Section web page.

FHWA foundation design manuals are also acceptable methods for use in foundation design. Subsurface investigations for all structures should be conducted in accordance with the *AASHTO Manual On Subsurface Investigations (1988)*. Materials classifications should be in accordance with the *ODOT Soil and Rock Classification Manual (1987).*

1.10.5.1 Foundation Design Process

A flow chart showing the overall foundation design process, related to plans development, is provided in *Figure 1.10.5.1-1*. It is important for the foundation and bridge designers to establish and maintain good communication and exchange of information throughout the entire bridge design process. Any questions regarding foundation design issues should be brought to the attention of the geotechnical designer as early as possible in the design process. For most typical bridge design projects two Geotechnical Reports are provided, the TS&L Foundation Design Memo and the Geotechnical Report. A description of the phases follows.

Figure 1.10.5.1-1 Bridge Design Process at 0, 30 and 90 percent



TS&L FOUNDATION DESIGN MEMO

The purpose of this memo is to provide sufficient data for developing TS&L plans and cost estimates and for permitting purposes. The memo is generally provided before the subsurface investigation is completed. It provides a brief description of the proposed project, the anticipated subsurface conditions (based on existing geologic knowledge of the site or as-constructed information) and presents preliminary foundation design recommendations such as foundation types and preliminary resistances. The potential for liquefaction and associated effects are also briefly discussed. The memo is to be provided no later than two-thirds of the way through the TS&L design process.

GEOTECHNICAL REPORT

This report is to be provided by the end of the Preliminary Bridge Design phase, which is usually 90 percent design. It provides the final foundation design recommendations for the structure and a Geotechnical Data Sheet for each structure. In order to conduct a proper foundation investigation and complete this report the geotechnical designer will need the following information:

- Bent locations and layout.
- Proposed roadway grade (fill heights).
- Anticipated foundation loads.
- Foundation size/diameter and depth required to meet structural needs.
- Allowable structure settlements (total and differential).
- Proposed retaining wall locations.
- Estimated scour depths (from Hydraulics Report).
- Construction or environmental constraints that could affect the type of foundation selected.

The report will contain the all geotechnical data on the site including final boring logs, Geotechnical Data Sheets, laboratory test results, foundation soil design parameters, recommended foundation types, sizes and resistances, and other recommendations. Construction recommendations are included along with project specific specifications, which are to be included in the contract Special Provisions. Seismic foundation design recommendations are provided including site characterization and soil coefficients, estimated ground acceleration and any liquefaction mitigation measures considered necessary (See *BDM 1.17*).

The geotechnical designer should review the final Plans and Special Provisions for the structure to make sure they are consistent with the design recommendations provided in the Geotechnical Report. Any discrepancies should be resolved and addendums to the report issued when necessary. A copy of the Geotechnical Report should be included in the project file and is made available to contractors through the Project Manager's Office when the project is advertised for bid.

1.10.5.2 Bridge Foundation Records

As-constructed records on existing bridge foundations may be found in the Salem Bridge Engineering Office from the following sources:

- Pile Record Books.
- **As-constructed** Bridge Plans (available through ODOT intranet).
- Microfilm Construction Records.
- Bridge Maintenance Files.

1.10.5.3 Spread Footing Foundation Design

Spread footings are considered early on in the design process as a possible economical foundation option when the foundation conditions are suitable. The design of spread footings is usually an interactive process between the geotechnical and structural designers. The bottom of spread footings should be at least 6 feet below the bottom of the streambed unless non-erodable bedrock is present. The bottom of spread footings should also be below the estimated depth of scour for the 500 year flood event. The top of the footing should be below the depth of scour estimated for the 100 year event. See *BDM 1.10.5.8* for scour design requirements. Spread footings are not to be constructed on soils that may liquefy under earthquake loading. When spread footings are recommended the geotechnical designer will provide the following design recommendations in the Geotechnical Report:

FOOTING ELEVATIONS

The elevations of the proposed footings will be provided along with a clear description of the foundation materials the footing is to be constructed on.

NOMINAL AND FACTORED BEARING RESISTANCES

The nominal and factored bearing resistances will be provided for various effective footing widths likely to be used. Resistance factors for all applicable load combinations should be consistent with the most recent version of *AASHTO LRFD Bridge Design Specifications*.

Bearing resistances corresponding to 1 inch of settlement (Service Limit State) should also typically be provided unless other settlement limits are established by the structural designer. The structural designer should communicate all footing settlement limits to the geotechnical designer. For soil conditions, the bearing resistances provided assume the footing pressures are uniform loads acting over effective footing dimensions B' and L' (i.e. effective footing width and length $((B\ or\ L) - 2e)$ as determined by the Meyerhof method. For footings on rock, the resistances provided assume triangular or trapezoidal stress distribution and maximum toe bearing conditions.

Minimum footing setback on slopes and embedment depths will be provided.

SLIDING STABILITY AND ECCENTRICITY

The following soil parameters will be provided for calculating frictional sliding resistance and active and passive earth pressures:

- Soil Unit Weight, γ (soil above footing base).
- Soil Friction Angle, ϕ (soil above footing base).
- Active Earth Pressure Coefficient, Ka.
- Passive Earth Pressure Coefficient, Kp.
- Coefficient of Sliding, $tan\ \delta$.

OVERALL STABILITY

The geotechnical designer will evaluate overall stability using the maximum footing load which can be applied to the design slope while maintaining resistance factor of 0.65 as outlined in *LRFD 11.6.2.3*.

1.10.5.4 Pile Foundations

When spread footings are unsuitable or uneconomical for foundation support, driven piles should be considered. Consult with the geotechnical designer to determine the most appropriate pile type, size and bearing resistance to support the desired pile loads. Typical pile types, sizes and structural resistances ($0.9f_y$ x Sectional area of pile) used on ODOT projects are listed below in *Table 1.10.5.4-1*. The geotechnical designer should provide the nominal resistances required to achieve the required resistance for each limit state.

Table 1.10.5.4-1 Steel HP – Pile Structural Resistance

SHAPE	STRUCTURAL RESISTANCE f_y = 36 ksi (kips)	STRUCTURAL RESISTANCE f_y = 50 ksi (kips)	STRUCTURAL RESISTANCE f_y = 60 ksi (kips)
HP 10x42	402	558	--
HP 10x57	541	752	--
HP 12x89	839	1166	1399
HP 12x117	1115	1548	1858
HP 14 x 117	1115	1548	1858
HP 16 x 141	1351	1877	2252
HP 16 x 183	1753	2435	2921
HP 18 x 204	1950	2709	3251

Table 1.10.5.4-2 Steel Pipe – Piles Structural Resistance

SHAPE	STRUCTURAL RESISTANCE f_y = 35 ksi (kips)	STRUCTURAL RESISTANCE f_y = 45 ksi (kips)
PP 16 x 0.375	580	745
PP 18 x 0.5	865	1113
PP 24 x 0.5	1162	1494

When steel piles are installed under environmental conditions meeting corrosion criteria as described in *BDM 1.10.5.4.1(10)*, specify a method of corrosion protection for the steel piles or

determine required cross-section of the steel piles throughout the minimum design life. Corrosion rates for sacrificial thickness are specified in *BDM 1.10.5.4.1(10)*.

Do not use precast prestressed or timber piles.

1.10.5.4.1 Piling Considerations

(1) PILE RESISTANCE

Nominal pile resistances will be provided according to *AASHTO LRFD* design procedures. The resistance factor will be provided according to the construction quality control method recommended in the Geotechnical Report (i.e. dynamic formula, wave equation, Pile Driving Analyzer, etc.). The geotechnical and bridge designers should confer to make sure the pile types and sizes selected take full advantage of the available geotechnical and structural resistances when possible.

(2) DOWNDRAG LOADS

Pile downdrag loads, due to soil settlement other than that caused by dynamic (seismic) loading, are added to the factored vertical dead loads on the foundation in the Strength Limit state. Load Factors for downdrag loads will be recommended by the geotechnical designer. Transient loads should not be included with the downdrag loads in either the strength or service limit state calculations. Downdrag loads resulting from liquefaction or dynamic (earthquake) induced soil settlement should be considered in the Extreme Event limit state pile design. Downdrag loads resulting from soil liquefaction are different than those caused from static loading and they should not be combined in the Extreme Limit state analysis.

At sites where downdrag conditions exist, the pile must overcome the frictional resistance in the downdrag zone during installation. This resistance should not be included in the calculation of available factored resistance since after installation it reverses over time becoming the static downdrag load.

(3) UPLIFT CAPACITY

In general, the uplift resistance is the same as the pile friction (side) resistance. Resistance factors and factored uplift resistances will be provided in the Geotechnical Report. Friction resistance in downdrag zones should be considered available for uplift resistance. The geotechnical designer should be consulted regarding the ability of the piles to resist uplift forces under various loading conditions (static or dynamic).

(4) MINIMUM PILE TIP ELEVATION

Minimum pile tip elevations (embedment depths) are typically required to meet one or more of the following design requirements:

- Lateral Load.
- Scour.
- Liquefaction.
- Uplift loads.

- Settlement or Downdrag.
- Required soil/rock bearing strata.

The required pile tips elevations should be shown on the plans and labeled as **Required Minimum Pile Tip Elevations**. Large lateral loads due to seismic, or other, conditions may result in the need for additional piling, or larger piles, in order to satisfy lateral deflection criteria or other requirements. This may in turn result in individual axial pile loads being much less than the maximum factored resistances available (either geotechnical or structural). Conversely, when pile tip elevations are needed to meet scour, uplift, or other requirements, the piles may need to be driven through very dense materials to nominal resistances much higher than needed for supporting just the axial loads. Close communication is needed between the geotechnical and bridge designers to determine the most economical foundation design under these conditions. See *BDM 1.10.5.8* for scour design requirements.

(5) PILE GROUP SETTLEMENT

Pile group settlement should be compared to the maximum allowable settlement and pile depths or layout adjusted if necessary to reduce the estimated settlement to acceptable levels.

(6) PILE GROUP EFFECTS

For pile group lateral load analysis use the P- multiplier methods described in *LRFD* and the FHWA Manual on the *Design and Construction of Driven Pile Foundations*.

(7) PILE SPACING

Use a minimum spacing of 3 feet for piles placed underwater. Above water pile spacing should be no closer than 2.5B.

(8) PILE TIP TREATMENT

Where pile tip reinforcement is required, specify commercial cast steel points or shoes.

Where closed-ended pipe piles are required, specify a welded end plate or a welded end plate with stiffeners having the same diameter as the pipe pile. An analysis was performed for a range of pipe pile sizes which verified sufficiency and the minimum dimensions for the end plate and stiffeners. This analysis was conducted using $0.9fy$ as the maximum load, which is the maximum stress the pile will undergo and what is allowed during pile driving. Provide dimensions for the end plate and stiffeners on plan sheets for each project. Deviation from the minimum dimensions below requires project specific shell and buckling analyses and driving stress from the geotechnical designer.

Table 1.10.5.4.1-3 provides the minimum dimensions for a welded end plate and a welded end plate with stiffeners. Use these dimensions with *Figures 1.10.5.4.1-1* and *Figures 1.10.5.4.1-2*. The figures are available online in the standard details at the following location Welded_Plate_Details. These details are available for project specific use.

Figure 1.10.5.4.1-1 Welded End Plate

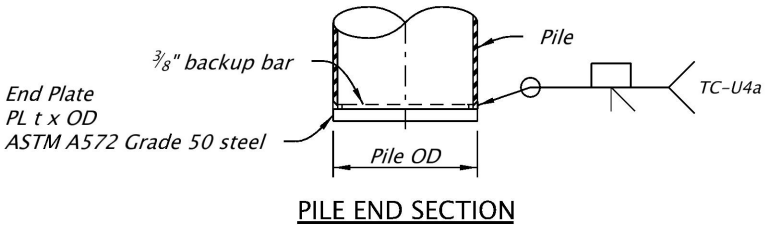


Figure 1.10.5.4.1-2 Welded End Plate with Stiffeners

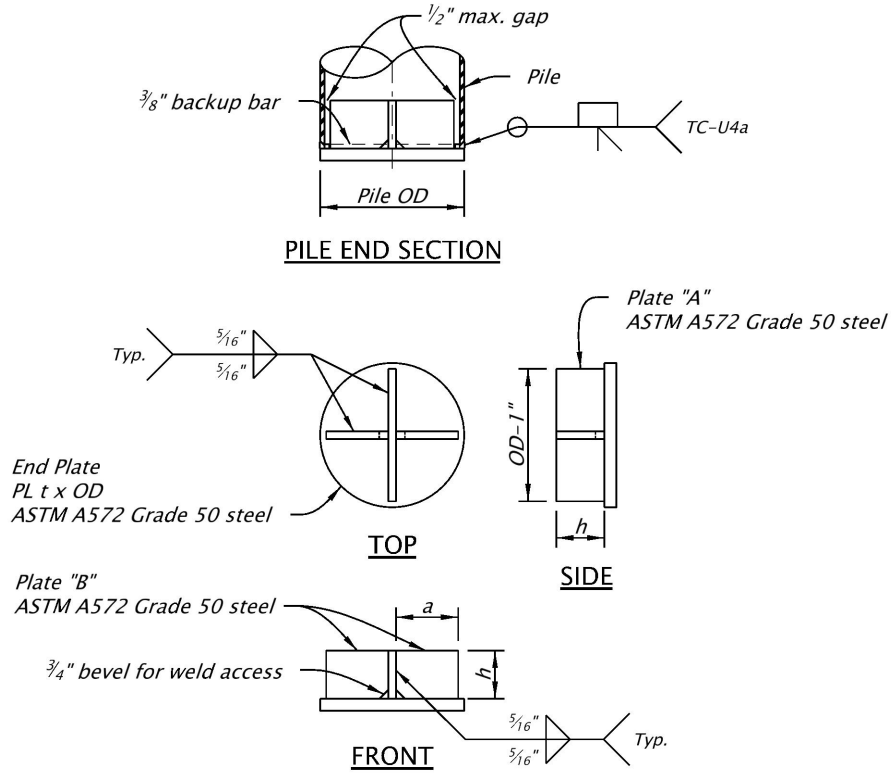


Table 1.10.5.4.1-3 Minimum Dimensions for Welded End Plates

	End Plate Thickness - No Stiffeners		End Plate Thickness with Stiffeners					
Pipe Pile Size	**Gr 2 Pipe**	**Gr 3 Pipe**	**Gr 2 Pipe**			**Gr 3 Pipe**		
			End Plate	**Stiffeners (Plates A & B)**	**Plate B Length**	**End Plate**	**Stiffeners (Plates A & B)**	**Plate B Length**
	t		**t**	**h x t**	**a**	**t**	**h x t**	**a**
PP12 ¾ x 3/8	2.25	2.5	1.00	4 x 0.375	5.500	1.25	4 x 0.500	5.250
PP12 ¾ x ½	2.50	2.75	1.00	4 x 0.500	5.375	1.75	4 x 0.500	5.125
PP16 x 3/8	2.50	2.75	1.00	6 x 0.375	7.125	1.25	6 x 0.375	6.938
PP16 x ½	2.75	3.00	1.25	6 x 0.375	7.000	1.50	6 x 0.500	6.750
PP18 x 3/8	2.50	2.75	1.00	6 x 0.500	8.125	1.25	6 x 0.500	7.875
PP18 x ½	3.00	3.25	-	-	-	-	-	-
PP24 x ½	3.25	3.75	-	-	-	-	-	-
t = thickness; h = height; a = length								

(9) PILE FOUNDATION DESIGN RECOMMENDATIONS

The geotechnical designer will provide final foundation recommendations in the Geotechnical Report, or earlier in the design process as needed. The following recommendations will typically be provided as a minimum:

- Pile Resistance: The nominal pile resistances (Rn) will be provided along with estimated pile lengths for one or more pile types. These values may be in tables or graphs of Rn versus depth may be provided. Modified Rn values will be provided as necessary to account for scour, and liquefaction conditions. The resistance factor will be provided along with the recommended method of construction control (i.e. dynamic formula, wave equation, etc.). Downdrag loads, when present, will be provided along with an explanation of the cause of the downdrag loads. The depth or thickness of the downdrag zone will be provided.
- The nominal pile uplift resistance will be provided either as a function of depth or for a given pile length (typically associated with the minimum tip elevation). The pile uplift resistance will be provided for normal static conditions and for any reduced capacity condition such as scour or liquefaction. The resistance factor will be provided.

- P-Y Curves: Foundation design parameters will be provided to develop p-y curves for lateral load analysis using either the LPILE or other soil response computer programs. Two sets of data may be required, one for static conditions and one for dynamic (liquefied soil) conditions.
- Required Pile Tip Elevations: Required minimum pile tip elevations will be provided along with an explanation of their basis. These tip elevations (minimum pile embedment) should be checked to see if they need to be modified to meet other design requirements, such as lateral loading requirements. Any changes to the recommended required tip elevations should be reviewed by the geotechnical designer.
- Special Provisions: The following foundation related items will be provided, as necessary, for Section *SP 00520* of the project Special Provisions:
    - Wave Equation Input (when WEAP is specified for driving criteria).
    - Recommended number of pile splices.
    - Pile tip treatment, tip reinforcement recommendations and specifications.
    - Recommendations regarding pile freeze, jetting, preboring or use of followers.

(10) PROTECTION FOR STEEL PILING

Assess all steel piling used in permanent structure applications for corrosion potential and design for the long term effects of corrosion. Steel pilings do not need a corrosion allowance when they are used in temporary works.

Reference *LRFD 10.7.5* for design requirements and guidance regarding steel pile corrosion assessment and protection. The design requirements and guidance provided in *LRFD* are further defined and supplemented in this section and the *ODOT GDM*.

Recommended pile corrosion assessment measures are provided below for corrosive and non-corrosive environments.

Buried steel piling corrosion requires oxygen and moisture. Corrosion risk is typically greatest from the bottom of pile cap/footing down to 3 feet below the lowest recorded/measured ground water elevation.

CORROSIVE AREAS

Protect all steel piling in corrosive areas with one or more corrosion protective system.

C CORROSIVE AREAS

When there is concern of microbial corrosion, conduct a Biological Activity Reaction Test (BART). A good warrant for this test is a site in proximity showing evidence of microbial corrosion. Contact the Bridge Corrosion Engineer with questions.

The following areas are considered corrosive areas for the purpose of this section:

- Marine areas, defined in *BDM 1.2.4.1.*

Project sites located beyond marine areas may still have a significant potential for pile

corrosion depending on site specific conditions. For example, if brackish water is present, or there are indications of potential corrosive conditions. Steel piling in waterways that extend above round such as in estuaries, lakes or streams may also be subject to significant corrosion.

- Pile in the atmospheric environment (as defined below) on Interstates and in snow and ice areas, as defined in *BDM 1.2.4.1*.

Consider exposure due to runoff, joint failure, splashing or overspray. This does not apply to sites where piles do not have any exposure (e.g. piles completely buried behind an MSE wall.)

Investigate the following corrosive sites with a thorough soil and groundwater testing program outlined in the *GDM*:

- Landfills or fill materials composed of cinders, ash or slag.
- Sources of mine or industrial drainage (acidic groundwater).
- Sites with stray electrical currents, such as electric (DC) transit systems, or high voltage power lines.

The circumstances or site conditions listed are highly corrosive and indicate the potential for severe corrosion. Corrosion rate depends primarily on electrochemical properties of the surrounding soil. Existing fill (especially man-made fill); disturbed native soils; and native soils containing cinders, slag, or ash are relatively likely to be considered aggressive compared other native soils.

Both surface and subsurface materials should be characterized to ensure representation of the strata at the site. Do not take water samples in flowing streams and rivers when the water level is elevated due to storm conditions as they may dilute the chemical concentrations.

Guidance on the extent of site investigations, including the soil sampling and testing required for corrosion assessment, are presented in the *ODOT GDM*.

Consider an area corrosive when it has one or more of the following conditions:

- Soil resistivity is less than 2000 ohm-cm.
- Soil pH is less than 5.5.
- Sulfate concentrations are greater than 1000 ppm.
- Chloride concentrations are greater than 500 ppm.

Test results should be representative of the environment, not a specific test location. Consult with the geotechnical engineer when needed.

NON-MARINE AREAS

Non-corrosive areas are all locations not designated as corrosive areas per *BDM 1.10.5.4.1*.

C NON-CORROSIVE AREAS

In non-corrosive areas, piles that are permanently buried and are always below the water table have a low potential for steel corrosion. Often no corrosion protection system is warranted.

Additional guidance and background information on the corrosion of steel piling in non-corrosive areas can be found in NCHRP Report 408 and *AASHTO R-27-01 (2015)*. The minimum pH criteria of 4.0 recommended in these reference documents does not supersede the 5.5 value used in *LRFD*.

Design buried steel piling in non-corrosive areas with a minimum sacrificial steel thickness defined in *Table 1.10.5.4.1-4*.

CORROSION PROTECTIVE SYSTEMS

Provide corrosion protection systems using one or more of the following: sacrificial thickness, protective coatings, cathodic protection.

Investigate the use of protective coating systems in the following order of preference: sacrificial thickness, protective coating, cathodic protection.

C CORROSION PROTECTIVE SYSTEMS

Other protective measures, such as concrete encasement or special steel alloys, may also be considered on a case by case basis. Contact the Bridge Corrosion Engineer for additional guidance.

Each system has advantages and disadvantages, as described below. Project budget typically drives the selection of a corrosion protection system. Some other

factors can include, but are not limited to: site corrosion potential, driving forces, structure loading, site environmental constraints, aesthetics and site accessibility.

Combining corrosion protection systems may extend the service life of the system. Pair this with a cost effective approach.

(1) Sacrificial Steel Thickness

C (1) Sacrificial Steel Thickness

Use a sacrificial steel thickness to account for steel section loss over the design life of the structure in non-corrosion and in corrosive areas only in soil embedded zones.

As a corrosion protection measure, sacrificial thickness allows for more rehab options in the future. It is also the most constructable option.

Do not use when the increase in pile size becomes uneconomical or unreasonable to for pile driving. Appropriate sites to use sacrificial thickness include, but are not limited to:

- Sites with variable or marginal test results (e.g. only failing one criteria).
- Sites with limited or deep exposure to native soil.

At a minimum, specify a thickness loss of 1/16 inch to account for possible corrosion loss occurring in the steel piles. Corrosion rates for use in determining thickness loss are specified in *Table 1.10.5.4.1-4*.

The corrosion rates in *Table 1.10.5.4.1-4* are based on information published in the WSDOT *BDM, 2014*, *CALTRANS March 2018, Corrosion Guidelines, version 3.0*, the *FHWA NHI-16-009 Design and Construction of Driven Pile Foundations - Volume I*, and the *Washington State Ferries Terminal Design Manual, 2016*.

Determine thickness loss over a minimum design life based on project design criteria. Use the reduced thickness in the pile design.

The potential for scour need not be considered when choosing a design corrosion rate as it relates to zones of exposure. It is assumed any significant scour would be repaired and the applicable zone of a structural element would not be changed.

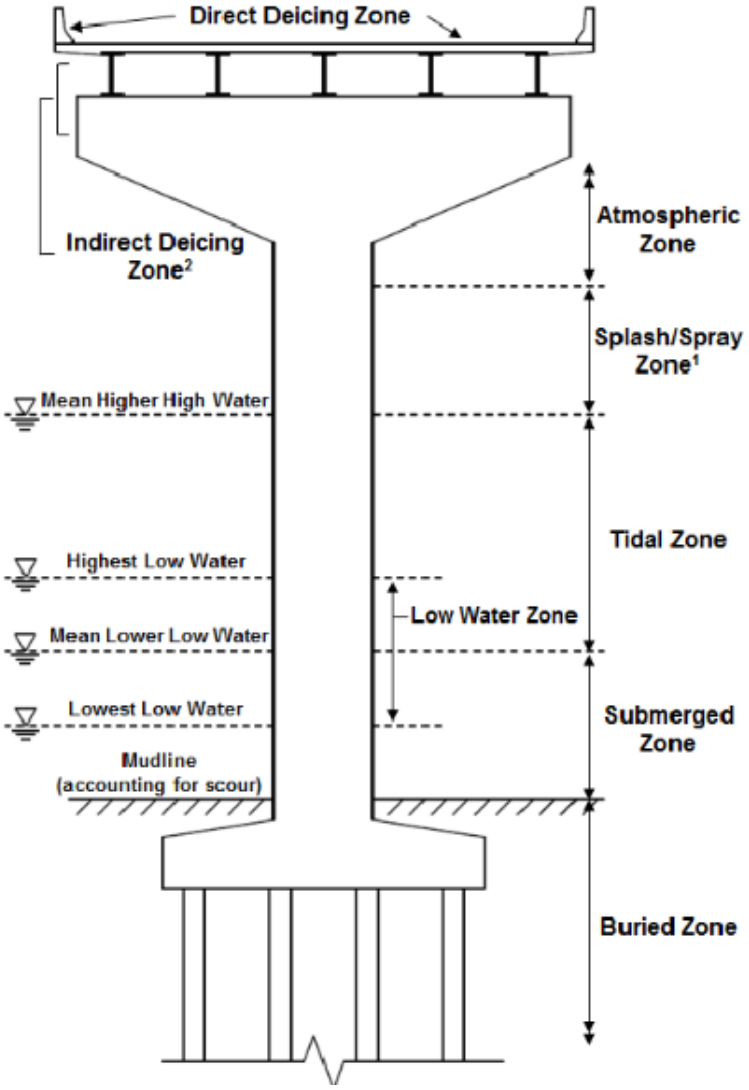
Table 1.10.5.4.1-4 Steel Pile Corrosion Rates

Pile Environment	Corrosion rate (inch/year)	
	Corrosive Areas	Non-Corrosive Areas
Soil embedded zone (undisturbed soil)	0.001	0.0005*
Soil embedded zone (fill or disturbed natural soils)	0.0015	0.00075*
Immersed zone	0.003	0.0015
Tidal zone	0.004	-
Splash zone	0.006	-
Atmospheric	0.002	0.001
* 1/16 inch sacrificial steel thickness is suitable for pipe piles, shells, and casings with a service life of 75 years or less.		

Definitions of the terms in *Table 1.10.5.4.1-4* are as follows and depicted in *Figure 1.10.5.4.1-1*:

- Immersed zone - continuously submerged in water. Immersed non-corrosive: environments with fresh water or are tested and found not to meet the marine area or corrosive area values.
- Tidal zone - between the Mean Low Water (MLW) and the Mean High Water (MHW).
- Splash zone - above the MHW plus five additional feet or as otherwise determined for a specific site.
- Atmospheric - above the splash zone or above ground line as applicable.

Figure 1.10.5.4.1-1 Typical exposure conditions of bridge elements



Double the corrosion loss for steel H-piling.

Steel H-piles have two surfaces on either side of the web and flanges that are exposed to corrosive conditions.

Apply the corrosion allowance only to the exterior surface of the pile for pipe piles, shells, and casings.

The interior of the pile will not be exposed to sufficient oxygen to support significant corrosion.

(2) Protective Coatings

Two types of coating systems are available: a 3-coat system and a 4-coat system.

Calculate section loss outside of coating service life and provide sacrificial thickness based on the section of pile with the highest corrosion rate not readily accessible to maintenance.

Account for variability in estimated tip elevations when designing coating lengths. This may increase the length of required coating.

Specify the 3-coat protective coating system for piles in direct contact with water. Coat piles full length at bents with pile tips within 25 feet of groundline.

Specify 4-coat protective coating system based on site preferences. Coat all other piles from final cutoff elevation to 25 feet below groundline.

C (2) Protective Coatings

A coatings effective life is generally assumed to be 15 years and there is no system maintenance. Coating is done off-site so installation does not require ground access to individual piles. Specify an appropriate coating system to withstand damage from handling and driving.

Areas not accessible to maintenance include embedded and immersed zones. Other may include steep and uneven terrain or sites with excessive access coordination and costs such as urban congestion, public land use, or railroads/utilities. When sacrificial thickness is not practical or too expensive, consider pile geometry that is conducive to future cathodic protection. (i.e. Choose pile configurations which minimize soil/water contact area, consider bracing details which reduce the number of anode beds, identify locations and/or install electrical attachment details, etc.).

The 3-coat protective coating system is comprised of a zinc-rich primer followed by two coats of moisture-cured urethane/urethane-tar. It comes in very limited color options, contact the Senior Structure Coating Engineer for additional information.

The 4-coat protective coating system includes the same 3-coat protective coating system followed by a top coat. The top coat is primarily an aesthetic coating which provides more color options, but limited additional protective qualities.

Use both *SP 00520* and *SP 00594* for protective coatings.

SP 00520 identifies where and what kind of coating system is to be used. *SP 00594* provides the preparation, application, materials, testing, measurement and payment for the specified coating system.

(3) Cathodic Protection

Use cathodic protection in conjunction with protective coatings on new steel piling installations to minimize anode consumption.

Use cathodic protection on existing steel pilings installations to address existing corrosion concerns.

C (3) Cathodic Protection

Cathodic protection should be considered in the most highly corrosive areas in new construction. Installation requires direct access to piles. Cathodic protection is a good fit for sites with stray currents or when driving conditions are not conducive to protective coatings.

Catholic protection does require regularly scheduled inspections of the exposed piling and maintenance, but if it is properly maintained it has the longest service life of the three systems.

Guidance for design and installation of anode systems for pilings in marine waters is provided by *NACE Standard SP0176*. Guidance for design and installation of anode systems for pilings in soil is provided by *NACE Standard SP0169*.

Make the electrical connection to the piling by brazing a brass stud on the downstream side of the piling. Provide the following project note on the plans:

> Remove coating from piling surface (approximately 4 by 4 inch area) and install ½ by 13 by 2 inch brass stud using an approved brazing process. Provide 2 brass nuts and 2 brass washers with stud. Recoat piling surface with protective coating after installation of brazed stud.

1.10.5.4.2 Piling Details

(1) STEEL PILE FOOTING EMBEDMENT TO DEVELOP FIXITY

Two types of pile head connection, i.e., pinned and fixed, are allowed for assumed boundary conditions to analyze piles or pile groups embedded in a reinforced concrete cap. This article describes how to achieve the boundary condition through pile embedment depth and steel reinforcing details in the pile cap.

For bridges in SDC A, when there is insignificant lateral load and tension capacity is not needed, pile embedment depth into the pile cap of 6 inches may be used. Assume a pinned boundary condition in structural analysis. Do not specify pile anchoring details, when pinned head piles are used.

For bridges in SDCs B, C, and D, when lateral load resistance is required, piles must be embedded adequately into pile caps with sufficient confinement reinforcing to develop the full bending moment capacity of the pile section.

Minimum required embedment depths for steel piles, cap dimensions, and steel reinforcing are shown in *Table 1.10.5.4.2-1*.

C1.10.5.4.2

Previous tests showed that a small embedment depth in the pile cap, such as 12 inches, can develop partial fixity. This partial fixity makes it difficult to accurately determine forces in the pile or displacement of the pile bent system. To reduce the uncertainty of the connection type, a very short embedment depth of 6 inches was chosen for the pinned connection assumption.

Designers may utilize the pinned boundary condition for some design situations to achieve desired displacement or reduce resultant force developed in pile section near the cap. For bridges located in low seismic zones, pinned head details can be specified. The shallow pile embedment simplifies steel reinforcing details in the pile cap and allows for easier pile cap construction. Research showed that a small size and number of reinforcing bars welded on the perimeter of pile extending into the pile cap could develop partial fixity.

Table 1.10.5.4.2-1 Minimum Pile Embedment and Design Details for Pile Cap

Shape				L_{emb} (in)**	Minimum Cap Size		U-bar ***
	M_p (K-ft)*	λ_{mo}	M_{po} (K-ft)		Width (in)	Height (in)	# legs
PP 12¾ x 3/8	285.6	1.20	342.7	20	36	40	4#7
PP 12¾ x ½	355.4	1.20	426.5	22	36	40	4#7
PP 16 x 3/8	478.0	1.20	573.6	22	36	40	6#7
PP 16 x ½	590.9	1.20	709.1	24	36	42	6#7
PP 18 x 3/8	625.7	1.20	750.8	24	40	42	6#7
PP 18 x ½	770.2	1.20	924.2	26	40	48	6#7
PP 20 x ½	978.2	1.20	1173.8	28	44	48	6#8
PP 24 x ½	1483.2	1.20	1779.8	32	48	52	8#8
HP 10 x 42	221.4	1.20	265.7	20	36	36	4#7
HP 10 x 57	304.8	1.20	365.8	22	36	40	4#7
HP 12 x 53	339.2	1.20	407.0	22	36	40	4#7
HP 12 x 74	481.3	1.20	577.6	26	36	42	4#7
HP 12 x 89	578.9	1.20	694.7	28	40	48	4#7
HP 12 x 117	782.8	1.20	939.4	32	40	52	6#7
HP 14 x 89	669.2	1.20	803.0	28	40	48	6#7
HP 14 x 117	889.2	1.20	1067.0	32	40	52	4#8
HP 16 x 141	1210.0	1.20	1452.0	36	44	54	6#8
HP 16 x 183	1599.6	1.20	1919.5	40	44	60	6#8
HP 18 x 204	1984.6	1.20	2318.5	42	48	62	8#8

* The plastic moment capacity of piles, M_p, was calculated using expected material properties, i.e. 1.3 x f'_c for concrete (f'_c = 3.3 ksi), 1.1 x F_y for steel piles (F_y = 45 ksi for pipe piles and F_y = 50 ksi for H piles). The moment capacity of pipe piles was based on a pipe pile section filled with concrete and was calculated according to the plastic stress distribution method *LRFD C6.12.2.3.3*.

**The minimum embedment length was calculated using specified minimum material properties.

***Example of U-bar details is shown in *Figure 1.10.5.4.2-1*.

The pile embedment depth shown in *Table 1.10.5.4.2-1* is a minimum required embedment depth regardless of seismic design category or stress level in the steel pile.

The plastic moment M_p and the overstrength plastic moment M_{po} may be used for steel pile and pile cap designs for the Extreme Event limit state. For strength limit state design, use specified minimum material properties to calculate pile plastic moment capacity.

Laterally loaded pile design with a fixed pile head is typically governed by the Extreme Event load combinations. Therefore, load and resistance factors for the Extreme Event limit state were used to determine the embedment depth shown in the table. The embedment depths were calculated considering equilibrium of the resultant of the upper compression block developed at the face of the piles against concrete taken to be equal to that of the lower compression block caused by the overstrength plastic bending moment as described in *FHWA/MT-13-001/8203 (MDT) report (2013)*.

When design parameters and details, such as pile material properties, pile section, additional reinforcing steel inside the pipe pile, etc., deviate from the parameters used for the minimum required embedment depth and details shown in *Table 1.10.5.4.2-1*. The embedment depth, L_{emb}, of steel piles can be calculated as follows:

$$L_{emb} = \sqrt{\frac{2 \times M_{po}}{D_{pile} \times \beta_1 \times (1 - \frac{\beta_1}{2}) \times \varphi_b \times \alpha \times f'_c}}$$

where,

M_{po} = overstrength plastic moment of pile taken as λ_{mo} x M_p (kip-ft)

λ_{mo} = overstrength magnifier = 1.20

D_{pile} = diameter or minimum width of pile (in.)

β_1 = stress block factor varied with concrete strength

ϕ_b = resistance factor for bearing on concrete = 0.70

α = concrete confinement factor = 1.50

f'_c = concrete strength (ksi)

Several methods for calculating flexural strength of pipe pile filled with concrete are described in *AASHTO Guide Specifications 7.6.2* (Exact Geometry and Approximate Geometry methods) and *LRFD 6.12.2.3.3* (the plastic stress distribution and strain compatibility methods). It was found that the plastic stress distribution method yielded slightly larger flexural strength of pipe pile filled with concrete compared to the other methods. The Approximate Geometry method generally resulted in the least flexural strength and may conservatively be used for pile design.

When bending moment demand at the pile-to-pile cap location is less than 0.60 M_p, unfilled pipe piles may be used. Specify the embedment depth shown in *Table 1.10.5.4.2-1*.

Specify concrete-filled pipe piles for piles resisting bending moment demand at the pile-to-pile cap location equal to or greater than 0.60 M_p or when the embedded piles are located in SDC C or D. To ensure that the full bending moment capacity of the pile section is developed, flexural capacity of the cap must be equal to or greater than the overstrength plastic bending moment of the pile plus dead load negative bending moment and bending moment due to the plastic shear.

Provide U-bars at the bottom of the embedded pile according to *Table 1.10.5.4.2-1*. Place U-bars as close to the bottom fiber of the pile cap as possible and within the bottom half of the embedment depth. Provide #5 spirals with a 4-inch maximum pitch near the top of the embedded pile within the location shown in *Figure 1.10.5.4.2-1*.

In addition to steel reinforcing design for shear, satisfy the joint design requirements according to *AASHTO Guide Specifications 8.13*.

Figure 1.10.5.4.2-1 shows minimum steel reinforcing in pile caps. Stirrups with a 4-inch maximum spacing are required to ensure confinement for concrete around the embedded pile head. In cases where a wider cap width is specified, ensure that stirrups provided in the confinement zone meet the following equation:

$$A_{sh} = 0.12 \times s \times b_{cap} \times \frac{f'_c}{f_y}$$

where,

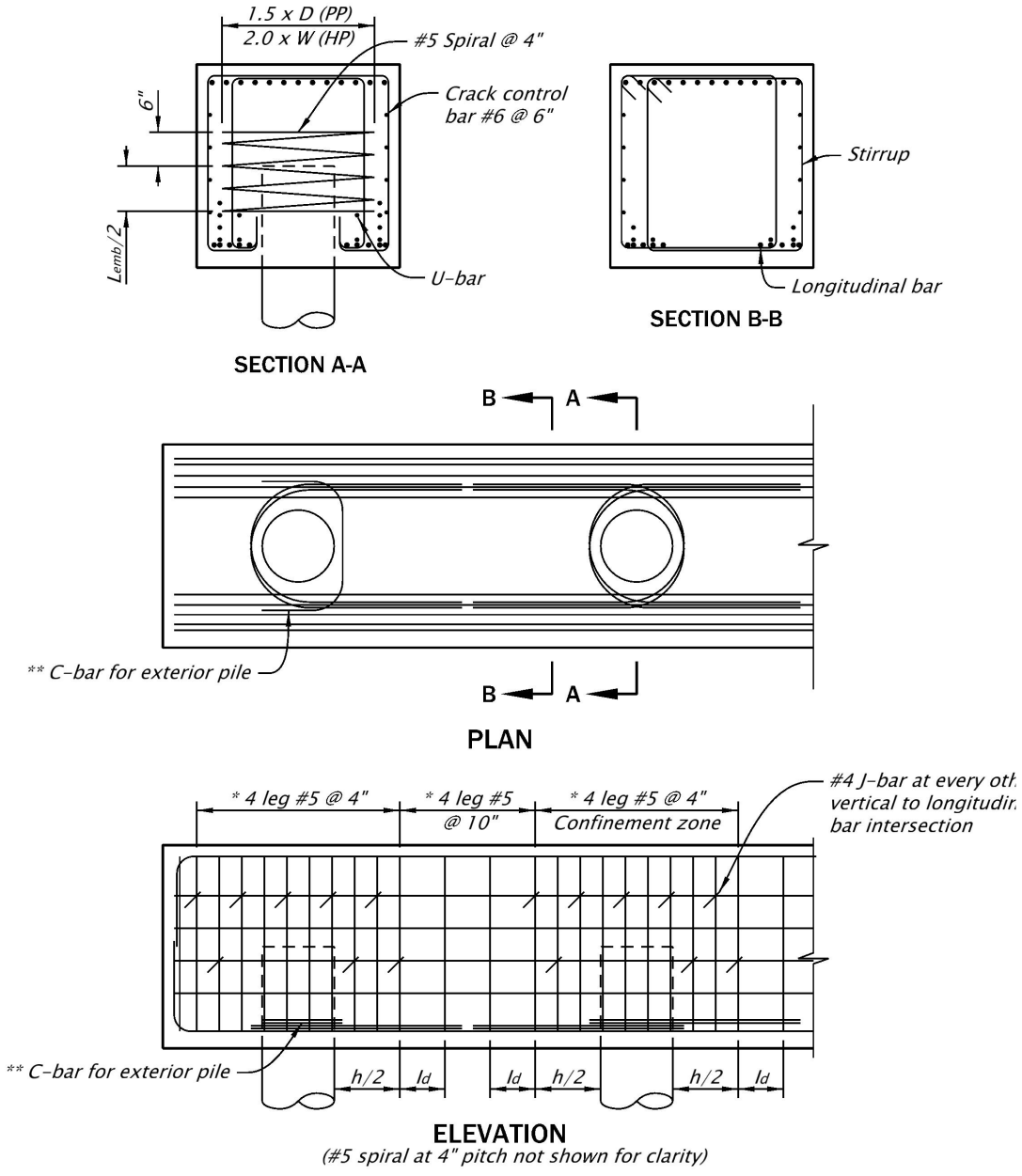
A_{sh} = total area of stirrup legs (in^2)

s = stirrup spacing (in.) (no greater than 4 in.)

b_{cap} = width of the pile cap (in.)

f_y = yield strength of rebar (ksi)

Figure 1.10.5.4.2-1 Minimum Steel Reinforcing Details for Pile Caps with Pile Moment Demand ≥ 0.60 M_p or Located in SDCs C or D



** Use #6 bars for PP 24x1/2 and HP 18x204.*

*** Provide C-bars at each exterior pile having the same bar size and number of U-bars. Provide hook length = 90 degree bend + pile diameter + the cantilever length of pile cap or 90 degree bend + pile diameter + h/2 + l_d, whichever is less.*

Fill the embedded portion of pipe pile with concrete plus 5 feet below the pile cap soffit. Do not specify coating on the surface of embedded pile. When protective coating is used, stop the pile coating 2 inches maximum from the pile cap soffit. When tension capacity of the embedded pile is required, provide pile anchoring details according to *BDM 1.10.5.4.2(4).*

When space is limited for U-bars, such as in footing pile caps, provide #5 spiral at 4-inch pitch around the embedded pile portion from the top of the bottom longitudinal reinforcing mat to 6 inches above the embedded pile.

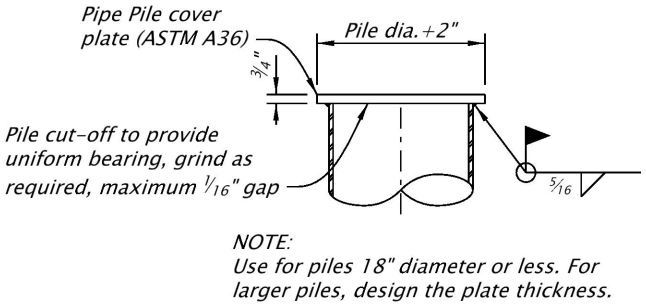
According to the *MDT report*, the damage threshold of concrete in the cap specimens during the tests was 0.75 M_P. For conservative design, 0.60 M_P was chosen as the loading limit before the rigorous steel reinforcing details are required. This is to ensure that plastic hinging will occur in steel pile section with limited concrete cracking in the pile cap. Use of the U-bars around the embedded pile followed the recommendations described in the *MDT report*. The top reinforcing spiral was required to avoid premature internal concrete cracking in the cap and increase the confinement. Transverse steel reinforcing in the embedded pile zone were also detailed according to the *MDT report*.

Several methods for casting concrete inside pipe piles include using formwork installed at the specified elevation or filling the pipe pile with gravel or controlled low strength materials (CLSM) up to the specified elevation before pouring the structural concrete.

(2) PIPE PILE COVER PLATES

Provide a welded cover plate as detailed below in *Figure 1.10.5.4.2-2*.

Figure 1.10.5.4.2-2 Pipe Pile Cover Plate

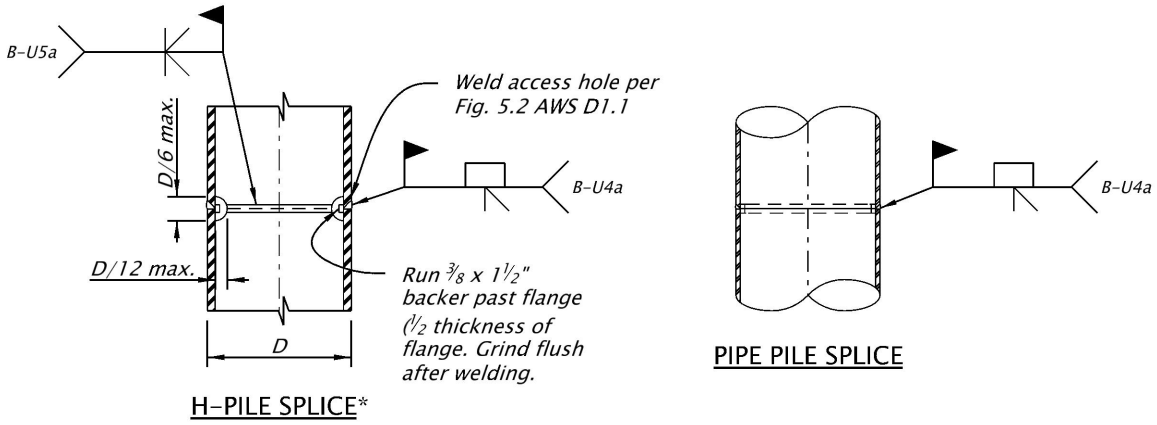


PIPE PILE (CLOSED END)

(3) STEEL PILE SPLICES

When splicing of steel piles is anticipated, show one or both of the following details on the plans.

Figure 1.10.5.4.2-3 Steel Pile Splices



H-PILE SPLICE*

PIPE PILE SPLICE

* – Manufactured A709 grade 36 H-pile splices may be used when located a minimum of 40 feet below the bottom of the footing and installed according to the manufacturer's recommendations.

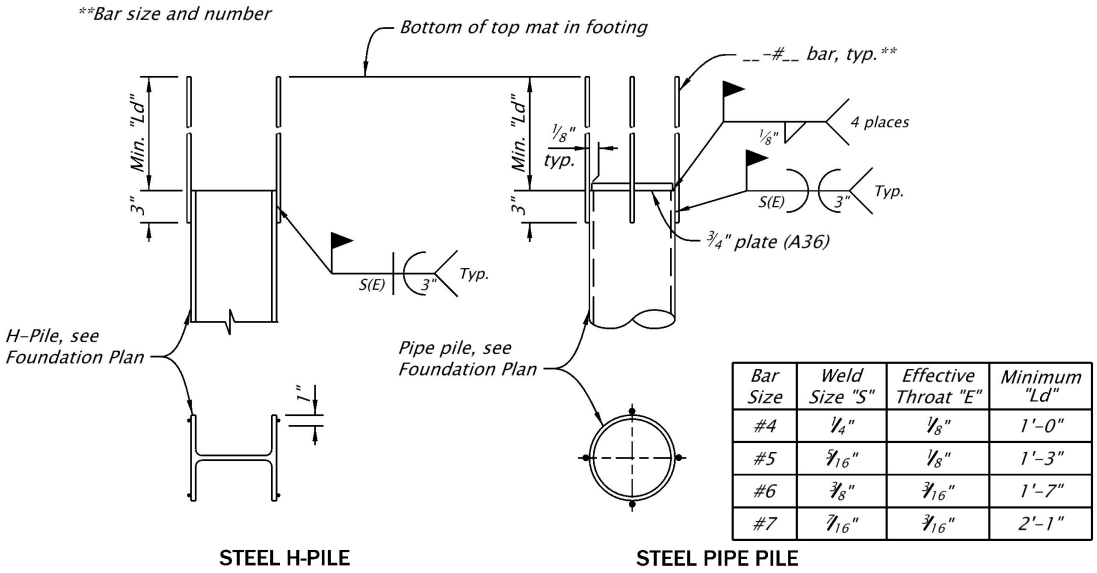
(4) ANCHOR PILES

Two methods of anchoring piles are shown in *Figure 1.10.5.4.2-4*.

When pile anchoring details are used, specify embedment depth according to *Table 1.10.5.4.2-1* and assume fixed boundary conditions in the structural analysis.

Other methods such as extending the top plate and using welded studs or other shear connectors may be appropriate. Submit proposed details to State Bridge Engineer for approval before use.

Figure 1.10.5.4.2-4 Anchor Pile Details (A) Steel H-Pile (B) Steel Pipe Pile (C) Filled Pipe Pile



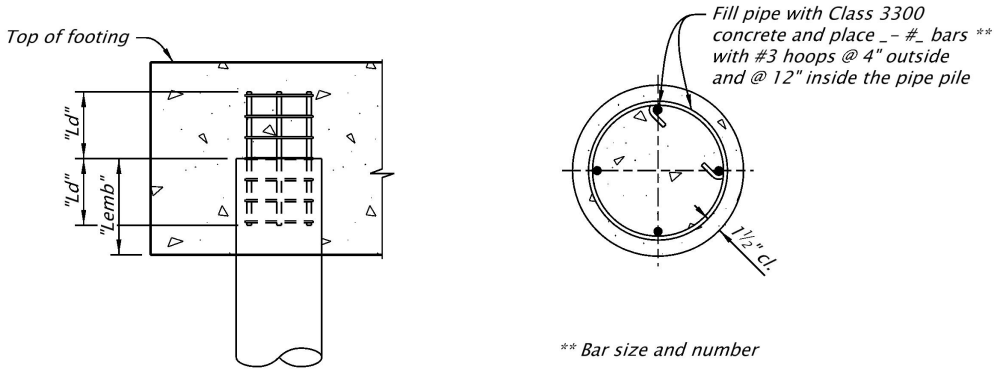
Bar Size	Weld Size "S"	Effective Throat "E"	Minimum "Ld"
#4	1/4"	1/8"	1'-0"
#5	5/16"	1/8"	1'-3"
#6	3/8"	3/16"	1'-7"
#7	7/16"	3/16"	2'-1"

STEEL H-PILE

STEEL PIPE PILE

* Provide ASTM A706, except ASTM A615 Grade 60 or ASTM A496 may be used if copies of the chemical composition analysis are submitted and approved as weldable by the Engineer.

ANCHOR PILE DETAILS



FILLED PIPE PILE ANCHOR DETAILS

1.10.5.5 Drilled Shafts

Use drilled shafts for bridge foundations only when:

- The geotechnical designer has recommended drilled shafts for the preferred foundation type AND
- The design is economical (relative to other deep foundation designs).

Consult with the geotechnical designer regarding site constraints, environmental issues, constructability and lateral loads before selecting drilled shafts for foundation design. Locate drilled shafts early in the design process so an exploration drill hole can be located as close as possible to all drilled shaft locations for design and construction purposes.

C1.10.5.5

A Drilled Shaft Task Force Group exists to aid geotechnical and bridge designers in resolving constructability issues, revising specifications, and successfully implementing new technology. The Drilled Shaft Task Force Group is led by the Senior Bridge Geotechnical Engineer and comprised of ODOT personnel and representatives from the drilled shaft industry. Consider engaging this group early in the design process.

1.10.5.5.1 Design Considerations

(1) DRILLED SHAFT DIAMETERS, COVER REQUIREMENTS, AND HORIZONTAL TOLERANCES

The geotechnical and bridge designers should confer early in the design process to decide the most appropriate shaft diameter(s) to use for the bridge given the axial and lateral loads, column diameter, subsurface conditions, and other relevant factors.

Common shaft sizes range from 3 to 12 feet in diameter. Large shafts are difficult to construct to precise horizontal tolerances. Do not design columns the same diameter as the shaft. Consider allowable horizontal tolerances, 3 inches for 6 feet shaft or smaller; and 6 inches for shaft larger than 6 feet, in shaft sizing and design. Provide additional shaft capacity to resist possible load demands in shaft that is not constructed at as-specified location.

Size drilled shaft diameters, concrete cover in drilled shafts, and column diameter using the *Table 1.10.5.5.1-1*.

Table 1.10.5.5.1-1 Drilled Shaft Dimensions

Diameter (feet)		Concrete Cover (inches)	Horizontal Tolerance (inches)
Shaft	Max. Column		
3	2	3	3
3.5	2.5	4	
4	3		
4.5	3.5		
5	4		
5.5	4.5		
6	4	6	
7	5		6
8	6		
9	7		
10	8		
11	9		
12	10		

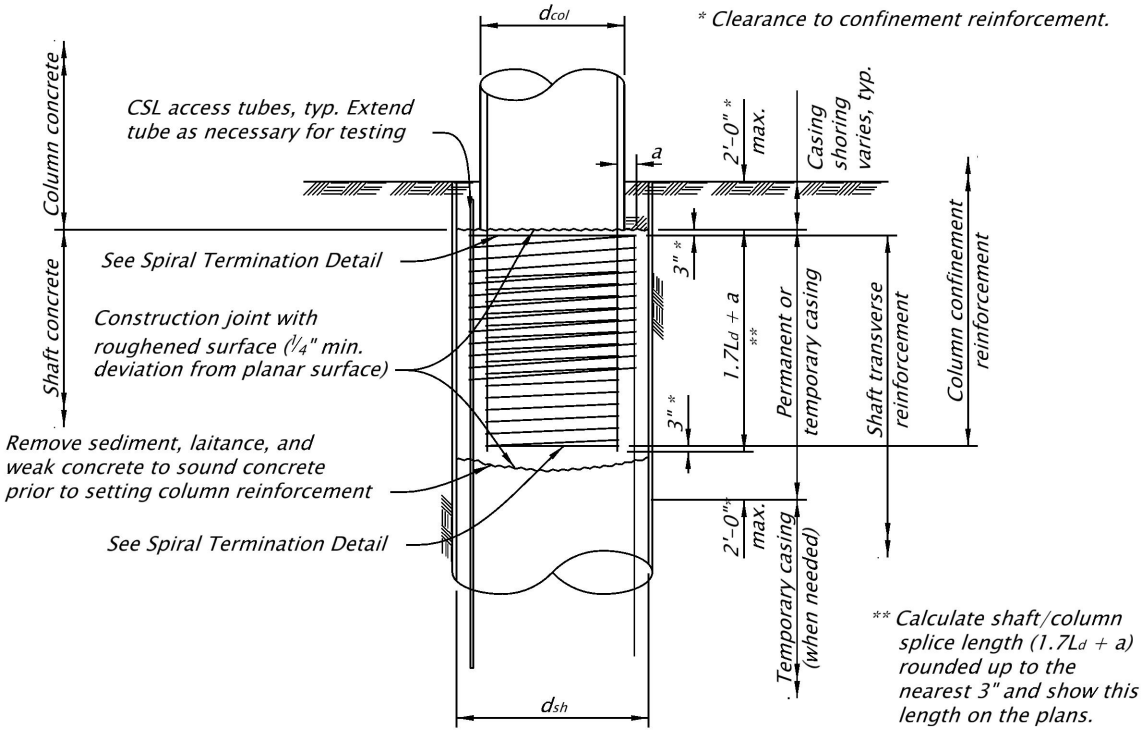
(2) NON-CONTACT SHAFT/COLUMN SPLICE

Detail shaft/column splice regions in accordance with *Figures 1.10.5.5.1-1* or *1.10.5.5.1-2*. The splice region is $(1.7Ld + a)$ rounded up to the nearest 3 inches. Note that Ld is the modified development length per *LRFD 5.10.8.2.1*. The non-contact splice detail allows the column to be adjusted horizontally when the shaft is slightly out of position (but still within the horizontal tolerance for the shaft). The shaft vs. column size limits are selected to ensure this adjustment can be made without increasing the tolerance more than the standard 1 inch for the column.

Non-contact splices require a lot of equipment, often in space-limited areas. Use of permanent casing should be considered for instances when the geotechnical designer recommends or identifies caving conditions, restricted space, and worker safety. Permanent casing changes the resistance of the drilled shaft. Use of permanent casing must be communicated with the geotechnical designer so new depth vs resistance graphs can be developed and provided to the bridge designer. Permanent casing may result in longer drilled shafts.

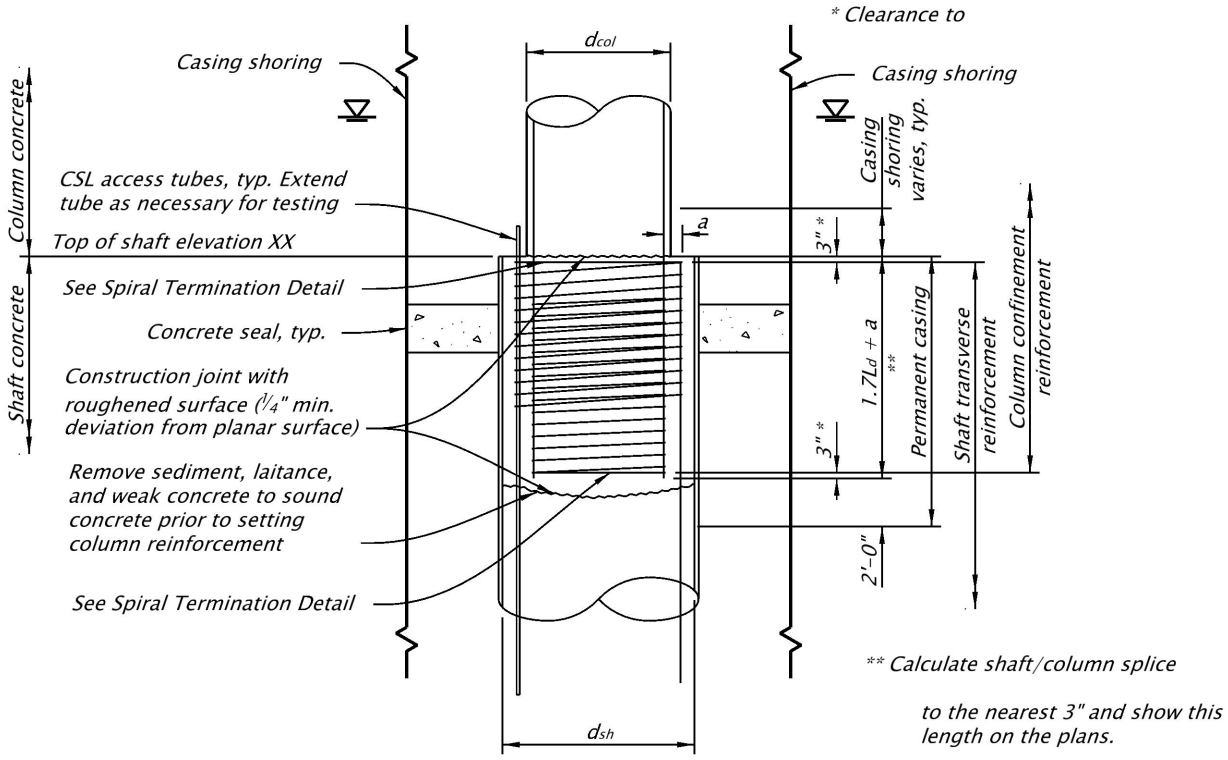
Often, during construction temporary casing cannot be extracted. Bridge designers should review the consequences of temporary casing becoming permanent casing. Communicate with the geotechnical designer these consequences so that an appropriate Drilled Shaft Installation Plan is approved during construction. Casing configurations shown on *Figures 1.10.5.5.1-1* and *1.10.5.5.1-2* are shown as examples only.

Figure 1.10.5.5.1-1 In-Ground Non-Contact Drilled Shaft Splice



$a = \frac{1}{2}$ (shaft spiral dia. – column spiral dia.)
d_{col} = column diameter
d_{sh} = shaft diameter
L_d = Modified tension development length per LRFD 5.11.2.1.1

Figure 1.10.5.5.1-2 In-Water Non-Contact Drilled Shaft Splice



a = ½ (shaft spiral dia. – column spiral dia.)
d_{col} = column diameter
d_{sh} = shaft diameter
L_d = Modified tension development length per LRFD 5.11.2.1.1

(3) DOWNDRAG LOADS

Downdrag loads, due to soil settlement other than that caused by dynamic (seismic) loading, are added to the factored vertical loads on the foundation in the Strength Limit state. Load Factors for downdrag loads will be provided by the geotechnical designer. Downdrag loads resulting from liquefaction or dynamic (seismic) induced soil settlement should be considered as a permanent load and included the Extreme Event Limit State shaft design.

(4) SHAFT UPLIFT RESISTANCE

Shaft uplift resistance is usually the same as the side friction resistance. Friction resistance in downdrag zones should be considered available for uplift resistance.

(5) SHAFT ROCK SOCKETS

Minimum shaft embedment depths into hard rock, or rock sockets, may be required due to one or more of the following design requirements or conditions:

- Lateral Load, due to earthquake loading.
- Scour.
- Liquefaction.
- Uplift loads.
- Settlement or downdrag.
- Required soil/rock bearing strata.

For rock sockets constructed inside shafts that will require either temporary or permanent casing, consider designing the diameter of the rock socket smaller than the diameter of the cased shaft above the rock socket. This is necessary to accommodate rock auger tools which are smaller in diameter than the nominal outside diameter of the cased shaft. Reduce the shaft diameters of rock sockets by at least 6 inches in these cases.

The required rock socket embedment depths should be shown on the plans. Under this condition, shaft tip elevations should be shown as **Estimated Tip Elevations** since they are likely to change depending on the actual elevation of the top of rock or hard bearing strata encountered during construction. The geotechnical designer should provide an additional shaft length that accounts for the uncertainty in the top of the bearing layer and this additional length should be specified in the Special Provisions. In these cases, add the additional reinforcement required for this additional shaft length into the estimated quantities provided in *SP 00512*. Also adjust the concrete quantities to include this additional length. Extra reinforcement length can quickly and easily be cut off to provide the proper cage length once the final tip elevation is determined.

(6) SHAFT SETTLEMENT

Refer to *AASHTO* methods to calculate the settlement of individual shafts or shaft groups. Compare this settlement to the maximum allowable settlement and modify the shaft design if necessary to reduce the estimated settlement to acceptable levels. End bearing shafts on soil will typically settle more than friction shafts in order to mobilize end bearing resistance.

(7) SHAFT GROUP EFFECTS

For group lateral load analysis use the p-y multiplier methods described in *AASHTO* and the FHWA Manual *Drilled Shafts: Construction Procedures and Design Methods*.

(8) SHAFT SPACING

Use a minimum spacing of 3 feet for drilled shafts.

(9) SHAFT FOUNDATION DESIGN RECOMMENDATIONS

The geotechnical designer will provide final foundation recommendations in the Geotechnical Report, or earlier in the design process as needed. The following recommendations will typically be provided as a minimum:

- Shaft Resistance: The nominal shaft resistance (Rn) will be provided along with estimated shaft tip elevations for one or more shaft diameters. This may be in the form of tables or graphs of Rn versus depth may be provided. Modified Rn values will be provided as necessary to account for scour, liquefaction or downdrag conditions. The resistance factors used will be provided. Downdrag loads, when present, will be provided along with an explanation of the cause of the downdrag loads. The depth or thickness of the downdrag zone will be provided.
- Shaft Settlement: Estimates of shaft settlement will be provided for the range of loads expected. The geotechnical designer will need to know the anticipated service loads on the shaft for these calculations along with any limiting settlement criteria.
- Shaft Uplift Resistance: When required for design, the nominal shaft uplift resistance will be provided either as a function of depth or for a given shaft length. The uplift shaft resistance will be provided for normal static conditions and for any reduced capacity condition such as scour or liquefaction. The resistance factors used will be provided.
- P-Y Curves: Foundation design parameters will be provided to develop p-y curves for lateral load analysis. Two sets of data may be required, one for static conditions and one for dynamic (liquefaction) conditions when they exist.
- Special Provisions: The following foundation related items will be provided, as necessary, for *SP 00512*:
    - Designation as either a **friction** or **end-bearing** shaft; for cleanout purposes.
    - Permanent casing (when recommended by geotechnical designer or otherwise required).
    - Crosshole Sonic Log testing requirements.

(10) POST INSTALLATION VERIFICATION TESTING

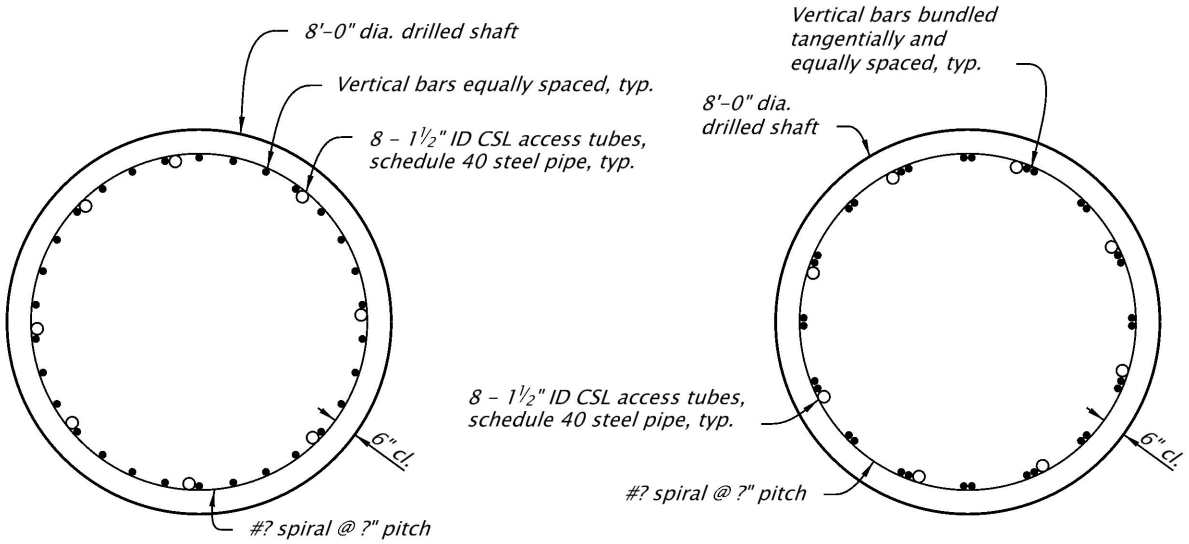
Crosshole Sonic Log (CSL) Testing

In general CSL tubes are installed in all drilled shafts unless otherwise recommended in the Geotechnical Report. CSL tubes may not be required in some cases where foundation conditions may be very favorable and there is redundancy in the foundation design. Consult with the geotechnical designer regarding the CSL testing that should be performed on the project.

The rule of thumb is one CSL tube per foot diameter of shaft, with no less than four tubes and rounding up. They are equally spaced around the shaft as shown in *Figure 1.10.5.5.1-3*.

CSL tubes are comprised of 1½ inch inner diameter, schedule 40 steel pipe, must be water tight and removable caps at the top for access. PVC access tubes can be used however are discouraged because debonding is more prevalent than with steel.

Figure 1.10.5.5.1-3 CSL Tubes in Drilled Shaft



Thermal Integrity Profile Testing

Thermal Integrity Profile (TIP) testing is a post installation integrity verification test and should be considered in addition to CSL testing in some cases. TIP should be used for drilled shafts which are non redundant, or large, or constructed underwater. Unlike CSL, TIP is able to provide rebar cage cover in addition to integrity of the drilled shaft core. TIP thermal sensors are spaced at 1 foot intervals. Quality control specifications for the use of TIP are being developed, contact the Senior Bridge Geotechnical Engineer for additional guidance.

(11) SHAFT REINFORCEMENT

Determine the moment to be transferred from the column to the top of shaft according to the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*. The maximum shaft moment depends on the soil-structure interaction and is generally larger than the top of shaft moment.

Design shaft transverse reinforcement for the lesser of the plastic shear or elastic seismic shear of the column. Since the shaft diameter must exceed the column diameter, the shaft essentially remains elastic under seismic loads. If so, there is no need to satisfy the volumetric ratio and spacing requirements for transverse reinforcement in *LRFD 5.13.4.6.3*.

As well as meeting plastic shear or seismic shear demands, ensure shaft transverse reinforcement within the non-contact splice region meets the requirements in *LRFD 5.11.5.2.1*.

Detail shaft reinforcing to minimize congestion and facilitate concrete placement. Space both longitudinal and transverse reinforcement to provide 5 inches minimum and 9 inches maximum

clear spacing between bars. In non-contact splice regions, transverse reinforcement spacing in the column can be as small as 3 inches in some cases. Provide 5 inches minimum clear spacing between transverse reinforcement in the shaft to minimize congestion. Transverse shaft reinforcement may include spiral bars, hoops or bundled pairs.

(12) SHAFT CONCRETE

Use Class 4000 – 3/8 concrete in all drilled shafts. Concrete for drilled shafts should generally have a high slump and relatively small aggregate size in order to properly flow through the shaft reinforcement and provide the required fluid pressures against the sides of the bore hole. This is necessary to develop the desired friction resistance. Placement of concrete may be by free fall (in dry holes) or by tremie pipe (in dry or wet holes). At the present time, free fall placement of concrete in dry holes is allowed to unlimited depths. Refer to the report *Effects of Free Fall Concrete in Drilled Shafts (ADSC Report No. TL112*) for more information.

(13) REINFORCEMENT CONNECTIONS

Do not specify hooked longitudinal bars at the top of the shaft (extending into footings or caps) that will conflict with temporary casing removal. Design and detail reinforcement considering the requirements of temporary casing.

(14) REINFORCEMENT SPLICING

For shafts constructed at locations where a minimum penetration into the rock (or hard strata) is required and the elevation of the top of rock is uncertain, consider adding additional lengths of reinforcement to avoid the need for splicing. Once the final tip elevation is determined, any remaining rebar length can be cut off and removed. Splicing of reinforcement is undesirable because it usually results in delaying the concrete pour which could lead to other problems. When splicing is required, provide splicing details on the plans.

(15) SHAFT ELEVATIONS

Show or list the **Top of Shaft** elevation on the plans for each drilled shaft. This elevation is the top of the drilled shaft concrete. Also show or list shaft tip elevations. When shaft tip elevations are anticipated to vary due to uncertainties in the top of the bearing strata then label these as **Estimated Tip Elevations** and show the required penetration depth into the bearing strata.

(16) PERMANENT CASING

The use of permanent casing may be beneficial in locations especially where the top of shafts are constructed in open water such as rivers or lakes. The use of permanent casing can simplify construction by eliminating the need for any temporary casing and forms. When permanent casing is desired it should be taken into account in the structural analysis of the bridge because it increases the stiffness and strength of the shaft and may significantly affect the overall response of a bridge subject to large lateral loads. It also affects the geotechnical side resistance. Consult with the geotechnical designer when permanent casing is to be used.

When permanent casing is specified remember to take OSHA requirements into account when determining casing lengths. OSHA may require casing to extend at least 2 feet above the ground surface during construction. This additional length may later be cut off and removed after the shaft is constructed.

When permanent casing is required, provide casing diameters, thicknesses and lengths in the special provisions.

(17) SHAFT DIAMETER FOR SEISMIC ANALYSIS

Drilled shafts are generally constructed slightly larger than the nominal diameter shown. For example, in soil conditions where casing is required, a 6 foot diameter shaft cannot be drilled inside a 6 foot diameter casing. A larger size casing diameter is required. Discuss with the geotechnical designer whether or not casing may be required and a larger shaft size should be checked in the structure stiffness analysis (i.e., seismic analysis). An oversize of 6 inches is recommended for shafts up to 6 feet diameter and 12 inches is recommended for larger diameter shafts.

(18) DRILLED SHAFT PRECONSTRUCTION MEETING

Preconstruction meetings are held prior to beginning drilled shaft construction. This meeting should be attended by the structural designer who designed the shaft.

1.10.5.6 Seismic Foundation Recommendations

The geotechnical designer shall provide the seismic ground motion values for the Cascadia Subduction Zone Earthquake and the 1000-year return period earthquake. Liquefaction potential is addressed along with recommendations regarding estimated lateral deformations of embankments and dynamic settlement and downdrag potential. Downdrag loads resulting from liquefaction or dynamic compaction (settlement) will be provided. Liquefaction mitigation measures and recommendations are addressed if necessary (see *BDM 1.17.4* for Liquefaction Mitigation Procedure).

1.10.5.7 Overall Stability Evaluation

The geotechnical designer shall evaluate the overall stability of the approach fills leading up to the bridge and all other unstable ground conditions, such as landslides or rockslides, that may affect the structure. This analysis shall include both static and dynamic analysis of slope stability as related to the service and extreme limit state designs. This analysis is to determine potential impacts to the bridge and approach fills which may result from embankment instability, landslide movements, settlement or other potential ground movements. A thorough geotechnical investigation, focused on slope instability, should be conducted in accordance with *the ODOT Geotechnical Design Manual (GDM).* Methods for evaluating overall stability and for estimating settlements and displacements are also described in the *GDM*. The overall stability

analysis should include both non liquefiable and liquefiable foundation soil conditions as appropriate. This evaluation should be completed as early as possible in the design process to allow for changes in location and modifications to the bridge design to accommodate slope instability conditions. Coordinate with the geotechnical designer to resolve any slope instability issues that will affect the final bridge design.

For the Service Limit State, the overall stability of bridge approach fills not supporting abutment spread footings should provide a minimum factor of safety of 1.3, (roughly equivalent to a resistance factor of 0.75). A factor of safety of 1.5 against overall stability should be provided for end bent spread footings supported directly on embankments or bridge retaining walls. For bridges that are located in landslide areas, or in areas that could be affected by slide movements, the slide should be stabilized to the same factors of safety as stated above for approach fills or as determined by the Region Tech Center Geotechnical Section and Senior Bridge Geotechnical Engineer.

For Extreme Limit State I (seismic loading) conditions, the overall stability and displacement of the approach fills should be evaluated. In addition, other potentially unstable ground conditions, such as landslides or rockfalls, should also be investigated and evaluated for their potential impacts on the structure due to earthquake forces. A minimum factor of safety of 1.1 should be provided for the pseudo static analysis of bridge approach fills, landslides and any other potentially unstable ground conditions that may affect the structure. This criterion applies to sites with or without liquefiable foundation soils. In addition to this requirement, ground displacements (lateral and vertical) should be estimated and evaluated in terms of meeting the seismic design performance criteria described in *BDM 1.17.1*. This performance criterion also applies to liquefiable or non-liquefiable foundation soil conditions. The Newmark approach is recommended for estimating the lateral displacements of approach fills, adjacent slopes, landslide masses or other ground features that may affect the structure. Other methods for estimating lateral ground deformations under seismic loading are presented in the *ODOT Geotechnical Design Manual*. When estimated ground displacements result in excessive deformation or damage to the bridge such that the performance criteria cannot be met, then mitigation measures should be pursued. The limits of liquefaction mitigation described in *BDM 1.17.4* also apply to all non-liquefiable soil conditions that require mitigation measures to meet the specified performance criteria.

1.10.5.8 Scour Design Requirements

Design bridge foundations according to the scour design criteria in *Table 1.10.5.8-1*.

In the case of bridges that overtop below the 100 year flood event, coordinate with the hydraulics designer to determine appropriate limit case scenarios.

C1.10.5.8

See *GDM* and *LRFD 2.6.4.4* for additional information on scour design information and footing elevation.

This section addresses scour during seismic events but does not address liquefaction. Refer to *BDM 1.17* for additional information on liquefaction and additional seismic related requirements for foundation design.

See *Hydraulics Design Manual 10.8* for definition of base flood, overtopping flood, check discharge and total scour. The scour elevation will provided by the hydraulic designer in the *Hydraulics Report*. See *Figure 1.10.5.8-1* for a depiction of scour definitions.

When drift or debris is a concern, coordinate with the hydraulic designer to review the bridge inspection reports and check-in with District Bridge Supervisor to confirm any ongoing debris issue, and then coordinate debris criteria with the project team when applicable.

Refer to the *GDM, Hydraulics Design Manual* and ultimately the *Hydraulics Report* on how scour countermeasures are incorporated into the foundation design. See *BDM 1.10.5.4.1* and *Hydraulics Design Manual 10.12* for additional information.

For Extreme Event II, the effects due to degradation of the channel should be considered.

Figure 1.10.5.8-1 Scour Components

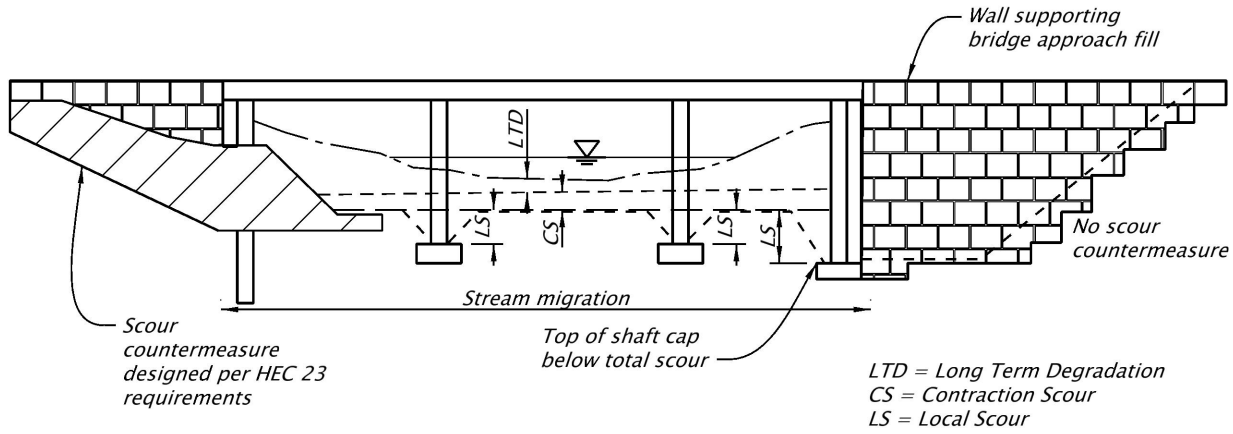


Table 1.10.5.8-1 Scour Design Criteria Based on Limit State

Limit State	Total Scour (percentage)	*Flood
Service	100	Q100
Strength	100	Q100
Extreme Event I Case 1	0	Q2
Extreme Event I Case 2	50	Q2
Extreme Event II	100	Q500
* Use this flood for calculating the WA load.		

CHANNEL MIGRATION

Consider the potential for channel migration and its effect on scour depth outside of the main channel when determining the potential depth of scour at the bridge abutment and at interior piers outside of the main channel within the design life of the structure.

C CHANNEL MIGRATION

The potential for channel migration conditions should be addressed in the *Bridge Hydraulic Report*. Refer to *BDM 2.14.7.3* for requirements and guidelines regarding bridge channel migration evaluations. Scour is not a force effect per se, but the change in conditions of the substructure may significantly alter the consequences of force effects acting on structures.

Consider foundation conditions with and without scour due to channel migration, and with aggradation for Extreme Event I.

Consider the potential for scour to undermine the channel slope, resulting in a slope failure, when assessing the potential scour depth. Include the potential of the scour-induced slope failure to cause lateral loading on abutment foundations as well as the nearby intermediate pier foundations.

Under channel migration conditions, neglect all foundation support (vertical and lateral) provided by the embankment material beneath the abutment down to the scour elevation associated with both the Design Flood (base flood) and Check Flood events (excluding local pier scour). Design the foundation to support the bridge dead loads (DW and DC only) under both design conditions as described in the *AASHTO LRFD Bridge Design Specifications*. When the waterway is subject to channel migration, evaluate the structure for stability assuming scour associated with both the Design Flood (base flood) and Check Flood events (excluding local pier scour) has occurred throughout the "waterway".

For single span structures, investigate the bridge for the case of backfill being washed out behind one abutment. For this case, use active soil pressures with no live load surcharge on the abutment.

Scour can be a detrimental effect or beneficial effect on seismic performance.

Two key concepts should be considered during this evaluation, channel migration and stream stability. See *HEC-20* for definitions of each. The channel migration definition should include vertical and horizontal changes over time. For the purposes of this chapter sticking to Channel Migration as a whole should be sufficient in addressing the concerns of washout. Alternatively there is stream stability which captures a stable channel vs an unstable channel (once that has the potential for vertical and/or horizontal migration) More information available in *HEC-20*.

Coordinate with the Hydraulics designer to determine whether any scour countermeasures or increase in structure size are recommended.

For sites with potential channel migration, investigate the condition that would produce the worst case unbalanced loading in the bridge, provided that case is feasible. (e.g. an abutment located outside of the main channel could end up within the channel, the abutment should be designed for scour as if in the main channel). This is often the case for strutted abutments where the passive resistance of the abutment backfill material is crucial to the stability of the bridge. Backfill material behind only one abutment could lead to unbalanced loads and failure of the bridge.

For channel migration at abutments supported on deep foundations, do not include debris loads on the end bent piles or shafts in this analysis.

1.11 Substructures

1.11.1 Retaining Structures, General

1.11.2 End Bents

1.11.3 Interior Bents

1.11.1 Retaining Structures, General

See *BDM 1.12.5* for Retaining Structures.

1.11.2 End Bents

GENERAL

Where end bents or retaining walls are located adjacent to roadway construction, locate the top of footings at or below the elevation of the bottom of the roadway subgrade. Locate the top of the footing a minimum of 1 foot below the surface of the ground. The effect of items such as utilities (see *BDM 1.12.6*), ditches and future widening should also be considered.

TERMINOLOGY

In this section and elsewhere in the *BDM*, the terms **end bent** and **abutment** are used interchangeably. **Integral Abutment** is the industry standard term used to describe abutments that provide a continuous connection between the superstructure and the substructure. However, for consistency on ODOT bridge drawings, all bridge support locations are referred to as **bents**. Refer to the glossary in the *Appendix* for definitions of the terms **Abutment**, **Bent** and **Pier**. A possible exception could include the rehabilitation of an existing bridge, where the original plans called out **abutments** or **piers**, etc. and it would be less confusing to keep the same terminology as the existing plans.

DESIGN

Lateral earth pressures at the end bent must be well thought out by both the bridge engineer and the geotechnical engineer. To more consistently model the behavior of the bridge and to ensure the design loads are constructed a detail has been developed for use in the plan set. The bridge engineer is responsible for including the completed detail, and pay limit diagram. The load diagram and associated notes in the plan set

C DESIGN

Historically a one foot neat-line with drain material has been used as a detail. This detail allows for easy calculation of the excavation and drain material quantities. However, the detail does not provide limits for the backfill at the end bents and wingwalls and while the specifications require granular structure backfill there is not consistent direction for the extent of the backfill. Thus, there are no assurances that the designed lateral earth

will be provided by the geotechnical engineer.

Provide access for inspection of bearings, shear lugs and backwalls for semi-integral abutments and access inspection for backwalls of integral abutment per *BDM 1.15.5*.

pressures are achieved in construction which will also limit deformation.

When Earthquake Restraining Systems and Earthquake Resisting Elements are used for passive restraint the engineering soil parameters and construction become more critical for the operational performance of the bridge. Increases in height to mobilize the passive soil pressure for seismic resistance may be necessary to achieve sufficient resistance.

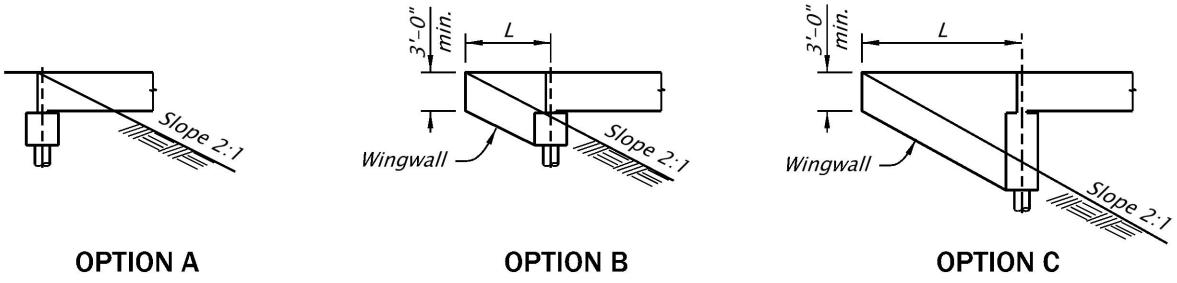
1.11.2.1 Determining Bridge Length

Options for the end bent in relation to the end fill intersection with the finish grade are shown in *Figure 1.11.2.1-1* and include:

- Option A, no wingwalls, but a longer structure than for options B and C.
- Option B, the structure length is shorter, but short wingwalls to retain the fill.

Option C, the structure length is shorter yet, but longer wingwalls and a taller abutment wall to retain the fill.

Figure 1.11.2.1-1 End Bent Location Options



OPTION A OPTION B OPTION C

C1.11.2.1

Generally, option B will provide the least cost, especially for prestressed slab spans. For option C, larger longitudinal forces from lateral soil loads must be resisted by the superstructure and substructure.

1.11.2.1.1 Slope Paving and Railroad Slope Protection

Where required, provide slope paving on the bridge end fill according to Bridge Standard Drawing *BR115*.

For a highway bridge crossing over a railroad, consult with the railroad to determine required slope protection.

C1.11.2.1.2

Slope paving is generally required where a roadway passes under a bridge. Also consider slope paving where a bridge crosses over a sidewalk or park.

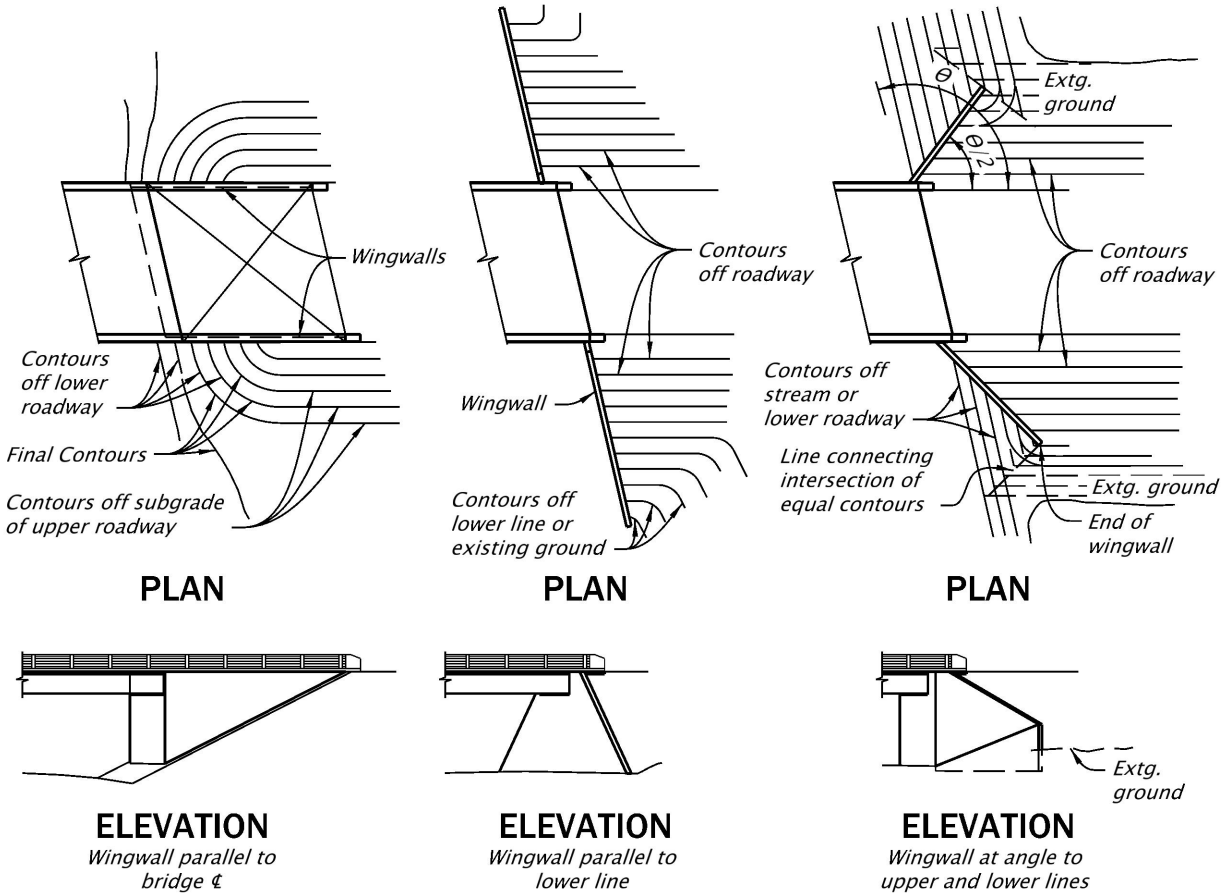
Rock slope protection may be required on the end fill slope of railroad undercrossings.

1.11.2.2 Wingwall Location

Locate wingwalls for end bents as follows:

- Walls parallel to the structure are used for filled or **false** (unfilled) bents. These are generally used for grade separation structures where the face of the bent is quite a distance back from the toe of the slope under the structure.
- Walls parallel to bridge bents are generally avoided due to safety or stream flow considerations.
- Walls at an angle to both structure and lower roadway or stream. The angle is generally half the angle between the structure and the lower roadway or stream center lines, as this usually leads to a minimum length wall. The end of the wall is determined by plotting final contours off the upper and lower profile lines. The point where the contours of equal elevation intersect determines the location of the end of the wall.

Figure 1.11.2.2-1 Wingwalls at End Bents



1.11.2.3 Wingwall Design and Construction

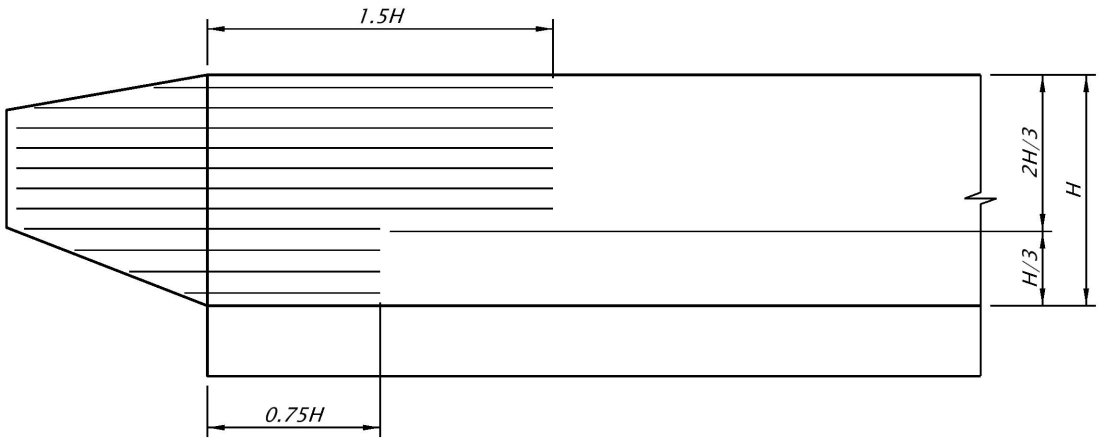
DESIGN

For cantilever wingwalls on abutments with relatively stiff footings (footing width is at least three times abutment wall thickness), the horizontal reinforcement in the fill face of the abutment wall resisting the moment caused by earth pressure on the wingwall need not extend farther from the wingwall-abutment juncture than the following:

- 1.5H for the top 2/3 of the abutment wall height.
- 0.75H for the bottom 1/3 of the abutment wall height.

Where abutment walls with wingwalls are designed with thickened tops for bearing seats or backwalls, those thickened portions should be designed to carry 1/2 to 2/3 of the bending moment in the upper half of the abutment wall. Reinforcing between the abutment wall and the wingwall should extend beyond the juncture enough to develop the strength of the bar reinforcement.

Figure 1.11.2.3-1 Wingwall Reinforcement



CONSTRUCTION

Consider all stages of construction when wingwalls are cantilevered from an abutment or pilecap. When the abutment or pilecap would be unstable or overstressed under the dead load of the wingwalls before the superstructure or backfill are placed, the **Bent Construction Sequence** on the plans should require that the concrete in the wingwalls not be placed until the superstructure or backfill are in place. Do not count on there being soil under the wingwall unless the wall has its own footing.

The height of the wingwall at the outer end of the wall should be a minimum of 3 feet. The slope of the bottom of the wall should be a maximum of 2:1.

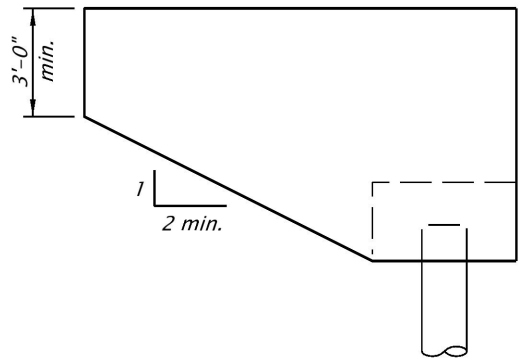
The *Special Provisions* and detail drawings should require that the embankment fill be placed to the elevation of the bottom of the wall before the wingwalls are constructed. In other words, bridge end bent wingwalls shall be cast against undisturbed material or well compacted backfill. The designer may want to use some discretion in this matter. A 24 foot wall would normally always need to be constructed on compacted fill, while a 6 foot wall could be constructed and backfilled at later time.

For walls shorter than about 8 feet, the bottom of the wall can be formed level, at the discretion of the designer or at the contractor's option. This adds some cost in materials, which may be offset by cost savings from easier construction. Potential benefits:

- Wingwalls are founded on level ground, no sloped or elevated bottom forms are required.
- Adds stability to abutment.
- Helps contain approach embankment at stream crossings when primary scour protection fails.

Due to concerns about stability and the potential for migrating of fresh concrete over the top of wingwall forms, the slope of the top of a wingwall should not exceed the maximum slope of the adjacent embankment nor 1.5:1 without a special stability investigation.

Figure 1.11.2.3-2 Wingwall Slopes



1.11.2.4 Bents on MSE Walls

Refer to the *ODOT Geotechnical Design Manual, Chapter 15* for the design of MSE Walls. Provide a concrete facing for all MSE abutment walls and wingwalls.

1.11.2.5 Integral Abutments

Use integral abutments wherever site conditions and structure geometry are suitable for such structures and the conditions and criteria described in this section are met. In integral abutments, expansion joints and bearings are eliminated and the superstructure is fully integral with the abutment. This results in numerous potential benefits including:

- Greater structure redundancy.
- Simplified construction.
- Reduced construction cost and time.
- Reduced maintenance cost.
- Stiffer longitudinal response at abutments.

For a continuous bridge with expansion end bent connections, the interior bents take all of the longitudinal and transverse force effects. By using integral abutments in place of the expansion end bent connections, some of the longitudinal and transverse forces are distributed into the integral abutment (piles and backfill soils), thereby reducing the net forces on the interior bents. Integral abutments can reduce the longitudinal and transverse force effect considerably in a continuous bridge as compared to a bridge with expansion joints at the abutments.

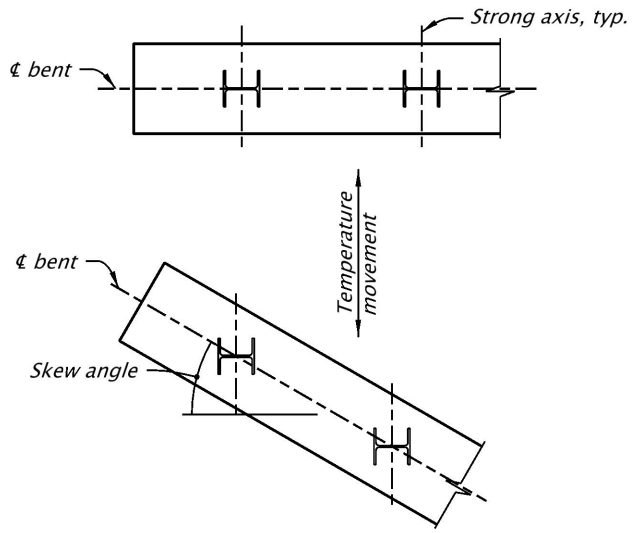
Use integral abutments under the following conditions:

- When the end bent is founded on steel pipe piles or H-piles. Do not place integral abutment foundations on top of, or through, MSE retaining wall reinforced backfill. For all other foundation types, see guidelines for semi-integral abutments.
- When bed rock is a minimum of 12 feet from the bottom of the pile cap. Avoid using pre bored piles when bed rock is close to the surface, since this type of construction has been uneconomical.
- When there is negligible potential of abutment settlement which does not affect the serviceability of the bridge.
- When the radius of horizontal curvature is greater than 1200 feet.
- When the skew angle is less than 30 degrees.
- When, for all service limit states, movement at the top of integral abutment piles does not exceed ±1.5 inches from the undeflected position. The corresponding range of pile movement is therefore 3 inches when the superstructure is made integral with the piles at the mean annual temperature.

Design Guidelines for Integral Abutments:

- Use a U-shaped abutment (wingwalls parallel to roadway alignment) when possible.
- Use H-pile with strong axis in the direction of temperature movement. See *Figure 1.11.2.5-1*.

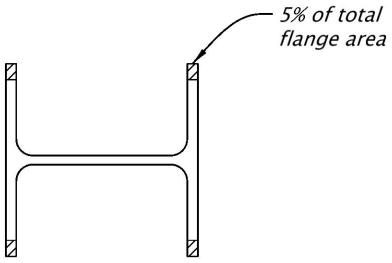
Figure 1.11.2.5-1 H-Pile Orientation



- Embed piles into the pile cap to develop moment fixity. See *BDM 1.10.5.4(1)* Piling Details.
- Preboring may be necessary in some cases where pile design stresses are excessive due to thermal movements and cannot be accommodated without special foundation design and construction. The cost of preboring for the piles should be compared to the benefits gained by using an integral abutment design. Increasing the number of piles or the use of larger piles in the abutment may decrease individual pile stresses to acceptable limits. When preboring is required, and cost effective, then consider preboring an oversized hole. The prebore dimensions should be at least 4 inches or more in diameter larger than the diagonal dimension of the pile and large enough to accommodate the estimated pile deflection. Backfill the area around the pile with loose sand conforming to the current *SP 00360.10* or as recommended by the geotechnical designer. Do not compact the sand backfill material. Bentonite or pea gravel backfill are not recommended since they may not provide for the long term flexibility required of the pile and soil system. The depth of prebore should be 10 feet or more or as required to provide the required pile flexibility to decrease pile stresses to an acceptable limit.
- Detail piles of integral abutments to resist uplift force from temperature differential between top and bottom of the pilecap. Refer to *Figure 1.10.5.4-6* for pile anchorage details.
- The design of integral abutment bridges with a grade change between abutments should consider both vertical and horizontal components of bridge longitudinal loads such as uniform temperature changes, creep, shrinkage, braking, seismic, and lateral earth pressure, on the resulting axial and flexural stresses in the piles.
- Develop a LPILE model using the full pile for soil and pile interaction. Evaluate pile deflections, bending moments and stresses using LPile computer program analysis.

- At the service limit state, H-pile flange yielding at each flange tip should not exceed five percent of the total flange area. See *Figure 1.11.2.5-2*. For steel pipe piles no yielding of section is permitted.

Figure 1.11.2.5-2 H-Pile Flange Yielding



- Consider the relative stiffness of the superstructure, substructure and any asymmetric span lengths in calculating end bent movement. Consider the potential for unequal thermal movements at end bents (integral abutments) due to asymmetric span lengths or changes in substructure stiffness.
- Consider torsion in components connected to integral abutments.
- Consider the combination of worst case events (except seismic) with temperature rise and fall.
- Specify placement and compaction requirements and an increased frequency of field density test requirements of the backfill material (minimum of two tests per stage of construction at each end bent) to achieve consistent soil stiffness behind both end bents.
- Consider the friction force between the bottom of the approach slab and structure back fill (expansion and contraction) in the superstructure design at the service limit state. Assume a friction coefficient of 0.54 unless specific measures are taken to reduce friction.
- Connect superstructure and end bents with a closure pour. Require a minimum of three days wait period between concrete deck placement and closure pour to release shrinkage stress in bridges with steel superstructures and include long term creep in your design for concrete superstructures. Include a note which requires backfill behind the abutment after closure pour.
- Where the range of abutment movement is less than ½ inch, the approach slab may be fixed to the superstructure and thermal movements accounted for by providing a saw cut in the approach pavement at the end of the approach slab. Where the range of abutment movement is ½ inch or more, provide an expansion joint between the approach slab and the deck so the approach slab is not dragged back and forth with thermal expansion and contraction. See *BDM 1.9.2.4.2* for additional information.
- In integral abutment bridge staged construction, a continuous abutment is capable of transferring traffic live load vibrations in one stage to the girders and the deck that are under construction in another stage. These vibrations can damage fresh concrete in the

deck. To minimize these vibrations, provide an expansion joint or closure segment in the integral abutment located between the stages of construction. See *BDM 1.9.1.3* for more discussion on vibrations.

- Specify deck casting sequences and deck closure pours at integral abutment connections and specify the range of temperature when the contractor may place the concrete on the plans and in the special provisions. Keep the range of temperature in the closure pour to not adversely affect the pile stress during temperature fall or rise.
- See design example in the following publication of the American Iron and Steel Institute *HIGHWAY STRUCTURES DESIGN HANDBOOK, Vol. II Chapter 5, Integral Abutments For Steel Bridges*, prepared for the National Steel Bridge Alliance by Tennessee DOT.

1.11.2.6 Semi-Integral Abutments

Recommendations for integral abutments also apply to semi-integral abutments, except as noted in this subsection.

Consider the use of semi-integral abutments, rather than integral abutments, on foundations that are stiff in the longitudinal direction, such as spread footings, drilled shafts, and concrete piles. These foundations do not provide the required flexibility in the longitudinal direction required for integral abutments. Also consider semi-integral abutments, rather than integral abutments, when the abutment is founded on top of or passes through MSE retaining wall reinforced backfill.

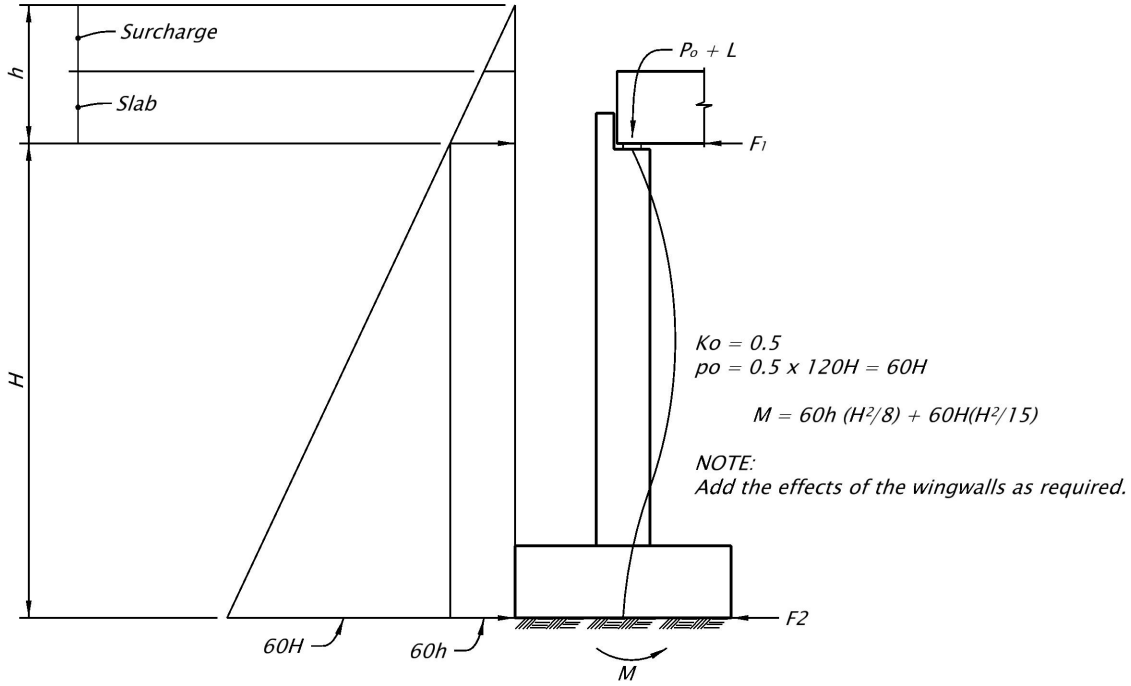
Two points that need to be evaluated on semi-integral abutments (especially on skewed bridges) are torsional forces affecting the bearings, and the effectiveness of shear keys used. When geometry requires a stiff footing, this type of construction is recommended.

For skewed bridges, consider the load path from thermal forces to the substructure. Skewed semi-integral abutments may rotate (finish condition).

1.11.2.7 Strutted Abutments

Abutments of single span bridges with the superstructure in place before backfilling may be designed using the strutting action to resist earth pressure overturning. For such abutments, apply soil pressure based on an at-rest or neutral condition of the soil. Footings for these abutments are not required to satisfy the **uniform bearing** under the dead load requirement. Investigate the bridge for the case of backfill being washed out behind one abutment, see *BDM 1.10.5.8* and *2.14.7.3.2*. For this case, use active soil pressures with no live load surcharge on the opposite abutment. A factor of safety against overturning of the whole structure of 1.25 will be considered adequate, and 125 percent of the allowable bearing pressure will be acceptable.

Figure 1.11.2.7-1 Strutted Abutment Loading



1.11.2.8 Pile Cap Abutment Details

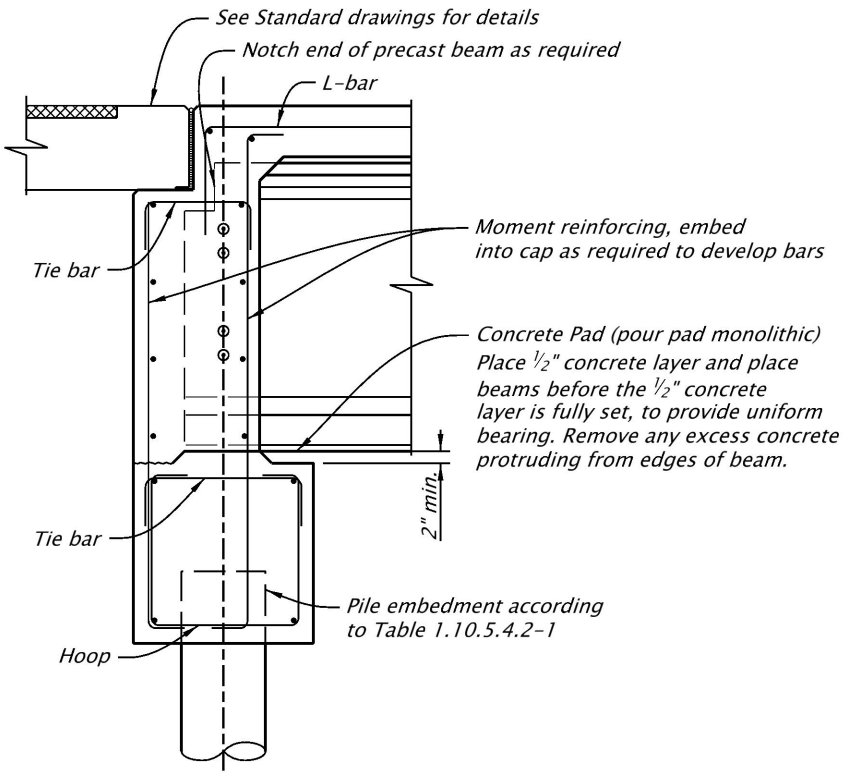
PILE CAP ELEVATIONS

Show the bottom of the pile cap elevations on the pile cap **Elevation** view. When the pile cap is sloped, show the elevation at each end.

FIXED (INTEGRAL) ACTION

Double row of reinforcing bars provides the connection between superstructure and substructure. Shear and moment are transferred. Pile embedment to develop fixity is required, when the number and size of piles are selected to resist a specified load.

Figure 1.11.2.8-1 Integral Abutment Detail

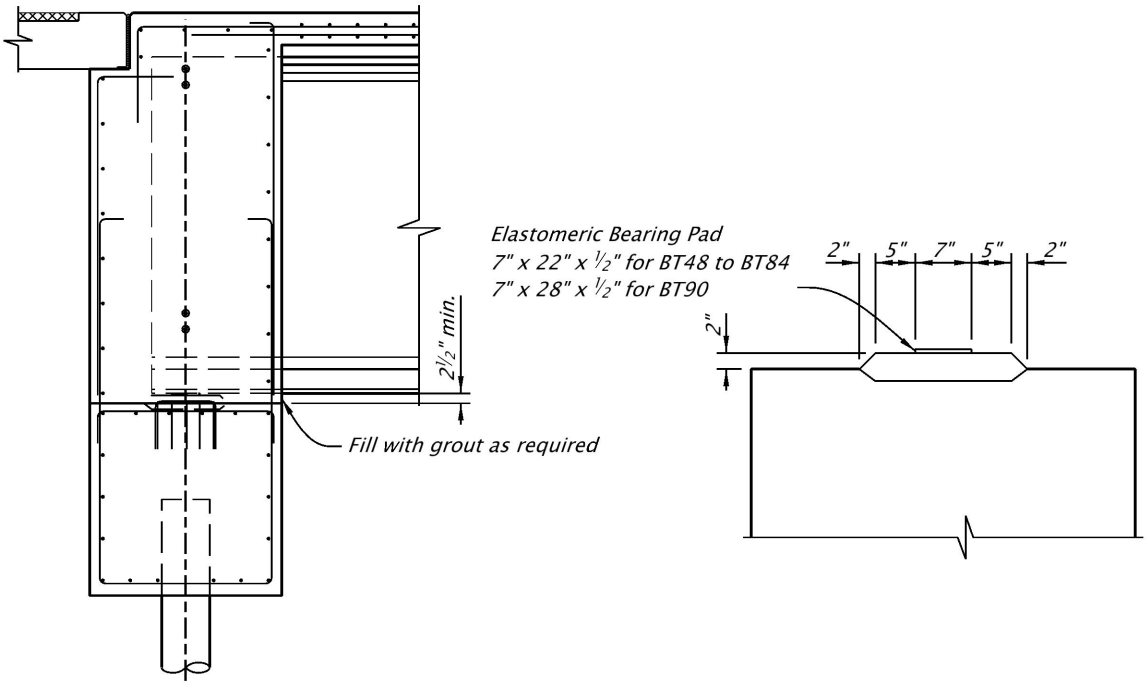


FIXED (INTEGRAL) ACTION WITH ELASTOMERIC BEARING PADS

This option allows the use of a ½ inch elastomeric bearing pad to be placed on top of the concrete grout pad. The precast beam can then be placed on top of the pad prior to the placement of the full width backwall. The beam should be placed just after a wet ½ inch grout layer has been placed under the bearing pad as specified in *BDM 1.14.1.6*. A double row of reinforcing bars provides the connection between superstructure and substructure. Shear and moment are transferred. Pile embedment to develop fixity is required, when the number and size of piles are selected to resist a specified load.

A reinforced concrete pad is required to resist temporary bearing loads. Hand placement of grout under the bottom flange of the beam may be required to fill the 2½ inch gap.

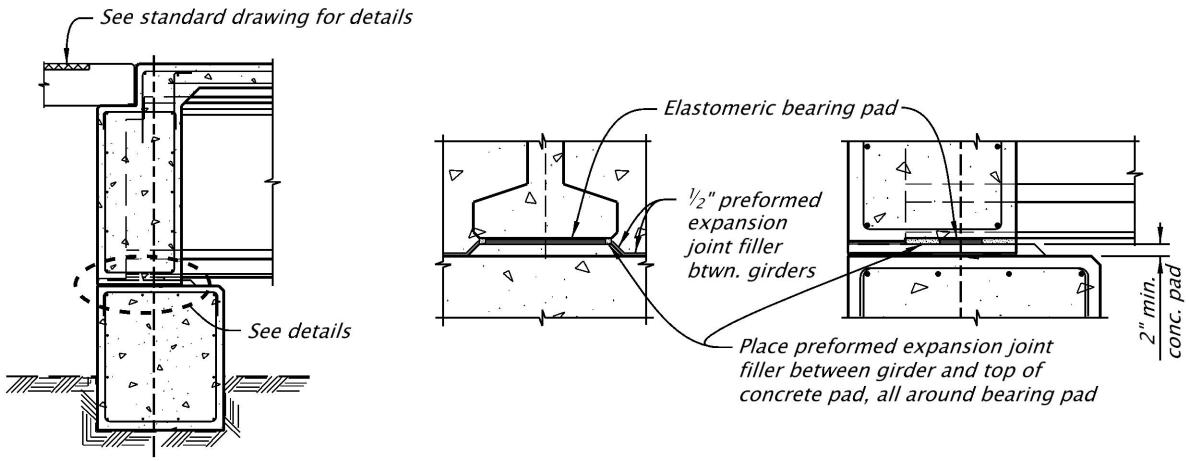
Figure 1.11.2.8-2 Grout Pad Detail



The performance of the ½ inch bearing pad under the vertical load and rotation resulting from deck load and diaphragm load was evaluated according to *LRFD 14.7.6.3.5b* for BT48 to BT90 girders. For BT48 to BT84 girders, a 7 by 22 inch pad is required. For BT90 girders, a 7 by 28 inch pad is required. Beam weight was not included in the end rotation calculations because the wet grout layer placed below the elastomeric pad at the time of beam placement eliminates any rotation of the pad due to beam end rotation from beam dead load.

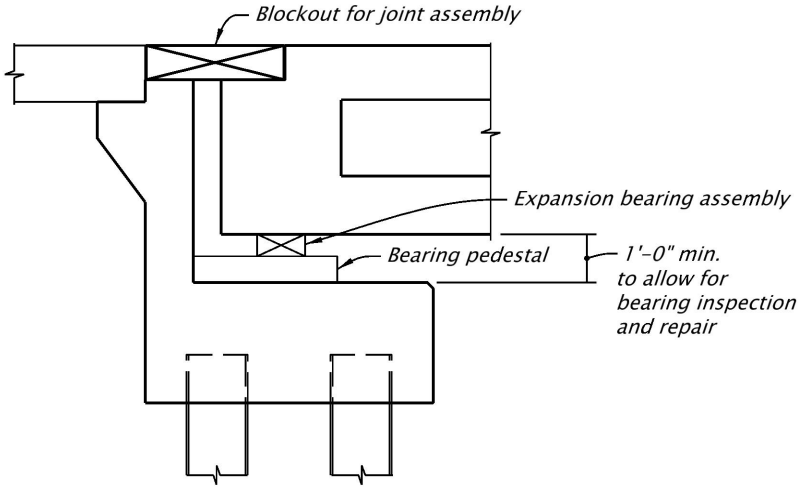
Expansion allowed (nominal amount of movement) – No reinforcement is provided between the superstructure and substructure. This type is appropriate when nominal movement is expected on a non-yielding type of foundation.

Figure 1.11.2.8-3 Bearing Detail



Expansion allowed (movement allowed as required) - No reinforcement is provided between the superstructure and substructure. This type is appropriate when movement needs to be accommodated in the design. Various types of bearings and joints can be used for the movement required.

Figure 1.11.2.8-4 Expansion Details



1.11.2.9 Abutment Details for Prestressed Slabs

See *BDM Appendix 1.11* for Prestressed Slab End Bent Design/Detail Sheets for more details.

SHALLOW ABUTMENTS (PILE CAP) – PRECAST SLAB OR BOX

Most common and most economical type of end bent. It requires the least amount of excavation and cast-in-place concrete.

Figure 1.11.2.9-1 Shallow Abutment - Precast Slab or Box detail

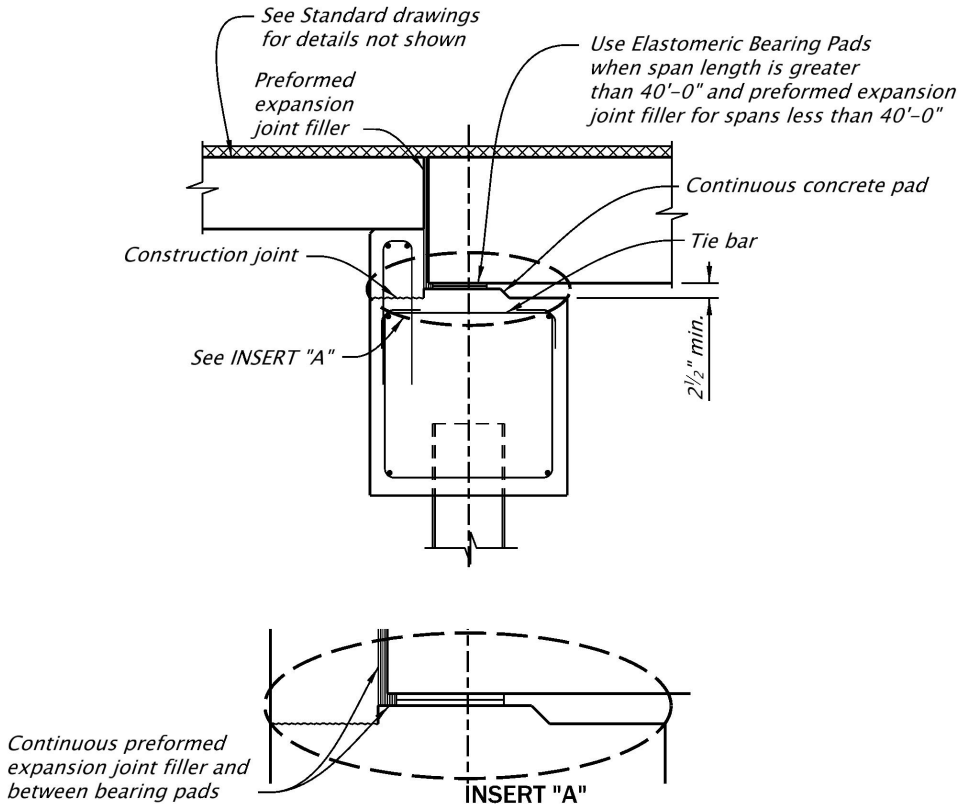
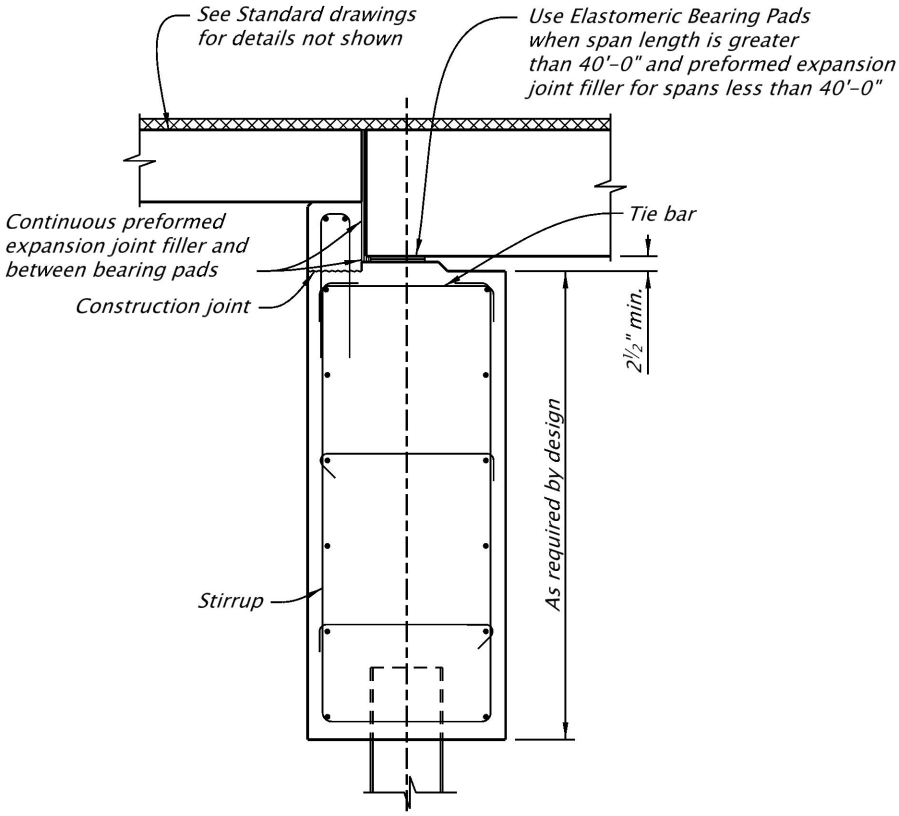


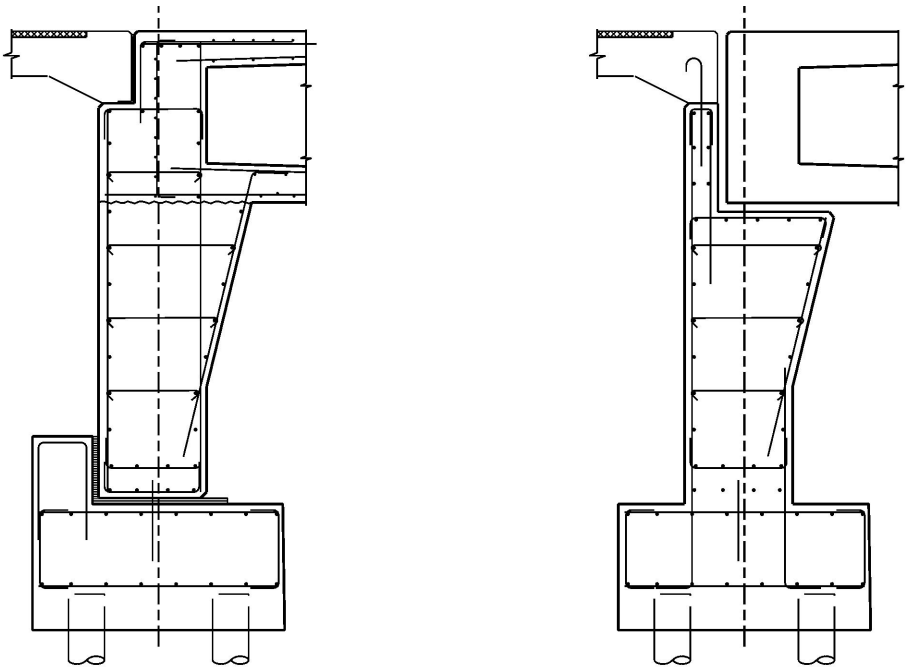
Figure 1.11.2.9-2 Partial Depth Abutment - Precast slab or box detail



1.11.2.10 Forming of Backwalls for End Beams

Details should be developed that will allow the removal of forming materials. Forming materials, including expanded polystyrene must be removed. Forming material is normally not yielding and can cause cracking as the structure expands and contracts.

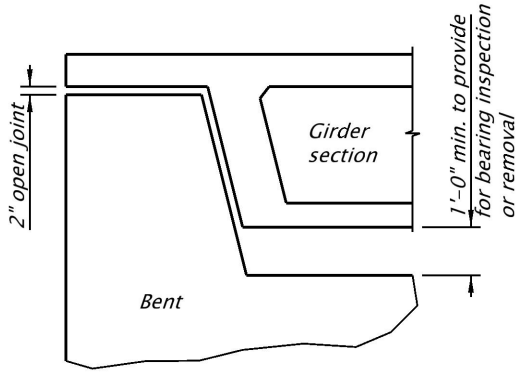
Figure 1.11.2.10-1 Backwalls for End Beam form detail



1.11.2.11 Bent Joint Details

Provide an open joint between the abutment and the deck-and-girder section, as shown below. Note on the plans of post-tensioned structures that when expanded polystyrene is used to form the joint, it must be removed before tensioning.

Figure 1.11.2.11-1 Bent Joint Detail



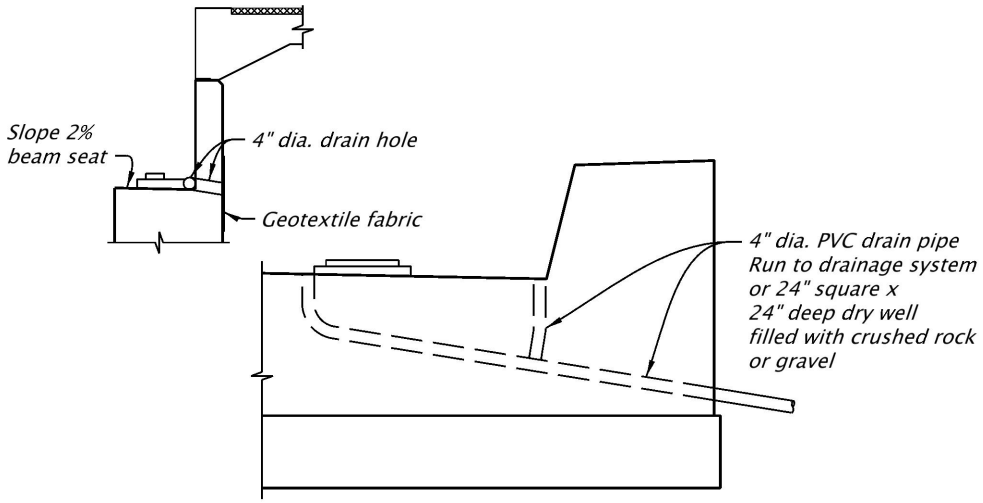
1.11.2.12 Backwall Reinforcement for Post-tensioned Structures

When detailing the vertical reinforcement for the backwalls of abutments for post-tensioned spans, the designer should take into account the location of the post-tensioning anchorages. Spacing of bars and splicing details should be such that the vertical bars do not have to be bent out of the way for the post-tensioning operation and bent back to their final positions.

1.11.2.13 Beam Seat Drainage

Slope the beam seats of abutments to drain away from the front face. Provide scuppers through the bearing pedestals and backwall or drainpipes at low points to pick up any water that might leak into this area.

Figure 1.11.2.13-1 Beam Seat Drainage Detail



1.11.2.14 Reinforced Concrete Approach Slabs

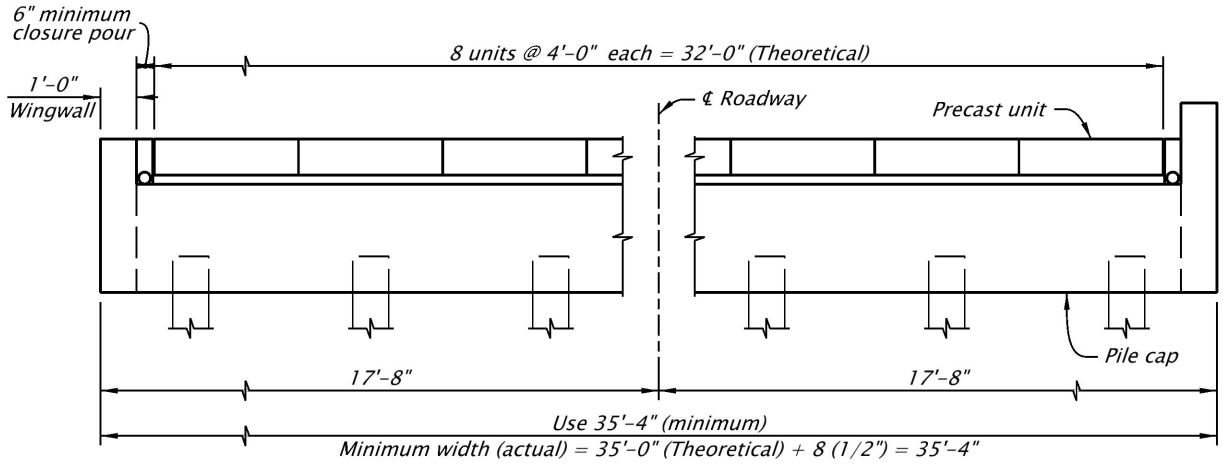
See *BDM 1.9.2.4.1* for approach slab requirement criteria.

1.11.2.15 Bent Width Provisions with Precast Units

All pile caps, crossbeams, abutments, etc. supporting adjacent precast units (such as slabs, boxes, integral bulb-T's, etc.) should be detailed for the total width of all units with an additional width of a minimum of ½ inch per precast unit. This is required because unit fit-up is not exactly true and **growth** in width occurs. The 6 inch minimum closure pour on each side of the exterior units at abutments as shown on End Bent Detail drawing may be used for

adjustment due to these misfits. The 6 inch dimension may be increased where necessary for wider roadways.

Figure 1.11.2.15-1 Bent Width Dimensions



1.11.3 Interior Bents

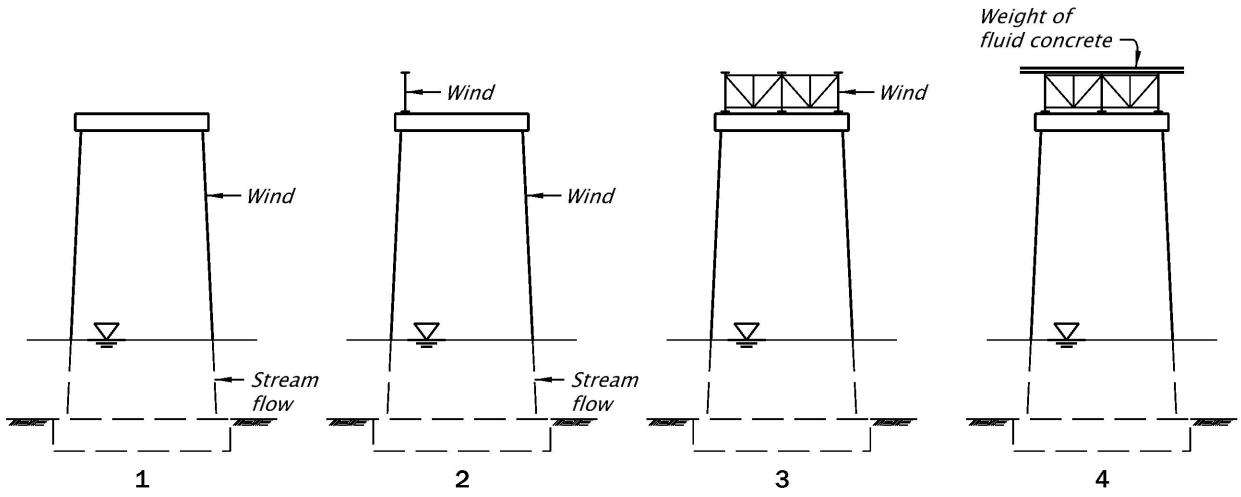
1.11.3.1 Interior Bents, Design and Detailing

DESIGN

Design structure for stability under all stages of construction. The following conditions, in particular, should be checked:

- Stream flow and wind load without superstructure.
- Dead load of one or more girders plus wind load and stream flow. Note: Contractor is responsible for stability of girder itself.
- Lateral system must be sufficient to insure stability of girders under wind load without deck.
- Top flanges must have sufficient support not to buckle under dead load of (fluid) concrete without the aid of deck forms.

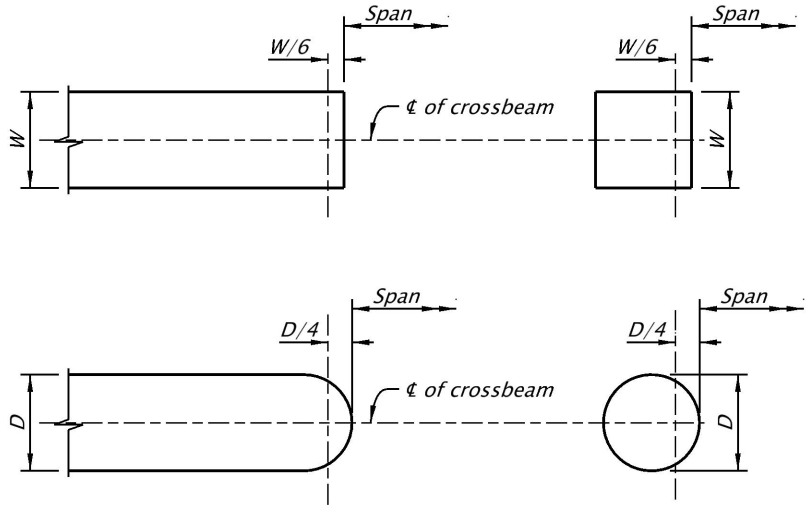
Figure 1.11.3.1-1 Construction Stability Conditions



EFFECTIVE SPAN LENGTH

When computing the maximum negative moment for a crossbeam on a column or pier, the crossbeam may be considered to be supported by a concentrated reaction, the distance show in *Figure 1.11.3.1-2* inside the face of the column or pier.

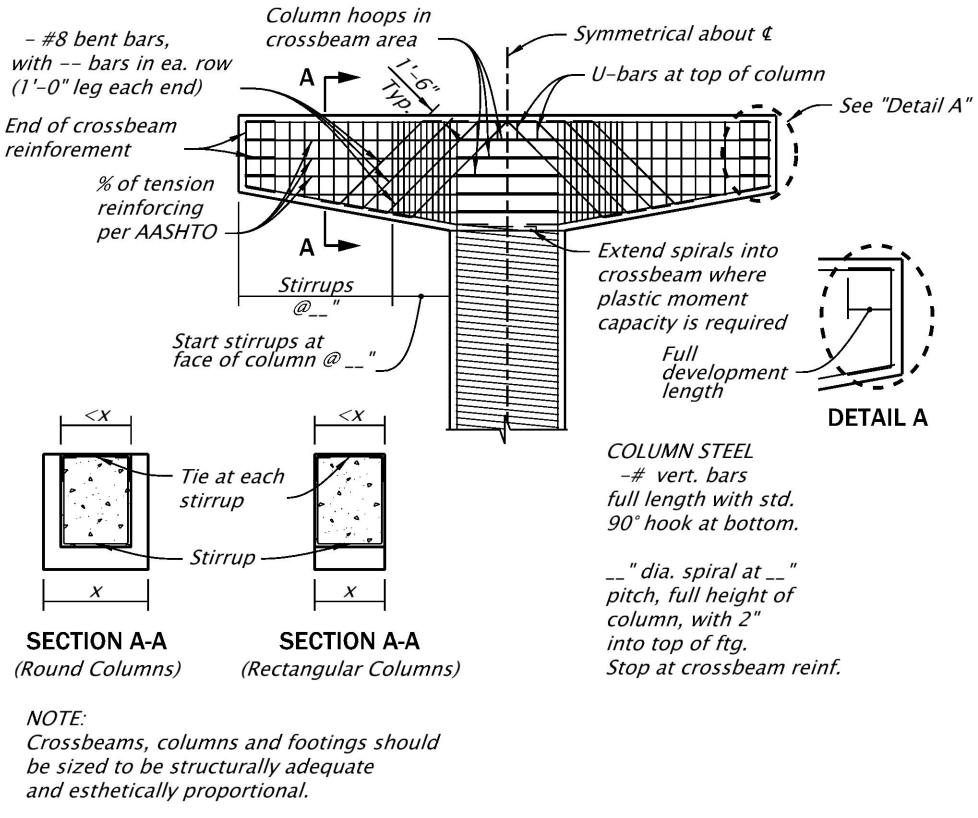
Figure 1.11.3.1-2 Effective Span Length



DETAILING

Provide all dimensions and details necessary for the reinforcing steel fabricator and contractor to construct it.

Figure 1.11.3.1-3 Structure Detailing Requirements



See *BDM 1.11.3.5* and *BDM 1.11.3.6* for details of column reinforcing.

1.11.3.2 Interior Bent Details for Prestressed Slabs

See *BDM Appendix 1.11* for Prestressed Slab Interior Bent Design/Detail Sheets.

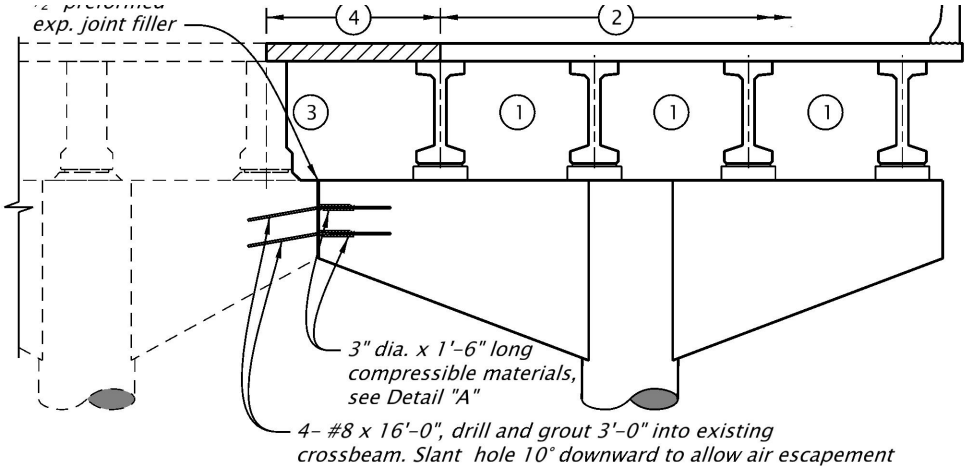
1.11.3.3 Structure Widening, Interior Bents

Generally, connections between structure bents should be detailed to tie the structures together, but prevent dead load and concrete shrinkage loads from being transferred to existing bents.

Example details are shown in *Figures 1.11.3.3-1* and *1.11.3.3-2*.

The method below allows the new crossbeam to deflect during the construction loadings with minor load transfer to the existing crossbeam.

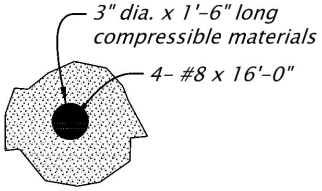
Figure 1.11.3.3-1 Structure Widening Detailing Example



ALTERNATE CROSS BEAM CONNECTION

POUR SCHEDULE
(INCLUDING CLOSURE POUR)

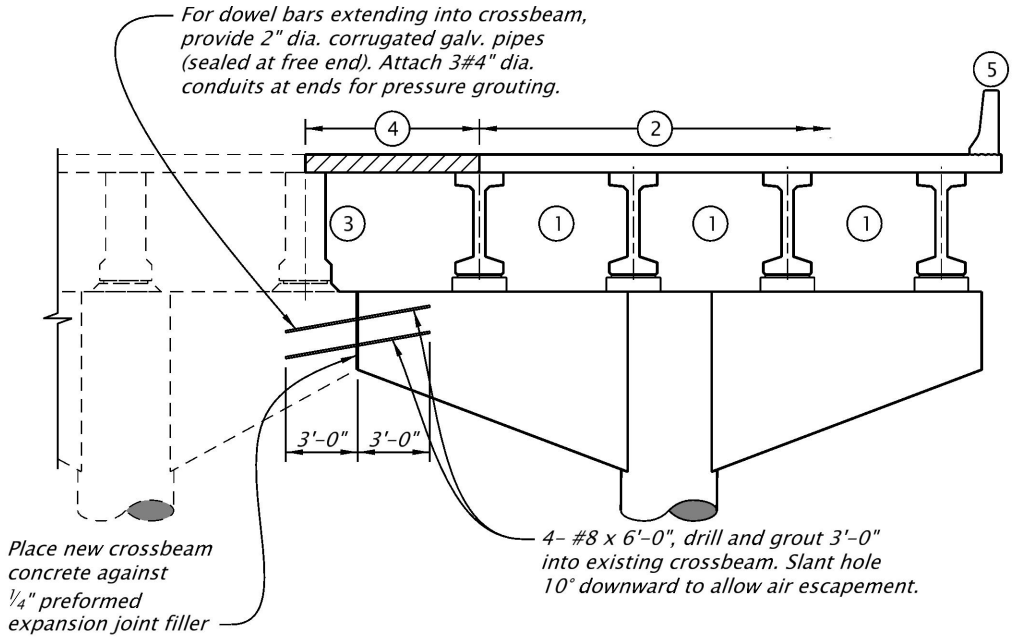
1. *Make pour in end beams and diaphragms.*
2. *Make pour in deck slab. Delay pour (2) a min. of 3 days after pour (1). A transverse deck construction joint may be made at any diaphragm beam. Delay pouring adjacent deck sections a minimum of 36 hours.*
3. *Make pour in end beams and diaphragm of closure pour section.*
4. *Make pour in deck slab of closure pour. Delay a minimum of 3 days after pour (3).*
5. *Make pour in bridge rail.*



DETAIL "A"

The method in *Figure 1.11.3.3-2* allows the widening construction to be completed before the connecting bars are grouted and able to transfer loading from the new crossbeam to the existing crossbeam.

Figure 1.11.3.3-2 Structure Widening Detailing Example



CROSS BEAM CONNECTION AND CLOSURE POUR DETAIL

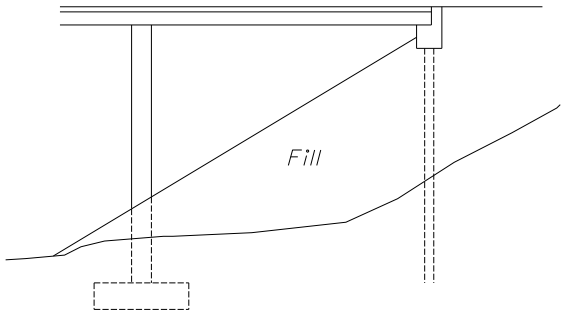
POUR SCHEDULE (INCLUDING CLOSURE POUR)

1. *Make pour in end beams and diaphragms.*
2. *Make pour in deck slab. Delay pour (2) a min. of 3 days after pour (1). A transverse deck construction joint may be made at any diaphragm beam. Delay pouring adjacent deck sections a minimum of 36 hours.*
3. *Make pour in end beams and diaphragm of closure pour section.*
4. *Make pour in deck slab of closure pour. Delay a minimum of 3 days after pour (3).*
5. *Pressure grout dowels in crossbeam.*
6. *Make pour in bridge rail.*

1.11.3.4 Columns in Slopes

Special attention should be given to situations where new fill could exert lateral pressure against bents other than the end bents. Such situations may require special construction sequence notes and special footing design including battered piling.

Figure 1.11.3.4-1 Additional Column Loading Situations



1.11.3.5 Column Design, General

See *BDM Appendix 1.2* for column loading criteria for vehicular impact, depending on type and location of barrier used (ODOT Instructions for *LRFD 3.6.5*).

For column designs controlled by seismic loading, provide shear and confinement reinforcement detailing according to *2nd edition of AASHTO Guide Specifications for LRFD Seismic Bridge Design, Section 8.6*.

For both tied and spiral columns, ensure adequate space for man access for tying and inspection.

Multiple interlocking spirals are the preferred choice for non-circular columns. Use 0.75 spiral diameters as the maximum center-to-center spacing of spirals. In this way, the smaller column dimension will dictate the larger column dimension. Closer center-to-center spacing of spirals is possible but would reduce the access space for tying and inspection. At least four vertical bars must be placed within the spiral overlap area. A photo log from FHWA is available showing how multiple spirals have been constructed.

Corners will normally be filleted or rounded. Using rectangular corners would normally require nominal corner vertical bars with ties developed within the core area. Such ties would interfere with bar tying and inspection. Therefore, design corners to be considered **expendable** in an earthquake, by detailing the rebar so that it is not developed within the core.

Bundled bars should only be oriented tangentially (both bars touching the spiral). Multiple concentric rings of bars are not a constructible option with multiple interlocking spirals, but may be used in detailing of circular columns.

Apply *LRFD equations (5.7.4.6-1, 5.10.11.4.1d-1, 5.10.11.4.1d-2 and 5.10.11.4.1d-3)* using volumes for a single spiral, using a theoretical minimum cover column with 2 inches of cover to determine gross area in these equations. The maximum spiral yield strength to be used in determining spiral spacing is 60 ksi. The heavier spiral confinement requirements for plastic hinge areas do not apply to tops of columns that are pinned.

Where columns are supported by drilled shafts, use a non-contact splice as shown in *Figures 1.10.5.5-1* or *1.10.5.5-2*. Ensure column diameter is less than shaft diameter according to *BDM 1.10.5.5(1)*. Provide confinement reinforcement meeting the requirements in *LRFD 5.10.11.4.1d* for column segments extending into drilled shaft as shown in *Figures 1.10.5.5-2* and *1.10.5.5-2*.

Specify *ASTM A706* reinforcement for vertical column bars when columns are supported on drilled shafts or when plastic hinging is anticipated in either the top or bottom of the column.

Specify ¾ inch maximum aggregate size in footings, columns and crossbeams. To maintain the shape of the spirals, use a maximum vertical bar spacing of 8 inches.

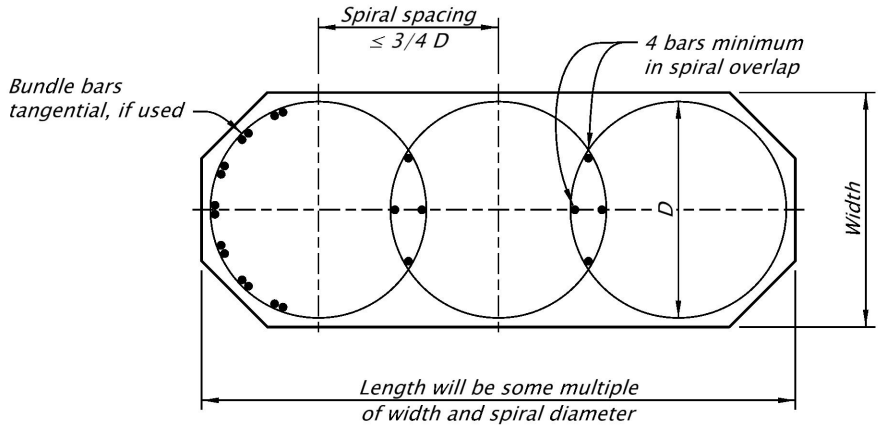
Containing an 8 inch diameter drainpipe within the column and taking it out between spread bars at the bottom is not an option since confinement requirements would be violated.

1.11.3.6 Spiral Reinforcing

Use spiral reinforcing for all columns. For column designs not controlled by seismic loading, extend spirals from a minimum 2 inches below the top of the footing to the bottom of the steel in the crossbeam or longitudinal beam.

Where plastic moment capacity is required between column-to-crossbeam connections, extend the spirals into the crossbeam to the top crossbeam steel.

Figure 1.11.3.6-1 Spiral Reinforcing

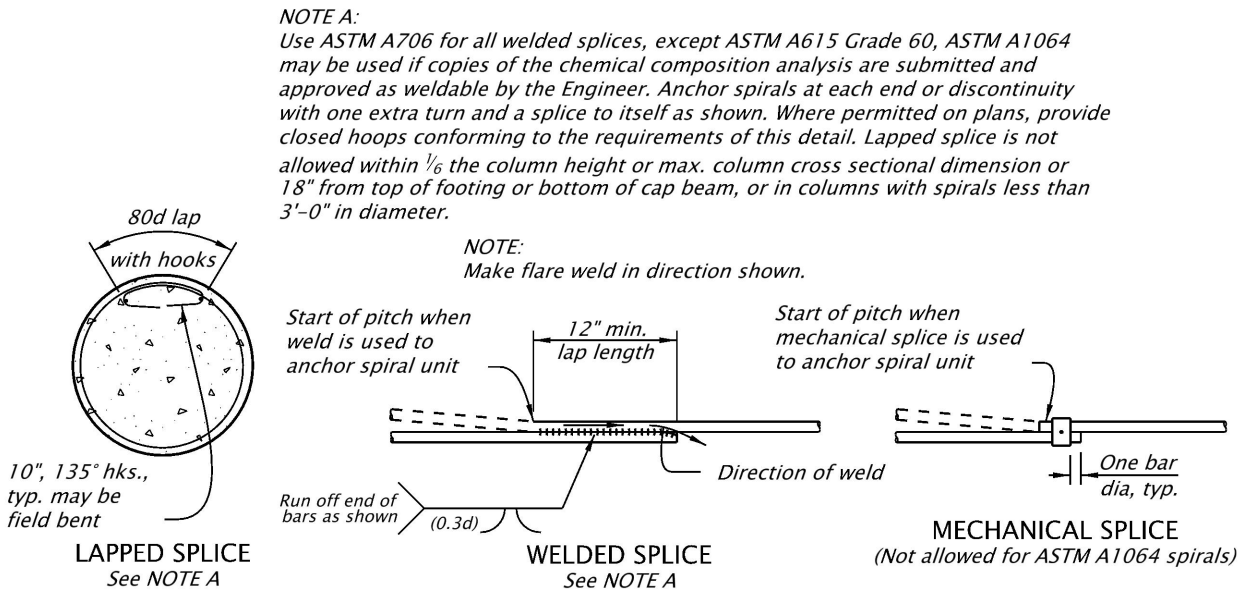


The following notes apply to the specification above and are for designer information only:

- Deformed bars (ASTM A615 grade 60 or ASTM A706) can be specified in sizes from #3 through #6.
- A706 is formulated to be weldable so submission of chemical analysis is unnecessary. It is also preferred because it is the most ductile.
- A1064 plain steel wire cannot be mechanically spliced because it lacks deformations. It is available only in sizes 5/8 inch diameter or less.
- ASTM A1064 and A615 grade 60 bars are available in coils. Average A1064 bar coils have a weight of approximately 1500 pound, and A615 deformed bar coils have a weight of 3000 pound to 4500 pound, depending on the size of the bar.
- For ease of handling, spirals are generally fabricated without splicing in weights up to a maximum of 200 pound per piece for diameters 8 feet and under.
- Coated spiral bars are fabricated using ASTM A706 bars. Stock lengths are generally 40 feet to 60 feet. Bars are spliced using the weld lap splice method. Maximum shipping mass is 200 pound for ease of handling and protection of the coating.
- Approved mechanical fasteners may be used provided the full strength of the bar is developed.
- Use of lapped splices should be avoided because of the 80d lap requirement and because hooks into the core will inhibit access for tying and inspection. Use of lapped splices is not permitted for spirals less than 3 foot diameter. Although the lap splice detail is structurally acceptable, and permissible by the code, it causes construction challenges. While casting concrete, the tremie gets caught in the protruding 10 inch hooks, making accessibility to all areas and its withdrawal cumbersome.
- The plans should state the type of spiral reinforcement used in computing reinforcing quantities. Normally the designer should assume A706 with welded splices.

Standard spiral splice and termination details are shown in *Figure 1.11.3.6-2*.

Figure 1.11.3.6-2 Spiral Splice and Termination Detail

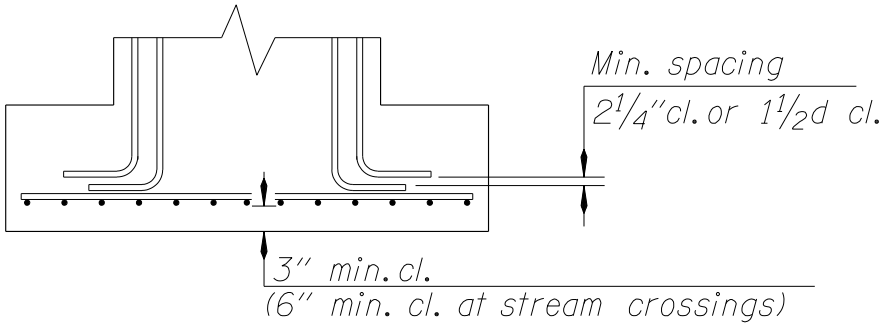


Weld reinforcing steel splices in accordance with ANSI/AWS D1.4 "Structural Welding Code Reinforcing Steel"

1.11.3.7 Column Steel Clearance in Footings

Column steel hooks are placed on top of the footing mat to avoid the need for threading footing steel through the column steel cage.

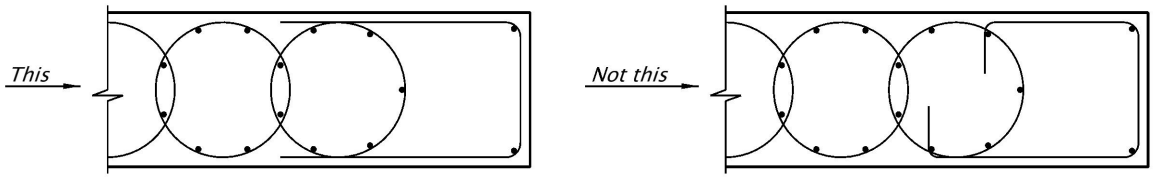
Figure 1.11.3.7-1 Column Steel Hooks Detailing



1.11.3.8 Column Hoops

Due to seismic requirements, use hoops and ties only to supplement spiral reinforcement for architecturally shaped columns to provide some confinement to concrete that is **expendable** in a major seismic event. Terminate these supplemental hoops and ties without the normal extension (hooks) into the interior mass of the column concrete. Because these architectural features are expendable and are not considered in the analysis and design we want to allow their failure. They should be detailed so they do not add undesired stiffness and strength.

Figure 1.11.3.8-1 Column Steel Hoops

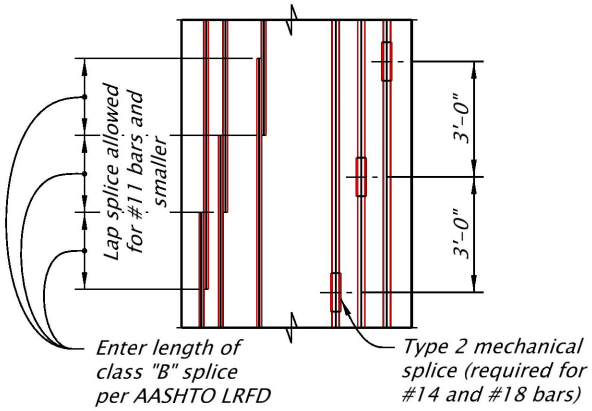


1.11.3.9 Vertical Bar Splices

Do not splice vertical column bars for columns less than 30 feet in length (no footing dowels). For longer columns, splices may be made as shown below in the middle half (preferably at mid-height) of the column (outside the plastic moment areas). Lap splice is allowed for #11 bars and smaller. For #14 and #18 bars Type 2 mechanical splice is required. Type 2 mechanical splice is required to develop at least 125 percent of the specified minimum yield strength and 100 percent of the specified tensile strength of the reinforcing bars.

The development requirements may require 180 degree hooks of the column verticals in the cap beam. Pay attention to how the column verticals, extended spirals, bottom cap beam bars, and post-tensioning ducts all fit together.

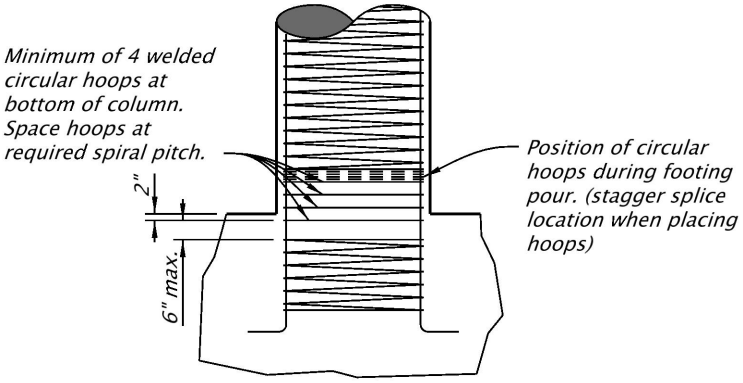
Figure 1.11.3.9-1 Vertical Bar Splices



1.11.3.10 Optional Hoop Detail at Bottom of Column

The detail below will facilitate more effective concrete placement in the core area of the footing. The 6 inch gap is used to facilitate placement of the top mat of reinforcement.

Figure 1.11.3.10-1 Optional Hoop Detail at Bottom of Column



1.11.3.11 Footing Reinforcing

Provide a mat of reinforcing steel (minimum of #5 bars at 12 inch centers each way) in the top of all footings. When calculated loads require larger amounts of reinforcement, the latter controls. Also provide U-bars at 12 inch centers around the periphery of the footing.

Extend spirals at least two inches into the footing. Place the footing top mat immediately below the spiral termination. Place additional spirals below the mat (use a 6 inch spiral gap) down to the vertical bar's point of tangency. Use the same spiral pitch at all locations.

See *Guide Spec. 6.4.7* for footing joint shear reinforcement for Seismic Design Category (SDC) C and D.

Figure 1.11.3.11-1 Example of Footing Reinforcing

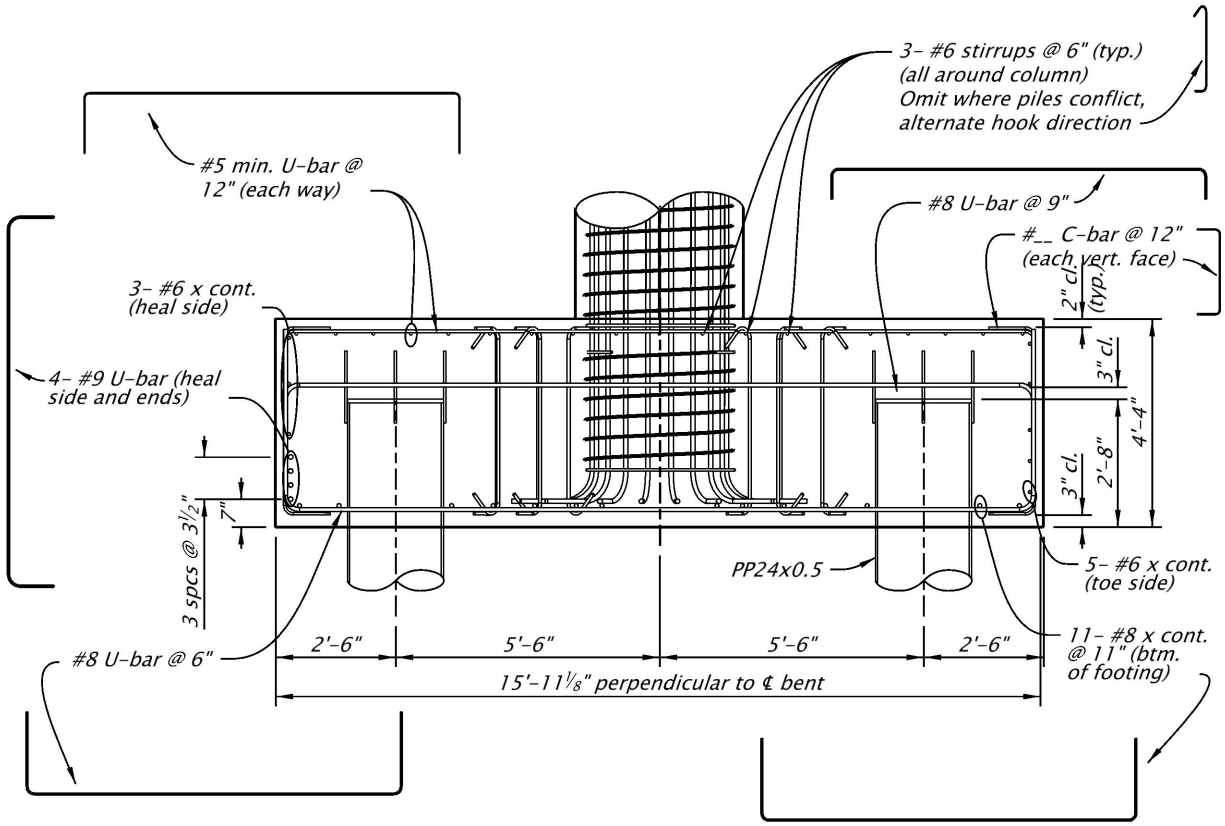
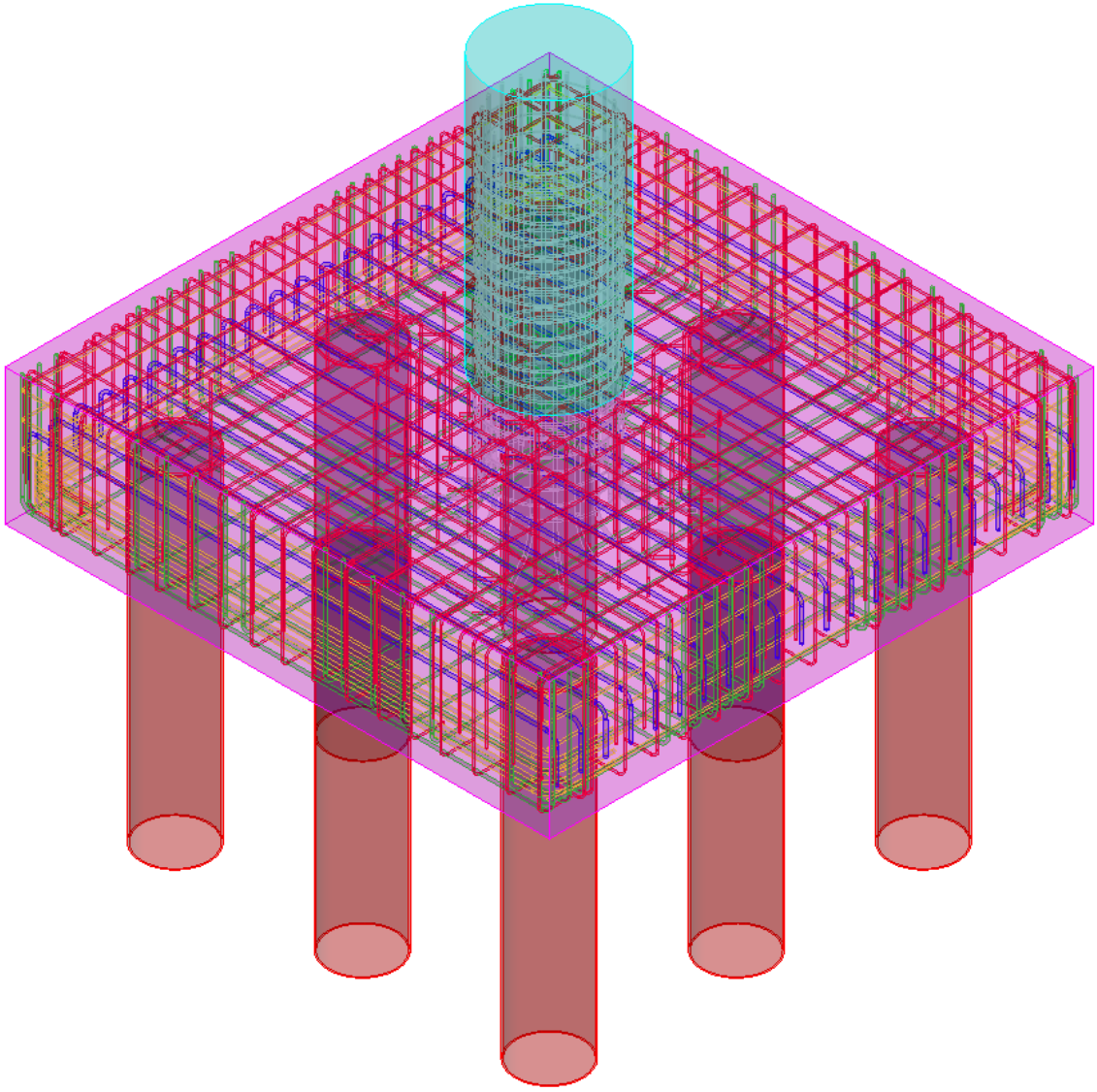


Figure 1.11.3.11-2 Example of Footing Reinforcing, Isometric View



1.11.3.12 Sloped Footings

General criteria for sloped footing tops are:

- The required footing thickness adjacent to the column should be at least 4 foot 6 inches. No minimum edge thickness is specified except as required for shear.
- The amount of concrete saved should be at least 10cy.
- The top may be sloped either two ways or four ways, but should not be steeper than 2:1.
- A horizontal area should be provided 6 inches to 12 inches wide outside the base of the column to facilitate forming the column.

1.12 Other Structures

1.12.1 Culvert Design

A single culvert span, or out-to-out sum of closely spaced culvert spans, of 20 feet or more is defined as a **bridge** and is included in the National Bridge Inventory (NBI). NBI culverts must be load rated per the *ODOT Load Rating Manual*.

Large culverts are defined as having a diameter or span 6 feet or greater.

Precast culverts are designed by the manufacturer according to *SP 00595*. Ensure the designs of large culverts comply with the following requirements:

- Precast elements consist of individual cells with continuous vertical joints, unless an engineered substructure is provided.
- For precast wingwalls, provide positive connections between wingwalls and reinforced concrete box culverts (RCBC) end sections with short closure pours or weldments.
- Provide reinforcement continuity between precast footings and concrete aprons.

Use the following criteria to determine the use of rail standards over culverts:

- Follow the guidance of *BDM 1.13.9* for guardrail layout in low fill.
- For culverts under 6 feet in span or diameter, use roadway standards.

- For culverts between 6 and 25 feet in span, use roadway standards unless the location is considered high risk.

C1.12.1

Refer to the *ODOT Highway Design Manual* and *Hydraulics Manual* for additional guidance. The Region Tech Centers will typically design or administer concrete culverts, metal pipe culverts and pipe arches.

Large culverts are processed like bridges. Request a structure number, drawing number(s), etc. for large culverts.

When selecting roadside safety hardware at culverts, the decision to use roadway or bridge standards is based on culvert length, fill depth, and risk.

Locations that require TL-5 are considered high risk.

- For culverts greater than 25 feet in span, use bridge standards unless fill depth is greater than 2 feet 8 inches.

ODOT standard guardrail requires a minimum embedment of 2 feet 8 inches. Lesser depths present an obstacle for continuous guardrail and is classified as low fill.

1.12.2 Tunnels (structural elements)

(Reserved for future use.)

1.12.3 Sound walls, General

Design sound walls according to the guidelines provided in *LRFD Section 15*, and *ODOT Geotechnical Design Manual, Section 16.6*.

Investigate the soil condition specific to the sound wall site and include in the Geotechnical Report.
The design and detailing requirements for sound walls on bridges and retaining walls adjacent to the traveling public are different than the ones away from the traveling public. The failure of these sound walls is a safety hazard for the traveling public.

C1.12.3

It is recommended that this be taken care of early in the project's development.

1.12.3.1 Sound walls mounted on Bridges

Do not locate sound walls on bridge structures where feasible alternative locations exist.

C1.12.3.1

Sound walls on bridge structures cause a disproportionate increase in bridge cost because of strengthening the deck overhang and exterior girder. These structures may cause increase in risk to traffic below during seismic events or in case of vehicular impact. In addition, sound walls on bridges interfere with normal maintenance inspection access and detract from the aesthetic quality of the structure.

Where feasible alternative locations do not exist and sound walls must be located on bridges, limit the total height, as measured from the top of bridge deck to the top of the sound wall, to 8 feet.

The 8 foot height limit is per the crash tested result and how the cargo truck interacts with the wall and barrier: NCHRP Report 350 Evaluation of the T501 Bridge Rail with Sound wall (tamu.edu). There is no crash tested result to support sound walls taller than 8 feet.

Only crash tested sound walls are allowed on bridges.

In lieu of crash-testing, design sound walls for vehicular collision forces according to *LRFD Section 15.8.4*.

On bridges where the sound wall does not meet crash test requirements of MASH Test Level 4, place sound walls a minimum of 4 feet beyond the gutter line of an ODOT approved standard bridge railing.

Sound walls may be combined with the traffic railing as long as the structural system meets the crash test requirements of MASH Test Level 4 criteria.

1.12.4 Traffic Structures

1.12.4.1 On-Bridge Sign & Illumination Mounts

1.12.4.1.1 Design Considerations

The following traffic structures may be located on bridges:

- Standard lighting poles.
- Camera poles.
- Structure mounted signs (signing for traffic passing under bridge).
- Miscellaneous small signs (signing for traffic on bridge).

In special cases where other (larger) traffic structures not listed above must be located on a bridge, connect them directly to a bent.

When a traffic structure is on a bridge, the bridge designer will be responsible for the connection between the traffic structure and

C1.12.4.1.1

Standard traffic lighting poles are the only traffic structures with standard bridge connection designs.

Placement of other traffic structures on bridges is discouraged due to increased costs, complexity and maintenance. The decision on whether the traffic structure is located on the bridge and the exact location of the traffic structure on the bridge will be made by the bridge designer in conjunction with the project team.

The *ODOT Traffic Structures Design Manual* has design loads and guidance for many common applications.

the bridge, including the anchor bolts, and will review or check the shop drawings associated with the bridge design responsibilities. The Traffic Structures Designer will have very limited involvement with the bridge structure.

Structure mounted signs should preferably not exceed 7 feet in height.

In urban areas the required sign legend may dictate a larger sign panel. The bridge designer should work with the project team to arrive at an acceptable solution, considering effects on aesthetics, sight distance, and related factors.

1.12.4.1.2 On-Bridge Sign Mounts

C1.12.4.1.2

Position all new side mounted signs on bridges such that the bottom of the sign or lighting bracket does not extend below the bottom of the bridges (bottom of girder flange). Maintain at least a nominal 2 inches between the bottom of the sign and the bottom of the bridge to account for

This is to ensure that the bottom of the sign is above the bottom of the bridge while accounting for construction tolerances and bracket arm sag.

Design bridge side mounted sign brackets to account for the weight of added lights, and for the wind effects on the lights to ensure bracket adequacy if lighting is attached in the future.

Do not place signs under bridge overhangs. Do not place signs directly under the drip-line of the structure. Provide a minimum 2 inches of clearance between the back side of the sign support and edge of the bridge.

1.12.4.1.3 On-Bridge Illumination Mounts

[Reserved for future use]

1.12.4.2 Truss and Monotube Cantilever Sign Bridges

Refer to *ODOT Traffic Standard Drawings* website *600 series drawings* for standard truss and monotube cantilever sign bridges. For new truss and monotube cantilever sign bridges design, see *ODOT Traffic Structures Design Manual* and *LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, 1st Edition* for further guidance.

See *ODOT Geotechnical Design Manual (GDM) Chapter 16* Foundation Design for Signs Signals, Luminaires, Sound Walls and Buildings for foundation design guidance for truss and monotube cantilever sign bridges.

C1.12.4.2

1.12.5 Retaining Structures

Retaining walls that support bridge bents will typically be designed by the bridge designer, and drawings will be included with the structure drawings.

For all other free-standing retaining walls, refer to the *ODOT Geotechnical Design Manual*.

Retaining wall project development activities require integrated and coordinated effort of bridge, geotechnical, geology, and roadway disciplines. Additional disciplines could be necessary dependent on the project. It is common for either the bridge engineer or geotechnical engineer to take the lead role in retaining wall design. Coordination in the design process includes:

- Determine if retaining wall is needed and wall location design - [Roadway, Retaining Wall designer].
- Subsurface investigation planning, execution, identifying ground water, soil and rock properties for design [Geotechnical, Geology].
- Identify structure types for consideration, loading conditions, perform analysis and design, estimate cost, and recommend structure type [Geotechnical, Bridge or both]
- Evaluate recommended structure type (technical and cost estimate check), evaluate constructability [Retaining wall designer, Retaining wall checker, other project reviewers].
- Project design changes that affect the retaining wall [Project team process coordination]. Coordinate with other design elements.

The Retaining Wall chapter of the *ODOT GHE CAD Manual* provides guidance for Contract Plan Sheet development for retaining walls:

- Proper plan sheet series by retaining wall category (within J series of a bridge, a separate J series for the retaining wall, or G series).
- Reference as needed to the *Bridge CAD Manual (BCM)*.
- Sources of information for title block, structure name, structure number, BDS drawing number, calculation book number, file V-number, when applicable.
- Plans content by retaining wall category and system type.
- Plans Checklist

1.12.6 Utilities

1.12.6.1 General

Permanently relocate utilities in advance of the project, when feasible.

When there is a utility relocation or modification on a project, utility design is to be provided by the utility owner. When utility work is included in an ODOT Contract, the utility owner is responsible for supplying plans, specifications, estimates, and calculations conforming to the documented requirements.

On new construction, the State provides the concrete inserts in the deck for hangers, holes through diaphragms, crossbeams and endwalls, and pipes under the approach slabs. When a utility owner requests additional conduits in a sidewalk or concrete rail, special attachment brackets, inspection walkways, etc., it is the expense of the utility owner.

The State and utility owners within project limits need an agreement before their utility work can be included in the project. The Utility & Railroad Coordinator in the Right of Way section coordinates such agreements.

Include provisions for two future 3 inch I.D. conduits on new NHS bridges for Agency Communication Infrastructure.

C1.12.6.1

When utilities are not relocated in advance of the project, it is preferred for the work to happen as part of the construction contract, installed by the contractor.

This avoids often difficult coordination between the Contractor and utility owner. This also avoids the need for the utility owner to conduct work after the contract is complete, which often time requires performing repairs of the constructed facility like patching newly placed roadways and bridge elements.

This is regarded as providing minimal accommodation which essentially has zero or negligible cost (**de minimis**, or below the threshold of actually costing the program) compared to not providing these items, and is acceptable per an opinion from the Oregon Department of Justice. All other costs for materials and labor related to the utility installation are the responsibility of the utility company.

The provisions include holes in transverse members and cast-in anchors.

For bridges inside incorporated city limits or in metropolitan planning areas, provide for future needs with two 12 inch diameter holes on each side of the bridge in addition to the specific utility requirements.

This applies to all bridges, including those carrying the freeway. While ODOT policy typically does not permit longitudinal installations of utilities within freeway right of way, there is an FHWA hardship exception process that could result in future installations.

1.12.6.2 Special Utility Considerations

C1.12.6.2

GENERAL REQUIREMENTS

Install wire line type crossings, such as fiber optic line, in conduit.

Provide transverse supports for utility lines as required.

Provide expansion fittings at each expansion joint or install on rollers as allowed by applicable safety codes.

ELECTRICAL LINES

Provide appropriate bonding jumpers across expansion fittings for electrical installations.

See *NEC 250.98*.

High voltage power distributions lines greater than 22,000 volts are not allowed.

In extraordinary circumstances where alternate crossings are not practical, high voltage lines can be considered. Additional cost to the utility will not be considered reason enough to place power lines on bridges.

Lines with 600 volts or less are allowed.

Line with voltage greater than 600 volts but less than 22,000 volts will be evaluated on a case-by-case basis.

Provide adequate shielding for electric power distribution lines.

This is to eliminate adverse effects of electromagnetic fields on radio signals, fuel injection systems, reinforcing and structural steel, and maintenance personnel.

Provide adequate circuit protection. Locate disconnects within 1000 feet of the utility's first anchorage to the bridge.

This is to reduce the risk of electric shock hazards and allow for disconnection of the line upon request from ODOT.

Conduit embedded in concrete is to be Sch. 80 PVC (preferred) or stainless steel.

Other embedded steel conduit is not allowed due to corrosion concerns. Where possible, PVC conduit is preferred to reduce the potential for damaging lines when pulling them through. See *NEC 334.15(B)*. Galvanized embedded conduit is not permitted due to dissimilar materials.

Hot-dip galvanize external steel conduit.

GAS LINES

The following language applies to gas lines or other lines carrying volatile materials.

Provide Sch. 40 steel pipe or approved equal. Install automatic shut-off valves at or near each end of the bridge.

Case lines full length of enclosed or box type bridges. Vent casings to the outside of the bridge at each end and at high points.

Protect lines that are accessible from ground from damage, both accidental and intentional.

Protection could include barrier and fencing with locked access.

Have gas line corrosion protection systems reviewed by the Bridge Section Preservation Unit. See *BDM 2.14.10.*

WATER LINES AND SEWER LINES

Case segments of lines whose failure could undermine the bridge footing(s) or be an environmental hazard.

Lines are to be hot-dip galvanized steel, ductile iron pipe, or approved equal.

HDPE pipe can be considered approved equal when the following conditions are met:

- When wrapped for UV protection, or when the pipe is encased (i.e., not exposed to UV or corrosive environment) and
- When the pipe is adequately supported.

Corrosion protection systems for utilities

may include cathodic protection, except when attaching to steel structures.

Provide transverse supports near each coupling for lines and line thrust blocks as required.

In enclosed or box type bridges, make provisions for a line failure. Provide drain holes or grating at low points in the cells. Provide low pressure sensing shut-off valves or fully encase the line to minimize leakage inside the bridge girders.

1.12.6.3 Utility Location on Bridges

C1.12.6.3

Locate the utility installation to avoid the following:

- Adverse effect on the appearance of the bridge.
- Restriction to inspectability and maintainability of the bridge.
- The risk of potential vehicle impacts when the bridge spans another roadway or railroad crossing.
- Modification of structural members.

On structures over freeways, locate the bottom of utility line a minimum of 18 feet above the roadway surface.

Do not hang from cross-frames, diaphragms, or main beams. Suspend utility lines from the deck.

Install the utility between girders or in the sidewalk or rail when applicable. Locate the utility as close as possible to the exterior of the bridge to allow access by snooper crane, when no other access is provided.

This may not be possible when staging of the bridge is not compatible. See *BDM 1.2.10* for safety and accessibility guidance.

Provide sufficient space around utilities for maintenance activities.

Maintenance activities include cleaning and repainting steel members.

Do not extend utilities and supports below the bottom of the superstructure, except when transitioning to a buried utility. Place transitions only at bents or abutments.

When the utility is placed on the outside of the rail or exterior girder on stream crossings, place it on the downstream side of the bridge.

This is to minimize the chance of damage from floating debris.

Do not hang utilities against the sides of decks that have no curb. When required to put utilities on the side, provide spacers from the deck so they do not trap debris.

Avoid exterior mounted utilities in snow and ice areas, as defined in *BDM 1.2.4.3*.

Do not place utilities below the outfall of bridge drainage.

Avoid attaching utilities to timber elements.

Many timber elements require replacement during the bridge's life.

Avoid going through end bents with a history of approach settlement.

This is common on shallow end bents without approach slabs. Excavation may increase settlement, settlement may cause the utility to shear, or the utility may get in the way of installing sheet pile or approach slabs in the future.

1.12.6.4 Design and Detailing Guidelines

C1.12.6.4

EARLY DESIGN TASKS

As an early design task, determine:

- Requirements for carrying existing and future utilities on bridges. See *BDM 1.12.6.1*.
- Requirements for accommodating utilities in the vicinity of other structures (i.e. box culverts, sound walls, retaining walls, or especially mechanically stabilized earth (MSE) walls).

During the site visit, identify any utilities present on, or in the vicinity of, the bridge.

Document utility decisions in the DCSA.

Coordinate with the Utility Specialist to determine project utility needs and location as early as possible in the design process. See *BDM 2.14.10*.

GENERAL CONSIDERATIONS

When the proposed utility weighs more than 90 pounds per linear foot or causes modification of a structural member, the utility company will be required to provide a load rating of the bridge per *BDM 1.3.2.2*.

The load rating will have the utility loading superimposed onto the bridge, so that it can be determined whether the bridge has sufficient loading carrying capacity for the installation of the utility. When available, provide a set of bridge plans for load rating use. All plans must be field verified, because not all As-Constructed bridge plans are accurate. See *BDM 2.10.8* and the *LRFR manual* for additional information.

Use existing utility accommodations located on the bridge, when possible.

REQUIREMENTS FOR ATTACHMENT TO BRIDGE

The following requirements apply to both new and existing bridges.

Design utility attachments to the currently accepted *AASHTO LRFD Bridge Design Specifications* as modified by this manual. Design utility lines in accordance to the governing industry standard for the type of installation.

Design utility installations so that a failure will not:

- Result in damage to the bridge.
- Be a hazard to traffic.
- Endanger the public.

Design individual members and the entire bridge for all loads imposed by the utility.

Utility attachments may exert large forces at the point of connection. Consider loads or movements that might be imposed on the utility by the bridge, such as from temperature, wind, or earthquake movements.

Restrain the utility line to prevent the line hitting bridge structural members. Use longitudinal **and** transverse supports or anchorages as needed.

Space utility supports for future utilities as follows:

- 2 feet minimum from abutment face.
- 8 foot maximum on center parallel to the utility line for Agency Communication Infrastructure.
- 10 foot maximum on center parallel to the utility line for other utilities.

Specify hot-dip galvanized steel utility supports, including fasteners and anchorages.

ATTACHMENT ANCHORING DESIGN GUIDELINES

Post-Installed Anchors

Follow the design requirements of *BDM 1.5.11.2*.

Design attachments that use a single anchor at each attachment point to remain serviceable if one of the other nearest attachments were to fail.

Mechanical anchors may be used when the following considerations are satisfied:

- Anchors are of a type that will maintain capacity under dynamic or vibratory type loads.
- Provide at least two anchors at 1.33 capacity-to-demand per attachment

for redundancy, or design attachments with a single anchor to provide a capacity-to-demand ratio of 2.0.

Review as-constructed plans and locate post-installed anchors to avoid conflicts with existing reinforcement.

Refer to *SP 00535, 00589* for additional information.

Cast-In Anchors

In the absence of specific instructions from the utility company, provide hot-dip galvanized expanded coil cast-in anchors with closed-back ferrule, threaded for rods or bolts installed in a new bridge deck.

When design of utility attachment using cast-in anchors is required, follow *BDM 1.5.11.3.*

ATTACHMENT DETAILING GUIDELINES

Locate holes in transverse members centered in the bay between the first interior and the exterior girders except for side-by-side girders.

Maintain the alignment of utility holes as straight as possible, both vertically and horizontally, to avoid difficulties in placing utility pipes.

Consider construction tolerances and variables in the design of brackets and hangers. Incorporate slotted holes, adjustable rod lengths, etc. into the attachment design.

Where utility holes are provided in the ends of the bridges for future utilities, form each hole with a pipe sleeve made of galvanized smooth steel pipe (1/4 inch min. thickness), or PVC pipe (Sch. 40 min.) of the same inside diameter as the utility hole.

When an approach slab is required, place a pipe through the sleeve to a point 5 feet minimum beyond the end of the approach slab, parallel to the centerline of the bridge. From the end of the pipe extension, angle the pipe at a 45 degree angle until the pipe is outside the shoulder.

Form a hole 1 inch larger in diameter than the pipe into the backwall or end beam. After the pipe is installed, fill the void around the pipe with a compressible material.

For side-by-side girders provide for future utilities through the end wall closure pours with one of the following options:

- Capped 8 inch diameter blockouts.
- Embedding a 6 inch diameter PVC pipe in the wall and extending it 8 to 10 feet beyond the bridge bent.

Limit the number and size of conduits in the bridge rail to assure ease of placement and proper consolidation of the concrete, and function of the bridge rail.

UTILITY LABELING

Utilities are to be labeled at each approach or first anchorage to the bridge and every 200 feet according to *American Public Works Association (APWA)* standards with color code and owner, contact information, etc. Adjust spacing to include one label in each bay bounded by beams and diaphragms.

See *BDM A1.11.2.9-1*.

Utility holes and pipes under approach slabs may need to be a larger diameter to accommodate joint splices, couplers, or bells at connections.

See *Appendix Figure A1.11.2.9-1*.

For side-by-side girders with CIP concrete decks, a space between two adjacent girders may be provided to allow for placement of a utility line.

Give special attention to details at expansion joint couplings because these tend to be much larger in diameter than the conduit.

See *SP 00589* for additional requirements.

1.12.6.5 Items for Design Submittal or Permit Review

Requests for attachments to bridges are submitted by Utility Companies to District. District makes the final decision on the permit application in consideration of comments and recommendations by the RBLE.

C1.12.6.5

Refer to *BDM 2.14.10* for roles and responsibilities and information.

Coordinate with District to ensure the utility permit includes requirements that the utility company will:

- Perform routine maintenance to the utility installation to ensure it remains in serviceable condition.
- Promptly respond and repair failing utilities.
- Remove utility apparatuses when no longer in service.

Maintenance access will not be allowed from the freeway for bridges carrying freeways or bridges carrying highways over freeways.

Provide utility plans and calculations stamped by an Engineer that is registered in the State of Oregon. Include the following in the submittal:

- Vertical, lateral, and longitudinal loading, as appropriate.
- Maximum and operating pressures for pressurized systems.
- Waterline thrust blocks.
- Loadings to be carried by the bridge and their location.
- Calculations for attachment connections or brackets.
- Details of the pipe, casing, vents, and shut-off valves.
- Utility hanger details.
- Utility labeling.
- Maintenance access plan.

Maintenance access plan should limit traffic impact for all classes of bridges.

1.13 Rails, Impact Attenuators and Protective Screening

1.13.1 Bridge Rail, General

C1.13.1

The current roadside safety crash test standard is the *Manual for Assessing Safety Hardware (MASH) 2016*.

After Dec. 31, 2019, all bridge rails on the National Highway System (NHS) must meet MASH, per AASHTO/FHWA Joint Implementation Agreement for Manual for Assessing Safety Hardware (MASH). When no MASH alternative exists, states can approve lesser standards, but should aim for MASH whenever possible.

Local agency projects off the NHS may use NCHRP 350, at the request of the local agency. Use *LRFD Chapter 13*, rather than this chapter, when local agency projects use NCHRP 350.

Bridge rails are roadway safety features. In most cases, the primary goal is keeping vehicles, bicycles and pedestrians on the road safely. Due to the dynamic nature of impacts, crash testing of all roadway safety features is performed to assure that critical failure modes such as vehicular stability and occupant compartment deformation are satisfied. MASH establishes the vehicles, speeds, and angles for the collisions required to achieve a passing crash test.

Prior to MASH, the controlling crash test standard was NCHRP 350, which itself replaced NCHRP 230. MASH crash test vehicles are taller than previous standards, requiring higher railings to maintain stability.

MASH requirements are directed at new construction. Thus far, AASHTO has not set acceptance criteria for retrofitting existing non-standard bridge rails, leaving it to the owner to establish warrants.

Due to the change to MASH, where this section is in conflict with *LRFD Section 13*, the *BDM* controls for all state-owned bridges and bridges on the NHS.

AASHTO is currently preparing to update *LRFD Section 13* to better accommodate MASH testing.

Under MASH, roadside safety features are evaluated based on six different Test Levels (TL), each including different vehicles and crash speeds and angles. The levels are intended to correlate to different levels of traffic.

More information on the different test levels is available in *LRFD 13.7.2*.

In Oregon, bridges on or over all interstate routes and National Highway System (NHS) routes require a **minimum** crash test rating of TL-4 for all bridge rails. ODOT promotes highway planning that replaces or upgrades railing on existing bridges on these routes to TL-4.

TL-3 is the lowest level FHWA permits on NHS routes.

FHWA allows existing bridge rails that are compliant with NCHRP 350 to remain in place under certain circumstances. All undamaged rails constructed after 1994 are assumed to be compliant. Contact the BDM Technical Resource for rails for pre-1994 rails when compliance cannot be established.

See *BDM 1.13.6* and *1.13.7* for more information on when NCHRP 350 compliant rails must be replaced.

To determine the appropriate TL for a bridge, follow the procedure of NCHRP Report 22-12(03), which uses a risk-based methodology. When this method does not correlate well with a specific bridge site, contact the BDM Technical Resource for alternatives.

In some situations, the test level reached through *NCHRP Report 22-12(03)* may not be appropriate due to other design features or conditions, such as wide sidewalks or very low speeds. In these cases, prepare a Design Exception to use a different test level.

In addition to the safety requirements of MASH, bridge rails should also meet OSHA requirements for the safety of maintenance and inspection personnel. *OSHA "Fall Protection – Walking-Working Surfaces"* requires a 42-inch high railing or other fall protection accommodations.

On state highways, 42-inch high bridge rails are preferred. *BR200* has transverse holes to enable workers to tie off for fall protection. Rail height is measured vertically from a 10-foot straight edge placed perpendicular to the lanes on the bridge deck to the top of the highest load resisting rail member.

1.13.1.1 Design Exception Process

Submit Design Exceptions per *BDM 1.2.2*.

Include in the Design Exception:

- Discussion of why an ODOT standard is not appropriate.
- Evaluation of appropriate test level.
- Crash test data documentation for alternate rail or one similar to the proposed design and conclusions that the rail performed acceptably in all three categories (structural adequacy, vehicle stability and occupant compartment deformation).

C1.13.1.1

1.13.2 ODOT Standard Vehicular Railings

Table 1.13.2-1 lists the ODOT standard bridge railings with current MASH test levels. Transitions are presented in *BDM 1.13.8*.

C1.13.2

More information about these rails are available in the standard drawing reports and on Projectwise.

Table 1.13.2-1 MASH Bridge Rails

Drawing No.	Description	Tested or Evaluated	MASH Test Level	OSHA Compliant
BR200	Concrete Bridge Rail Type **F**	Evaluated	3	With Anchorages
BR208	3-Tube Curb Mount Rail	Tested	4	Yes
BR214	Concrete Parapet with Steel Post	Evaluated	4	No
BR216	Sidewalk Mounted Combination Bridge Rail	Evaluated	4	Yes
BR222	Vertical Concrete Parapet, 42-Inch	Evaluated	4	Yes
BR285	Type **F** Concrete Rail Rectangular Tube Retrofit	Evaluated	4	Depending on existing
BR290	3 foot 6 inch Type **F** Rail	Evaluated	5	Yes

Table 1.13.2-2 lists other ODOT standard bridge railings that have not yet been evaluated or tested under MASH. Use of these rails on a state highway requires a Design Exception.

More information about the specific rails is available in the standard drawing reports.

Table 1.13.2-2 ODOT Standard Bridge Rails without a MASH Crash Test Equivalency

Drawing No.	Description	Crash Test Standard	Estimated NCHRP 350 Test Level
BR206	2-Tube Curb Mount Rail	NCHRP-350	4
BR212	Concrete Post and Beam Bridge Rail	Unknown	-
BR220	Flush Mounted Combination Bridge Rail	NCHRP 230	4
BR221	32 inch Vertical Concrete Parapet	NCHRP 230	4
BR226	2-Tube Side Mount Rail	NCHRP 230	4
BR233	Thrie-Beam Rail and Transition	NCHRP 230	2
BR250	Pedestrian Rail on Sidewalk Mounted Concrete Parapet	None	2
BR253	Sdwk. Mounted Conc. Parapet with Chain Link Fencing	None	2
BR256	Pedestrian Railing on Type **F** Concrete Bridge Rail	NCHRP 350	4
BR260	Chain Link Fencing on Type **F** Concrete Bridge Rail	NCHRP 350	4
BR266	Modified Type 2A Rail	NCHRP 230	2
BR273	Thrie Beam Rail Retrofit for Curb and Parapet Rail	NCHRP 350	4

1.13.2.1 Rail Orientation

Orient bridge rail posts and joints normal to grade in the longitudinal direction and vertical in the transverse direction for ease of construction and aesthetics.

Apply modifications to all rail components so they are all oriented the same (i.e. concrete parapet and attachments).

C1.13.2.1

When deck superelevations exceed 8 percent, consideration can be given to orienting posts normal to grade in both directions to be more aesthetically pleasing.

1.13.2.2 Rail Aesthetics

Maintain the minimum concrete cover requirements when using architectural treatments on concrete bridge rails. Cover can be increased to accommodate formliner patterns. Architectural treatment on the traffic face of a traffic barrier is only permitted within the limits of NCHRP Report 554. When that standard is met, no design exception is required.

1.13.2.3 Other Crash Tested Rails

Bridge rail designs from any state or local agencies that have been crash tested to MASH may be used with a Design Exception. Submit requests to the BDM Technical Resource for frequently used rails to be added to ODOT's Bridge Rail standard drawings.

1.13.2.4 Non-Standard Rails

On a case-by-case basis, rails that have not been crash tested may be permitted on state-funded projects with the submission of a Design Exception. The rails must be designed according to *LRFD Section 13*, and usage must be limited to TL-2 locations. Contact the BDM Technical Resource early in the project when this option is considered for a state-owned bridge.

1.13.3 Bicycles and Pedestrians

C1.13.3

Use bicycle and pedestrian railing or combination railing on the outside of structures that are specifically designed to carry bicycle and pedestrian traffic. Combination railing alone may be used in conjunction with a raised curb and sidewalk where vehicular design speeds are less than 40 mph. Separate bicycle and pedestrian traffic from vehicular traffic by a combination rail when vehicular design speeds exceed 40 mph and provide a pedestrian railing on the exterior of the walkway.

Bicycle and pedestrian rails must meet or exceed 42 inches in height. Opening sizes on bicycle, pedestrian and combination rails are defined by *LRFD 13.8.1*.

At locations with high bicyclist traffic, rail height can be increased to 48 inches and up to 54 inches.

Use curbs above the level of the sidewalk under all pedestrian railings where there will be significant pedestrian, vehicular or boat traffic under the structure. Runoff from sidewalks does not need to be carried off a structure, see *BDM 1.9.5* for additional guidance.	This requirement is intended to keep debris from being kicked under the railing onto traffic below.
Objects that are placed in the range of 27 inches to 80 inches may become protruding objects for pedestrians in the walkway, and shall not extend more than 4 inches into the adjacent walkway from the nearest vertical face or detectable edge.	Parapets, railings and fencing adjacent to the walkway can have impacts to the pedestrian access route. ADA requirements need to be reviewed carefully to ensure these items do not create protruding objects or restrictions in the clear width available to pedestrians.

1.13.4 Joints in Concrete Bridge Rail

SCORING JOINTS

Place scoring joints at 15 foot maximum centers, equally spaced between Type 'B' joints and expansion joints. Show the location of each joint on the deck plan without dimensions. Do not consider these joints to be end segments for the purpose of overhang calculations.

The joint spacing must equal or exceed the critical length, L_C, of the yield line failure pattern (see *LRFD A13.3.1*) for a vehicle impact within a wall segment. The bottom two longitudinal bars are continuous through scoring joints but terminate two inches before all other joint types.

TYPE 'B' JOINTS (AT INTERIOR BENTS WITH CONTINUOUS DECK)

To reduce shrinkage cracks in the rail, form a joint using ¼ inch preformed expansion joint filler through the rail. These joints also reduce the tendency of the rail to act compositely with the superstructure. Do not consider these joints to be end segments for the purpose of overhang calculations.

AT BRIDGE DECK EXPANSION JOINTS

Provide rail joints at every bridge deck expansion joint to prevent cracking or spalling of the rail or structure. Show rail details at expansion joints on the drawings. Skew rail joints to match the deck joint for skew angles up to 20 degrees. For skew angles in excess of 20 degrees, orient the rail joint normal to the rail.

C1.13.4

For typical ODOT standard concrete rails, space joints in the range of 10 to 15 feet.

See *BDM 1.9.2.2.2* for overhang reinforcement design information.

For ODOT Standard Rails, L_C is listed on the ODOT Bridge Engineering website – Software Tools for Design. It is typically in the range of 8.5 to 12.5 feet.

Do not leave rail joints as open joints, including joints between the bridge end and bridge approach slab. See expansion joint standard drawings for the rail joint material and arrangement. For expansion joints greater than two inches, use expansion joint cover plates to prevent vehicle snagging.

It is a potential problem when water passes through the joint and erodes the embankment. RR overcrossings may have additional requirements for joints in rails, depending on the RR.

1.13.5 Bridge Modernization Projects

C1.13.5

On Bridge Modernization projects, replace bridge rails to meet the required MASH TL criteria at the bridge site. The structural designer works with the project team to select the best rail for a given site. Considerations include:

- Roadway geometry.
- Traffic volume.
- Speed.
- Truck traffic.
- Accident history.
- Pedestrian needs.
- Sight distance.
- Occupant risk.
- Aesthetics.
- Corridor context.
- Maintenance.
- Inspection.
- Cost/benefit and related factors.

For new bridges, design the overhang to resist the test level of the rail or TL-4, whichever is greater.

The intent is to reduce the risk of future strengthening over the life of the bridge.

Standard deck overhang design is provided on *DET3170* and *DET3171*. When designing deck overhangs per LRFD, rail design capacities are available for ODOT standard rails on the ODOT Bridge Engineering website – Software Tools for Design. Using the calculated parapet resistance, M_c and T, of the parapet at its base per *LRFD A13.4.2*. LRFD rail design capacities will give overly conservative results in most cases.

Instead, for *LRFD A13.4.2* design case 1 for concrete parapet railings in TL-4 locations, design the deck overhang to resist the lesser of a vehicular impact moment, M_{CT}, and coincidental axial tension force, T_{CT}, calculated as follows, for the end and wall conditions:

$$M_{CT,end} = \frac{1.25 \times F_t \times H_e}{L_{c,end} + H_e + X}, \text{ and}$$

$$T_{CT,end} = \frac{1.25 \times F_t}{L_{c,end} + H_e + X}.$$

$$M_{CT,wall} = \frac{1.25 \times F_t \times H_e}{L_c + 2H_e + 2X}, \text{ and}$$

$$T_{CT,wall} = \frac{1.25 \times F_t}{L_{c,end} + 2H_e + 2X}$$

Where:

H_e = Effective height of the vehicle rollover force (ft).

L_c = Critical length of yield line failure pattern (ft).

X = Lateral distance from toe of barrier to deck design section (ft).

F_t = Transverse force specified in Table 1.13.5 for MASH TL-4 impact load for different barrier heights (kips).

LRFD A13.4.2 describes the design approach for deck overhangs supporting concrete parapet railings. The design approach does not specify vehicular-collision loads. Overhangs are designed for the maximum inelastic force effects generated and transmitted by the railing resisting a vehicular impact. This results in an extremely conservative deck overhang that is not in accordance with observed field behaviors.

Based on observations of impacted bridge railings from crash testing, an overhang designed with typical deck reinforcing of #5 at six inches does not fail when a railing failure occurs due to a collision. See *TTI Test Report No. 9-1002-5*, Figures 3.1-3.3, 6.3 and 6.4. In the absence of a more rigorous method for designing overhangs, many states use the loads shown here, based on finite element impact simulations from NCHRP funded research.

The end condition, at bridge ends and expansion joints, controls design due to the limited distribution length.

Table 1.13.5-1 Design Forces and Designations for Standard Barrier Heights (in)

Standard Barrier Heights	36 inch	39 inch	42 inch
F_tTransverse (kip)	67.2	72.3	79.1
F_LLongitudinal (kip)	21.6	23.6	26.8
F_vVertical (kip)	37.8	32.7	22
$L_t$andL_L(ft)	4	5	5
H_e(in)	25.1	28.7	30.2

When designing or checking overhangs, do not reduce capacity due to development length of the deck bars.

Impact loading is different in effect from sustained loading. Testing shows that typical development length calculations do not apply.

1.13.6 Bridge Retrofit Projects

C1.13.6

1.13.6.1 Process

C1.13.6.1

Establish required test level for the site. Upgrade the rail when any of the safety warrants exist (*BDM 1.13.6.3*), regardless of project scope. Otherwise, when project scope impacts the rail, assess the existing bridge rail for structural adequacy, vehicle stability, and occupant compartment deformation, in the post-project configuration (*BDM 1.13.6.2*).

When a deck overlay will reduce the relative height of the rail, this negatively impacts the structural adequacy of the rail.

See *BDM 1.9.2.2.2* for bridge deck overhang strengthening requirements.

Based on the deficiencies and required test level, select a retrofit or replacement option. When an appropriate standard drawing or detail is not available, it is permissible to design one for a specific project. A Design Exception is required in these cases.

Research is on-going into the behavior of resin bonded anchors under impact loading. Anticipate new standard rail attachment details upon completion of this research.

1.13.6.2 Assessment of Existing Bridge Rails

C1.13.6.2

STRUCTURAL ADEQUACY

Structural adequacy is a function of a barrier's ability (including deck overhang) to effectively contain and redirect a vehicle. Vehicles may not penetrate, underride, or override the barrier. It is possible to use engineering analyses, based on yield line theory or plastic design procedures, to compute the load capacity of rigid or semi-rigid barriers. Then compare that with known design forces for the desired test level. All aluminum tube rails are structurally inadequate.

For more information on the assessment of existing bridge rails, see NCHRP 22-14(03).

Since structural adequacy also requires the vehicle not override the barrier, barrier height is critical. For required height based on test levels, see *Table 1.13.6.2-1*.

Table 1.13.6.2-1 Minimum Bridge Rail Heights

Test Level	Minimum Height (inches)
TL-2	27
TL-3	29
TL-4	36
TL-5	42

These heights are not the same as *LRFD Table A13.2.1* due to MASH testing.

VEHICLE STABILITY

In order to ensure the safety of vehicle occupants, a vehicle must remain upright during and after a collision with the barrier. MASH evaluates this based on a maximum pitch and roll of the vehicle during testing. This is a common failure mode for bridge rails.

Whenever possible, assess this through comparison with previously crash tested rails.

Curbs and sidewalks that project in front of the face of rail negatively impact vehicle stability. Comply with *LRFD 13.7.1.1* for sidewalk height.

Many states only permit unprotected curbs and sidewalks, where design speeds are 45 mph or less.

OCCUPANT COMPARTMENT DEFORMATION

Occupant compartment deformation is commonly associated with snagging of the vehicle on the rail. Consider extensions or recessions beyond two inches as potential snag points; rails comprised of separate unconnected elements.

Evaluate snag hazard based on post setback distance, ratio of contact width to height, and vertical clear opening based on *LRFD Section 13*.

Improved vehicle design has reduced occupant compartment deformation, making this a less critical factor for evaluating barriers under MASH. Rails that satisfied this criteria under NCHRP 350 should be expected to meet MASH testing as well.

1.13.6.3 Safety Warrants

C1.13.6.3

Rails are considered candidates for retrofit or replacement as part of a bridge retrofit category project when any of the following are true:

- Three or more accidents or a fatality has occurred in the past five years at the bridge site.
- The rail is in a condition state four.

• The rail height is less than 27 inches.	Existing rails may be raised to meet height requirements, provided the existing strength capacities are not reduced by the revised configuration.
• The rail has aluminum components.	Aluminum rails are structurally inadequate under most impact conditions. In some cases it is possible to simply replace aluminum tubes with steel tubes without significant cost.
• The bridge is located on an interstate or high-speed, high-volume facility (for this purpose high volume is considered AADT of 30,000 or higher and has shoulder widths of less than three feet).	
• A safety assessment by the Region Traffic Engineer recommends an upgrade.	Consult the Region Traffic Engineer for concurrence, when postponing or deferring a rail upgrade on a bridge retrofit category project. The Region Traffic Engineer will perform a safety assessment to analyze existing safety hazards, crash history, site geometry, identify high risk sites, and determine when the implementation includes the appropriate solution. See *HDM 106.4, 111* and *119* for additional information on safety assessments. Document their concurrence in the DCSA.

1.13.6.4 Deck Anchorage

Options for anchoring a replacement bridge rail to an existing deck include through bolting and resin bonded anchors. When through bolting, oversize the drilled holes to create a ¼ inch annular space, seal the bottom of the bolt hole and grout the annular space around the bolt with high strength grout.

C1.13.6.4

This is required to ensure shear loads from impact transmit evenly to the deck.

See *BDM 1.9.2.2.2* for deck overhang capacity information.

1.13.7 Bridge Preservation Projects

Bridge preservation projects only require bridge rail modifications when:

- The relative height of the existing bridge rail is reduced below 29 inches (32 inches on interstates and other divided highways).
- The project is part of a 3R or 4R project on an NHS route and the existing bridge rail is not NCHRP 350 compliant.

In these cases, correct the deficiency or submit a Design Exception.

Correct transitions and blunt ends per *BDM 1.13.8*.

C1.13.7

Where projects negatively impact bridge rail height, *BR285* is a reasonable, cost-effective repair.

Where the existing rail is aluminum, assess the site risks and consider retrofitting the rail.

1.13.8 Bridge Rail Transitions

C1.13.8

Rail transitions are required on rail installations. They provide a controlled variation in stiffness from the approach guardrail to the more rigid bridge rail. The current transitions are crash-tested and have very close post spacing.

Research indicates transitions that passed NCHRP 350 testing will continue to meet MASH criteria *(NCHRP Project 22-14(03)*).

Select a transition, in conjunction with the roadway designer, appropriate for the speed.

In low speed locations (45 mph or less) where approach rail is not used, slope the bridge rail end down (end must be outside of the clear zone) or shield it using a crash cushion. See *HDM 405.4* for additional guidance.

All projects must upgrade transitions when unprotected ends or unconnected transitions are present per Technical Bulletin RD20-01(B). *RD20-01(B)* applies unless the work is considered a single function project that does not permanently modify the traveled way.

Problems have arisen when the first post off the structure conflicts with the bridge end. Consider any post conflicts and detail a solution in the contract plans. Prepare a design exception when adjusting the post spacing.

Possible remedies include:

- Remove concrete to allow room for the normal post to fit.
- Add a concrete pad (with anchor bolts) to the existing concrete, and add a base plate to the first post.
- Mount a structural steel spacer block to a vertical face of a rail end block, in place of a post.

1.13.9 Rails over Culverts & Railroads

1.13.9.1 Rails over Low Fill Culverts

When applicable, use Standard Drawing *RD470* and *RD471* for guardrail over low-fill culverts. When span lengths are over 25 feet, use Standard Drawing *BR266*, Modified Type 2A Rail, when the fill height above a box culvert or rigid frame is less than the standard embedment of timber guardrail posts.

Using this standard drawing eliminates the need for transitions.

1.13.9.2 Rails over Railroads

Use cast-in-place concrete rail without openings on structures over railroads. Use rail with a minimum height of 42 inches for structures in snow and ice areas, and rails with a minimum height of 30 inches elsewhere.

Extend rail to the limits of the railroad right-of-way or a minimum of 25 feet beyond the centerline of the outermost existing track, future track or access road, whichever is greater.

C1.13.9

C1.13.9.1

BR266 uses the same system reported in the *Transportation Research Record No. 1198*.

During the test, the steel posts yielded about 32 inches, which is similar to ODOT's timber post system. The crash test report claims this system is acceptable for fill heights from zero to three feet.

C1.13.9.2

Snow and ice zones are defined in *BDM 1.2.4.3*.

Existing locations with insufficient height or non-solid barrier (e.g., 3-tube rail) may require in-filling gaps over facilities below. Possible solutions are use of splashboards or providing tubes between primary rail tubes equal to the thickness of the spacers used for primary rail tubes. See *BR245* for splash board details or BR 09515.

Refer to *UPRR-BNSF Guidelines for Railroad Grade Separation Projects* for additional information on requirements at railroad crossings.

1.13.10 Temporary Barriers

FHWA requires that temporary bridge rails meet TL-3 performance criteria using successfully crash tested systems. Ordinarily temporary bridge rail is constructed from pin and loop median barrier secured against sliding and overturning as shown in Standard Details *DET3295* and *DET3296*. Restraints are not required when the barrier can be displaced 5.5 feet or more away from the traffic side(s) without infringing on a traffic lane, a work area, or beyond the edge of the deck.

Coordinate with the Traffic Control Plans (TCP) designer to confirm the assumed layout of temporary barrier on the bridge agrees with the final layout shown in traffic control plans, and to determine when the detail plans must note the reflectorized barrier. Modify *DET3295* accordingly.

Use resin bonded anchor bolts with fully threaded rods in accordance with *ASTM* F1554 grade 36. Use anchor bolts for through bolting in accordance with *ASTM* A307 or *ASTM* F1554 grade 36. Use nuts in accordance with *ASTM* A563 or *ASTM* A194. Use flat washers in accordance with *ASTM* F436 and plate washers shall be in accordance with *ASTM* A36 or *ASTM* A709 grade 36.

Install four anchor bolts per barrier on the traffic side, as shown in Standard Details *DET3295* and *DET3296*. Do not drill into or otherwise damage the tops of supporting beams or girders, bridge deck expansion

C1.13.10

The ODOT anchored barrier is adapted from barrier used in a Lincoln, Nebraska crash test, documented in report *TRP-03-134-03* dated August 22, 2003. The current standard detail does not reflect the validated test results from Nebraska's 2019 crash test.

For more information regarding temporary barriers, see *Section 2.6.1* and *3.4.25* of *TCP Design Manual*.

joints or drains. Install anchor bolts and nuts so that the maximum extension beyond the face of the barrier units is ½ inch. Snug tighten the nuts on the anchor bolts. For through bolted installations, snug tighten the double nuts on the underside of the deck against each other to minimize the potential for loosening.

Omit one anchor bolt within a single barrier unit when a conflict exists between the anchor bolt location and a bridge deck expansion joint or drain. Install each adjacent barrier units with the standard four anchor bolts.

Other temporary rail options: At least one crash tested proprietary steel safety shape rail system exists, which could be a contractor option for temporary rail use. Example: see FHWA Acceptance Letter B-165.

1.13.11 Protective Screening or Protective Fencing

Oregon law mandates the installation of protective screening. *ORS 366.462* states:

1. The DOT shall construct fences on all freeway overpasses that are built on and after Nov. 4, 1993. The fences shall be designed to deter persons from throwing objects from the overpasses onto the freeways.
2. The DOT shall construct fences on existing freeway overpasses that involve the greatest risk factors.

1.13.11.1 General

Provide protective screening at the following locations on projects that modify the rail or when required by the facility owner:

- All structures crossing freeways (interstates and similar controlled access highways with at least four lanes) that carry vehicles or pedestrians.
- Pedestrian structures which cross high-speed facilities (posted speed equal to or greater than 55 mph) and are located within half mile of a school, playground, park, athletic field, shopping center, or other facility likely to generate pedestrian traffic.
- Structures with sidewalks which cross high-speed facilities (posted speed equal to or greater than 55 mph) and are located within half mile of a school, playground, park, athletic field, shopping center, or other facility likely to generate pedestrian traffic.
- All other structures (with or without sidewalks) crossing high-speed facilities (posted speed equal to or greater than 55 mph) with regular pedestrian traffic.
- All other structures where the fall

C1.13.11.1

Contact the Bridge Standards and Program Manager for a list of applicable freeways.

height is 50 feet or greater.

- On both sides of all overhead structures crossing railroad right-of-way.

Portland and Western Railroad (PNWR) and Central Oregon and Pacific Railroad (CORP) do not require any fencing.

- In front of ITS elements attached to bridge structures.
- When the opening between parallel bridges is 10 feet wide or less. Required only on the interior railings of each structure.

Increasing the bridge rail height to a minimum of 54 inches is also acceptable.

Protective screening need not be provided at freeway ramp structures that typically do not have any provisions for pedestrians.

When a project impacts the bridge rail, install protective screening when any of the situations above apply.

1.13.11.2 Protective Screening Design Criteria

C1.13.11.2

Design protective screening to deter persons from throwing objects from the overpasses onto freeways. Design protective screening using the following criteria:

The design criteria in this section also applies to exclusionary fencing.

- Use design loads per *AASHTO 13.8.2* and dead load per standard drawings.
- Able to carry pedestrian rail loading when pedestrians are not separated.
- Openings three inch square or less. Minimize projected area to less than 30 percent of total area.

A two inch chain link mesh is generally acceptable, with a one inch mesh being used for special cases.

Use details that are difficult to climb (e.g. no handrail).

- No more than one quarter inch opening between the bottom of screening and top of concrete surface.

Ensure the bottom of screening has sufficient stiffness to prevent permanent large deflections.

- Minimum eight feet high from top of walk surface. Minimum 10 feet high at railroad overcrossings from top of walk surface. When ornamental screening has a variable height, maintain minimum height at all locations crossing over travel lanes. Minimum 54 inches high when the opening between parallel bridges is 10 feet wide or less.

Protective screening limits and extents:

- Provide protective screening over all travel lanes plus a minimum of 10 feet beyond the travel lanes on each side.

  Coordinate with roadway to consider future widening when determining screening limits.

- Where on or off ramps also cross under a structure, ensure screening extends a minimum of 10 feet beyond the end of any ramp travel lanes.

  In areas where aesthetics is a consideration and when screening does not extend to the end of the structure, provide a transition panel (sloped panel or partial height panel) at the end of each run of screening as an aesthetic termination.

- For divided highways, continue protective screening uninterrupted through the median.

  For unusually wide medians and divided highways with a significant elevation difference for each direction, protective screening may be interrupted through the median with the use of transition panels.

- Provide protective screening on both sides of a structure, even when a sidewalk is provided on only one side. Where twin structures cross a high-speed facility, provide protective screening for the center opening between structures.

- Provide protective screening to the greater of the following on structures over railroads:
    - Railroad right-of-way or
    - 25 feet minimum from centerline of nearest existing or future track or railroad access road.

1.13.11.3 Design Deviations

Submit a design deviation when protective screening is not provided for structures specified in *BDM 1.13.11.1.*

Submit a design deviation when protective screening details deviate from standard drawings, as is the case for retrofit installation.

Submit a design deviation request to exclude screening at locations that present a sight distance hazard.

Provide the following with a design deviation request:

- Basis for the proposed design deviation.
- A description of pedestrian activity including width of sidewalks and proximity to pedestrian sources such as schools, playgrounds, parks, shopping center or athletic fields.
- The history of incidents and signs of graffiti at the bridge site or sites in the vicinity.
- The distance to adjacent bridges also

C1.13.11.3

Due to public safety concerns, public or legislative oversight of design deviations for protective screening is likely.

Screening may obscure sight distance on horizontally curved structures; of signage; and the intersection sight distance at ramps, cross-streets, or driveway accesses off the end of the structure. Non-signalized intersections increase this potential hazard. Specialized designs may be considered to address sight distance issues, such as adjusting screening limits.

crossing the facility and whether they have screening.

In addition, when the basis for the design deviation is for sight distance provide the following:

- Concurrence from the Region Roadway Manager.
- A plan of the bridge showing sight lines obstructed by the proposed screening.
- The approximate cost of widening the structure when widening would avoid a sight distance hazard.

1.13.11.4 Screening Type Considerations

When placing a barrier between the sidewalk and roadway, use protective screening in place of a pedestrian rail along the outer edge of the structure.

When no separated barrier exists, place screening behind or attached to the combination rail along the outer edge of the structure.

When installing ITS elements on a structure, use Type 'A' fencing.

Use Type 'A' fence *BR240* on pedestrian bridges.

VERTICALLY CURVED SCREENING

Consider curved screening when a sidewalk is present. Curved screening does not require end treatment. Curved screening can be used in combination with Type A, B, C and D fence.

C1.13.11.4

Protective screening affects bridge inspection using under bridge inspection trucks (UBITs). Details should be discussed with the Region Bridge Inspector (RBI) prior to installation over bridges with limited inspection access.

Certain sweepers will not fit through curved fence enclosures. Region 1 sweepers measure 10 foot 5 inches. Standard Drawing *BR240*, Type 'A' Fence has provisions to allow access. Contact the District Bridge Supervisor to determine an acceptable type of fence.

Curved screening is an additional deterrent because it forces the thrower into the roadway in order to clear the screening. Curved screening may cause additional sight obstruction. Curved screening may require additional height to accommodate bicycles and, in some cases, horses with riders. Curved screening is not preferred in the Portland area due to aesthetic concerns.

HORIZONTALLY CURVED STRUCTURES

On structures with tight horizontal curves, consider using straight screening rather than curved. When chorded screening is used on a tight curve, ensure any **gap** between the bottom of screening and the curved edge of the bridge does not exceed three inches. Such **gaps** may be closed using plates attached to concrete surfaces near the bottom of the screening.

It is difficult to construct curved screening on a tight curve and obtain proper fit of the chain link fabric.

1.13.11.5 Screening Aesthetics

C1.13.11.5

AESTHETIC CONSIDERATIONS

Chain link is the most economical screening available and used as the default screening type. However, chain link has very low aesthetic value.

There are proven low-cost methods available for improving the aesthetics of chain link screening:

- End treatment - Providing a special termination section at each end of each screening run is a low-cost and effective aesthetic enhancement. This can be as simple as tapering the ends (for example, see *DWG 65137*) or a reduced-height panel. Any end treatment with a height less than the minimum required must start at least 10 feet beyond any travel lanes or ramps (25 feet from tracks or access road for railroad crossings).
- Color – Use of vinyl-coated chain link can improve the appearance of chain link at a very modest increase in cost. Possible colors are black, navy blue, or dark green, depending on location. Hot-dip galvanize screening before vinyl-coating.

Designers are encouraged to seek input from others (designers, district, and local community) when using aesthetic concepts outside these methods.

ORNAMENTAL SCREENING:

ODOT has received requests to install ornamental screening on structures. See Standard Detail *DET3243* and *DET3244*. Address the following before a request can be processed:

- Funding – Ornamental screening can be included in bridge modernization and retrofit projects when deemed an important architectural item by the project team and supported by the environmental study. For retrofit to an existing structure (not associated with an ODOT project), include possible funding sources with the proposal.
- Permits – When an entity outside ODOT proposes to install a feature in ODOT right-of-way, it must obtain a permit from the district office.
- Design – Do not create a distraction for drivers with ornamental protective screening. Review design outside of ODOT's normal standards through a review process with the district maintenance, Region Tech Center and others to assure it is appropriate and meets ODOT clearances and standards.

- Maintenance – Establish maintenance responsibility in case of damage or deterioration. Districts receive funding to maintain ODOT standards. When designed and installed by forces outside of ODOT, resources are required to maintain it, which should include a bond, city or county taking responsibility.

Powder coat ornamental screening. Paint can be used, when requested, for a specific reason and agreed to by maintenance. Also see *BDM 1.2.5.10, Structure Appearance and Aesthetics, Ornamentation*.

1.13.12 Impact Attenuators

Attenuators are required in areas where hazardous objects cannot be removed from the possible paths of vehicles. Such locations include gore points of diverging roadways and columns in medians.

Avoid geometry where attenuators will be required on structures.

1.14 Bearings and Expansion Joints

1.14.1 Bearings

1.14.2 Expansion Joints

1.14.1 Bearings

1.14.1.1 Design, General

Provide provisions for bearing replacement, including temporary jacking and support for all manufactured bridge bearings. There is a potential of bearing failure during the service life of a bridge, which requires that provisions for bearing replacement be provided in the design drawings. Providing temporary jacking support (design, detailing and construction) on existing structures is complex and increases the maintenance cost and life cycle cost of a bridge. Including consideration of jacking and temporary support in the original design will reduce future rehab cost and ease future bearing replacement. This work may require pilecap or crossbeam widening, or widening under each girder. Show grout pad locations in the contract drawings for temporary jacking support and a bearing replacement sequence and minimum jacking loads. Check the adequacy of all affected structural elements during bearing replacement and stability of the structure. See *BDM 1.3.4* for temporary loading and *BDM 1.2.6.1.6* for inspection and maintenance considerations.

1.14.1.2 Elastomeric Bearing Pads

Elastomeric bearings are used to accommodate movements on short to medium-span structures. The three types of pads include:

- plain pads.
- laminated pads reinforced with fabric (fiberglass).
- laminated pads reinforced with steel.

Plain pads are made from elastomer molded or extruded into large sheets, vulcanized and then cut to size.

Do not use cotton duck pads or random Oriented Fiber Pads bearing for slabs and box beams construction. Use plain elastomeric (neoprene) pads instead.

Fabric reinforced pads are made from alternate layers of elastomer and fabric (usually fiberglass) in large sheets, vulcanized and then cut to size. Fabric reinforced pads are restricted to short to medium spans with little or no skew.

Steel reinforced pads are made from alternate layers of elastomer and steel cut to size and then vulcanized. A thin cover layer of elastomer encapsulates the steel to prevent corrosion. The exposed edge voids in the pads caused by the steel laminate restraining devices are shop sealed with an appropriate caulking material.

Use **Method A** to design elastomeric bearings. Where there is a need to use **Method B**, specify in the *Special Provisions* and contract drawing that the **Method B** was used. Elastomeric bearings designed using **Method B** require extra testing.

Use the following movements for pad thickness design:

$$ES + LF1 * (CR + SH) + LF2 * (TF \ or \ TR)$$

Where:

ES = elastic shortening movement

CR = creep movement $CR = (ES) * (CF)$

SH = shrinkage movement

TF = temperature fall movement

TR = temperature rise movement

CF = creep factor

LF_1 = from *LRFD 3.4.1*

LF_2= TU, Load Factor from *LRFD Table 3.4.1-1*

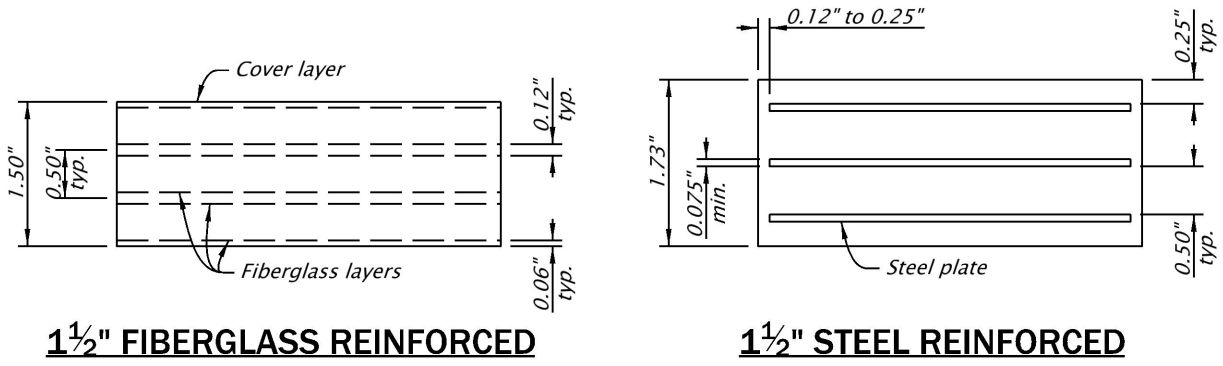
Use proper signs and the Service Limit State Load Factor that produces the largest movement in each load combination.

The final elastomer thickness is two times the design movement. Size the nominal pad thickness in multiples of ½ inch, from ½ inch to 6 inches maximum. The actual finished thickness will vary depending on the type of reinforcement. Fabric has a negligible thickness. Steel plate thickness may vary with the manufacturer, but be a minimum of 14 gauge.

The creep factor above is taken as 1.5 for both prestressed and post-tension concrete structures. Shrinkage movement is calculated using 0.0004 times the total length of the structure. For prestressed concrete structures 40 percent of this movement takes place within the first thirty days after manufacture. Therefore, the amount of creep and shrinkage movement for these structures, after placement, can normally be taken as 60 percent of the total.

Values for shortening of post-tensioned, cast-in-place concrete bridges have been determined by field measurements by the ODOT Bridge Section. See *BDM 1.5.8.1.* Compare the design values with the field measured values and use the more conservative values.

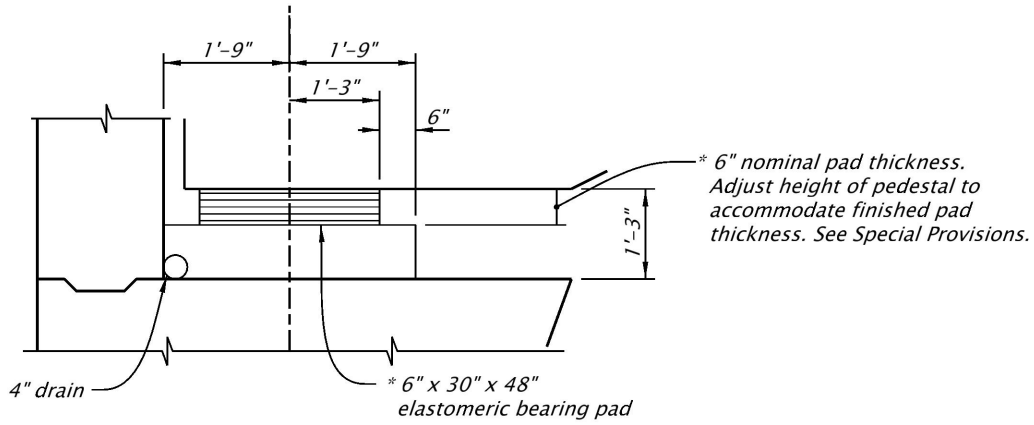
Figure 1.14.1.2-1 Standard Bearing Pad Detail



Pad thickness called for on detail plans is the total thickness of the elastomer required. When bearing pad elevations are shown, list the assumed finished pad thickness. Use circular elastomeric bearing pads for curved steel girders.

Examples are shown below in *Figure 1.14.1.2-2*.

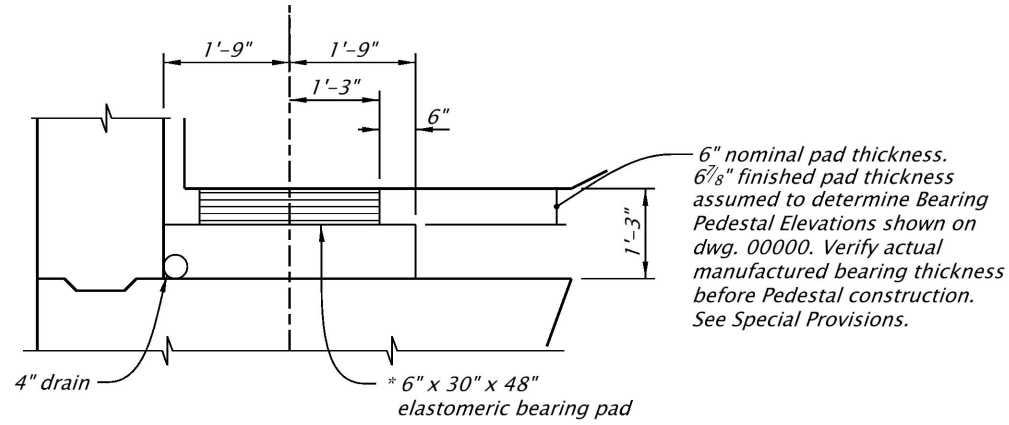
Figure 1.14.1.2-2 Elastomeric Bearing Pad Examples



SECTION

ELASTOMERIC BEARING PAD

(No Elevations Shown)



SECTION

ELASTOMERIC BEARING PAD

(Elevations Shown)

For prestressed slab and box beam bearing pad sizes, use *Figure A1.11.1.7-4* (end bents) or *Figure A1.11.3.2-3* (interior bents).

1.14.1.3 Proprietary Pot, Disc, Slide, Radial, or Spherical Bearings

These bearings are normally used on long-span and post-tensioned concrete structures where the design movement cannot be accommodated with elastomeric bearings.

When provided to allow longitudinal movement for concrete superstructures, design bearings to accommodate the anticipated effects of shrinkage, creep and elastic shortening (where applicable) as well as temperature.

Use the following movements for proprietary bearings:

Shortening: $ES + LF1 * (CR + SH) + LF2(TF)$

$ES + LF1 * (CR + SH) + LF3(EQ)$

Lengthening: $LF2 * (TR)$

$ES + LF1 * (CR + SH) + LF3(EQ)$

Where:

ES = elastic shortening movement

CR = creep movement $CR = (ES) * (CF)$

SH = shrinkage movement

TF = temperature fall movement

TR = temperature rise movement

CF = creep factor

EQ = Maximum design earthquake displacement (movable bearings)

LF_1 = from *LRFD 3.4.1*

LF_2= TU, Load Factor from *LRFD Table 3.4.1-1*

LF_3= Load Factor from *LRFD Table 3.4.1-1*

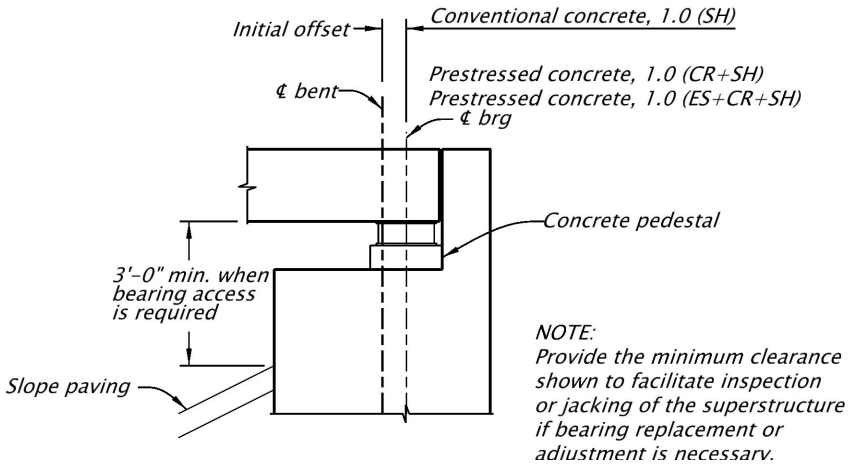
Use proper signs and the Service Limit State Load Factor that produces the largest movement in each load combination.

The creep factor above is taken as 1.5 for both prestressed and post-tension concrete structures. Shrinkage movement is calculated using 0.0004 times the total length of the structure. For prestressed concrete structures 40 percent of this movement takes place within the first thirty days after manufacture. Therefore, the amount of creep and shrinkage movement for these structures, after placement, can normally be taken as 60 percent of the total.

Values for shortening of post-tensioned, cast-in-place concrete bridges have been determined by field measurements by the ODOT Bridge Section. See *BDM 1.5.8.1*. Compare the design values with the field measured values and use the more conservative values.

Detail the initial position of expansion bearings so that the bearing will behave satisfactorily after the design movement has taken place.

Figure 1.14.1.3-1 Bearing Position to Allow Movement



Performance Specifications for Approved Proprietary Bridge Bearings are now covered by the *Standard Specifications*. Approved bearings are listed in the *Qualified Products List*, which is available on the ODOT website.

The designer must check the shop drawings, specified test results, and certifications for compliance with these specifications.

When proprietary bearings are used, show the following details and information in the contract plans:

SCHEMATIC DRAWING

A schematic drawing of the bearing showing the method of attachment of the upper and lower units to the superstructure and substructures, respectively. See *Figure 1.14.1.3-2* for an example.

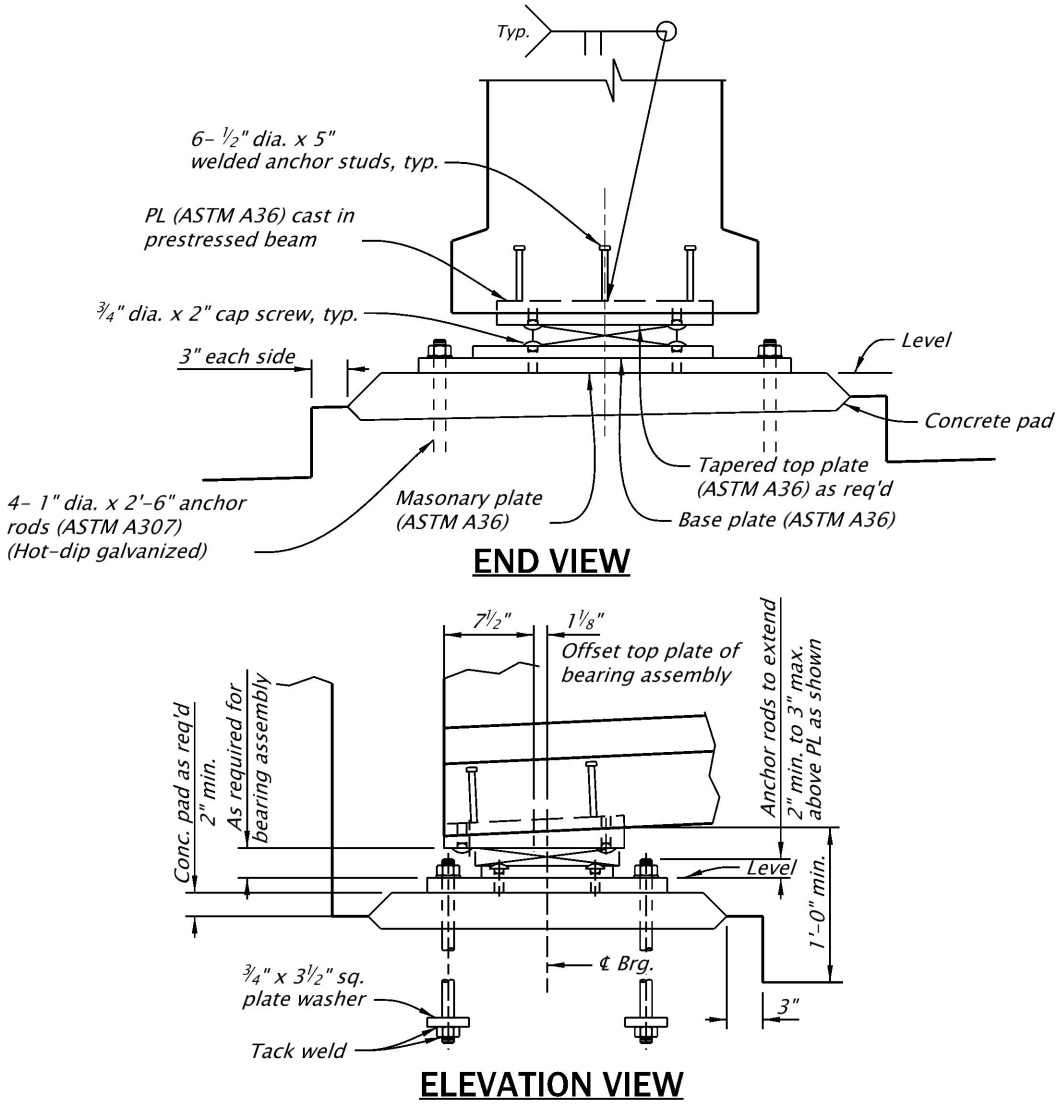
DESIGN NOTES

List design notes for:

- Required clearance to edge of concrete support.
- Maximum allowable concrete bearing stress.
- Minimum rotational capacity of bearing (not less than 0.015 radian).
- Any restriction as to type of bearing (pot, disc or spherical).
- Reference to bearing schedule for load and movement capacity.
- Reference to *Standard Specifications* for painting.
- Reference to the *Qualified Products List* for approved bearings.

Paint all exposed surfaces of the bearing devices except teflon, stainless steel, machine finished or polished bearing surfaces, as set forth in *SP 00594*. Provide a primer coat only for portions to be in contact with concrete and for steel to steel contact surfaces.

Figure 1.14.1.3-2 Schematic of Bearing Showing Attachment



1.14.1.4 Bearing Replacement

Consider the potential of expansion bearing replacement during the life of the structure in sizing of crossbeams and bents. Provisions may need to be made for jacking locations.

When a bent is accessible (close to the ground, out of traffic, etc.) it may be assumed that a falsework jacking bent can be constructed and no special provisions on the bent are required.

See *BDM 1.3.4* for information on temporary loading. See *BDM 1.16.3.3* for information on falsework.

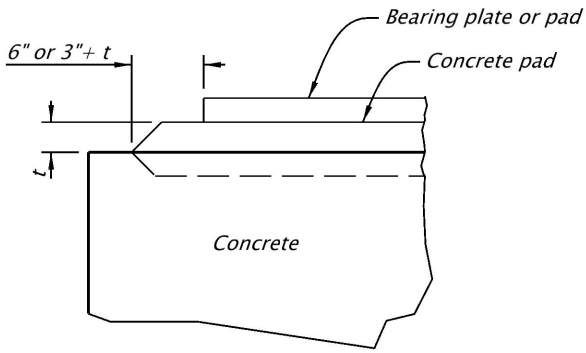
When the bent is not easily accessible, provide provisions for jacking, such as a wider crossbeam or strengthened diaphragm beam.

1.14.1.5 Reinforced Concrete Bearing Seats

CLEARANCE

The minimum horizontal clearance from the edge of a bearing plate, or 1 inch and thicker elastomeric bearing pad, to the edge of a concrete bearing seat shall be 6 inches, or 3 inches plus the thickness of grout under the bearing, whichever is greater. Where the bearing is skewed with the bent, this dimension may be reduced at the corner of the pad. Locate anchor bolts a minimum of 6 inches clear of the nearest face of concrete.

Figure 1.14.1.5-1 Concrete Bearing Seat Clearance

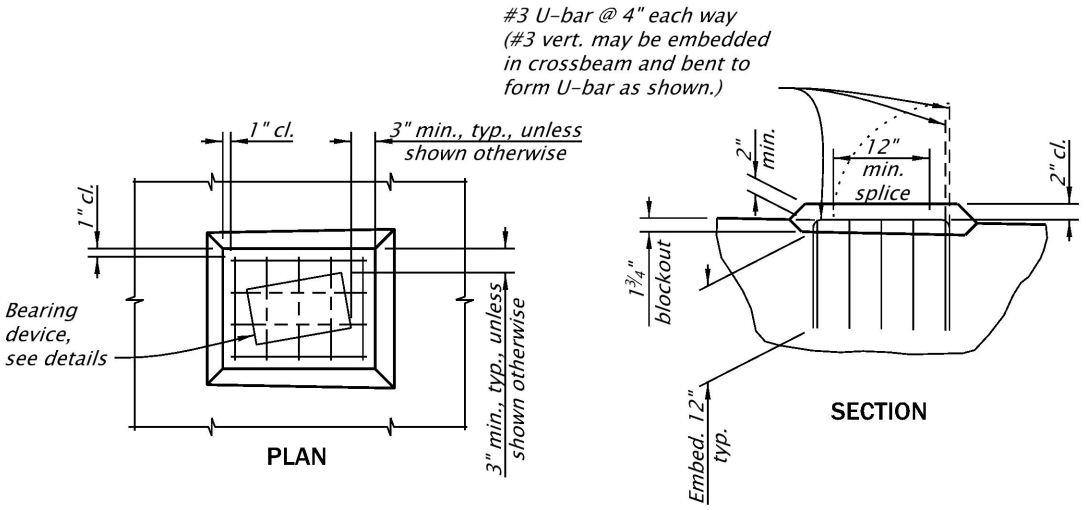


ADDITIONAL REINFORCEMENT

Generally, detail a reinforced concrete buildup, as shown below, under the bearings of all prefabricated beams, except precast slabs and box beams less than 70 feet in length.

Certain bearings may require no concrete buildup but have the bearing surface ground to grade.

Figure 1.14.1.5-2 Typical Concrete Pad Details



TYPICAL CONCRETE PAD

Scale: 1/2"=1'-0"

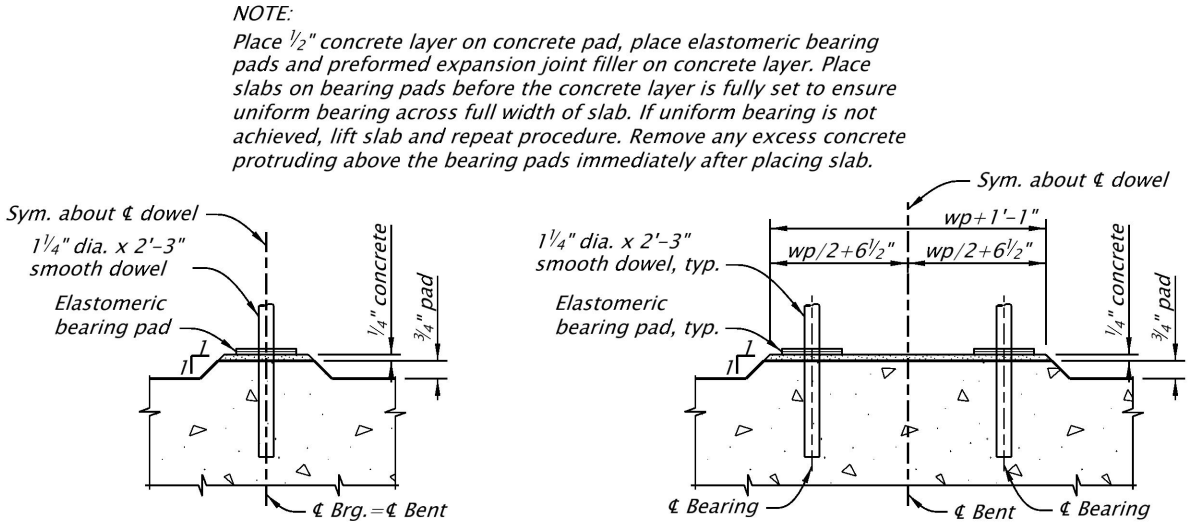
1.14.1.6 Unreinforced Bearing Seats (Prestressed Slabs and Boxes)

GENERAL

For prestressed slabs and boxes, provide bearing details as shown in *Figure 1.14.1.6-1*.

Set precast concrete slabs over 40 feet in length on elastomeric bearing pads. Do not allow cotton duck pads as a replacement for elastomeric bearing pads.

Figure 1.14.1.6-1 Bearing and Concrete Pad Details



CONSTRUCTION PROCEDURE

STEP 1. Pour 1½ inch concrete pad, allow concrete to cure for three days or until concrete obtains design strength.

STEP 2. Place ½ inch concrete layer as shown in *Figure 1.14.1.6-1*.

1.14.1.7 Bearing Schedule

Include the following items in the Bearing schedule:

- Location of bearing (bent number).
- Number of bearings required (number per bent).
- Bearing fixity (fixed, guided or non-guided).
- Final dead load (load/bearing).
- Vertical design capacity (dead load + live load + impact, load/bearing).
- Horizontal design capacity of fixed and guided bearings (not less than 10 percent of the vertical design capacity).

The specification requires each guided bearing to resist the entire horizontal load at any one bent. Use no more than two guided bearings per bent or hinge. Where more than two guided

bearings are required, provide devices independent of the bearings to resist horizontal loads. Use non-guided bearings at these locations.

Design movements for:

- Mean temperature.
- Temperature rise.
- Temperature fall.
- Creep, shrinkage and elastic shortening.
- Change in bearing centerline per specified temperature increment.

Ensure the top bearing plate dimensions are adequate to compensate for the initial bearing offset shown.

Provide additional bolted plates with the top and bottom plates of the bearing assembly to facilitate removal of bearing for repair or replacement and to provide a level surface for the bearing unit.

Figure 1.14.1.7-1 Bearing Schedule

BEARING SCHEDULE

BENT	NUMBER REQUIRED	TYPE	DESIGN LOAD CAPACITIES IN KIPS PER BEARING			INITIAL OFFSET	CALCULATED MOVEMENTS			MOVEMENT PER 10° F TEMP. CHANGE	MINIMUM MOVEMENT CAPACITY FROM INITIAL POSITION		
			VERTICAL	LATERAL	LONGIT.		30° TEMP. RISE	40° TEMP. FALL	CREEP, SHRINKAGE & ELASTIC SHORTENING		TEMP. RISE DIRECTION	TEMP. FALL DIRECTION	TOTAL
1 & 5	*4*	*Guided*	*1000*	**600*	*---*	*3"*	*⅞"*	*1¼"*	*4½"*	*⅜"*	*1½"*	*7½"*	*9"*

**Reduce design load to 200 kips for PTFE surface only.*

1.14.2 Expansion Joints

1.14.2.1 Definitions

Armored Joint - Steel armoring to protect the vertical edges of a joint opening. The armor may be steel shapes.

Asphaltic Plug Joint (APJ) Systems - A closed expansion and contraction joint system composed of aggregate and flexible binder material placed over a steel bridging plate.

Closed Expansion Joint - A joint in which a seal material is placed to prevent water or debris from entering the joint. This includes poured joint seals, compression joint seals, asphaltic plug joint systems, preformed strip seals, and modular bridge joint systems.

Control Joint - A joint created by sawing a groove in a surface and filling it with a poured material, creating a weakened vertical plane that controls the location of cracking developed due to restraint stresses.

Filled Joint - A filled joint using a preformed joint filler placed prior to the concrete pour. Hot applied joint sealant is placed on top of the joint filler.

Hot Applied Joint Sealant - A hot-poured asphaltic material used for sealing cracks and joints from water penetration to prolong pavement and joint life.

Modular Bridge Joint Systems (MBJS) - A closed expansion and contraction joint using a series of continuous preformed strip seals inserted into steel shapes to seal the joint.

Poured Joint Seal - A closed expansion and contraction joint sealed with a rapid-cure poured joint sealant (2 part silicone).

Precompressed Foam Silicone Joint (PFSJ) Seal - A closed expansion and contraction joint system consisting of a preformed, pre-compressed, silicone-coated, self-expanding foam joint system bonded to joint faces using epoxy adhesive.

Preformed Compression Joint Seal - A closed expansion and contraction joint sealed with a continuous preformed elastomeric compression gland.

Preformed Joint Filler - A preformed expansion joint material having small extrusion and substantial recovery after release from compression.

Preformed Strip Seal System - A closed expansion and contraction joint using a continuous preformed elastomeric gland (strip seal) inserted into an extruded or formed steel retainer bar with steel anchors.

1.14.2.2 Design Considerations for Joint Seals and Systems

Consider integral abutment or semi-integral abutment wherever criteria in *BDM 1.11.2.5* and *BDM 1.11.2.6* are met. Design expansion joints to provide for the effects of temperature, shrinkage and creep.

Use skew angle a minimum of ±5 degrees different from snow plow angle for all joints except asphaltic plug joints. Normally the angle of attack of snowplows is skewed 30 degrees to the roadway alignment. Snowplow blades can fall into the joint where the skew angle of the joint matches the snowplow's angle, resulting in danger to the snowplow driver or traffic. Consider the effect of skew angles on future widening of the structure.

STRIP SEAL SYSTEM

Use preformed single strip systems to seal deck joints with up to 4 inch range of movement (1½ inch minimum installation width). For joints of greater anticipated movement, use a modular bridge joint system. It is not recommended to use a modular bridge joint system solely to provide for possible seismic movements.

PREFORMED COMPRESSION SEAL

Preformed compression seals may be specified for joints with a design movement of up to 1½ inches. Specify a seal size to ensure that the seal remains in compression throughout the service life.

POURED JOINT SEAL

Poured joint seal may be specified for a design movement up to 1½ inches.

ASPHALTIC PLUG JOINT SYSTEM

Asphaltic plug joint systems are suitable for joints between two pavements with asphalt concrete pavement and may be specified where following conditions are satisfied:

- Maximum range of design movement up to 1 ½ inches (total).
- Maximum bridge skew less than 45 degrees.
- Maximum lateral movement at joint ¼ inch.
- Maximum vertical movement at joint (uplift) ¼ inch.
- Maximum superelevation of 6 percent.

Asphaltic plug joint systems do not perform well under following conditions:

- Where traffic is accelerating or decelerating, such as intersection with traffic lights or stop signs.
- Bridge with a curved horizontal alignment.
- Longitudinal joint between two structures. Skid resistance of this joint diminishes with time and it may become a hazard to motorcyclist and bicyclists.

PRECOMPRESSED FOAM SILICONE JOINT SEAL

Specify precompressed foam silicone joint seals for joint rehabilitation only. PFSJ seals may be specified for joints with a design movement up to 2 ½ inches. Limit a joint installation width smaller than 3 inches to ensure joint performance. Field verify the joint width. This joint system

can be field-spliced, which makes it suitable for staged construction and partial joint repair. Specify factory-fabricated bends for inside corners at the gutter line. Field verify a bend angle required for the existing inside corner and show the bend angle on project plans. When joints cross sidewalk on a bridge, specify a PFSJ seal system that is suitable for pedestrian traffic and provide joint details according to *RD722*. Splicing between bridge and pedestrian PFSJ seals are possible.

FILLED JOINT

Use filled joints for short span bridges with pin end bent connection. These joints are the least expensive joint and easy to repair.

CONTROL JOINT

Use a control joint to control the location of cracking at the end of bridge approach slab and in pavement over joints. For rehabilitation projects, control joints can be used in continuous concrete bridge deck over intermediate bents when it is necessary to control cracking in a particular location. Use ½ inch sawcut width to accommodate hot applied joint sealant installation. The control joint may be specified for contraction movement up to ¼ inch.

Check the *Qualified Products List* for the currently acceptable materials and joint systems.

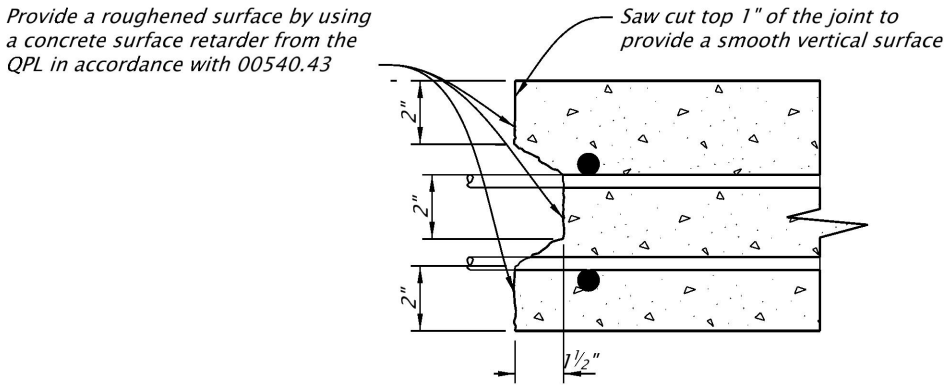
CONSTRUCTION JOINTS

Minimize the number of deck construction joints to avoid potential leaks through the deck. However, it is often necessary to provide deck construction joints to avoid shrinkage or deflection cracking. See *BDM 1.6.2.15* for more information on joints on steel structures, and *BDM 1.11.2.5* and *1.11.2.6* for options to eliminate joints.

Normally for non-continuous spans, deck concrete placements are full length or stopped at a transverse beam. The construction joint surface is normally vertical and roughened, according to *SP 00540.43(a)*, between placements.

Normally, shear keys at construction joints are unnecessary. Show construction joints with a roughened surface finish unless shear keys are required and shown on the plans. For continuous spans or for emergency situations, provide a shear key with a roughened surface between placements. Show typical key details on the plans as detailed in *Figure 1.14.2.2.1-1*.

Figure 1.14.2.2.1-1 Shear Key Detail



1.14.2.3 Expansion Joint Setting

Use a minimum change of joint width due to shrinkage of ¼ inch per 100 feet for the full length of non post-tensioned concrete segments (both pretensioned and conventional).

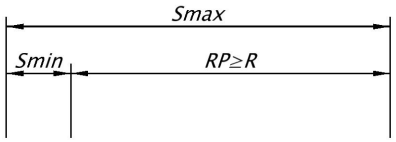
Use a minimum change of joint width due to creep and shrinkage of ½ inch per 100 feet for the contributing length of post-tensioned segments.

Use the following equations for calculating thermal effects:

Steel Girder Superstructure: $R = LF * (TR + TF)$

Concrete Superstructure: $R = LF * (TR + TF) + LF * (CR + SH)$

Figure 1.14.2.3-1 Calculation of Thermal Effects



Where:

$Smin$ = Minimum serviceable seal width

$Smax$ = Maximum serviceable seal width

R = Required seal range

RP = Provided seal range $(Smax - Smin)$

CR = Creep movement $CR = (ES) * (CF)$

SH = Shrinkage movement

TF = Temperature fall movement

TR = Temperature rise movement

ES = Elastic shortening

CF = Creep factor

LF = Load Factor from *LRFD Table 3.4.1-1* and *LRFD 3.4.1*

Use the Strength Limit State Load Factor that produces the largest movement in each load combinations.

Table 1.14.2.3-1 Creep and Shrinkage Factors

Thermal Effect	Conventional Concrete	Prestressed Concrete	P/T Concrete
Creep: Creep Factor	--	1.5	1.5
Portion of Creep to use	--	50 percent	70 percent
Shrinkage: ultimate	0.0004	0.0004	0.0004
Portion of Shrinkage to use	60 percent	60 percent	60 percent

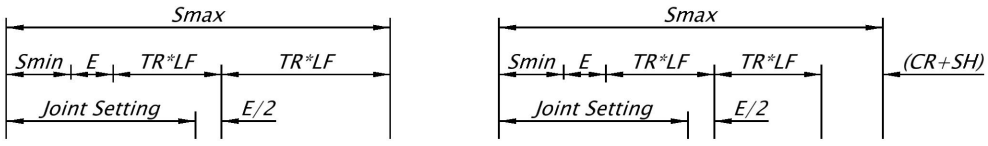
For the compression seals shown on Drawing *BR140* *Smin* and *Smax* are the width of the seal under a compressive force of 50 and 10 pounds per inch, respectively. In skewed joints, *Smin* and *Smax* may be limited by the allowable shear deformation of the seal. For the seals shown on *BR140*, always limit shear deformation of the seal to 10 degrees.

JOINT SETTING AT MEAN TEMPERATURE

In most cases, the range of serviceable seal width provided by a standard joint seal (RP) will be somewhat larger than the range required by design (R). Equally distribute this excess [$E = RP - R$] for expansion and contraction.

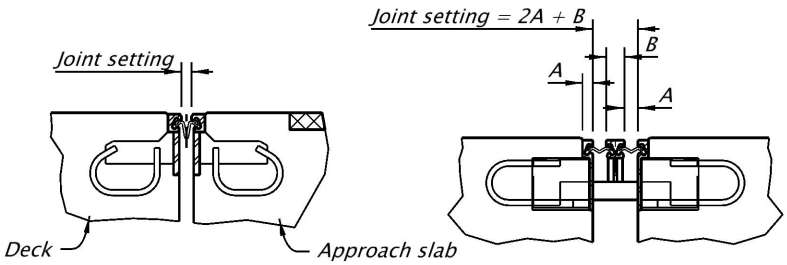
The following schematics show joint settings for the two design cases above.

Figure 1.14.2.3-2 Joint Setting Schematic



Use the form in *Figure 1.14.2.3-3* to call out joint settings on the plans.

Figure 1.14.2.3-3 Joint Setting Call-outs



Decrease Joint setting ___ inches for every 10^0F of structure temperature above ___^{0}F.

Increase joint setting ___ inches for every 10^0F of structure temperature below ___^{0}F.

Expansion joints are normally set after pretensioning is complete, so elastic shortening is not included in the joint setting width.

1.14.2.4 Details for Expansion Joints

See Standard Drawings *BR139, BR140, BR141, BR145, BR157, DET3138 and DET3150* for joint details.

For joints that span a pedestrian walkway, provide joint details according to standard drawings and details. Cover joint openings 3 inches or wider with a cover plate. Ensure that surfaces on the cover plate comply with ADA slip resistance and surface requirements.

Drawings *BR141, BR145 and DET3150* show the depth of metal to be 8 inches, with a plate being welded to the 2 inch deep rail section.

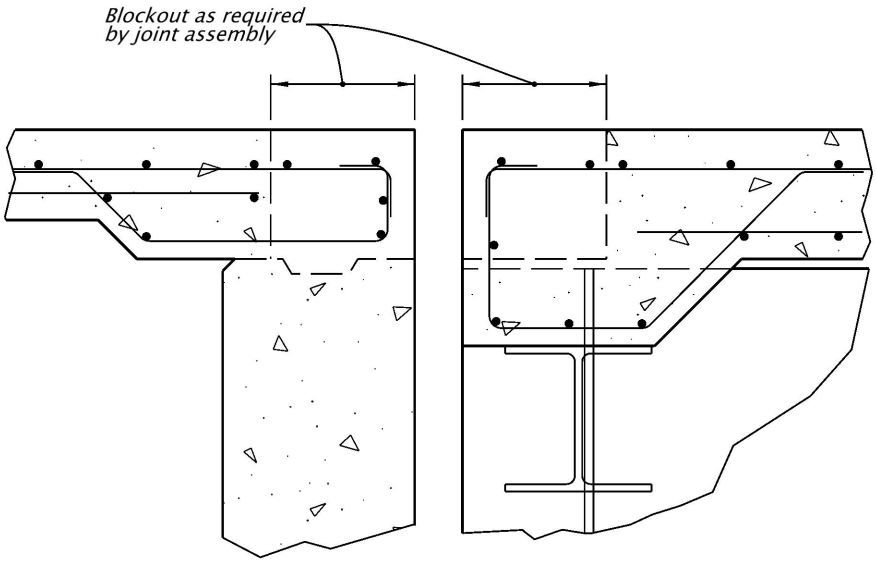
For modular joints, the bottom of the rail section must be the same depth as the bearing boxes, as noted as **Point F** on *DET3150*.

Refer to *BDM 1.9.2.1.2* for reinforcing details.

EXPANSION JOINT BLOCKOUT

Show a blockout detail on the plans to allow the expansion joint assembly to be placed a period of time after the final deck pour. Providing a blockout makes the adjacent deck pour easier, provides smoother deck transition to joint, and allows the majority of the superstructure shrinkage to occur prior to joint assembly placement.

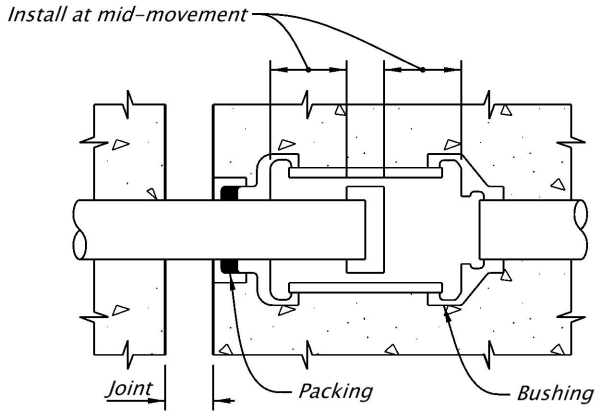
Figure 1.14.2.4-1 Expansion Joint Blockout



ELECTRICAL CONDUIT EXPANSION JOINT

At those locations on the structure where an electrical conduit crosses an expansion joint, show a detail similar to *Figure 1.14.2.4-2* on the plans.

Figure 1.14.2.4-2 Electrical Conduit Expansion Joint



1.14.2.5 Measurement and Payment of Joints

List joint types and estimated quantities of joints in *SP 00585*.

Normally, filled joints do not need bid items, because the payment for filled joints is included in payment for constructing bridge elements next to the filled joints. Hot applied joint sealant in filled joints is also included.

Control joints shown on *BR165*, which includes sawcutting and hot applied joint sealant, are paid with paving work. For rehabilitation projects, new control joints are sometime specified. When the control joint is not included in payment for paving work, *SP 00585* has an option to list the new control joints as a bid item.

Hot applied joint sealant is usually paid with paving work. When replacement of hot applied joint sealant is specified for rehabilitation projects and is not paid with paving work. Use *SP 00585* to add the hot applied joint sealant work to the list of bid items.

1.15 Repair and Strengthening

The terms **strengthening** and **repair** are sometimes used interchangeably to describe an action, but they are not the same. Strengthening is the addition of load capacity beyond the level provided for in the original design. Repair is the restoration of the load capacity to the level of the original design.

Whether repair or strengthening is called for, perform at least one site visit to the structure and identify/verify defects that may exist, to tailor solutions to problems found.

1.15.1 Strengthening of Bridges

Bridge strengthening is performed to increase capacity, such as, when a load rating factor for a bridge falls below 1.0. Design bridge strengthening to resist the live load given in *BDM 1.3.3.4*.

When material testing is performed, take concrete compressive cores from each concrete grade as indicated on the as-constructed plans, with at least three samples from each grade.

C1.15.1

See *ODOT LRFR* for additional information on load rating. For every bridge, we strive to have all traditional legal vehicles (Type 3, Type 3S2 and Type 3-3), as well as specialized hauling vehicles (SHVs) and Emergency Vehicles (EVs), with critical rating factors greater than 1.0. When any of them are less than 1.0, load posting signs are required to be installed to address the capacity issue until the bridge is shored, strengthened or replaced.

When load rating factors are below 1.0 or when the Bridge Inspection Report indicates quality issues, consider conducting material testing according to *The Manual for Bridge Evaluation Sections 5.3 and 5.4*. Additional sampling locations or tests may be required for large bridges or to address localized problems.

Material sampling of steel structures is rarely performed because of the well documented material properties, consistent material strengths and difficult to repair sampling locations. In some cases Brinell Hardness Testing may be used to increase confidence in correlating tensile properties to a known grade of steel in lieu of taking tensile test samples.

Material sampling of timber structures is discussed in the *BMDM*.

Repair all concrete sampling locations with hand patching materials from *SP 02015.20 or SP 02015.30* of the *QPL* designated for vertical and overhead application.

When a load rating factor is less than 1.0 for legal loads, a design deviation is required to support a Bridge Modernization or Retrofit category project with reduced design criteria that will not address all the rating factors less than 1.0. Consider a do nothing, or a reduced design criteria, approach only in rare cases where there are extenuating circumstances.

Legal loads are defined in *ODOT LRFR 1.5.1.2.*

Factors to be considered in the design deviation approval process may include:

- Estimated cost of repair or strengthening.
- Existing permit truck volume and potential for future increases.
- Existing girder cracking.
- Number of lanes and shoulder widths.
- Alternate routes available.
- Existing bridge detailing.

When strengthening is performed, address all longitudinal tension reinforcement deficiencies by adding reinforcement or by refined analysis.

It is not required to post bridges with longitudinal tension reinforcement deficiencies if they are in good condition without extensive cracking in the area of the deficiency. See *ODOT LRFR* for discussion on longitudinal tension.

1.15.1.1 Strengthening Methods and Details

Use of strengthening methods, not listed in the following sections, require approved design deviation (used to document use of innovative materials).

C1.15.1.1

BDM 1.15.1.1.1 thru *BDM 1.15.1.1.5* are preferred methods for strengthening girders in flexure and shear. Alternative methods are encouraged.

1.15.1.1.1 Section Enlargement

For bridge elements that require section enlargement with concrete cover less than 4 inch thick and are situated in a marine area per *BDM 1.2.4.1*, use nylon drywall anchors

C1.15.1.1.1

or stainless steel screw anchors to attain composite section.

Design interface shear reinforcement details to ensure composite section behavior between new and old concrete.

Adding depth and width to beams can increase flexural and shear capacity.

Steel dowels can be used for achieving the composite section by drilling holes into the existing concrete substrate, installing reinforcing bars or bolts, and anchoring the steel dowels with epoxy resin.

Consider high strength rebar for flexural and shear reinforcement in new concrete section, when space is limited. The high strength rebar reduces size and quantity of new rebar. See *BDM 1.5.5.15* for additional information on high strength rebar.

1.15.1.1.2 Post-Tensioning

Ensure that force due to post-tensioning is not excessive to avoid unintended cracking.

Account for any long-term relaxation unless provision for future tightening is included.

C1.15.1.1.2

Post-tensioning is an active means to restore or increase flexure, shear, and anchorage capacity of bridge structural elements.

When longitudinal post-tensioning is used as part of a strengthening system, understand that long-term relaxation of the post-tensioning system may reduce the effectiveness of the strengthening. When strengthened structural elements are located in a marine area per *BDM 1.2.4.1*, consider using corrosion-resistant high-strength bars, strands, and anchorage assemblies. Use of corrosion-resistant ducts filled with grout to encase P/T strands is also an option. See *BDM 1.5.8* for additional information.

1.15.1.1.3 Internal Shear Anchors

Limit internal shear anchor (ISA) size to ¾ inch. Do not use larger sizes unless the availability of multiple contractors has been verified.

C1.15.1.1.3

Specialty contractors are generally available for drilling 1-inch diameter holes up to 60-inches in depth. Internal shear anchors for shear strengthening can be installed either from above or below the girder. Installation from above may be easier and considered where practical.

Install internal shear anchors at a maximum 30 degree angle perpendicular to concrete surface.

Calculate the required embedment depth to develop anchor rod ultimate strength according to *ACI 318 17.4.5* with modification according to *BDM 1.5.11.2.2*. Provide adequate bar length greater than the calculated embedment length at both ends.

Calculate the shear capacity from internal shear anchors in normal weight concrete as follows:

$$V_{s_ISA} = \frac{A_s f_y d_{v_ISA}(cot\theta + cot \propto)sin \propto}{s_{ISA}}$$

where:

Effective shear depth, $d_{v_ISA} = L_{eff} sin\alpha$

Effective length, $L_{eff} = L_{anchor} - 2L_{d_ISA}$

Development length, $L_{d_ISA} = \frac{A_s f_y}{\tau_{uncr} \pi d_a}$

A_s = area of internal shear anchor (in.^{2})

f_y = minimum specified yield strength of ISA (ksi)

d_a = diameter of ISA (in^2)

τ_{uncr} = characteristic bond stress (ksi) (see *Table 1.5.11.2.2-3*)

L_{anchor} = total length of ISA (in.)

α = angle of inclination of ISA to longitudinal axis (degrees)

θ = angle of inclination of diagonal compressive stresses (degrees)

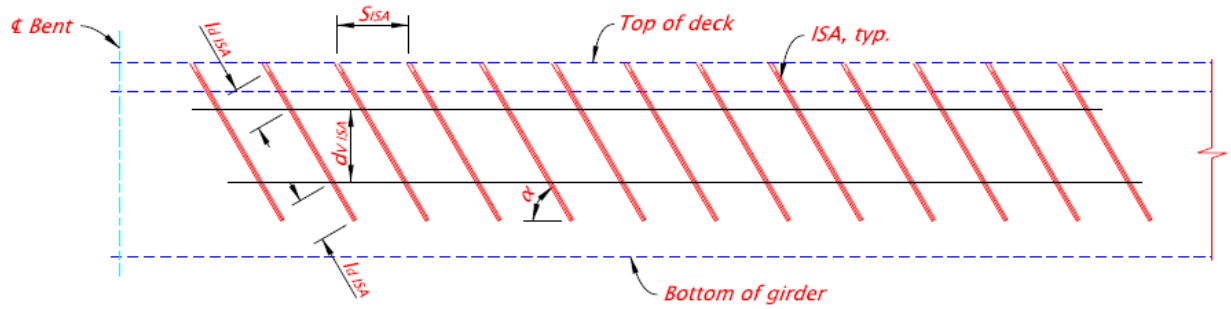
s_{ISA} = spacing of ISA (inch), minimum spacing equal to or greater than 6 x bar diameter or 6 inches.

Parameters in the above equations are shown in *Figure 1.15.1.1.3-1*.

This angle provides 96 percent of the capacity compared to 45 degree anchors and is much easier to install.

Internal shear anchors require development length at each end of the rod. The effective length of an internal anchor is the length remaining after subtracting the development length at each end of the bar.

Figure 1.15.1.1.3-1 Internal Shear Anchors



1.15.1.1.4 Bonded Strengthening Systems

C1.15.1.1.4

SURFACE MOUNTED SYSTEM

C SURFACE MOUNTED SYSTEM

Do not use a precured laminate for a strengthening system.

Design CFRP wet-layup strengthening according to *ACI 440.2R-08* or *AASHTO Guide Specifications for Design of Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements*.

Do not use CFRP wet-layup unless the critical load rating factor is greater than 0.80.

Strengthening with CFRP wet-layup can be considered a long-term (more than 20 years) strengthening solution.

Use intermittent strips with 4 inch minimum gap on girder webs for shear strengthening.

This is to allow for inspection of the bare concrete between the strips.

Do not specify more than three plies of CFRP wet-layup.

This is to avoid crack-induced debonding failure mode.

Provide positive anchorage at the ends of CFRP wet-layup. Anchorage using CFRP wet-layup transverse to the loaded direction is not acceptable.

Proper surface preparation is critical to ensure a successful CFRP application.

NEAR SURFACE MOUNTED SYSTEM

Near surface mounted (NSM) system can be used for flexure, shear, and anchorage strengthening of bridge structural elements.

Electrical isolation details are required when using CFRP.

Design NSM-CFRP system according to *ACI 440.2R-17* or *AASHTO Guide Specifications for Design of Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements, First Edition*.

C NEAR SURFACE MOUNTED SYSTEM

NSM system includes cutting shallow grooves into existing concrete substrate and embedding reinforcement surrounded by epoxy resin inside the groove. NSM is less prone to delamination, environmental degradation and allows for effective inspection of concrete surfaces compared to the CFRP wet-layup system. Consider the following reinforcement materials for near surface mounted application:

- CFRP bar.
- High strength bar.
- Stainless steel bar.
- Titanium alloy bar.

Consider using NSM system for the following bridge elements:

- Cap beams.
- Girders.
- Deck overhang.
- Bridge deck.
- Footing.

CFRP is a conductive material, that can develop galvanic corrosion, when installed in concrete elements reinforced with steel rebar.

Bond strength between CFRP bars, epoxy resin, and concrete is required to develop the tensile capacity. CFRP has linear properties up to rupture. On the other hand, metal reinforcement has a yield point with large strain before fracture and can be bent for mechanical anchorage into concrete substrate. Mechanical anchorage at terminations is added for metal reinforcement by using standard 90 degree hooks at both ends to provide anchorage.

NSM system using stainless steel and titanium alloy bars was investigated through tests for strength and long-term performance

at OSU. See *ODOT SPR 750* and *SPR 775* research reports for test information. The research studies found that the materials are suitable for strengthening bridge structural elements. However, NSM system with titanium alloy bars (NSM-TiABs) is more efficient than the system with stainless steel rebar due to higher strength.

See *AASHTO Guide for Design and Construction of Near-Surface Mounted Titanium Alloy Bars for Strengthening Concrete Structures* for design guidance and design examples with the following modifications:

ASTM B1009 describes material standards for NSM system with titanium alloy bars.

Consider smaller diameter bars for strengthening thin structural elements, since required tail length for end hooks is shorter. For instance, use #3 bars for deck overhang strengthening, where a tail length 5-inches.

- Reduce nominal cross-sectional area using a factor of 0.96 due to surface deformations to enhance bond.
- Use a specified yield strength of 130 ksi for design.
- Specify a maximum system length of 19 feet (excluding the hooks) for #5 bars or smaller and a maximum system length of 18 feet for #6 bars.

  The stock length for titanium bars is 20 feet total including the lengths of hook tails. Lengths up to 35 feet are possible, but not recommended. The additional length results in a much higher unit cost.

- Limit the use of #6 titanium alloy bars for members lightly reinforced concrete beams.

  This requirement is due to the required 12 inch hook tails embedded in the beam that can conflict with existing steel reinforcing.

Use NSM-TiABs for localized strengthening such as poor anchorage, specific deficient areas, etc.

Even though titanium alloy bars (TiAB) have the strength advantage over CFRP, stainless steel, and other high strength bars, the unit cost of titanium alloy bars is much higher than the others.

Concrete substrate for NSM system needs to be in good condition without significant cracking, spalling and delamination. For structural elements with extensive concrete damage, section enlargement with high strength rebar would be more appropriate.

When concrete cover repair is required, additional steps for strengthening would negate advantage of NSM construction.

1.15.1.1.5 Other Strengthening Systems

External stirrups (vertical rods) are not considered adequate for permanent strengthening.

Do not use bonded or bolted steel plates attached to the sides of concrete girders for shear strengthening.

C1.15.1.1.5

External stirrups can be used for temporary shear strengthening of concrete girders.

1.15.1.1.6 Strengthening Plan Details

The EOR of a bridge design project is responsible for strengthening design, except for CFRP wet-layup and CFRP-NSM systems. For CFRP strengthening systems, provide the following information on plan sheets for contractor's CFRP strengthening designers:

- Existing reinforcing details including material properties, concrete section, and relevant reinforcing details for capacity calculation.
- Locations and limits of deficiencies in structural element.
- Required total capacity from existing section and strengthening system.
- Conceptual strengthening details
- Required minimum material properties of CFRP.
- Required locations of CFRP reinforcement termination for full development.
- A reduction factor of 0.85 (ψ_f) required in the strength contribution of CFRP reinforcement.

C1.15.1.1.6

Include the following data of CFRP and composite materials used for the design of the CFRP strengthening system in General Notes sheet:

- Section properties.
- Ultimate and design tensile strength.
- Tensile modulus of elasticity.
- Ultimate strain.

When the CFRP strengthening system used in construction is different from design plans, document CFRP and composite material properties listed above in as-constructed drawings.

The material data are necessary for future evaluation and load rating of the bridge.

1.15.2 Repair of Bridges

All the strengthening methods in *BDM 1.5.1.1* can be used for repair as well.

C1.15.2

Bridge repair projects are typically limited to isolated portions of the bridge and address specific needs such as substructure issues and collision damage.

1.15.2.1 Concrete Repair

Identify known locations or bridge elements on the plan sheets that require concrete repair due to corroded reinforcing, concrete spalling, alligator cracking, surface rock pockets or construction joint rock pockets.

C1.15.2.1

Whether a structure is a few years old or a historic structure that is several decades old, the intent of a bridge rehabilitation or bridge preservation is not to rebuild bridge members but to maintain the original as built geometry and capacity. In some situations, maintaining the original aesthetic should be considered. There are situations where superstructure members will require strengthening based on deficient load rating which is covered under *BDM 1.15.1*.

Repair locations should be identified on a large scale, specifying broad areas rather than each specific location. Refer to *SP 00542* which specifies further investigation during construction. Refer to *BR 500* for details.

Alligator cracking will eventually spall and allow moisture to migrate to the reinforcing.

1.15.2.1.1 Existing Concrete Removal

Refer to *SP 00542* for existing concrete removal. The portion of *BDM 1.15.2.1.1* in single column format is commentary.

C1.15.2.1.1

The contractor will locate damaged concrete areas having visible spalling or delamination due to corrosion of reinforcement or by sounding with a 16-ounce masonry hammer for visibly loose, dislodged, or loosened concrete. The contract will verify locations of steel reinforcing with a rebar locator to determine location of steel reinforcing.

One of the steps for damaged concrete repair is sounding the concrete surface for delaminated concrete due to rebar corrosion. Concrete structures with large aggregate size can give a similar sounding to delaminated concrete. Avoid excessive removal to deep depths if there is no reinforcing in the removal work area. Some structures, primarily historic structures have had aggregate sizes as large as 6 inches. Locations can be verified with a rebar locator to determine if steel reinforcing is present.

The contractor will mark the extent of boundary repair area for saw-cutting perimeter to ½ inch depth. Saw-cutting perimeter provides a right-angle square edge to apply mortar. The contractor will saw cut boundary area in square or rectangular shape where applicable, avoid stepped type boundary shape.

Saw-cutting along the perimeter of repair area prevents the repair edge from having feathered surfaces that will have the potential to crack. Stepped shapes require more work for pumping grout and have the potential for more air voids in patch area.

The contractor will remove the damaged concrete within the marked boundaries with nominal 15-pound class pneumatic hammer, high pressure water jet blasting, chipping guns, manual picks, and chisels. Limit the deepest depth of removal to ½ inch past steel reinforcement that has apparent corrosion pitting or scaling.

The contractor should not remove concrete below rebar when there is no apparent corrosion to avoid debonding reinforcement resulting from hitting rebar with chipping hammer during concrete removal. The contractor should avoid chasing debonded rebar past designated repair boundary.

Contractor should limit removal depth of deep rock pockets to 2-inches. While the actual depth of the rock pockets may extend further into the member, do not repair beyond the initial 2-inches.

Some construction joints may have the rock pocket located around the entire section. The entire section should not be removed.

Refer to *BR 500* for details.

For rock pockets at construction joint, crack injection should be used for joints narrower than ¼ inch or shallower than 1½ inches. If crack injection cannot be used, remove and repair the area with 2-inch deep and 2-inch wide on each side of joint centerline and around perimeter of column.

The encased repair eliminates moisture infiltration to the column steel reinforcing. Excessive removal at construction joint locations due to soft material can be reduced by using a smaller weight chipping hammer or even a rock hammer. Excessive heat can occur during curing of crack injection material for joints wider than ¼ inch.

1.15.2.1.2 Surface Preparation

Refer to *SP 00542* for concrete surface preparation. The remainder of *BDM 1.15.2.1.2* is commentary.

C1.15.2.1.2

After removal of damaged concrete, the contractor will prepare the surface for repair patches and additional cover by abrasive-blast or waterblast. The contractor will prepare concrete surfaces according to *SP 00542*.

The contractor will provide removable watertight formwork for pumped repair mortar (pressure grouted) application per *SP 00542*.

Substrate concrete should be saturated for 24 hours by either filling forms full of water or packing a saturated foam or burlap material into the form. Forms should remain in place for three days after placement of repair mortar.

The 24-hour saturation (presoaking) ensures that the existing concrete surface will have a saturated surface dry condition during placement of repair mortar. Some structures, generally historic structures, tend to have a higher porosity concrete. Dry substrate concrete will draw moisture out of the repair mortar and increase the chances for cracking. Saturation duration is not temperature dependent.
Leaving forms in place for three days helps prevent cracking, regardless of strength levels being reached prior to that duration.

1.15.2.1.3 Anode Application

For zinc anode installation refer to *SP 00542*.

Embedded galvanic zinc anodes are used in reinforced concrete rehabilitation to prevent the corrosion of steel reinforcement from recurring at a current site or at a new site outside the removal boundary area (also referred to as the halo effect). Use anodes in the patch area to provide protection to the bars extending outside repair boundary in the existing contaminated concrete.

C1.15.2.1.3

See *BR 520* for details. Anodes are constructed with a sacrificial zinc core encased within a cementitious mortar shell. Incompressible material within the encasement shell allows for expansion of the zinc core. Anodes are fastened to the exposed rebar with tie wires, which causes a galvanic reaction to occur and preserve the reinforcement. Anodes are available in different sizes e.g.:

- 2.5 inch diameter x 1.1 inch height
- 1¼ inch width x 1 5/16 inch height x 4 inch length
- 1¼ inch width x 2 ¼ inch height x 4 ¼ inch length.

Do not specify galvanic anodes in repair materials with electrical resistivity greater than 15,000 ohm-cm.

Greater resistivity repair materials may be used by multiplying the spacing value with a spacing correction factor provided by anode manufacturer.

Many repair materials with polymer, fly ash, and silica fume-based cannot be used in conjunction with sacrificial anodes. Portland cement concrete is a suitable material for anodes due to its high alkalinity with PH above 10.

DESIGN FOR ANODE:

Do not place anodes at every repair location. Place anodes at locations with potential for reoccurring corrosion. Bars with no signs of corrosion do not require anodes. Position anodes around outer perimeter of repair areas and at a grid pattern.

C DESIGN FOR ANODE:

Possible locations for anode placement are as follows:

- Reinforcing that has signs of pitting or scaled corrosion and has resulted in a loss of rebar section
- RCDG joint/beam expansion joint location were the expansion joints fails and moisture will reactivate the corrosion process
- Concrete submitted to high chlorides due to environmental conditions

Bridge location will influence the areas that have reoccurring corrosion, whether it is in a marine environment or exposure to deicing materials, etc. Locations with exposure to chlorides tend to corrode faster.

Determine required anode spacing by calculating the steel density ratio (SDR). SDR is defined as the total steel reinforcing surface area (ft²) within square foot of concrete repair (regardless of depth).

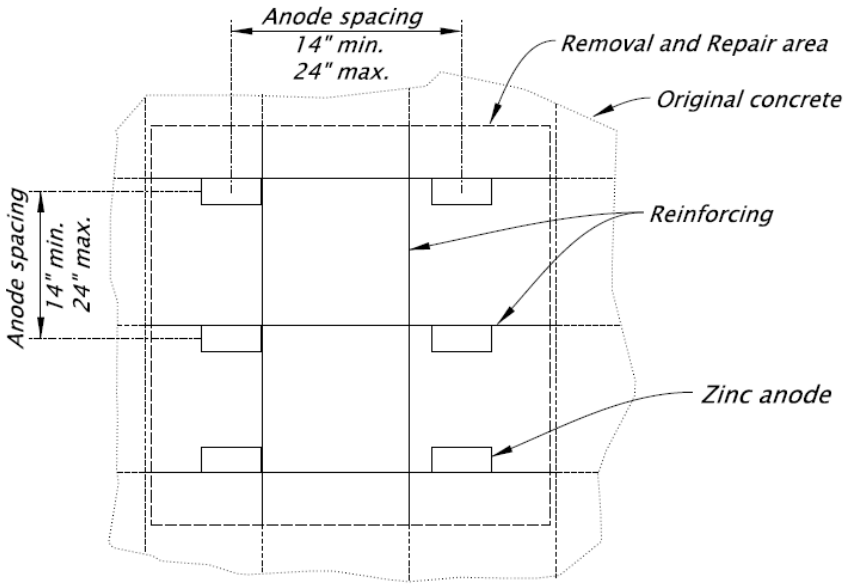
Use the steel density tables shown in *Tables 1.15.2.1.3-1* thru *1.15.2.1.3-3* to determine number of anode and spacing. The tables are based on product data from anode manufacturers listed in *SP 00542*.

When no calculations are performed, anode coverage for reinforcing without actual steel density ratio calculation can be placed in a 24-inch grid, on center. Maintain 14 inches minimum spacing and 24 inches maximum spacing (see *Figure 1.15.2.1.3-1*).

The following design examples can be used for anode puck design.

Anode puck placement calculations can be used for estimating purposes. Design is done by the manufacturer and is based largely on finding of field investigation. Approved zinc anodes are on a preapproved list in *SP 00542*.

Figure 1.15.2.1.3-1 Anode Puck General Placement



Example 1: Beam repair of stirrup

Assumptions:

- 18″ x 33″ Beam has #4 stirrup bars @ 9 inch on center.
- Stirrup legs on one side of beam has corrosion.
- Repair area is 20″ x 48″ (see *Figure 1.15.2.1.3-2*).
- Corroded bars are showing signs of pitting or scaling.

Solution:

NOTE: This example assumes continuity between all the stirrups from either a horizontal bar at top or bottom of beam. If no continuity exists between each stirrup, it will require either providing continuity by wiring bars together or wiring one anode between adjacent stirrups.

Calculate steel Density Ratio per foot of area = $\pi D_{\text{bar diameter}}$/bar spacing = π x 0.5 inch / 9 inch o.c. = 0.17 required SDR.

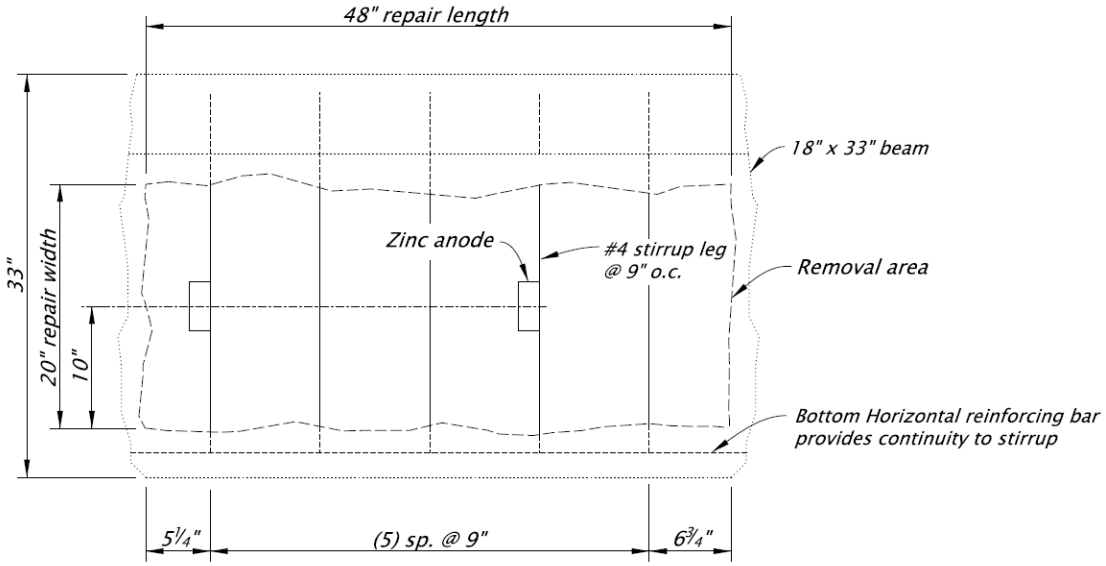
Choose a table that best describes the corrosion condition of reinforcing from *Table 1.15.2.1.3-1, 1.15.2.1.3-2, or 1.15.2.1.3-3.*

Determine anode spacing using *Table 1.15.2.1.3-2*, corroded bar, moderate corrosion.

SDR < 0.30 so maximum spacing = 28 inches.

Determine number of anodes based on maximum anode spacing: Horizontal spacing = (48 inch - 5 ¼ inch-6 ¾ inch length / 28 inch spacing) = 1.28 spaces (2 anodes) x Vertical spacing = (20 inch width / 28 inch) = 0.71 (1 anode). Requires 2 x 1 = 2 anodes.

Figure 1.15.2.1.3-2 Anode Puck Placement for Beam Repair with Stirrups (one -directional reinforcing) Elevation View



Example 2: Deck steel repair

Assumptions:

- Deck has #5 bars @ 8 inch and #5 @ 10 inch.
- Repair area is 2′-0″ x 4′-0″ (see *Figure 1.15.2.1.3-3*).
- High corrosion environment.

Solution:

Calculate steel Density Ratio / foot of area = $\Sigma(\pi D_{\text{bar diameter}}/\text{bar spacing}) = \pi \times 0.625$ inch / 8 inch o.c. + π x 0. 625 inch / 10 inch o.c. = **0.44** required SDR.

Determine anode spacing using *Table 1.15.2.1.3-3*, corroded bar, high corrosion.

SDR 0.31 – 0.60 so maximum spacing = 13 ".

Determine number of anodes based on maximum anode spacing: Horizontal spaces = (48 inch-3 ¼ inch – 4 ¾ inch length / 13 inch spacing) = 3.07 (4 anodes) x Vertical spaces = (24 inch width / 13 inch) = 1.85 (3 anodes). **Requires 4 x 3 = 12 anodes**.

Figure 1.15.2.1.3-3 Anode Puck Placement for Deck Steel Repair (two-directional reinforcing) Plan View

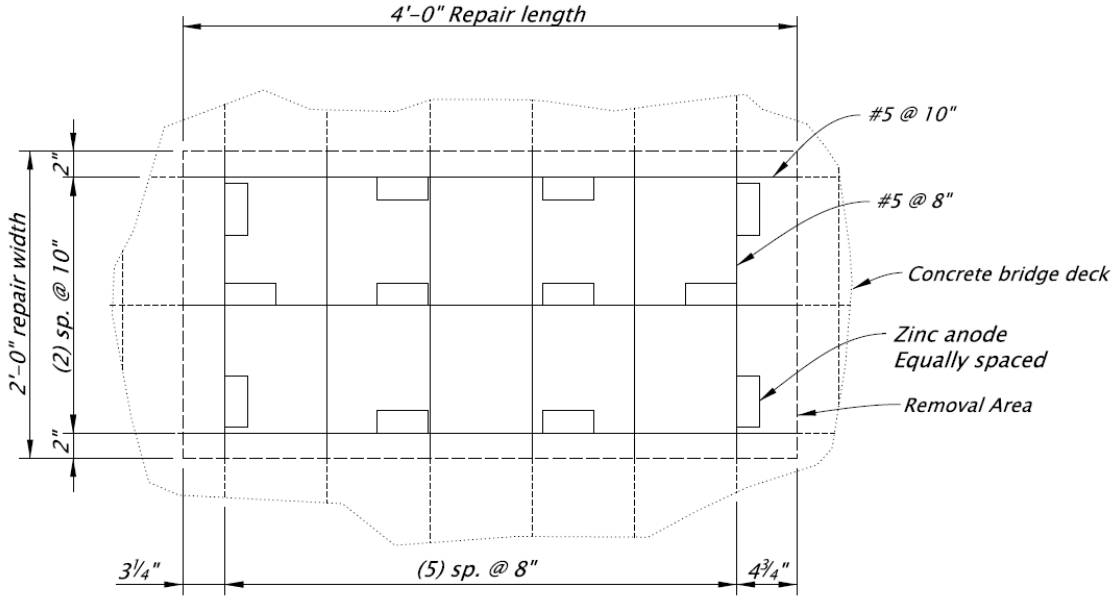


Table 1.15.2.1.3-1 Steel Density Table – Non-Corroded Bars, Low Corrosion

Steel Density Ratio	Maximum Spacing (in)
< 0.30	30
0.31 – 0.60	28
0.61 - 0.90	26
0.91 – 1.20	22
1.20 – 1.50	20
1.51 – 2.00	17

Table 1.15.2.1.3-2 Steel Density Table – Corroded Bars, Moderate Corrosion

Steel Density Ratio	Maximum Spacing (in)
< 0.30	28
0.31 – 0.60	19
0.61 - 0.90	15
0.91 – 1.20	13
1.20 – 1.50	11
1.51 – 1.80	10
1.81 – 2.10	9

Table 1.15.2.1.3-3 Steel Density Table – Corroded Bars, High Corrosion

Steel Density Ratio	Maximum Spacing (in)
< 0.30	19
0.31 – 0.60	13
0.61 - 0.90	10
0.91 – 1.20	9
1.20 – 1.50	7
1.51 – 1.80	6
1.81 – 2.10	5

1.15.2.2 Expansion Joint Repair

1.15.2.2.1 Design Considerations

Evaluate conditions of existing joints before beginning design work. Conduct a minimum of one site visit and consult the District Bridge Maintenance personnel or Region Bridge Inspectors.

1.15.2.2.2 Joint Surfaces

Limit repair work when joint surfaces are in good condition.

For deteriorated joint surfaces beyond minor repair, specify one of the following materials for joint surface replacement:

- Structural polymer concrete with a minimum compressive strength of 10 ksi, a minimum tensile strength of 1 ksi, and a minimum modulus of elasticity of 2,500 ksi
- High Early Strength Concrete (HESC) with synthetic fibers
- Ultra High Performance Concrete (UHPC)

When the joint surfaces need a re-construction to adjust joint opening, consider locating removal boundaries away from the joint opening where the splice length for new and existing reinforcing bars are adequate. Specify elastomeric nosing over the new joint surfaces when HESC is used.

C1.15.2.2

C1.15.2.2.1

It is crucial to know existing joint conditions and details before joint repair design. Often times, details shown in the as-constructed plans do not reflect actual conditions of the joint.

C1.15.2.2.2

Minor spalls can be patched. Removing and replacing existing concrete will require work including concrete removal, existing reinforcing steel retention, new steel reinforcing placement and new concrete placement.

Structural polymer concrete and HESC have a shorter cure time. When the materials are properly mixed, traffic could return after 4 hours from a placement. Even though UHPC has a longer cure time compared to structural polymer concrete and HESC, the material has superior durability along with high compressive, tensile, and bond strengths.

Do not specify elastomeric nosing material to narrow the joint opening without structural reinforced concrete support.

Elastomeric nosing material is considered non-structural.

1.15.2.2.3 Poured Joint and Compression Joint Seals

Ensure that size of the new replacement joint seal is suitable for the current joint opening and movement range. Replace poured joint and compression joint seals with either poured joint or pre-compressed foam silicone joint (PFSJ) seal. When the joint opening is too large or too small, but the movement range is still within the seal type capacity, rebuild the joint opening to fit available joint seal sizes.

C1.15.2.2.3

Compression joint seal installation requires a full-width roadway closure. Poured joint and PFSJ seals can be installed in stages and have similar service life compared to compression joint seals.

1.15.2.2.4 Strip Seals

Strip seals can accommodate large joint movement and have a longer service life than other joint types. Consider repairing strip seals instead of replacing or modifying, when possible.

For older strip seals and when replacement gland is not available, use either of the following options depending on the steel edgebeam condition:

- Significantly deteriorated condition: replace the entire system with a new strip seal.
- Acceptable condition: replace existing gland with PFSJ seal.

Specify a gland replacement full width of the roadway when the following conditions are met:

- Steel edgebeams are in acceptable condition.

C1.15.2.2.4

- Replacement glands are available.
- The joint opening and movement range are acceptable.

PFSJ seals may be specified for replacing deteriorated strip seal gland, even if steel edgebeams are still in acceptable condition.

Request approval from Bridge Maintenance District before beginning the design work.

Possible justifications for replacing PFSJ seals include a full closure is not possible, risk reduction and construction time reduction.

Some Regions have had more success replacing strip seal glands than other Regions. Gland replacement work is difficult to accomplish and can take a long time. PFSJ seals can be installed in stages, which make the replacement easier without a full closure compared to conventional full-width gland replacement work. PFSJ can accommodate large movement and large joint opening.

For steel edgebeams with minor damage, steel edgebeams can be repaired by cutting and removing the damaged piece and splicing a new section onto the existing section by welding. Ensure that the gland is removed before the welding takes place. Specify a full-width gland replacement.

It is typical to find steel edgebeam sections damaged in wheel rut lines. See *BDM 1.9.3.1.5* for information on rut repair.

1.15.2.2.5 Asphaltic Plug Joint

Specify the same joint type for Asphaltic Plug Joint (APJ) seal replacement. Include details and notes to ensure even and smooth joint surfaces on both sides of the joint opening. The condition is crucial for APJ seal to perform.

C1.15.2.2.5

APJ seal requires significant notching into the bridge deck and approach slab to install the joint seal in the first place. When a different expansion joint seal is specified, significant surface preparation is required.

1.15.2.2.6 Precompressed Foam Silicone Joint Seal

Ensure that size of the replacement seal is suitable for the current joint opening and movement range. When the joint opening is too large or too small, but the movement range is still within the seal type capacity, rebuild the joint opening to fit available joint seal sizes.

C1.15.2.2.6

1.15.2.3 Keyway Repair

(Reserved for future use)

C1.15.2.3

1.15.2.4 Epoxy Injection

Epoxy injection is not considered a strengthening method for either flexure or shear, however, it improves corrosion protection by preventing moisture intrusion.

When large size active cracks are not desirable, address the structural issue that causes the cracking using a strengthening method.

Epoxy inject cracks with widths larger than 0.016 inches, but smaller than 0.25 inches where the bridge is:

- Located in a Snow and Ice area, defined in *BDM 1.2.4.3.*
- Located in a Coastal Area, defined in *BDM 1.2.4.2.*
- Showing signs of corrosion.

Use surface seal for cracks smaller than 0.016 inches. Specify grout for cracks larger than 0.25 inches.

C1.15.2.4

Injection of cracks smaller than 0.016 inches is difficult and is only marginally effective. Most epoxy manufacturers limit the size of cracks to be smaller than 0.25 inches due to excessive heat generated during the cure process after epoxy installation. Depending on the level of stresses in concrete and steel reinforcing and deformation of structural members, new cracks could occur next to epoxy injected cracks under subsequent loading.

1.15.3 Existing Rebar and Concrete Cover Investigation

Obtain as-constructed drawings and evaluate existing bar size, location, spacing, and cover thickness during design. Ensure that strengthening system is possible to be constructed, especially the strengthening methods that require drilling or cutting into existing concrete section.

C1.15.3

Give clear instructions to the contractor concerning how potential conflicts are to be either avoided or resolved.

It is often difficult to avoid existing deck steel or existing flexural steel. Possible solutions include but are not limited to:

- Locate existing bars and measure concrete cover using high precision rebar detector or Ground Penetrating Radar (GPR) before drilling holes or groove cutting.
- Expose the top mat of reinforcement before drilling.
- Relocate drilled hole or groove to an equivalent location, when a conflict is discovered.
- Add reinforcement, when cutting existing bar cannot be avoided.

FRP COMPOSITES

(Reserved for future use)

1.15.4 Reserved

1.15.5 Inspection and Maintenance Accessibility

Provide safe access to structures for bridge inspectors and maintenance personnel. Consider the structure type and associated inspection needs to determine appropriate access elements.

Provide access facilities that meet the *Oregon Occupational Safety and Health Code Chapter 437* rules *Division 2, General occupational safety and health Subdivision D, Walking-Working Surfaces*.

When a location on a bridge structure is intended for public access, provide appropriate pedestrian or combination railing according to *BDM 1.13.3* and *1.2.10.1*.

C1.15.5

FHWA mandates that bridges be inspected every 24 months. Inspectors require access to within 3 feet of all bridge components for visual inspection. Maintenance personnel need to access members for repair.

See *BDM 1.14.1.1* for additional information on bearing replacement access.

See *BDM 1.15.5.2* for additional information on access.

1.15.5.1 Fall Protection

ODOT policy is to use—the safest access method for any given situation. Use the hierarchy described below for evaluating and placing access and fall protection systems at a site.

1. First evaluate the feasibility of eliminating the fall hazard.

C1.15.5.1

A fall protection system can consist of a fall prevention system or a fall arrest system.

Access and fall protection should be one of many factors considered when deciding a bridge layout, but should not dictate structure arrangement (e.g., span length). Although it provides advantages to those engaged in maintenance activities it is often not feasible to eliminate, through design, situations where workers will be near the kind of drops or steep slopes described in *BDM 1.15.5.1.1*.

2. Where eliminating the fall hazard is not practical, then evaluate the feasibility of fall prevention systems.	Fall prevention systems include, but are not limited to, physical barriers, rails, walkways and covers. Where installing fall prevention systems increases the likelihood of transient activity, consider ground access instead. In urban environments, a combination of hazard elimination and ground access for bearing inspection will frequently be the preferred alternative. If a fall prevention system can be installed (e.g. railing or walkway) vs using mobile access equipment, the fall prevention system is the preferred method. Temporary fall protection systems can be approved on a site-specific basis. Anchorage points may be considered as a cost-effective alternative to fall prevention systems at accessible locations for steep slopes described in *BDM 1.15.5.1.1* (e.g. partial viaducts). Ideally this would eliminate the need to install a fall prevention system such as railing, fencing, etc..
3. Where a fall prevention system is not practical, provide a personal fall arrest system (limits falls to an acceptable force and distance).	
4. Evaluate the feasibility of ground access where height from the ground to the bearings is 15 feet or less, and inspection and maintenance can be performed with a ladder.	When inspection and maintenance can be performed via ground access, evaluate the possibility of eliminating a fall protection system based on site conditions. See *BDM 1.15.5.2* for ladder access requirements. Evaluate site conditions for when fall prevention systems on abutments is not justified (e.g. aesthetics, public access, urban proximity, frequency of access). At these sites, ladder access is a good alternative. Wingwalls do not need a fall prevention

when inspection and maintenance access is not required.

1.15.5.1.1 Fall Protection Warrants

Install fall protection under the following conditions where maintenance and inspection activities will occur during the service life of the structure:

- Along a vertical face or steep slope, defined as one upon which normal footing cannot be maintained without the use of devices due to the pitch of the surface, weather conditions, or surface material.

- When the potential vertical drop at the face of wall exceeds 6 feet, and access to the top of the feature is intended solely for the use of maintenance and operations.

C1.15.5.1.1

Consider slopes 3H:1V and steeper to be steep unless more detailed information can be documented.

1.15.5.1.2 Fall Protection Treatment Selection

Acceptable fall prevention systems include pipe rail fence (*BRXXX*), chain link fencing, and roadside barrier.

Acceptable fall arrest systems include horizontal fall arrest lifelines (*BR190* & *BR191*) and anchorage systems.

C1.15.5.1.2

Fall prevention systems and fall arrest systems serve very different functions. The design loadings and inspection requirements between fall prevention and fall arrest systems are significantly different. Consider the following site factors when selecting a fall protection system:

- Location of clear zone.
- Intended access (e.g., pedestrian, maintenance only).
- Frequency of access.
- Structure type, inspection needs.

Pedestrian and roadside barrier solutions that are not listed but meet the height and design criteria requirements of *BDM 1.15.5.1.3* may be adequate for fall protection.

See *RD770* for metal handrail details. *RD770* requires modifications to be suitable to use for fall prevention.

Seek concurrence from the appropriate Region bridge contact concerning the specific fall protection system proposed. Document decisions related to fall protection use and treatment type in the DCSA.

Each Region has a different process and reporting structure for bridge maintenance activities. The goal is to contact the manager in charge of performing Region bridge maintenance activities for input and concurrence.

Provide fall protection that is aesthetically appropriate for the site.

In many cases, this may involve extending the system along the full length of the wall even though portions of the wall may have less than 6 feet of vertical drop.

1.15.5.1.3 Design Criteria for Fall Protection Systems

C1.15.5.1.3

Design fall prevention systems that meets *OSHA 1926.5029(b)* using the following criteria:

- 42 inches high, measured from the top of the working level.
- Capable of resisting 200 pound force applied at the top, from any direction.

For bridge rail height requirements related to inspection and maintenance, see *BDM 1.13.6* and *BDM 1.13.7*.

This loading is based on *OSHA 1926.502(b)(4)*.

Design fall arrest systems using the following criteria:

- Design criteria in *BR190* and *BR191*
- Or an engineered system that complies with *OSHA 1910.28* and *ANSI Z359.1* standards.
- Use galvanized or stainless steel wire rope.
- Use galvanized or stainless steel cable connections and turnbuckles having a minimum ultimate strength at least as great as the cable strength.

Consider using stainless or other corrosive resistant materials (e.g., phillystran) for wire rope and connections in marine and coastal environments. Consider the environmental conditions, service life, existing structure materials, possible theft and interaction of dissimilar metals when choosing materials.

1.15.5.2 Access

Work platforms are not required on all structures. Provide work platforms only when it is appropriate for the structure type and site layout. Consider the inspection needs to determine appropriate access elements. Contact the State Bridge Operations Engineer about structure inspection needs.

Fixed ladders more than 24 feet in height require a personal fall arrest system.

Appropriate places to consider work platforms include (but are not limited to) the following:

- Wide and high bridges where the reach of the arm of an under bridge inspection truck (UBIT) is not long enough for proper inspection and maintenance of the bridge members.
- Critical areas that require close inspection such as nonredundant steel tension members, hinges, splices, hangers, expansion joints, complex bearings, utility lines, navigation lights, and areas that require frequent maintenance.
- Vertical abutments and MSE abutments needing bearing and shear lug inspection.
- Vertical abutments and MSE abutments with integral abutments needing backwall inspection.

- Between nonredundant steel tension members.

C1.15.5.2

Work platforms are considered a fall protection system and are preferred over permanent anchors and fall arrest systems, see *BDM 1.15.5*.

Work platforms is an elevated working or walking surface. An access walkway refers to access provided at the ground level.

Consider work platforms combined with other facilities such as ladders, manholes and safety cables.

Currently ODOT has access to a UBIT with an arm reach of 62 feet. See *BDM 1.5.7.6* for additional information on concrete box girder access.

FHWA has recommended maintenance walkways between all steel girders. This is a costly item and should be reviewed on a case-by-case basis. Maintenance walkways were provided on the Santiam River

Bridge (Steel Alternate) Bridge 08123D, Drawing 47448. The detailed W5x15 walkway beams are not readily available. A W8x18 alternate is recommended, as this was substituted on the John Day River Bridge, Bridge 00108D.

The following conditions do not require work platforms:

- Elastomeric bearings.
- When inspection access is from the ground.
- When access to bearing is 6 feet or less from the ground.
- When a UBIT will be used and can access all critical areas.

In certain instances, consider a work platform over UBIT access, such as in urban areas where mobility constraints make UBIT access difficult or impractical.

1.15.5.2.1 Design Criteria for Elevated Work Access

C1.15.5.2.1

Provide work platforms that meet all minimum clearances requirements under the structure and that do not infringe or reduce minimum required waterway openings.

Provide work platforms with sufficient headroom and width for inspection personnel to carry bulky equipment between walk rails without difficulty.

Provide work platforms that meet the following width and headroom requirements:

See *BR115 Slope Paving*.

- 3 foot minimum width.
- 5 foot minimum width when the height from the ground to the bearings is 15 feet or less and ground access is expected.

Width is the available space for an inspector or maintenance worker to walk as needed for inspection and maintenance activities. The ground measurement should be taken from where a ladder will be set, generally in front of the bearing.

- When bearing access is not required, there is no minimum headroom requirement.
- 3 foot minimum headroom for girder bridges, when bearing access is required.
- 5 foot minimum headroom for bridges having a solid bottom, such as a concrete box girder, when bearing access is required.
- 4-foot minimum headroom otherwise.

Headroom is the minimum height from the walkway surface to the bottom of superstructure.

Provide access to the work platform using one of the following, and ensure appropriate security measures are installed:

- 3 foot wide walkway along the top of the wingwall. Provide fall protection when the vertical drop exceeds 6 feet.
- Cast-in-place steel U-bar ladder steps from the ground level (under the bridge) up to the walkway. In urban environments, place the first U-bar ladder step approximately 12 feet from the ground. Access to this first step will be by portable ladder. Ensure there is an adequate bench for the ladder to seat.

See ladder bench requirements below.

When a fall arrest system is used in conjunction with a work platform, provide the following:

- Attach the cable to either the backwall or cap (approximately 4 feet above the top of walkway) or to the bottom of the girders.
- Limit the cable attachment to precast prestressed girders to the center 4 inches of the bottom flange.
- Locate the cable at least 2 feet horizontal distance away from the

vertical drop.

- Design the fall arrest system using the criteria given in *BDM 1.15.5.1.3*.
- Where potential maintenance activity is anticipated locate the cable to avoid interference with potential bearing replacement and girder jacking operations.

When a handrail is used in conjunction with a work platform, design the railing to be removable in sections to facilitate maintenance work.

For bridges in urban environments, use deterrent fencing and provide locked gates at each entrance to the walkway. Design fencing to be removable in sections to facilitate maintenance work. See *BDM 1.2.8, Bridge Security Design Considerations.*

Discuss security elements with local maintenance staff to meet site specific needs.

When the height from the ground to the bearings is 15 feet or less, and ground access is expected ensure the following:

- There is a 5 foot minimum bench at the top of slope to support a portable ladder. Where such a bench is not practical, provide cast-in-place steel U-bar ladder steps.
- Provide enough ladder steps so that an inspector can get within 3 feet of any bearing.
- Ladder requirements of *OSHA 1926.1053* are met.

1.16 Construction

1.16.1 Bridge Raising

Provide enough information in the contract document to enable the construction contractor's engineer to design supporting elements for a bridge raising and stability of the structure during this operation.

See *BDM 1.3.4.2* for loading criteria related to bridge raising.

Temporarily pin concrete barriers that protect the bridge from damage from adjacent traffic. Provide at least 1 foot clearance between the barrier and the bridge or falsework elements. Do not allow traffic in the lane adjacent to the columns that **chip-in** is in progress.

When using the chip-in method take a concrete sample of each column to verify the column's concrete strength.

C1.16.1

There are two common construction procedures employed in raising a bridge: falsework or **chip-in** construction. The **chip-in** method of bridge raising removes concrete at the mid-point of each column to provide enough room to place a jack and shims. The remaining concrete is removed and the reinforcing steel severed. After the bridge deck is brought to the desired elevation, the severed reinforcing steel is spliced and the void between the two portions of the column is filled with non-shrink concrete.

1.16.2 Accelerated Bridge Construction (ABC) Guidelines

Oregon has a long history of employing ABC methods to quickly deliver bridge projects using a variety of techniques. Some were assembled or erected on temporary falsework located adjacent to an existing structure and skidded into place. This method allowed contractors to close the facilities to vehicular traffic for a relatively short time (a few days or weekend) and skid the bridge over after quickly demolishing the existing bridge at night and working through weekends. Other bridges over navigable waterways were replaced using barges to float new and whole superstructures into place (also known as switch out when an existing structure is replaced). For wider structures that can accommodate staged construction, precast concrete or concrete filled steel grid deck panels were installed using a partial closure of the roadway during off peak travel times.

A few Oregon ABC projects were designed with rapid construction in mind to limit traffic interruptions, but most were selected either based on VE proposals by contractors, incentive/disincentive provisions, or design-build contracts. Generally, the project schedules specified a relatively short window for closing or disrupting traffic operations on the facilities. The incentive/disincentive provision for each project was normally based on user delay costs as a function of AADT, detour length and other variables. Those projects have demonstrated ABC as an effective and efficient solution to alleviate congestion and long detours where conventional methods such as off-site detour, on-site detour, stage construction or slight realignment of the roadway were difficult or not feasible. They also resulted in improved public safety through a shortened work zone exposure.

1.16.2.1 ODOT encourages and supports ABC Projects

ABC methods can be defined as using prefabricated bridge elements, combining elements into systems, or moving a complete bridge span to quickly deliver a project and re-open a highway to traffic. Use of any of these methods are encouraged and supported by ODOT. A compiled list of past Oregon projects that described the ABC featured elements is provided here at the end of the section for reference.

Construction activity results in delays to the public and incurs additional financial burdens on the people who must contend with the effects. This essentially results in a temporary tax on the affected neighborhood. Because of this, consider ABC methods even when it does not result in the lowest overall construction cost. Designers are encouraged to consider traffic delay costs and other user costs associated with a project to support stronger consideration of ABC methods. The ABC AHP Decision Making Program presented in *BDM 1.16.2.3* is available to assist in developing support for ABC.

Prefabricated elements consisting of deck panels, beams or girders, bent caps, pier columns and segments have been demonstrated successfully. Systems may consist of bridge components assembled and connected together to form a major portion or complete bridge span. Bridge

movements such as incremental launching, skidding, and transport by self-propelled modular transporters (SPMT) of a partial/complete superstructure span are also found to be acceptable methods of construction. The guidance provided here will help designers and owners decide when and where ABC is appropriate as a method of project delivery. Although the Engineer of Record is responsible for the design as well as for developing a unique method of construction/movement to fulfill ABC requirements, the owner needs to be assured that quality and durability is not being compromised by the specific rapid construction technique being considered.

1.16.2.2 Contracting Methods Allowed

A contract for specifying ABC method of delivery is allowed and will continued to be allowed under the current design-bid-build specifications. A contractor may propose an alternate method of construction for approval by the EOR/owner as part of the Cost Reduction Proposal provisions in *SP 00140.70* of the Oregon Standard Specifications for Construction. The third option allowing ABC is provided under the design-build contract provisions. More discussions and guidance are provided elsewhere and will not be elaborated here.

1.16.2.3 Decision Making Framework

A successful ABC project is dependent on deciding correctly at the beginning of a project planning to assess when and where ABC would be most efficient and effective. The criteria in the flowchart, *Figure 1.16.2.3-1*, for specifying a short window of closure may make ABC delivery the method of choice.

Figure 1.16.2.3-1 Flowchart for Determining the Applicability of ABC

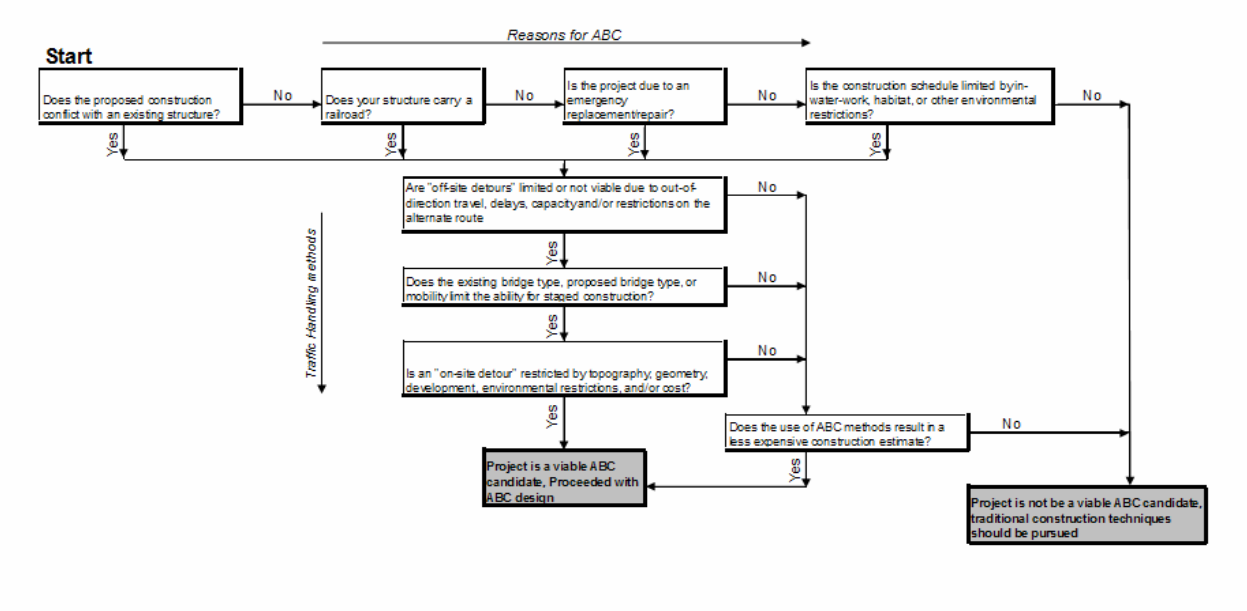


Table 1.16.2.3-1 is intended to help guide discussions when comparing ABC with conventional construction.

Table 1.16.2.3-1 Comparison of ABC with Conventional Construction Methods

ATTRIBUTES	**Accelerated BC (ABC)**	**Conventional BC (CBC)**
1. Complexity	-Engineer less familiar with design required for accelerated bridge construction techniques -May require more surveys to establish control points -May require pick points for prefabricated bridges -May require more complex construction techniques -May need new specs -May add risk to contractor -May require special equipment -Good with D/B and A+B with incentive/disincentive	-Engineer more familiar with design required for conventional construction techniques; therefore, considered less complex. - Contractors more familiar with methods used in conventional construction, therefore considered less complex -Standard specs exists
ATTRIBUTES cont.	**Accelerated BC (ABC) cont.**	**Conventional BC (CBC) cont.**
2. Schedule	-Facility to reopen for traffic in hours or over weekends -Slightly longer design schedule due to complexity (see above) -Need more overall planning and coordination -Parallel construction off CPM -Typically can be done off-line and shorter field erection season, pending ABC method chosen. -Approach or utility work may control schedule if not outside CPM -Good with incentive/disincentive -Constructible connection details for precast elements such as bent caps, footings & pile heads require flexibility for field closure pours. -May require coordinated demolition plan for change-over structures	- Typical field construction season in months or years -Typical design schedule -Often bridge work is controlling in CPM -Sequential activities typical and limitations may exist -Public delay cost may be high

	-May require tight control of scheduling on critical items -The contract plan or designed details should be simple or the precast element detail may not fit. -May require industry participation in PBES/ABC to ensure successful transition to field application. -Include contractor on design or constructability review team.	
3. Budget	-May be more expensive in construction cost due to non-typical construction methods -May increase design cost -Limited historical bid item data -ABC can significantly reduce the costs to highway users associated with traffic queues and detours during the bridge installation. -The contract bid cost for an ABC project may be more than a conventional bridge project but overall may be much less when the savings due to reduced traffic impacts and delays are factored in.	-Typical estimate given condition and conventional required structure type. -Typical standard project costs. -Incentives and disincentives may be included to accelerate construction and reduce traffic impacts but they may not be effective and could adversely impact project costs. -Careful analysis is needed to effectively apply incentive/disincentive methods to accelerate bridge projects.
4. Design Quality	-Design quality could be just as good as that of conventional -Limited design criteria for some elements -Construction loads may control design and need check -Require to show full connection details	-Design quality is expected to be good from standard and best practice.
ATTRIBUTES cont.	**Accelerated BC (ABC) cont.**	**Conventional BC (CBC) cont.**
5. Construction Quality	-Individual prefabricated elements are of higher quality under shop-controlled environment. -Construction quality could suffer in the field assembly due to time pressure.	-Construction quality depends on the contractor and inspection staff.
6. Disciplines required	-May require more upfront coordination between technical and non-technical disciplines and public relations.	-Standard project design and construction teams

7. Experience needed	-ABC experience is desirable especially regarding knowledge of ABC construction methods, new technologies and implementation of new design and details. -Additional research effort and resources may be required. -May require specialty construction experience.	-Standard project design experience. - Standard bridge construction experience.
8.Public Communications	-May require more early and upfront communication with the public for temp/short road closures -May need to develop a communication plan with stakeholders	- Typical
9. Demolition of existing structure	-Require full demolition plan -May need to provide staging place near site for off-line demolition -Coordination for change-over structures -May not require temporary structure to be in place for long duration	-Typical construction with either road closure or requires staging -Require full design of temporary structures for longer duration in place
10. Quality Control	-ABC elements should be verifiable during construction -May require constructability review	-Typical
11. Owner Staff	-Some additional effort may be expected of the owner staff in design or review of non-conventional details/procedures. Also may require more staff in a much more condensed timeframe.	-Standard

1.16.2.4 Analytic Hierarchy Process (AHP) Tool

ODOT has a tool for assisting project decision makers named *ABC AHP Decision Making Program*. This program allows the project team to analyze various applicable and weighted criteria in a paired-wise comparison. With input provided either by the designer or the project team, it captures the decision based on the controlling criteria and computed utility value for each criteria.

We encourage all project designers and leaders to take advantage of this useful tool as part of their decision making process to determine whether ABC is preferred over conventional construction. This program may be used with input provided by the bridge engineer alone if they have all the available information and feels comfortable to determine the relative importance between any two given criteria. When a project is complex and involves issues or concerns by other disciplines, it would be appropriate for the project team to provide input and

thus build consensus in their decision making process. Input can be collected with a survey form or entered directly into the program data fields either during or after the project kick-off meeting or when more information become available for them to better gauge the relative importance between any given paired criteria or sub-criteria.

1.16.2.4.1 Instructions for using the "ABC AHP Decision Making Program"

The AHP Program (in short) must be first loaded onto a personal desktop or laptop computer and must include the **dotNetFx40_Full_x86_x64.exe**. It is recommended the AHP Program be copied into a separate folder. It is assumed one is familiar through reading the manual (included in the CD folder) or attended the training. In summary, here are the logical steps to get started in running the program:

- Individual or team to establish the applicable criteria and sub-criteria for ABC decision. Refer to *Figure 1.16.2.4.1-1* and mark the ones that apply to the specific project in question. Reminder: Always save your work.
- Optional step: Use the survey form to assign the relative value for each paired-wise criteria comparison OR skip to next step.
- Run the Program by clicking on **AHPTool.exe** file. This will open the program under Tab 1 (Decision Hierarchy) and de-select the non-applicable criteria and sub-criteria determined in Step 1. User can add a new criterion or remove one from the default by using the **add child** or **remove** button on the right.
- Then click on Tab 2 (Pairwise Comparison) and enter the relative values from Step 2. Reminder: Always save your entries.
- Click on Tab 3 (Results)
- To use Tab 4, please read and follow instructions in the Manual.

1.16.2.4.2 Established Criteria and Sub-criteria for ABC decision

See *Figure 1.16.2.4.2-1*. Most transportation project decision making require some criteria that are important and specific to each site. Five main level criteria have been established and they seem to be the standard criteria used by several states for decision with ABC projects. Within each main level criterion is further defined by a sub-criterion that further expands to differentiate its elements. The definitions for each criterion are provided in *Table 1.16.2.4.2-1*.

Figure 1.16.2.4.2-1 Criteria for ABC Decisions

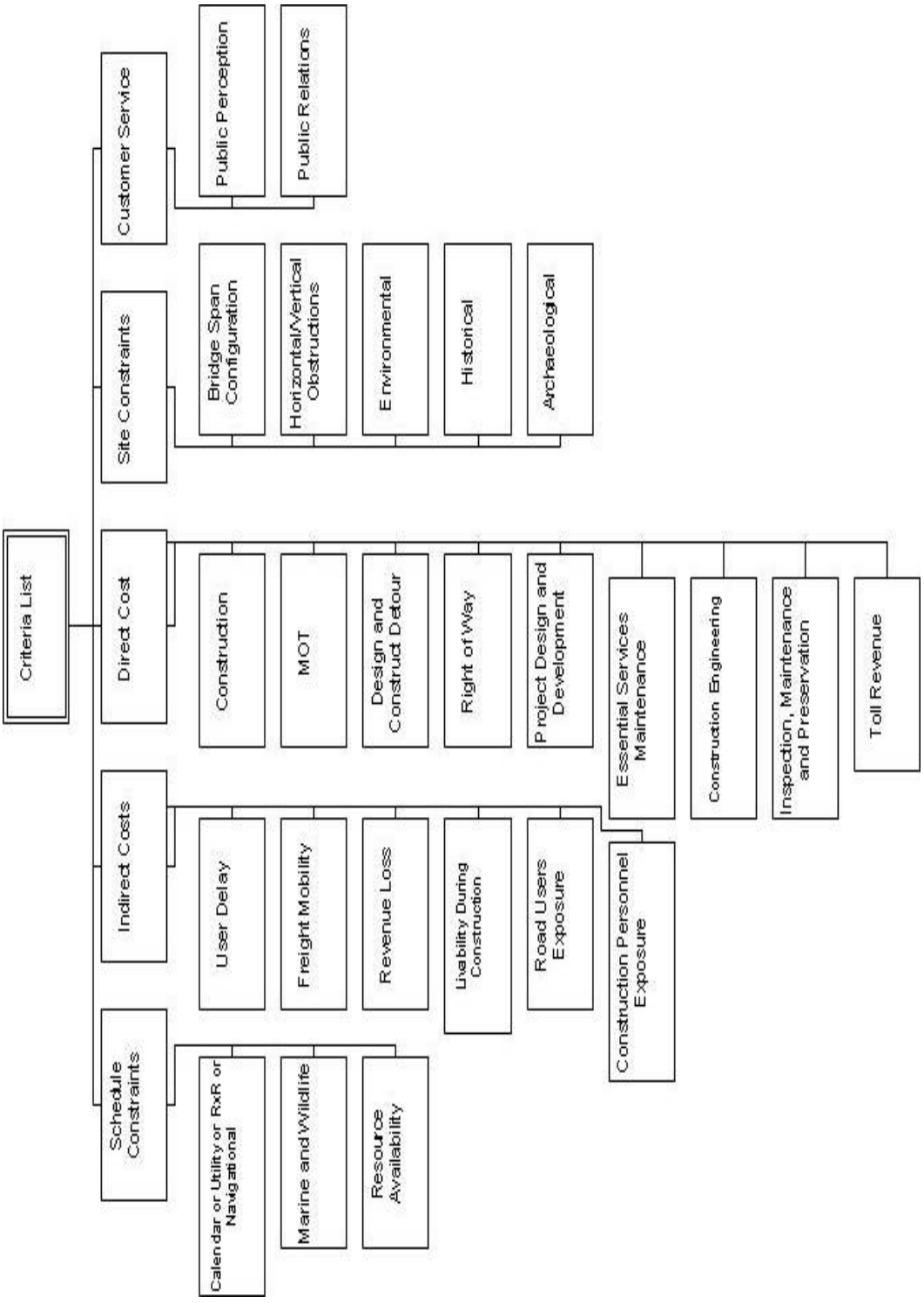


Table 1.16.2.4.2-1 Definition List of ABC Criteria

Main criteria	Sub-criteria	Definition
Direct Costs	Construction	This factor captures the estimated costs associated with the construction of the permanent structure(s) and roadway. This factor includes premiums associated with new technologies or innovative construction methods. Premiums might result from factors such as contractor availability, materials availability, and contractor risk. It may include incentive/bonus payments for early completion and other innovative contracting methods.
	Maintenance of Traffic (MOT)	This factor captures the maintenance of traffic costs at the project site. MOT costs may impact preference due to its impact on total costs. This factor includes all costs associated with the maintenance of detours before, during, and after construction. Examples of this factor include; Installation of traffic control devices, maintenance of detour during construction including flagging, shifting of traffic control devices during staged construction, restoration associated with the temporary detours upon completion of construction.
	Design and Construct Detours	This factor captures the costs to design and construct temporary structures and roadways to accommodate traffic through the project site.
	Right of Way (ROW)	This factor captures the cost to procure ROW. This factor includes either permanent or temporary procurements/easements.
	Project Design and Development	This factor captures the costs associated with the design of permanent bridge(s) and costs related to project development based on the construction method.
	Maintenance of Essential Services	This factor captures the costs associated with the need to provide essential services that may be impacted by the construction selected. Examples of this factor include alternate routes or modes of transportation to provide defense, evacuation, emergency access to hospitals, schools, fire station, and law enforcement, etc. This criterion is for situations where measures needed to be implemented beyond those already considered in the MOT and Design and Construct Detours criteria.
	Construction Engineering	This factor captures the costs associated with the owner's contract administration of the project.
	Inspection, Maintenance and Preservation	This factor captures the life cycle costs associated with the inspection, maintenance and preservation of individual bridge elements.

	Toll Revenue	This factor captures the loss of revenue due to the closure of a toll facility.
Main criteria cont.	**Sub-criteria cont.**	**Definition cont.**
Indirect Costs	User Delay	This factor captures costs of user delay at a project site due to reduced speeds and off-site detour routes.
	Freight Mobility	This factor captures costs of freight delay at a project site due to reduced speeds and off-site detour routes.
	Revenue Loss	This factor captures lost revenues due to limited access to local business resulting from limited or more difficult access stemming from the construction activity.
	Livability During Construction	This factor captures the impact to the communities resulting from construction activities. Examples include noise, air quality, and limited access.
	Road Users Exposure	This factor captures the safety risks associated with user exposure to the construction zone.
	Construction Personnel Exposure	This factor captures the safety risks associated with worker exposure to construction zone.
Schedule Constraints	Calendar or Utility or RxR or Navigational	This factor captures the constraints placed on the project that might affect the timing of construction as a result of weather windows, significant or special events, railroad, or navigational channels.
	Marine and Wildlife	This factor captures the constraints placed on the project by resource agencies to comply with marine or wildlife regulations. Examples include in-water work windows, migratory windows, and nesting requirements.
	Resource Availability	This factor captures resource constraints associated with the availability of staff to design and oversee construction. For example, a state may be required to outsource a project, which may result in additional time requirements.
Site Constraints	Bridge Span Configurations	This factor captures constraints related to bridge span configurations. This element may impact owner preference regarding bridge layout, structure type, or aesthetics.
	Horizontal/Vertical Obstructions	This factor captures physical constraints that may impact construction alternatives. Examples include bridges next to fixed objects such as tunnels, ROW limitations, sharp curves or steep grades, or other urban area structures that constrain methods or bridge locations.

	Environmental	This factor captures the constraints placed on the project by resource agencies to minimize construction impacts on natural resources including marine, wildlife, and flora.
	Historical	This factor captures historical constraints existing on a project site.
	Archaeological Constraints	This factor captures archaeological constraints existing on a project site.
Main criteria cont.	**Sub-criteria cont.**	**Definition cont.**
Customer Service	Public Perception	This factor captures both the public's opinion regarding the construction progress and their overall level of satisfaction.
	Public Relations (PR)	This factor captures the costs associated with the communication and management of PR before and during construction.

1.16.2.5 Steel Structures

Steel structures are excellent examples of prefabricated bridge elements and systems. They are proven to be cost effective for ABC applications. Steel girders are prefabricated and prime-coated as needed in the shop and shipped to the job site. A short closure of the affected highway facility may be required to erect them. Complete arches and trusses have been erected successfully on barges and floated into place.

Bridges built with plate girders (straight or curved) can accommodate precast concrete panels or steel grid deck systems for rapid construction. Sample connection details can be found at www.fhwa.dot.gov/bridge/prefab/

1.16.2.6 Concrete Structures

1.16.2.6.1 Prestressed and Precast Concrete (PPC) vs Cast-In-Place

PPC bridge elements are typically mass produced in a PCI-certified plant under factory-controlled conditions. This results in a high quality product. PCC products can be transported to the jobsite for just in time delivery, thus avoiding re-handling or the need for storage space that might be difficult to provide in urban areas. Traffic impact at the project site can be minimized and erection can normally be done during off peak hours.

In Oregon, construction cost for PPC girders is lower than CIP girders. Unless there is a compelling reason like curvature, aesthetics, and longer span requirements, PPC girders are preferred. There is economy of scale for larger projects requiring repetition of similar spans. For elements other than girders, there is opportunity to develop effective standard elements for

connecting them into bridge systems. In the erection of PPC elements, proven connection details are critical for long term performance. The FHWA has developed a manual on proven connection details found here: www.fhwa.dot.gov/bridge/prefab/

1.16.2.7 Full Depth Deck Panels, Approach Slabs or Approaches and Wingwalls

Full depth deck panels are used by many states. Connection details for both steel and concrete girders exist. A survey of details used by various states was published in 2006 as NCHRP 12-65. ODOT has constructed multiple projects using full-depth precast concrete deck panels, but has not yet settled on a standard detail. ODOT does not allow partial-depth precast deck panels. ODOT has existing standards for approach slabs/approaches and wingwalls that can be readily converted into ABC.

PRECAST APPROACH SLABS

- Consider issues regarding subgrade compaction and the contractors' ability to construct the surface of the subgrade to a smooth level condition prior to placement.
- Consider the ability of precast slabs to accommodate differential settlement (especially if subgrade is not level).
- Consider the design of the connection detail to pile cap/abutment wall and any joint construction.

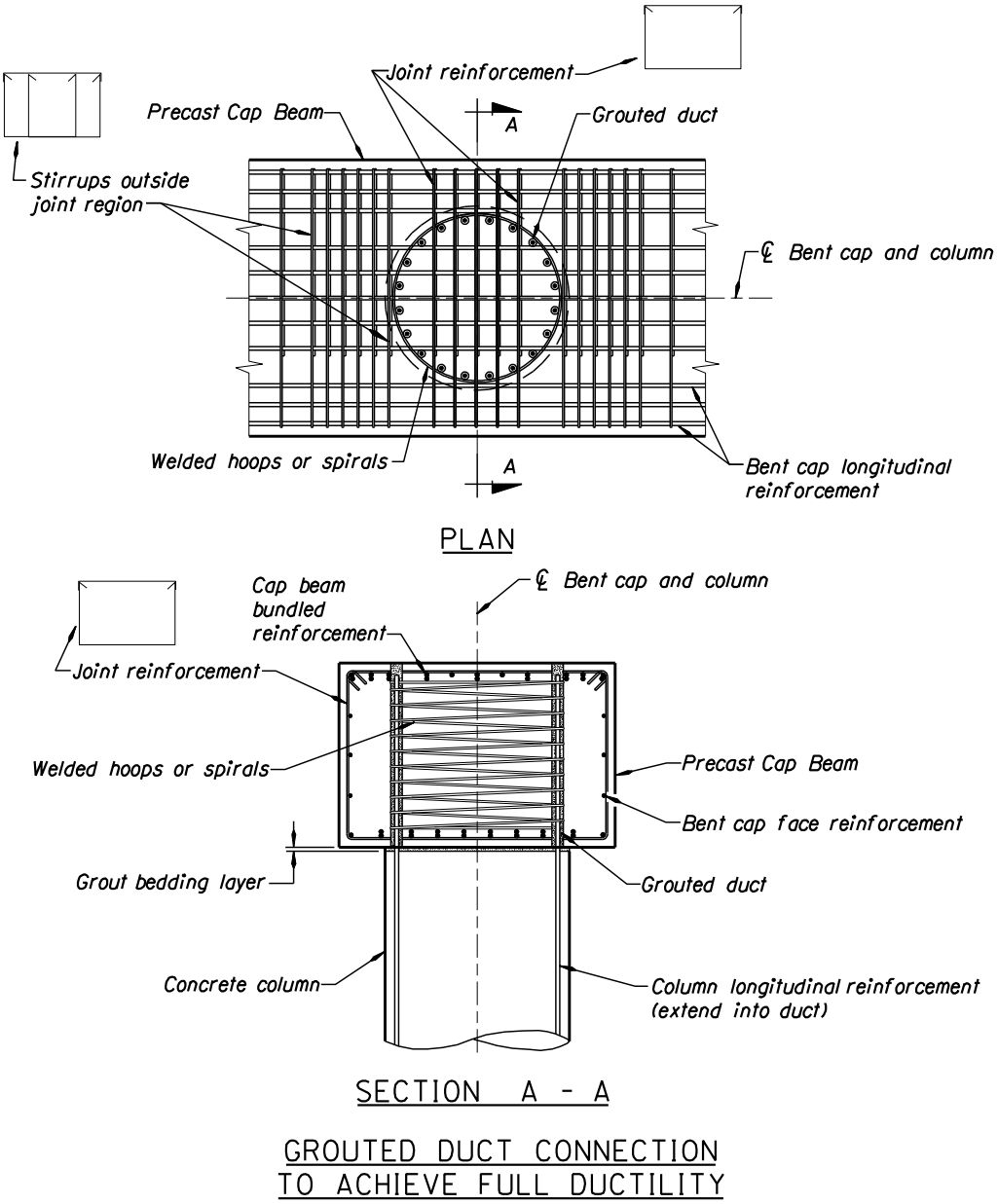
1.16.2.8 Precast Connections in Seismic Regions

When assembling prefabricated bridge elements on site, a detailed assembly plan may be needed. For single span bridges assembly is typically not complicated. For multi-span bridges, designing and detailing of connections has to be treated with the same importance as designing the rest of the structure.

Submit new connection proposals for approval to Bridge Design Manual Standards Engineer. This requirement is intended to ensure information about good connection details are subsequently distributed to other design groups. Research into new connection details has been ongoing with special concern for finding details that perform well under seismic loading. The *NCHRP 12-74* research project *Development of Precast Bent Cap Systems for Seismic Regions* identified a number of bent cap-to-column details that hold promise for seismic applications. Their conclusions were released in 2011 as *NCHRP Report 681*. ODOT is willing to implement a few of these details only for bridges in low-to-moderate seismic regions (Seismic Zones 1, 2 or 3):

- Grouted Duct – Grouted duct connections consist of bent caps which have corrugated ducts to accept reinforcement extending from supporting substructure elements.

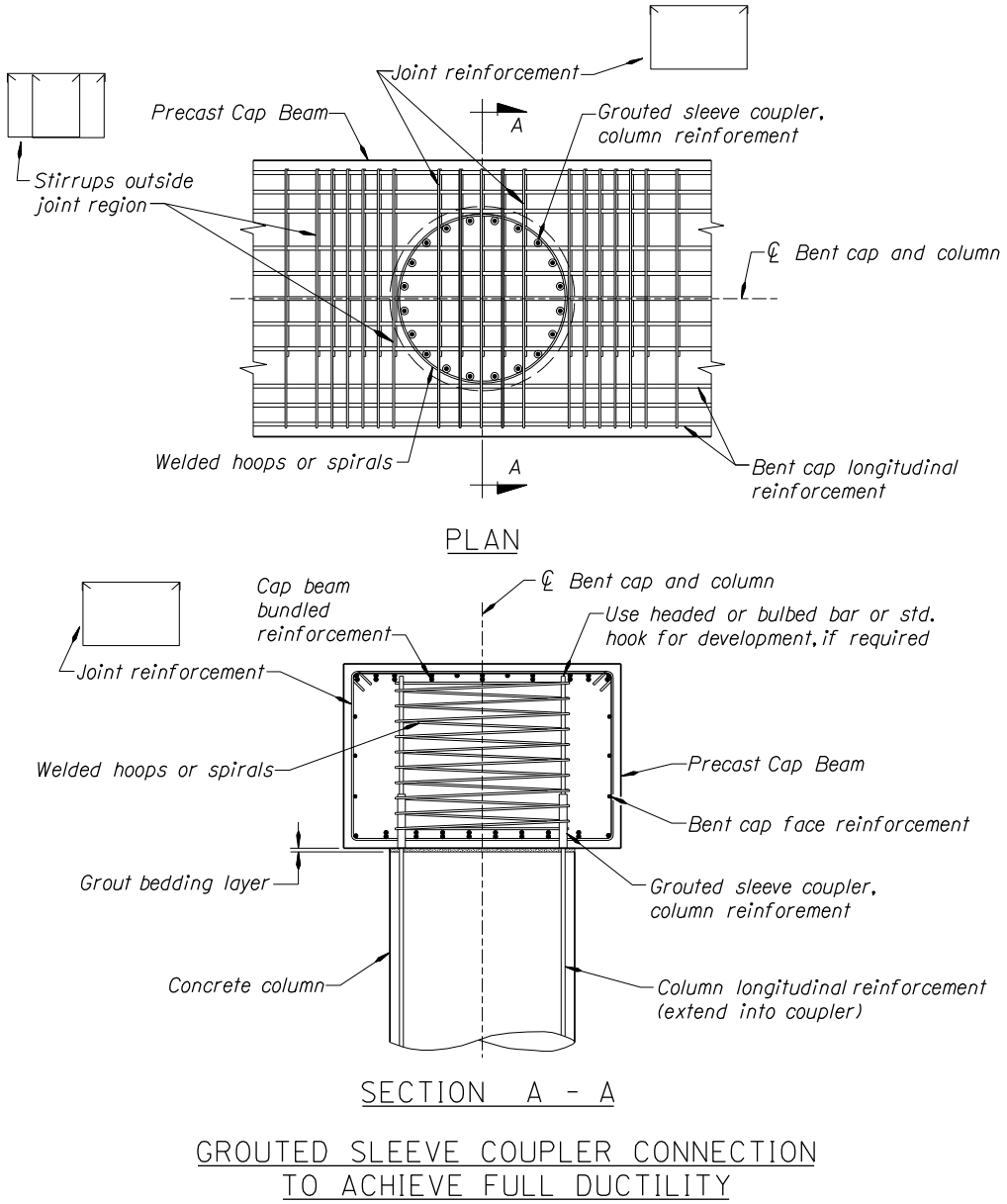
Figure 1.16.2.8-1 Grouted Duct Connection to Achieve Full Ductility



GROUTED DUCT CONNECTION
TO ACHIEVE FULL DUCTILITY

- Grouted Sleeve Coupler (Coupler in Cap) – Proprietary grouted sleeve couplers are used to connect reinforcing bars in precast concrete components. These couplers are placed in the bottom-half of the precast bent cap and are designed to withstand forces at overstrength as is often required in plastic regions.

Figure 1.16.2.8-2 Grouted Sleeve Coupler Connection to Achieve Full Ductility



1.16.2.9 Use of Self-Propelled Modular Transporters (SPMT)

SPMTs can support and move heavy loads using a flat-bed mounted on multi-axle, independent suspension and steering wheel lines. They have the ability to maneuver in difficult and uneven terrain with unmatched precision and distortion control of the payload.

SPMTs can move complete superstructure spans from a staging area (e.g. gore area or off the shoulders) to the final bridge location. A bridge move can be performed on weekends or at night using the SPMTs to erect a structure into final position within a matter of hours. FHWA has a user guide manual as a resource for anyone contemplating an ABC project using SPMTs. The guide is available free of charge at http://www.fhwa.dot.gov/bridge/pubs/07022/.

1.16.2.10 Geotechnical Considerations

Geotechnical designers need to consult closely with the bridge designer and the project team regarding the use of ABC methods at a particular bridge site. Continue to coordinate these efforts as necessary during the bridge design process.

Driven piling is normally the most rapidly constructed foundation type. However, piles are not suitable at every location. The most suitable foundation type for a bridge replacement or widening project depends on several factors including the subsurface materials and conditions, construction or environmental constraints and cost. Refer to the *ODOT GDM; Chapter 8* for additional guidance regarding the selection of foundation types. Once the most suitable foundation type is selected for a site, thought should be given to how the foundation construction can be expedited. This should include how to minimize traffic impacts due to foundation construction. At some locations the foundations (and substructure elements) may be constructed under, or away from, the existing bridge thereby avoiding, or minimizing, any traffic impacts. If this scenario is possible, the time required for foundation construction may be less significant because it does not directly affect traffic. At sites where foundation construction will directly impact traffic and multiple foundation types are possible, consideration should be given to the foundation system that can be constructed in the least amount of time and with the least impact to traffic. Some general guidance regarding the use of various foundation systems in ABC applications are described below.

1.16.2.10.1 Spread Footings

CONVENTIONAL SPREAD FOOTINGS

- Requires excavation to suitable foundation materials which may result in the need for large excavation areas or temporary shoring and possibly dewatering.
- Requires setting rebar, a concrete pour and curing time (and form work, if needed).

PRECAST REINFORCED CONCRETE SPREAD FOOTINGS

This type of ABC foundation system is currently under development. Design and construction standards and specifications do not currently exist. This type of foundation may be considered at sites where conventional spread footings would be appropriate. Precast spread footings (PSF) are currently recommended only for shorter, single span bridges at this time. Issues to consider in the application of precast spread footings would include:

- Need for construction of a concrete footing leveling slab beneath the precast footing (excavation/shoring, sloping bearing strata, presence of groundwater, etc.).
- Design of the connection between PSF and leveling slab.
- Design of the connection between the PSF and columns or abutment walls.
- Constructability issues when placing PSF directly on compacted soils.
- *LRFD* resistance factors for bearing and sliding resistance based on construction method, and settlement analysis.

1.16.2.10.2 Driven Piles

Often the quickest foundation construction method and can generally have the least impact and disruption to traffic.

- Consider using fewer, higher capacity, piles per bent to expedite construction, however:
    - Using higher capacity piles may result in significantly higher foundation costs due to the need for larger pile driving hammers, leads and cranes and possible effects on the cost of work bridges due to these higher loads.
    - Using less than five piles per bent may result in a reduced *LRFD* resistance factor due to less redundancy.
    - May be most appropriate for sites with relatively short end bearing piles.
- Requires assessment of pile top alignment tolerances for precast pile cap connection:
    - Standard specifications (*SP 00520.41(f)*) allow for a horizontal alignment tolerance of 6 inches from the plan location. If a smaller tolerance is required this reduced tolerance must be specified in the special provisions. Consult with the project geotechnical engineer regarding allowable horizontal tolerances for driven piles.
    - Should piles be installed in prebored holes to meet the specified tolerances? Keep in mind the final pile alignment is only as good as the prebore hole alignment. In soils where large cobbles or boulders are present, or where preboring will encounter a bedrock unit with a sloping surface, prebored holes should not be augered but instead excavated using core drilling equipment. Augers tend to wander uncontrollably in these materials and borehole alignment is very difficult to maintain.
    - Consider the time and cost of preboring.
    - Consider the risk of not preboring (possibly include preboring as an anticipated item).

- Minimize the potential for in-lead splices, particularly on pile with a wall thickness of greater than ½ inches such that extensive welding and welding QA/QC is not required.
- Increasing estimated lengths in variable subsurface conditions will help reduce the likelihood of an in-lead splice for pile shorter than 60 feet. For longer pile consider specifying that the pile be fabricated (spliced) on site prior to putting in the leads, taking into account the cost of using larger size leads and cranes and other concerns similar to those discussed above when using fewer high capacity pile.
- Piles can be installed in existing travel lanes, in stages under traffic control, and covered over with temporary steel cover plates to keep travel lanes open to traffic until the time for substructure construction.
- At water crossings consider a trestle pile design which eliminates the need for a cofferdam (if an above ground pile cap is permissible). Potential for drift buildup should be assessed relative to the use of a trestle pile system. A web wall may be required if drift potential is significant.

1.16.2.10.3 Drilled Shafts

- Usually takes the most time to construct. Drilled shafts are often the best method for rapid in-water foundation construction, since they may omit the need for a cofferdam (unless required for environmental considerations).
- Consider fewer, higher capacity, shafts per bent, (note that appropriate modifications to *LRFD* resistance factors are required for bents with less than two shafts).
- Higher potential for increased risk of time delays due to problems with shaft construction or negative NDT results.

1.16.2.10.4 Micropiles

- Usually more expensive than other foundation types.
- Suitable for certain ground conditions, particularly manmade unconsolidated rock fragment fills and low overhead clearance areas.
- May be installed to tight tolerances and drilled through pavement sections.
- Consider environmental concerns relative to spoils recovery since water is typically used to flush out cuttings.

1.16.2.11 Accelerated Embankment Construction

The time required for embankment construction, (either an all new roadway embankment or a widening section) depends primarily on the volume of material required, the type of embankment materials used, the level of contractor effort and the subsurface conditions at the site. Other factors such as access, retaining wall construction and weather can also play a role and affect the speed at which an embankment can be constructed. Embankment construction may be accelerated in a number of ways. In areas where very soft ground conditions exist there is potential for significant settlement and stability issues. Consideration should be given

to extending the bridge structure over these areas. This may result in a better overall design with less environmental impacts and a shorter construction period.

For ABC projects, the geotechnical engineer is responsible for evaluating the site conditions and project requirements to determine the most effective way of expediting embankment construction with the least impact to traffic flow and mobility. Refer to *ODOT GDM; Chapter 9* for more design guidance on the analysis and design of embankments. ABC projects often replace bridges in the same location (same horizontal alignment) as the existing bridge with the new bridge being wider. Therefore approach embankments also need to be widened. The grade may also be raised resulting in a further increase in embankment widening. Depending on the site constraints (available access/ROW, adjacent structures, wetlands, etc) this widening can often be accomplished with minimal traffic impacts. The geotechnical engineer plays a key role in the design of these widened sections to help determine the best approach for expediting construction while taking all appropriate geotechnical design requirements into account.

The need for retaining walls on a project should be carefully reviewed. Typically an embankment can be constructed quicker than a retaining wall. Retaining wall needs are typically driven by roadway **typical section** needs that may not have been optimized to reduce the need for retaining walls. For example, the slope immediately behind a guardrail could be steepened from the typical 1V:3H or 1V:4H to steeper slopes if longer (8 foot) guardrail posts are used rather than the typical 6 foot post lengths. Often typical fill slope rates of 1V:2H are considered in typical sections. Steeper slopes, when permitted, may omit or reduce the need for a wall. Use of stone embankment material may allow fill slopes to be constructed as steep as 1V:1.5H. If so, 8 foot metal guardrail posts may be needed to assist in penetrating the stone embankment material.

Retaining walls may be proposed in some areas to avoid, or minimize, environmental impacts. The need for walls in these areas should be closely evaluated, in consultation with the appropriate environmental specialists, to determine the underlying reasons for requiring a wall and whether or not it is the best solution for the specific location.

Some suggested considerations for embankment construction on ABC projects are summarized below:

- Use **All-Weather Materials** (stone embankment) instead of common **borrow** materials where available and appropriate. This allows construction to rapidly proceed regardless of wet weather conditions and can greatly reduce the total embankment construction time.
- Soft Ground Conditions (settlement and stability issues)
    - Lightweight fill material such as geofoam.
    - Geogrid reinforced embankments.
    - Ground improvement techniques.
    - Surcharge, with or without vertical wick drains.

1.16.2.12 QA/QC, Quality Control for Prefabricated Concrete Elements

1.16.2.12.1 Types

A variety of prefabricated concrete elements have been used on many projects. Prestressed concrete elements have been used since the 1960s. Use of non-prestressed prefabricated concrete elements dates back even earlier. The types of prefabricated concrete elements used on ODOT projects have included:

- Prestressed slabs and box beams.
- Prestressed girders.
- Prestressed columns.
- Prestressed arch ribs.
- Prestressed piles.
- Bridge railing.
- Bridge approach slabs.
- Pile caps/abutments.
- Stay-in-place deck forms.
- Culverts.
- Manholes and utility vaults.

1.16.2.12.2 Prestressed Elements

When precast concrete elements include prestressing, *SP 00550* of the standard specifications apply. *SP 00550.05* requires fabricators to be certified under the PCI Plant Certification Program. PCI certification ensures that industry best practices are followed. The member tolerances specified in *SP 00550.04* are those recommended by PCI.

For non-standard prestressed concrete elements, the existing *SP 00550* Oregon Standard Specifications for Construction will likely be adequate without modification. The designer may need to create a unique bid item since the available bid items only cover our current standards.

Verify new or modified details with local precasters (Knife River and R.B. Johnson Co.) before design plans are final. Also confirm with the Structural Materials Engineer whether standard inspection procedures are adequate.

1.16.2.12.3 Non-Prestressed Elements

Specify concrete elements that are not prestressed under *SP 00540*. Since there is not a nationally recognized certification program for non-prestressed elements, determine some minimum qualifications for fabricators. Minimum qualifications may include:

- Submission of a Quality Control Plan.
- Names and qualifications of key personnel.
- History of similar projects.
- Procedure for tracking material certifications.

The nature and complexity of the project will determine which items above should be included in the minimum contractor qualifications. Solicit input from the Structural Materials Engineer before finalizing any contract special provisions.

In addition to project qualifications, it may be desirable to require the contractor to identify the form material and forming details. Lifting and shipping details may also be required. For unique lifting and shipping situations or large elements, it may be necessary to require verification of lifting and shipping details. Such verification could be achieved with review by a professional engineer or by testing. Especially where there is potential for items to be fabricated by a contractor with little or no experience with precasting concrete, include special provision language to clarify any requirements that ensure safe and adequate lifting and transport details. In some cases, it may be desirable to add lifting and shipping verification as part of the contractor's Quality Control Plan.

Where precast concrete elements are specified under *SP 00540*, a special provision will be needed to address measurement and payment. Most structural concrete is paid on a cubic yard basis. However, precast concrete elements are typically paid either on a per length basis or per each.

Standard fabrication tolerances for structural concrete are provided in *SP 00540.40*. These tolerances are based on typical cast-in-place concrete construction. For precast elements tighter tolerances may be achievable and desirable. Consult with the Structural Materials Engineer to determine reasonable tolerances for your specific application.

Inspection of precast concrete elements is required both during the precasting operation and during placement in the field. The Structural Materials Engineer is responsible for inspection of precast elements and should be notified when precast concrete elements are to be used. This will help ensure ODOT staff is scheduled to be available for such inspections and whether any adjustment to the *ODOT Nonfield-Tested Materials Acceptance Guide* is needed.

1.16.2.12.4 Connection Issues

Current state-of-the art does not support connection of precast cap elements in high seismic locations. This is currently being researched at the national level.

Connection of precast elements may involve the use of grout pockets to emulate cast-in-place construction. Where grout pockets are used, manufacturer's recommendations should be followed regarding when grout should be extended with aggregate. For many grout products, aggregate is recommended when the pocket size reaches 2 inches or more.

1.16.2.13 Cost Considerations

It has been determined by numerous projects nationally that accelerating a project delivery will reduce the costs to highway users associated with traffic queues and detours during the bridge installation. Utah DOT has demonstrated that ABC can be successful and the initial costs of innovation are absorbed on the first few projects when there is some assurance that more projects using the same technology are being planned for the near future. The use of ABC should be justified on a specific project by analyzing the user cost savings compared to the estimated cost of various methods of rapid construction (see HYRISK discussion below).

ODOT has posted a *Work Zone Traffic Analysis Tool* that considers such topics as traffic delays and operations, and long detours. Guidance on Incentive/Disincentive Program for designers is also available.

1.16.2.13.1 Incentive/Disincentive Program

Requirements related to reduced traffic impact and time must be clearly specified in the contract documents. Innovative contracting strategies to achieve accelerated construction include incentive/disincentive, a financial bonus or penalty for delivery before or after a time set in the contract; A+B bidding, cost-plus-time based on the combination of contract bid items (A) and the time bid for construction multiplied by daily user cost (B); lane rentals, assessed rental fees for lanes taken out of service during temporary lane closures for construction; and no-excuse bonus, a modified incentive with no time adjustment for problems such as delays due to weather or utility conflicts regardless of who is responsible.

Incentives and disincentives for early completion give contractors a financial reason to change their conventional practices to accelerate construction. Contractors cannot count on incentives and, therefore, may not reduce their bid price in anticipation of receiving incentives. Disincentives are necessary but may result in higher bid prices because of the risk to contractors that they will not be able to meet the reduced construction timeline. In some accelerated bridge project case studies, it was found that by providing the right incentive/disincentive, contractors were able to lower overall total project costs when compared to conventional delivery methods.

1.16.2.13.2 Maintenance of Traffic Costs

Traffic management and user delay-related costs associated with bridge construction activities will significantly influence the selection of the most cost-effective bridge technology.

Elaborate traffic control plans can significantly add to the cost of replacement, especially when the traffic control plan changes significantly during the project due to development, local expansion, or other projects in the area. Cost savings from the reduced duration of the traffic control plan through the use of ABC method of delivery can be estimated based on the reduced number of days of traffic control cost times the average daily operating cost of such measures for comparable bridge projects.

1.16.2.13.3 Contractor's Operation Costs

In general, contractors bid projects with the plan to complete onsite construction as quickly as possible to increase profits. This is particularly true for projects with incentives for early completion. The contractor's costs, including overhead costs to staff projects, are reduced when the duration of the construction project is reduced. Also, construction crew safety in the work zone is increased with reduced exposure times.

1.16.2.13.4 Owner Agency's Operation Costs

Agency overhead costs to staff projects, e.g., construction engineering and inspection support, are reduced when the duration of construction projects is reduced. Prefabricated bridges, with their rapid onsite installation, can significantly reduce these project costs.

The use of prefabricated bridges to accelerate construction cannot be approached in a conventional manner by the owner. The owner will need to commit to working multiple shifts, odd hours, and under the same constraints as the contractor.

The manufacturers of prefabricated components may be able to offer lower unit costs if they can spread their fixed costs over many bridges or reuse formwork repeatedly. Bundling projects provides an attractive incentive for a contractor to acquire new or special equipment when he can recoup his investment on multiple applications.

1.16.2.13.5 Available Tool: HYRISK

How much will it cost highway users if a bridge is closed or detoured? The bridge with the longer detour requires additional time and mileage costs to negotiate the detour, and incurs the most cost to users. The cost of bridge construction alone fails to capture the total cost of the project.

A method that blends bridge construction cost and the users economic losses associated with a bridge construction is discussed in *Table 1.16.2.13.5-1* using HYRISK algorithm to compute the economic impact to a community.

AADT and detour length are extracted from the NBI record for the bridge. The assumed 2008 cost per distance traveled was equal to $0.44/mi ($0.27/Km). It is assumed that the project would have one year duration of the detour.

Table 1.16.2.13.5-1 HYRISK Matrix

Detour Mileage Cost (DMC) = Duration * Length Detour (L) * Cost/Length (CpL) *ADT Sample Bridge Project (Br # 00138)	
Duration of facility for construction (D) in days	365
Detour length (L) in km	26
Cost per Mile per Vehicle driven of detour length (CpL)/km	$0.27
Annual Average Daily Traffic (AADT)	330
Time cost per person (TcP)/hr	$16.31
Occupancy rate (person) per vehicle (O)	1.56
Time cost per truck (TcT)/hr	$29.50
ADTT (Truck Traffic as a percentage of AADT; i.e. 10 percent this case)	.10
Speed of Traffic on Detour (DS) in km/hr	64
Detour Mileage Cost (DMC) = D*L*CpL*ADT	$845,600
Detour Time Cost (DTC) = D*L*[(O*TcP)*(1-ADTT)+(ADTT*TcT)]	$1.265 M
Total Community Cost associated with bridge closure $T1_{cost}$=DTC+DMC	$2.11 M

1.16.2.14 Listing of bridges replaced using ABC techniques

Contact the BDM Technical Resource for *BDM 1.16.2.14* to request a project be added to the list. Plans for these projects can be found using *BDS (Bridge Data System)*.

Table 1.16.2.14-1 List of Bridges Replaced Using ABC Techniques

BDS Structure Number	Year Built	Region	Project Title	ABC Technique Used
BR22163	2014	5	Whiskey Creek, Wallowa County	PS Slabs on steel pile caps – 2- week road closure.
BR22057	2014	2	US 26 West Humbug Creek Bridge	Used precast and prefabricated elements. Precast deck panels.
BR22105	2014	5	OR 203 Branch of Ladd Creek Culvert	Inverted rigid frames with precast prestressed slabs as top panels, 30-foot spans. Used short bypass 2-lane detour.
BR00711	2013	2	US101B Lewis & Clark River Bridge	Partial ABC with CIP deck. Used some precast and prefabricated elements.
BR21548 BR21549	2012	5	US 26 Dean and Dog Creek Culverts	Inverted rigid frames with precast top slabs to form a single cell box, 16-foot span. Used centerline shoring.
BR21252	2012	5	US30 Burnt River (UPPR)	Precast PS Deck Panels with UHPC joints and connections. Steel diaphragms. FHWA Demo Project for Life Grant.
BR21439	2012	3	Hwy 1 Kane Creek Frtg Road LT (Old Stage Road)	Precast pile caps, end panels and wingwalls.
BR21493	2011	1	Sellwood Bridge, Willamette River	Slide bridge into place using "Shoo Fly" technology. SellwoodBridge.org
BR21188	2010	2	US26 Volmer Creek Bridge	All precast or prefabricated elements (staged construction).
BR21189	2010	2	US26 Johnson Creek Bridge	All precast or prefabricated elements (staged construction).
BR20584 BR20585	2008	3	OR 38 over Elk Creek Bridges near Elkton	1 steel plate, 1 Bulb-T girder bridge built on temporary falsework adjacent to the existing. Skidded on tracks during two weekend road closures.
BR20586	2008	3	OR 38 Bridge over Hardscrabble Creek, Douglas County	Bridge built adjacent to the existing and skidded into place.
BR02398	2008	5	Kimberly Bridge OR19, Grant County	Rapid replacement of 2 approach spans using precast pile caps on a long structure with 20-day full road closure.
BR01132 F	2007	3	Hwy 241 Isthmus Slough (east side)	Constructed substructure around existing bridge.
BR19273	2007	3	Depot Street Bridge over the Rogue River, Jackson County	306-foot concrete arch built adjacent to existing bridge and skidded into place. Road closed for 5 days.
BR20136	2007		Sauvie Island Bridge over Columbia River, Multnomah County	365-foot steel tied arch. SPMT used to skid and load bridge on barges and floated span into place.
	2007	1	OR 47 over Dairy Creek Overflow Bridge, Washington County	Used steel pile cap and reused salvage precast, prestressed slabs. Road closed for 14 days.
BR20238	2006	2	US 20 Bridge over Hayes Creek, Eddyville, Lincoln County	Used precast cap and slabs. Bridge constructed during a 72-hour road closure.
BR00711	2004	WSDOT Lead	Lewis & Clark Deck Replacement	SPMT used to replace superstructure 5478' L X 34" W, 34 panels. Conventional method duration was 4 years. Using ABC, done in 6 months with full road closure.
BR01660	2002	4	OR 26 Mill Creek Bridge Deck Replacement, Wasco County	3-span continuous truss with deck panels. Panels replaced sequentially with partially concrete filled exodermic steel grid deck. 540-foot deck replaced in 24 days under flexible road closure schedule.
BR07333	1997	1	I-5 (Interstate) Bridge over Columbia River, Multnomah County	Accelerated replacement of 2 trunnion assemblies and span/counterweight cables. Contractor awarded $1.4+M incentive ($100K/day) for early completion in less than 7 days; 14 days ahead of the required 21-day schedule.
BR18074	1997	5	Imnaha Bridge over Little Sheep Creek	Single span, concrete-filled grid deck over steel curved girder bridge. Built first half of new bridge and switched traffic over; demolished existing bridge and built second half with skidding to connect the two halves. Longitudinal concrete closure-pour in the middle
BR02529	1973	1	Freemont Bridge over Willamette River, Multnomah County	Arch span was floated on barges and moved into place using strands jacking.
			Sam Jones Bridge	Full depth precast deck panels.

1.16.3 Bridge Temporary Works

1.16.3.1 Design Considerations

C1.16.3.1

Temporary works include any temporary construction used to construct highway related structures but are not incorporated into the final structure.

Temporary works required for construction of permanent structures include:

- Diversion bridge.
- Work bridge.
- Falsework.
- Formwork.
- Shoring.
- Cofferdams.
- Temporary retaining structures.

See *BDM 1.3.4* for temporary loading.

The following tables include common Special Provisions and Standard Specifications.

Table 1.16.3.1-1 Special Provisions

Title	Section Number
Temporary Roadbed and Surfacing	00230
Diversion Bridges	00250
Agency Provided Temporary Bridges	00251
Temporary Work Bridges	00252
Temporary Work Access and Containment	00253
Temporary Bridge Jacking	00255
Temporary Retaining Walls	00256

Table 1.16.3.1-2 Standard Specifications

Title	Section Number
Structure Excavation and Backfill (shoring and cofferdams)	00510
Structural Concrete (Falsework)	00540
Trench Protection	00405.41(f)
Temporary Trench Plating	00405.46(c)
Jacking Pits	00406.41(b)

1.16.3.2 Diversion Bridges

Diversion bridges are bridges with a maximum service life of five years to carry traffic while an existing structure is replaced. Diversion bridges have the same requirements as that of a permanent structure, except as specified in this section.

Design diversion bridges using latest edition *AASHTO LRFD Bridge Design Specifications*. For seismic design requirements, refer to *BDM 1.17.2.1*.

1.16.3.2.1 Hydraulics Requirements

Hydraulic requirements apply to diversion structures designed by ODOT, and those designed by a contractor.

The hydraulics report provides recommendations for the diversion bridge. The data will include seasonal limitations, flow area of the structure, and minimum elevation of the diversion roadway.

Prepare a brief statement about the proposed location of the diversion. Other information about the diversion may include a discussion of maintenance needs such as monitoring for debris or scour. The diversion structure must conform to the temporary water management plan regarding fish passage.

Only use a dry season diversion during the dry season. The hydraulics report defines the start and end of the dry season. The design and check floods are based on the maximum predicted discharges for the months the diversion will be in place. Use the 2-year flood as the minimum design flood event.

An all-year diversion may be used throughout the year. The all-year diversion must pass the 5-year flood event at a minimum. Use the 10-year and 25-year check flows to determine the risk of damage when they occur during the time the diversion is in place.

The minimum road elevations for dry season and all-year diversions are the elevations at which the roadway will not overtop during the 2-year or 5-year flows, respectively. *Section 3.9* of the *ODOT Hydraulics Manual* furnishes more detailed guidance on requirements for either duration detour.

Consider and address other issues, such as maintenance needs, fish passage, navigational clearance, or other site-specific needs.

The crossing of FEMA floodways with temporary structures requires special consideration. Temporary structures must meet additional hydraulic requirements when they are in place across the floodway between November 1 and May 31.

During the design process, contact Region Tech Center staff for assistance as soon as possible when the structure is to cross a floodway during these months.

Section 3.9 of the *ODOT Hydraulics Manual* furnishes more detailed guidance on FEMA policy requirements.

1.16.3.2.2 Structural Requirements

Design all structures on public roads, temporary or permanent, to carry all anticipated loads, and forces. Temporary structures must also resist lateral loads caused by hydraulics, debris, ice, wind and other applied forces when they exist. Design diversion bridges over waterways assuming scour depths and design flood in accordance with *BDM 1.10.5.8*, the *ODOT Hydraulics Manual* and *Hydraulics Report*.

Mechanically connect members of the diversion bridge together. Design mechanical connections with a minimum capacity to resist a load in any direction, including uplift on the stringer, of not less than 500 pounds. Install all associated connections before traffic is allowed to pass beneath the span. All members at a connection need to resist the developed connection force. Design the substructure to resist all applied combined axial and lateral loads and the minimum connection design force.

Contractor designed diversion bridge will follow all required design steps as the design of permanent bridges. Provide necessary data to the contractor in *SP 00250* to accelerate design such as:

- Geotechnical report.
- Hydraulics report.
- Environmental study and limitations.
- In water work window.

Furnish information on the plans not limited to following:

- Minimum structure width, length and number of traffic lanes.
- Permit load (for permit load route).
- Minimum vertical and horizontal clearances when over crossing existing highway.
- All project specific requirements (utilities, sidewalk, railroad, etc.).

1.16.3.2.3 Agency Provided Diversion Bridge

ODOT has one-lane and two-lane diversion bridges ready to erect at different locations. Provide a drawing showing the bridge footprint and foundation drawings.

Use *SP 00251* for these diversion bridges.

C1.16.3.2.3

Contact the Bridge Operations Engineer for availability, scheduling and technical information of these bridges.

1.16.3.3 Falsework

1.16.3.3.1 General

See *BDM 1.3.4.3* for falsework loading.

Temporarily pin concrete barriers that protect falsework from damage from adjacent traffic. Provide at least 1 foot clearance between the barrier and falsework elements.

For falsework spans over roadways and railroads, mechanically connect all falsework stringers to the falsework cap or framing. Install all associated connections before traffic is allowed to pass beneath the span. See *SP 00540.41* for loading.
For falsework over navigable waterways, obtain US Coast Guard (USCG) approvals and permits, as required. Provide details, clearances, requirements and details required by the USCG.

Provide, as a minimum, the following design calculations and detailing of falsework drawings, for a falsework supported by existing columns of a structure for widening projects or maintenance work:

- Complete connection details.
- Location of resin bonded anchors with a note to locate the existing reinforcing prior to drilling holes with GPR or approved equal method.
- Provide note for the contractor to field verify the location of holes prior to connection fabrication when using resin bonded anchor rods or thru holes for bolted connections to support endplates or bracket connections.
- Loading concurrent with *SP 00540*.

C1.16.3.3

C1.16.3.3.1

Refer to *SP 00540.41* and *SP 00540.42* for additional design requirements at highway and railroad traffic openings.

Refer to Caltrans Falsework Manual for more guidance and example problems.

1.16.3.3.2 Bridge Deck Falsework

Do not drill or punch holes through the girder flanges or the web of interior girders for temporary work.

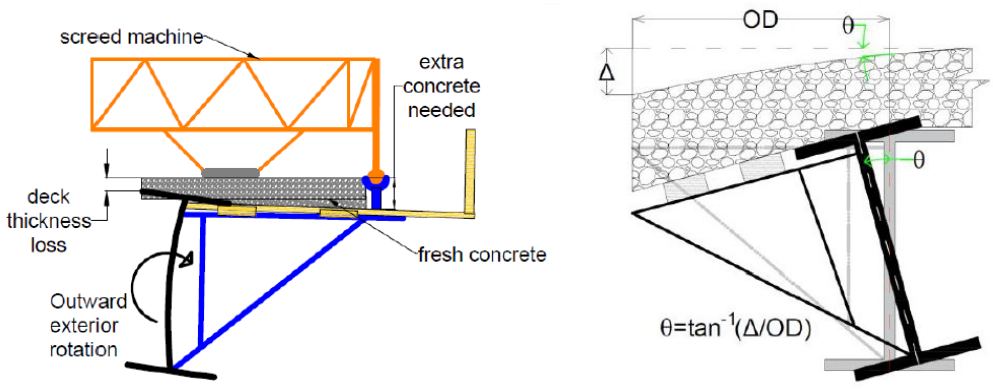
During deck placement operations there is a tendency for exterior girder to laterally deflect and rotate. When the deflection at the overhang tip (Δ) is larger than 3/16 inch, the displacement and rotation can lead to thin decks and an upward shift in bar mats which can decrease concrete cover. The tendency to laterally deflect and rotate is greater for shallow girders with large deck overhang.

C1.16.3.3.2

Deck forms for interior girders are usually set on joists hung on from top flanges or supported by post from bottom flanges. The use of embedded hangers welded to top flanges or shear studs projecting from top flanges is not recommended.

The *AASHTO Construction Handbook for Bridge Temporary Works* has design examples for cantilever deck forming for steel girders and deck overhang brackets.

Figure 1.16.3.3.2-1 Girder Rotation



Horizontal loads applied to the girder flanges by the falsework will produce a torsional moment in the girder. To prevent possible overstressing of the permanent diaphragm connections and distortion on girder web, the falsework design must include temporary struts and ties to resist the full torsional moment and to prevent appreciable relative vertical movement between the edge of deck form and the adjacent steel girder.

Include the following requirements in the specification to control the manner falsework loads are applied to the steel girder, and thus prevent undesirable distortion of the permanent structure.

The contractor is responsible to provide sufficient temporary bracing to minimize lateral deflection and rotation of exterior steel girders during deck placement. Do not assume partially or fully installed permanent bracing as shown in the plans is sufficient to minimize lateral deflection and rotation of exterior girders during deck placement.

Construct falsework and concrete forms on steel structures such that loads applied to girder webs:

- Are applied within 6 inches of a flange or stiffener.
- Do not produce local distortion of the web.
- Provide temporary struts and ties to resist lateral loads applied to girder flanges and prevent appreciable vertical movement between the edge of deck form and the adjacent steel girder.

- Temporary bracing shall not be welded to the steel girders or its attachments including the studs.

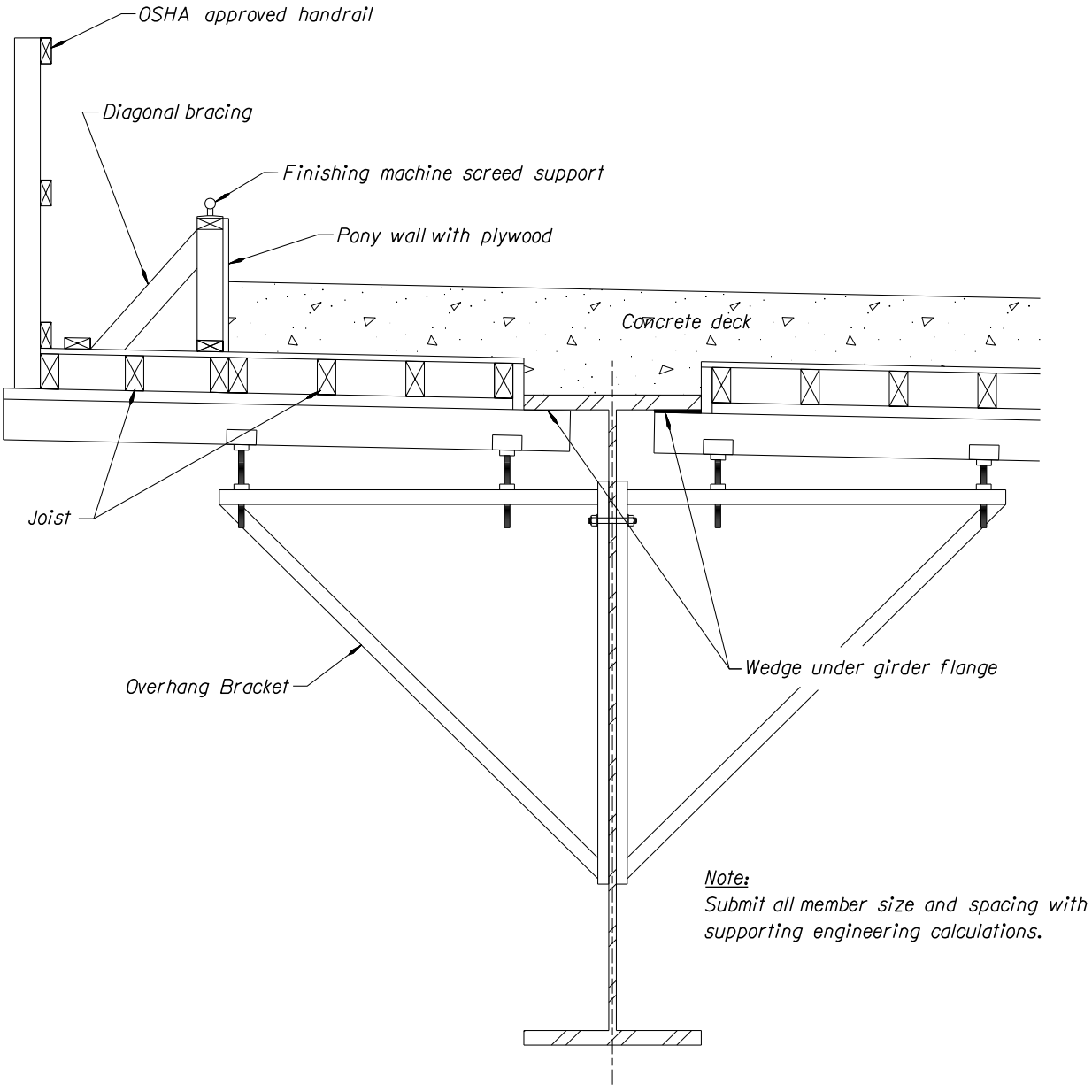
When drilled holes are used to attach the falsework brackets, note the following issues:

- Leaving bolt holes unfilled can reduce the fatigue performance of the member.
- Unfilled holes need subsequent bridge inspection.
- Moisture entrapment and acceleration of main member deterioration due to rust.

When drilled holes are used to attach the falsework brackets, the contractor is responsible to fill the hole with a high-strength *ASTM AF3125* Type 3 fully tensioned bolt, rotational capacity testing not required. The use of button-head twist-off bolts (*ASTM F1852* or *F2280*) is also allowed and may improve the aesthetics.

The contractor may provide double overhang brackets to minimize lock in stresses in exterior girders. *Figure 1.16.3.3.2-2* illustrates typical deck forming details using opposed overhang brackets attached to a steel girder.

Figure 1.16.3.3.2-2 Typical Deck Forming Detail



1.16.3.3.3 Piling

When using piling to support the falsework, the falsework plans shall specify the minimum required bearing capacity and the required depth of penetration for the piling. Provide the field method for determining the required pile bearing capacity.

Also, the falsework drawings shall show the maximum horizontal distance the top of a falsework pile may be pulled in order to position the pile under the cap. The falsework plans shall show the maximum allowable deviation of the top of the pile, in the final position, from a vertical line through the point of fixity of the pile.

The calculations shall account for pile stresses due to combined axial and flexural stress and secondary stresses. Show the stresses and deflections in load supporting members in the design calculations.

1.16.3.3.4 Spread Footings

When spread footings are used to support falsework, the falsework plans shall specify the minimum required bearing capacity, depth of embedment for the footings, and maximum allowable settlement. Design spread footings to adequately resist all imposed vertical loads and overturning moments.

Provide the soil parameters and groundwater conditions in the design calculations for the spread footings. Provide design calculations for allowable bearing capacity and settlement.

Show the estimated footing settlement under the imposed design loads on the plans. Provide provisions for addressing the effects of footing and falsework settlement.

1.16.3.3.5 Bracing

Do not attach bracing to concrete traffic barrier, guardrail posts, or guardrail.

Design all falsework bracing systems to resist the horizontal design loads in all directions with the falsework in either the loaded or unloaded condition. The falsework plans must show all bracing, connection details, specific locations of connections, and hardware used.

Thoroughly analyze falsework diagonal bracing. Give particular attention to the connections. The allowable stresses in the diagonal braces may be controlled by the joint strength or the compression stability of the diagonal.

Design cross-bracing members and connections to carry tension, as well as compression, to prevent falsework beam or stringer compression flange buckling. The falsework plans must show all components, connection details and specific locations. Install bracing, blocking, struts, and ties required for positive lateral restraint of beam flanges at right angles to the beam in plan view. When possible, set bracing in adjacent bays in the same transverse plane. However, when because of skew or other considerations, it is necessary to offset the bracing in adjacent bays, the offset distance shall not exceed twice the depth of the beam.

Provide bracing to withstand all imposed loads during erection of the falsework and all phases of construction for falsework adjacent to any roadway, sidewalk, or railroad track which is open to the public. Install all details of the falsework system that contribute to horizontal stability and resistance to impact, including the bolts in bracing, at the time each element of the falsework is erected and shall remain in place until the falsework is removed. The falsework plans shall show provisions for any supplemental bracing or methods to be used to conform to this requirement during each phase of erection and removal. Include wind loads in the design of such bracing or methods.

1.16.3.4 Shoring

For structural shoring, see *BDM 1.16.3.3 Falsework*. For the purposes of this manual, shoring is used interchangeably with falsework.

For geotechnical shoring, refer to the *ODOT Geotechnical Design Manual Section 15.3.26* for the design of temporary shoring, and defined shoring systems. Shoring is exclusive of cofferdams.

See *SP 00510.04* for plan requirements. Modify the special provision to include acceptable types of shoring as allowed. Consult with the geotechnical designer to determine safe slopes, minimum shoring lengths, and when there are unusual soil, clearance, or site conditions that may make shoring construction difficult.

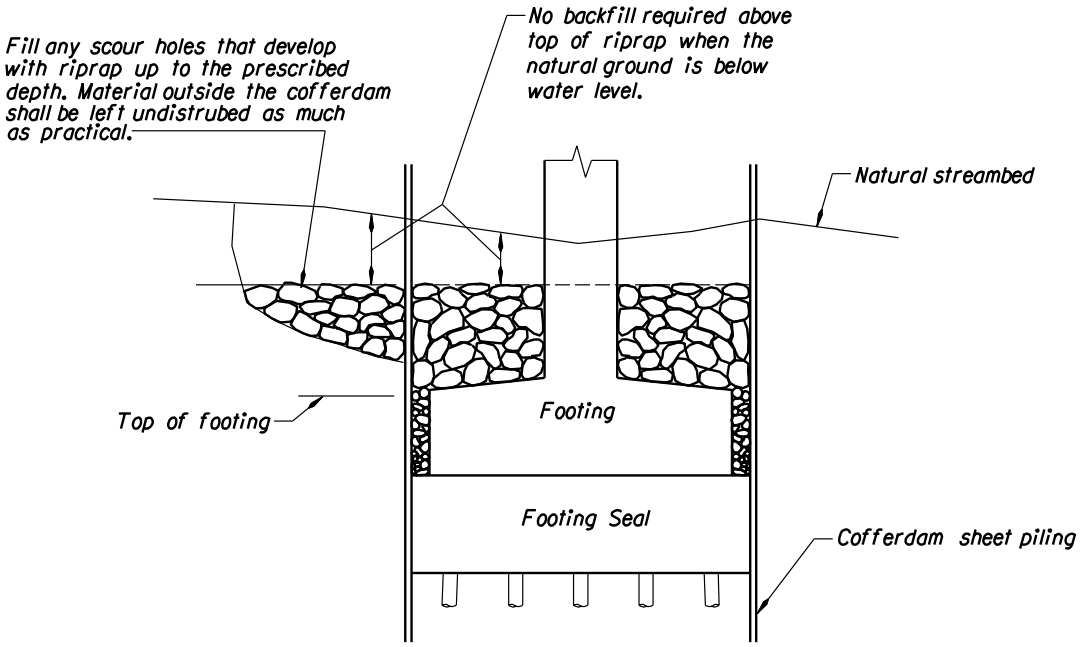
Provide shoring stationing on the project plans and list stationing in the *Special Provision*.

1.16.3.5 Cofferdams

1.16.3.5.1 Earth Pressure

When cofferdams are required and passive earth pressures are assumed in the design, show a detail similar to *Figure 1.16.3.5.1-1* on the plans. Material outside cofferdams should also be undisturbed and backfilled with riprap when disturbed.

Figure 1.16.3.5.1-1 – Typical Cofferdam Detail



1.16.3.5.2 Cofferdam Seals

Only use seals when the sheet piles cannot be driven to sufficient depth to cut off the water pressure.

The sheet piling must penetrate and form a seal in the soil so that there is no water flow under the sheet piling. In practice there will be some water entering the cofferdam. Energy is dissipated as the water flows down around the bottom of the sheet piles. Develop a flow net to determine the actual hydrostatic forces. The equipotential flow lines will show a reduction in the hydrostatic uplift forces. The hydrostatic uplift forces will be resisted by the friction between the soil and the sheet piles and the buoyant weight of the soil plug. Additionally, horizontal hydrostatic forces are present and must be designed for. These are special conditions and require detailed hydraulic and foundation studies.

SEAL DESIGN CONSIDERATIONS

The seal forms a plug at the bottom of the cofferdam. It uses a combination of seal mass and friction between the seal concrete and piling to resist the hydrostatic forces.

Scour protection for the footing influences the location (depth) of the footing and must be incorporated into the design. The Hydraulics Unit provides this information.

The top of the footing should be below the 100-year scour depth and the bottom of footing below the 500-year scour depth. The Hydraulics Unit provides these elevations.

Normally the friction or bond between the seal concrete and steel piling is assumed to be 10 psi for the surface area of the embedded pile. Check with the geotechnical designer for bond values of other pile types.

An uplift capacity of driven piling should also be obtained from the geotechnical designer to include in the overall stability or factor of safety of the system.

The minimum factor of safety of the system should be one. Note: The actual factor of safety is greater because the bond between the seal and sheet piling has been neglected.

A general rule of thumb, or good starting point, for seal thickness is 0.40 times (head of water plus an estimated seal thickness) for spread footings and 0.25 times (head of water plus an estimated seal thickness) for pile supported footings.

Use a minimum depth of seal of four feet, where piles are calculated to resist uplift in order to reduce seal depth.

Design pile footings that includes a seal for bending and shear ignoring any beneficial effects of the seal. This is due to the uncertain quality of the seal concrete and because the seal may be reduced or eliminated during construction.

There are two ways of looking at the cofferdam system when determining the seal thickness. Each should result in the same seal thickness:

- Method 1: Assume there is some leakage around the seal and the actual water level inside the cofferdam is at the top of the seal. Base the hydrostatic uplift force on the depth of water to the top of the seal, but because it is submerged the weight of the seal must be determined using the buoyant weight. There are examples of this design shown in *BDM Appendix A1.16.3.5.2.*
- Method 2: Assume the seal prevents any leakage and the hydrostatic uplift depth is to the bottom of the seal. Use the full weight of the seal to resist the uplift forces.

COFFERDAMS WITHOUT SEALS

There may be some locations and soil types where a seal may not be required for footing and column construction. The normal sequence of construction for a cofferdam without a seal includes:

Water level is the same inside and outside the cofferdam:

- Construct cofferdam - normally driven interlocking steel sheet pile.
- Cut vent holes in the sheet piling. Place vent holes at the maximum design water level elevation and allows water to enter the cofferdam. Cut a vent hole at the design elevation to prevent cofferdam failure.
- Excavate material inside the cofferdam to the bottom of the footing elevation. Excavation may also be done after dewatering, when there is no seal required, when the internal bracing is in place.
- Place internal bracing. Usually horizontal bracing consists of wales, frames, or struts to

resist the horizontal hydrostatic forces.

- Drive footing piles, when required. This may also be done after dewatering and after excavation.

Water is removed from the cofferdam:

- Install continuous pumping system. Cofferdams are never completely watertight and a sump system is normally installed to keep the cofferdam relatively dry.
- Cut piles to the specified elevation, when used.
- Construct footing and column in the dry.

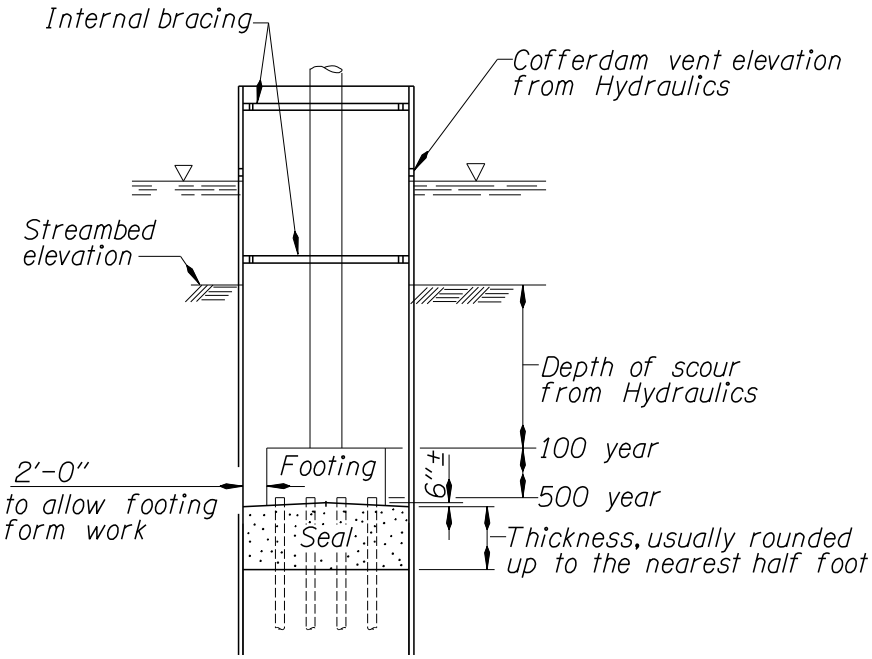
Cofferdam is flooded:

- Remove internal bracing.
- When agreed to by the environmental section, riprap is placed before or after the sheet piling is removed. It may be desirable to place riprap inside the cofferdam. Check with the geotechnical designer.
- Extract sheet piling.

COFFERDAMS WITH A SEAL

A seal is usually an unreinforced mass of concrete that seals the bottom of a cofferdam and allows construction of the footing and column inside of a dewatered or dry cofferdam. (See *Figure 1.16.3.5.2-1*)

Figure 1.16.3.5.2-1 – Seal Thickness Detail



The normal sequence of construction of a cofferdam with a seal depends on water level.

When water level is the same inside and outside the cofferdam:

- Construct cofferdam - normally driven interlocking steel sheet pile.
- Cut vent holes in the sheet piling. Place vent holes at the maximum design water level elevation and allows water to enter the cofferdam. Cut a vent hole at the design elevation to prevent cofferdam failure. The contractor may propose to use a lower vent elevation and thinner seal, when the anticipated water elevation is lower at the time of construction.
- Excavate material inside the cofferdam to the bottom of the seal elevation.
- Place internal bracing. Usually horizontal bracing consists of wales, frames, or struts to resist the horizontal hydrostatic forces.
- Drive footing piles, when required.
- Place seal concrete.
- With a tremie: A tremie is a long pipe that extends to the bottom of the seal and prevents the concrete from segregating as it passes through the water, as well as permitting a head to be maintained on the concrete during placement. The bottom of the tremie is kept submerged in the mass of concrete to minimize water intrusion into the mix.
- With a concrete pump: Similar principle to the tremie.

When water is removed from the cofferdam:

- Dewater cofferdam, only after the concrete has gained sufficient strength to resist hydrostatic loads.
- Install continuous pumping system. Cofferdams are never completely watertight and a sump system is normally installed to keep the cofferdam relatively dry.
- Cut piles to the specified elevation, when used.
- Prepare seal for footing construction. Level and clean, as needed, for constructing footing forms.
- Construct footing and column in the dry.

When cofferdam is flooded:

- Remove internal bracing.
- Place rip-rap before or after the sheet piling are removed. It may be desirable to place rip-rap inside the cofferdam. Check with the geotechnical designer.
- Extract sheet piling.

1.17 Seismic and Tsunami Design

1.17.1 Design Philosophy

1.17.2 Specification Interpretations and Modifications

1.17.3 Reserved

1.17.4 Liquefaction Evaluation and Mitigation Procedures

1.17.5 Costs

1.17.6 Instrumentation

1.17.7 Dynamic Isolators

1.17.8 Seismic Restrainer Design (New Designs And Retrofits)

1.17.9 Tsunami Design

1.17.1 Design Philosophy

Design all new bridges for full seismic loading according to the 2nd edition of *AASHTO Guide Specifications for LRFD Seismic Bridge Design* (*AASHTO Guide Specifications*) as modified herein.

Unless specifically modified in this document, each section of the *2nd edition of AASHTO Guide Specifications* must be used in its entirety.

Notify and consult ODOT Bridge Section for decisions involving deviations to the standard seismic design practices described in this manual. Submit a Design Deviation Request and obtain approval by the State Bridge Engineer for deviations from the following guidelines. Include the approved Design Deviation Request and any related files in the Calculation Book and reference the Design Deviation in the General Notes section of the Contract Plans.

At the end of the design process, complete and submit a copy of the Seismic Design/Retrofit Data Sheet to ODOT Bridge Section mailbox (ODOTBridgeEngineeringSection@odot.oregon.gov).

NON-CONVENTIONAL BRIDGES: *AASHTO Guide Specifications 3.1* states that the seismic provisions provided therein are applicable for conventional bridges. For seismic design of non-conventional bridges, develop a Project Specific Seismic Design Criteria (PSDC). Obtain approval by the State Bridge Engineer for PSDC no later than Design Verification Package submittals.

1.17.2 Specification Interpretations and Modifications

1.17.2.1 Bridge Operational Categories

For the purpose of seismic design, highway bridges are prioritized in one of the following categories based on the desired post-earthquake operational performance:

- Ordinary bridges.
- Recovery bridges.
- Critical bridges.

ORDINARY BRIDGES: Ordinary bridges are expected to experience major damage and may be closed following Upper Level Motion, but no span is expected to collapse during this motion.

All bridges not designated as Recovery or Critical are designated as Ordinary.

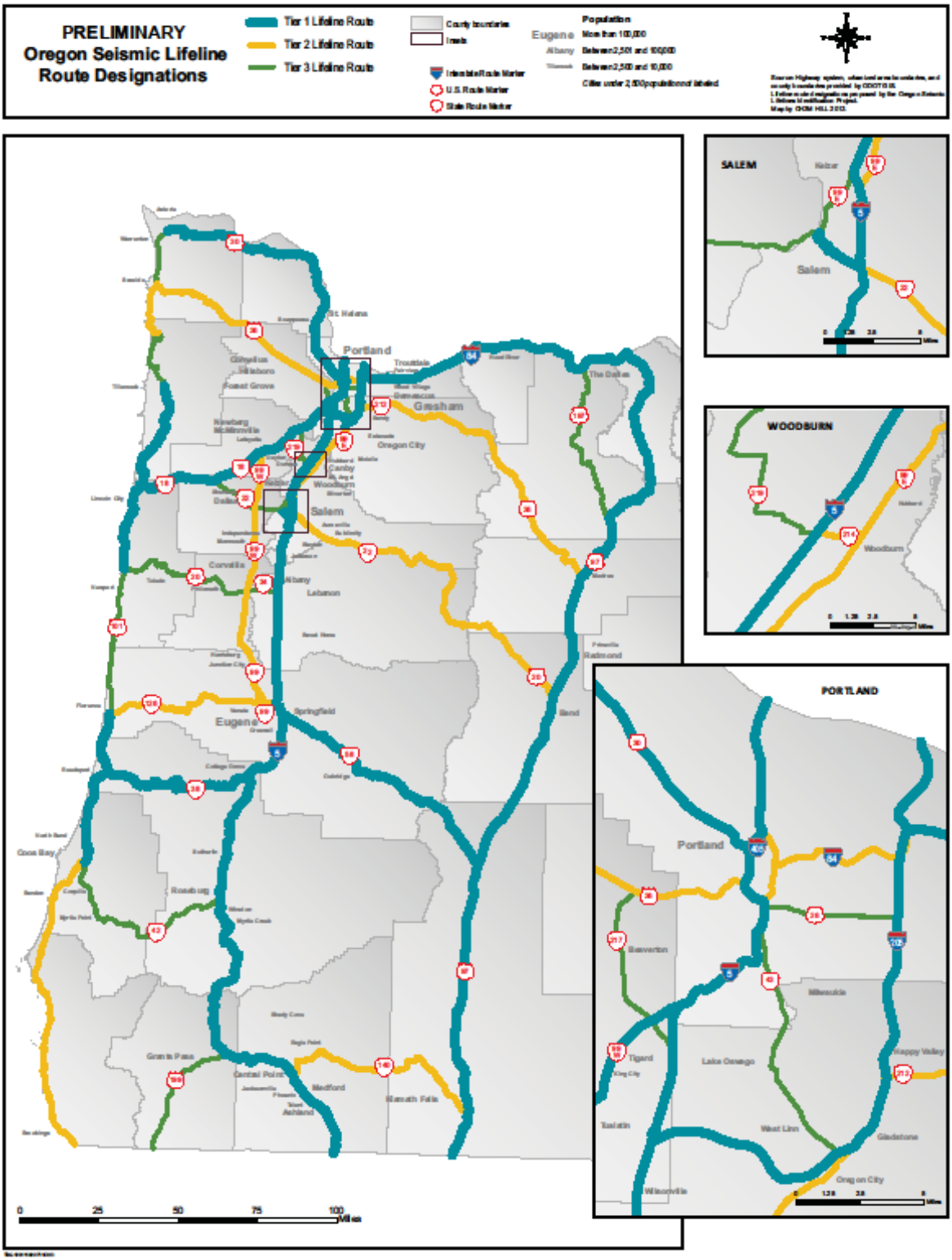
RECOVERY BRIDGES: As a minimum, recovery bridges are expected to remain open to emergency vehicles and for security/defense purposes after inspection and clearance of debris following the Ground Motion of a full rupture of Cascadia Subduction Zone Earthquake (CSZE).

Highway bridges carrying the State Seismic Lifeline Routes are designated as Recovery bridges. Refer to *Figure 1.17.2.1-1* for highway routes designated as Seismic Lifeline Routes. An interactive map of Seismic Lifeline Routes is available at ODOT TransGIS.

CRITICAL BRIDGES: Critical bridges are expected to remain open to all traffic after inspection and clearance of debris following the CSZE and Upper Level Motion. These bridges are expected to be usable by emergency vehicles and for security/defense purposes after an earthquake larger than the Upper Level Motion.

Critical bridges will be designated by the sponsoring ODOT district in consultation with Bridge Engineering Section. By 2022, ODOT has not designated any of the new bridges as Critical.

Figure 1.17.2.1-1 Seismic Lifeline Routes



1.17.2.2 Seismic Performance Levels

Seismic design and detailing must provide one of the following seismic bridge performances:

- Life Safety.
- Operational.
- Fully Operational.

LIFE SAFETY: Significant damage is sustained during an earthquake and service is significantly disrupted, but life safety is assured. The bridge may need to be replaced after a large earthquake.

OPERATIONAL: Damage sustained is minimal and full service for emergency vehicles should be available after inspection and clearance of debris. Bridge should be reparable with or without restrictions on traffic flow.

FULLY OPERATIONAL: Damage sustained is negligible and full service is available for all vehicles after inspection and clearance of debris. Any damage is repairable without interruption to traffic.

1.17.2.3 Seismic Performance Criteria

Design bridges to meet the performance requirements specified in *Table 1.17.2.3-1*.

Table 1.17.2.3-1 Performance Requirements for Oregon Bridges

Ground Motions	**Bridge Operational Categories**		
	Ordinary	**Recovery**	**Critical**
Lower Level (Full rupture CSZE)	N/A[1]	PL2 – Operational	PL3 – Fully Operational
Upper Level (1,000 yr.)	PL1 – Life Safety	PL1 – Life Safety	PL2 – Operational
[1] – Ground motions of a 1,000 year earthquake are expected to be higher than ground motions of a full rupture of Cascadia Subduction Zone Earthquake anywhere in Oregon. Contact Bridge Engineering Section if encountering an unusual circumstance.			

Refer to *Table 1.17.2.3-2* for description of the Performance Levels and earthquake damage.

Table 1.17.2.3-2 Performance Levels and Associated Damage Descriptors

Bridge Performance Levels		
PL1: Life Safety	**PL2: Operational**	**PL3: Fully Operational**
Significant damage is sustained during an earthquake and service is significantly disrupted, but the potential for collapse is minimized. The bridge may need to be replaced after a large earthquake.	Damage sustained is minimal and access for emergency vehicles is available after inspection and clearance of debris. Bridge should be reparable with or without restrictions on traffic flow.	No damage (or very minor damage that does not require immediate attention) is sustained and full service is available for all vehicles immediately after the earthquake.
Significant damage includes permanent offsets and cracking. Exposed, buckled, and possibly some fractured reinforcing steel. Repair may be possible, but will require invasive measures that may include column replacement. At a minimum, reinforcing bar segments are replaced or plastic hinge relocation techniques employed, if repair is attempted. Beams may be unseated from bearings but no span is expected to collapse. Similarly, foundations are not damaged except in the event of large lateral flows due to liquefaction, in which case inelastic deformation in piles may be evident. Undesirable failure modes such as shear failure in reinforced concrete are avoided.	*Minimal damage* includes minor inelastic response and narrow flexural cracking in concrete. Exposed reinforcing steel, but not visibly buckled. Damage requires repair using minimally invasive techniques that range from simple patching of cover concrete and epoxy injection to grouted steel jackets. Permanent deformations are not apparent and repairs can be made under nonemergency conditions with the possible exception of superstructure expansion joints which may need removal and replacement. Replacement of strip seals and other non-load-carrying elements of modular joints may be necessary, but structural elements, including blockouts, have experienced only minor damage.	*Very minor damage* consists of minor cracking of concrete, possible incipient crushing or flaking of concrete cover.

Refer to *Table 1.17.2.3-3* for Post-Earthquake Residual Displacement Limits.

Table 1.17.2.3-3 Performance Levels and Post-Earthquake Residual Displacement Limits[1]

Case No.	Engineering Design Parameters	Bridge Performance Levels		
		PL1: Life Safety	PL2: Operational	PL3: Fully Operational
1	Superstructure-to-abutment permanent vertical offset	No Limits	≤ 6 inches	≤ 1inch
2	Superstructure-to-abutment permanent horizontal offset	No Limits	≤ 6 inches	≤ 1 inch
3	Approach fill settlement limit of approach slab (longitudinal differential settlement)	≤ 1/20	≤ 1/50	≤ 1/250
4	Approach fill settlement without an approach slab	≤ 12 inches	≤ 6 inches	≤ 1 inch
5	Lateral soil movement	≤ 36 inches	≤ 12 inches	≤ 6 inches

[1] – Displacement limits provided in this table are driven mainly by geometric parameters required for the expected rideability and accessibility standards for a given performance level which, as a minimum, should provide the life safety of traveling public. Bridge type and size may not always provide the structural capacity to tolerate this magnitude of deformations and should control over these limits.

Illustration photos of engineering design parameters provided in *Table 1.17.2.3-3* are shown on *Figure 1.17.2.3-1.*

Figure 1.17.2.3-1 Illustration Cases of Engineering Design Parameters



Case 1 (Photo courtesy of Mark Yashinsky)



Case 2 (Photo courtesy of Mark Yashinsky)



Case 3 (Photo courtesy of Caltrans)



Case 5 (Photo courtesy of Mark Yashinsky)



Case 4 (Photo courtesy of Mark Yashinsky)

POST-EARTHQUAKE STRUCTURAL CAPACITY CHECK

For PL2 and PL3, when significant permanent displacements are identified by the geotechnical engineer, perform bridge performance evaluation in the post-earthquake condition during the design phase of the project. Ensure the structural capacity of all affected bridge members. Use the following load combinations and load factors:

- PL2: Strength II Load Combination using Oregon Legal Loads as defined in *ODOT LRFR 1.5.1.2* and EV3 with IM = 25 percent.
- PL3: Strength I Load Combination.

To satisfy the "Life Safety" criteria, comply with the following requirements and guidelines:

Seismic Design Categories (SDC) A, B and C

- Meet all design requirements for SDC A, B and C according to the *AASHTO Guide Specifications*.

Seismic Design Category (SDC) D

- Meet all design requirements for SDC D according to the *AASHTO Guide Specifications*, except as modified below:
  - The maximum concrete strain in confined section of the columns (ε_{cc}) does not exceed 90 percent of the ultimate concrete strain (ε_{cu}), computed by Mander's model.
  - The maximum strain of reinforcing steel does not exceed the ε^{R}_{su} as defined on *Table 8.4.2-1* of the *AASHTO Guide Specifications*.
  - The maximum strain of prestressing steel does not exceed $\varepsilon^{R}_{ps,u} = 0.03$

The above guidelines are applicable even for the other Seismic Design Categories, if Pushover Analysis will be used instead of the implicit equation.

To satisfy the "Operational" criteria, comply with the following requirements and guidelines:

Seismic Design Categories (SDC) A, B, C and D

- Verify the "Operational" performance for Cascadia Subduction Zone Earthquake when potentially liquefiable soils are present on site.

Seismic Design Categories (SDC) A and B

- No structural analysis is required for "Operational" criteria.

Seismic Design Category (SDC) C

- Satisfy *equation 4.8-1* of the *AASHTO Guide Specifications* ($\Delta^{L}_{D} < \Delta^{L}_{C}$) for each bridge bent, where Δ^{L}_{C} is determined from the *equation 4.8.1.1* of the *AASHTO Guide Specifications* (displacement capacity for SDC B).

Seismic Design Category (SDC) D

- Meet all design requirements for SDC D according to the *AASHTO Guide Specifications*,

except as modified below:

- Ensure the maximum concrete strain in confined section of the columns does not exceed $\varepsilon_{cc} = 0.005$.
- Ensure the maximum strain of reinforcing steel does not exceed $2*\varepsilon_{sh}$, where ε_{sh} is defined on *Table 8.4.2-1* of the *AASHTO Guide Specifications*.

Ensure the maximum strain of prestressing steel (for 270 ksi strands) does not exceed $\varepsilon_{ps,EE} = 0.0086$

1.17.2.4 Seismic Ground Motions

The Structural Engineer/Designer should rely on the project geotechnical designer to provide the seismic hazards, ground motions, deformations and additional permanent loads.

Determine the spectral response parameters for the Upper Level Ground Motions using the USGS 2014 Seismic Hazard Maps with Seven Percent Probability of Exceedance in 75 years (1000-year Return Period). The probabilistic hazard maps for an average return period of 1000-year and 500-year are available at ODOT Bridge Section website, but not available on USGS website.

To aid in providing consistency and efficiency, Bridge Section has developed an excel application for constructing the probabilistic response spectrum using the general procedure (three-point curve). Latitude, Longitude, and Site Class are the only required input. Version 2014.16 of this excel application has been released to incorporate the updated Site Coefficients associated with the 2014 hazard maps and can be obtained at the following link:

https://www.oregon.gov/ODOT/Bridge/Pages/seismic.aspx

Replace *AASHTO Guide Specifications Table 3.4.2.3-1* with *Table 1.17.2.4-1* and *Table 1.17.2.4-2*.

Table 1.17.2.4-1 Values of Site Factor, Fpga, at Zero-Period on Acceleration Spectrum

Site Class	Mapped Peak Ground Acceleration Coefficient (PGA)1					
	PGA ≤ 0.1	PGA = 0.2	PGA = 0.3	PGA = 0.4	PGA = 0.5	PGA ≥ 0.6
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.2	1.2	1.2	1.2	1.2
D	1.6	1.4	1.3	1.2	1.1	1.1
E	2.4	1.9	1.6	1.4	1.2	1.1
F2	*2	*2	*2	*2	*2	*2

Table 1.17.2.4-2 Values of Site Factor, Fa, for Short-Period Range of Acceleration Spectrum

Site Class	Mapped Spectral Acceleration Coefficient at Period 0.2 sec (S_S)[1]					
	$S_S \leq 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S = 1.25$	$S_S \geq 1.5$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.3	1.2	1.2	1.2	1.2
D	1.6	1.4	1.2	1.1	1.0	1.0
E3	2.4	1.7	1.3	*3	*3	*3
F[2]	*2	*2	*2	*2	*2	*2

Replace *AASHTO Guide Specifications Table 3.4.2.3-2* with *Table 1.17.2.4-3*.

Table 1.17.2.4-3 Values of Site Factor, F_v, for Long-Period Range of Acceleration Spectrum

Site Class	Mapped Spectral Response Acceleration Coefficient at Period 1.0 sec (S_1)[1]					
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 = 0.5$	$S_1 \geq 0.6$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.8	0.8	0.8	0.8	0.8	0.8
C	1.5	1.5	1.5	1.5	1.5	1.4
D3	2.4	2.23	2.03	1.93	1.83	1.73
E3	4.2	3.33	2.83	2.43	2.23	2.03
F2	*2	*2	*2	*2	*2	*2

Notes:

[1] – Use straight-line interpolation for intermediate values of PGA, S_S, or S_1.

[2] – Perform a site-specific geotechnical investigation and dynamic site response analysis for all multi-span bridges in Site Class F.

[3] – Consider performing a ground motion hazard analysis or dynamic site response analysis for multi-span bridges.

Determine the spectral response parameters for the Lower Level Design Earthquake using the Ground Motions of a full rupture of Cascadia Subduction Zone Earthquake (CSZE). The full rupture CSZE hazard maps are available at the ODOT Bridge Section website.

The CSZE is a deterministic event, and a deterministic Design Response Spectrum must be generated. To allow for consistency and efficiency in design for the CSZE, an application for generating the Design Response Spectra has been developed by Portland State University. Latitude, Longitude, and $V_{s,30}$ are the only required input for running the application. This application can be accessed at the following link:

https://www.oregon.gov/ODOT/Bridge/Pages/Seismic.aspx

1.17.2.5 Consideration for Other Loads and Hazards

Use the load factor for the Live Load on Extreme Event Load Combination I, γEQ = 0 (*LRFD 3.4.1*), unless the bridge is designated by Bridge Section as non-conventional or Critical.

For bridges subject to liquefaction, perform seismic analyses for both liquefied and non-liquefied conditions. Refer to *Chapter 13 of GDM* for combination of inertial loads and liquefaction or liquefaction-induced lateral spreading.

For bridges subject to scour, perform seismic analyses for both scoured and non-scoured conditions. Use the scour depths provided in *BDM 1.10.5.3* for the scoured condition.

1.17.2.6 Modifications to AASHTO Guide Specifications for LRFD Seismic Bridge Design

The following items summarize ODOT's additional requirements and deviations from *AASHTO Guide Specifications for LRFD Seismic Bridge Design*:

- Design all bridges to satisfy the Type-1 Global Seismic Design Strategy (ductile substructure with essentially elastic superstructure), *AASHTO Guide Specifications 3.3*. However, in the case of a steel substructure, design the bridge according to the latest edition of the *AASHTO LRFD Bridge Design Specifications*.

  Type-2 Global Seismic Design Strategy (essentially elastic substructure with ductile superstructure) is not permitted by ODOT.

  Type-3 Global Seismic Design Strategy (elastic superstructure and substructure with a fusing mechanism between the two) can be considered if approved by the State Bridge Engineer. Include a clear description of the selected Seismic Design Strategy in the calculation book for the structure.
- The following types of Earthquake Resisting Systems (ERS) or Earthquake Resisting Elements (ERE) provided in *AASHTO Guide Specifications 3.3* are permissible ERS or ERE for ODOT bridges:
  - Type 1, 2, 3, 4 and 5 on *Figure 3.3-1a.*
  - Types 1, 2*, 3, 7, 8, 9, 10, 11**, 12 and 14 on *Figure 3.3-1b.*

    * Near ground plastic hinge is limited to less than 4 feet below ground surface.

    ** To use this Earthquake Restraining Element the following must be applied:

    1. Liquefaction induced-lateral spread and slope instability are deemed unlikely to occur under the design earthquake.

2. For the Life Safety criteria a maximum of 70 percent of the passive abutment resistance can be used in seismic analyses in accordance with *AASHTO Guide Specifications*.
3. For the Operational criteria a maximum of 30 percent of the passive abutment resistance can be used in seismic analyses.

- Obtain approval from the State Bridge Engineer before utilizing any of the following types of Earthquake Resisting Systems (ERS) or Earthquake Resisting Elements (ERE) provided in *AASHTO Guide Specifications 3.3*:
  - Type 6 on *Figure 3.3-1a.*
  - Types 4, 5, and 6 on *Figure 3.3-1b.*
  - Types 1 and 2 on *Figure 3.3-2.*
- The following types of Earthquake Resisting Systems (ERS) or Earthquake Resisting Elements (ERE) provided in *AASHTO Guide Specifications 3.3* are not permissible ERS or ERE for ODOT bridges:
  - Type 13 on *Figure 3.3-1b.*
  - Types 3, 4, 5, 6, 7, 8 and 9 on *Figure 3.3-2.*
  - Types 1, 2, 3 and 4 on *Figure 3.3-3.*
- Identify the ERS for bridges in SDC B (*AASHTO Guide Specifications 3.5*) when $0.25 \leq S_{D1} < 0.30$.
- Pushover analysis can be used instead of the implicit equations to determine the Displacement Capacity for SDC B and C as prescribed on *AASHTO Guide Specifications 3.5*. When pushover analysis is performed, provide SDC D Level of Detailing regardless of the design SDC.
- Satisfy the balanced stiffness and balanced frame geometry requirements for all bridges in SDC C and D (*AASHTO Guide Specifications 4.1.2* and *4.1.3*).
- For bridges with two or more spans in SDC B, C, or D, use analysis Procedure Number 2 (Elastic Dynamic Analysis) to determine seismic demand (*AASHTO Guide Specifications 4.2*).
- Use Procedure 3, (Nonlinear Time History Analysis) if the geotechnical engineer is performing a site-specific hazard motion analysis and any of the following exists:
  - The maximum bridge span length is more than 300 feet or the total bridge length is more than 1,800 feet.
  - Bridge geometry does not allow for the balanced stiffness or balanced frame geometry requirements be satisfied.
  - Special bearing and damping devices (isolation bearing, shock transmission units, etc.) and non-conventional expansion joints are expected to be installed.

  Nonlinear Time History Analysis is the most expensive seismic analysis procedure; however, the extra design cost is often offset by construction cost savings and can be effectively used to manage risk.

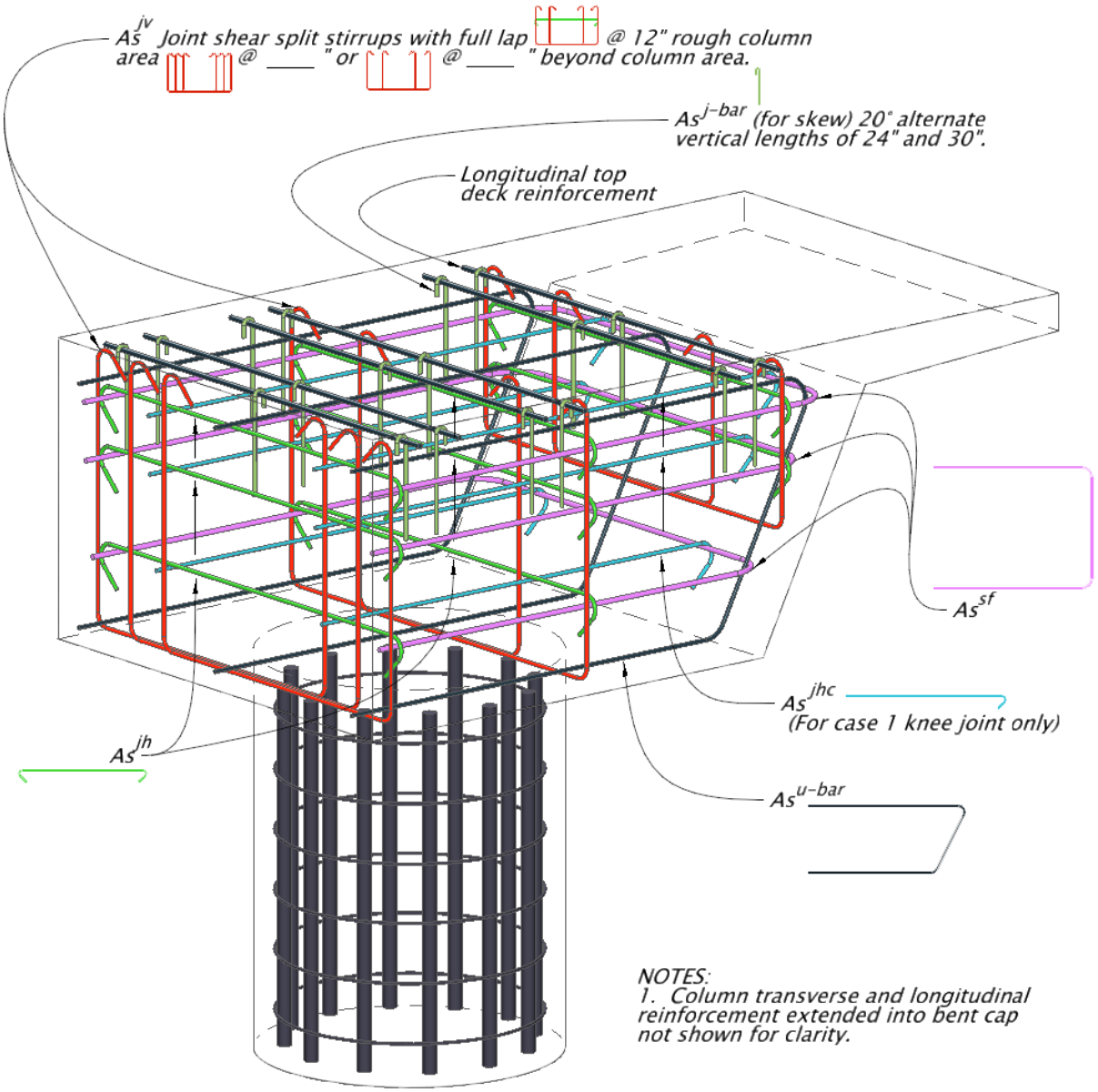
  Consult the project geotechnical engineer to determine if there are geotechnical factors that may lead to pursuing the development of ground response and non-linear time history analyses. Geotechnical factors may include:

  - Bridge is within 6 miles of an active fault.
  - Soils at bridge site are defined as Site Class "E" or "F" soils.
  - Soil profile supporting the bridge varies significantly among bridge bents.
- Use a Damping Ratio of 5 percent (*AASHTO Guide Specifications 4.3.2*) on all new bridges for seismic loading. The application of the reduction factor, R_D, is not allowed without approval from the State Bridge Engineer.
- Use Design Method 3 (Limited-Ductility Response in Concert with Added Protective Systems) for designing the lateral seismic displacement demand (*AASHTO Guide Specifications 4.7.1*) only upon approval from the State Bridge Engineer.
- 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 *(AASHTO Guide Specifications 4.2)*, design and detail the capacity protected members to resist the lesser of:
  - 2.0 times the elastic forces resulting from the demand displacement.
  - Column/pier overstrength plastic moment and shear.
- Design Longitudinal Restrainers (*AASHTO Guide Specifications 4.13.1*) in accordance with *BDM 1.17.8*.
- Participation of the approach slab, wingwalls, and backwalls in the overall dynamic response of bridge systems may be considered in seismic design of bridges using *BDM 1.10.4.2*.
- Select the Foundation Modeling Method (FMM) (*AASHTO Guide Specifications 5.3.1*) according to *BDM 1.10.4*.

  Do not allow uplift or rocking of spread footings in all SDCs.
- Perform Liquefaction Assessment for all bridge sites according to *Chapter 6* of the ODOT Geotechnical Design Manual.
- Use the provisions in *AASHTO Guide Specifications 7.2* in conjunction with the forced-based seismic design procedure utilized in the *AASHTO LRFD Bridge Design Specification* and requirements of this section of the *BDM*.
- Provide minimum shear reinforcement for bridges in SDC A, when $0.10 \leq SD1 \leq 0.15$, according to the requirements of *AASHTO Guide Specifications 8.6.5* for SDC B, in addition of satisfying the requirements of *AASHTO Guide Specifications 8.2*.
- Do not use wire rope or strands for spirals in bridge columns. containing potential plastic hinge regions. Deformed welded wire reinforcement (*AASHTO Guide Specifications 8.4.1*) may be used with approval from the State Bridge Engineer.
- Do not use high strength reinforcing bars with yield strength exceeding 60 ksi for sprials, hoops, or longitudinal column reinforcement where plastic hinging is expected.
- The same size vertical bars may be used inside and outside of interlocking spirals (*AASHTO Guide Specifications 8.6.7*).
- Provide minimum longitudinal reinforcement (*AASHTO Guide Specifications 8.8.2*) of 1 percent for columns in SDC B, C and D.
- Extend the vertical column bars into oversized drilled shafts according to *BDM 1.10.5.5*, in lieu of *AASHTO Guide Specifications 8.8.10*.

- Revise the third bullet of *AASHTO Guide Specifications 8.13.4.1.1* as follows:
    - Exterior column joints for box girder superstructures and other superstructure types if the cap beam extends beyond the joint far enough to develop the longitudinal cap reinforcement.

Figure 1.17.2.6-1 Three-Dimensional Representation of Knee Joint Reinforcing



1.17.2.7 Bridge Widenings

Design selected bridge portions for seismic loading as directed by the flowchart shown in *Figure 1.17.2.7-1*. Design by the same criteria as for "New Bridges".

POTENTIAL FACTORS AFFECTING SEISMIC PERFORMANCE OF BRIDGE WIDENINGS – **The following considerations refer to the flow chart in *Figure 1.17.2.7-1*.**

Consideration 1

- Widening without adding new columns will make a bridge more vulnerable to seismic loads. Clearances for railroads or highways under structures may prevent adding new columns.

Consideration 2

- Widening on both sides will increase the potential for the new portion to be able to resist seismic loads for the full widened structure.
- Widening on one side only may actually result in a completed structure that is more vulnerable than the original structure.
- If widening is on one side only, is there a possibility another future widening could be placed on the opposite side?
- It will not normally be practical for a widening to resist the total seismic load (existing and widening) when widening on only one side; however, there could be exceptions.

Consideration 3

- A formal seismic analysis may be required to answer this question. A “yes” answer to Consideration 3 assumes only minimal work (such as column jacketing) will be needed for the existing structure.
- Although the existing structure may have inadequate capacity, it will have some capacity that can probably be taken advantage of.
- If existing columns are not stressed beyond the elastic range they will probably not need a Phase 2 retrofit.
- The existing structure will have to go through the same deformations as the new portion even though the capacity may not be included in the seismic analysis.

Consideration 4

- Structures which are connected must have compatible deflections at connections.
- We are usually not concerned about the seismic load generated from one structure colliding with an adjacent structure; however, there could be exceptions.
- Providing a joint between the widening and existing structure will probably increase the potential for the new portion to resist seismic loads. If the widening adds enough width for at least two lanes and the longitudinal joint would not be in a travel lane, a joint should be considered.

Consideration 5

- Base isolation is strongly encouraged, especially when bearing replacement is required anyway.
- When footing strengthening is required, Phase 2 will probably not be practical due to the high cost. If cost is the primary decision factor, a realistic estimate of Phase 2 retrofit cost should be prepared. Don't say it costs too much without knowing how much too much is!
- The closer footings are to the ground surface, the more practical Phase 2 will become.

Consideration 6

- If you can't see the new portion acting separately, do not waste time assuming it will!
- Widenings with only one new column per bent vs. multiple columns on the existing structure probably do not need to be modeled separately.
- When widening with 2 or more columns or with drilled shafts, it is probably reasonable to model the new structure separately.
- Consider the potential for another future widening. Perhaps size the footings larger than necessary.

Consideration 7

- Is it even possible to close the structure to replace it? Can it be replaced in stages? Is it historic?
- A new structure will usually be far superior to a "band-aided" structure.

Consideration 8

- FHWA requirements take effect when the new structure has more travel lanes than the existing structure. Widenings that add only shoulder width or median width are not affected. FHWA requirements may assist in convincing region of including Phase 2 seismic retrofit, but it is not intended to force a Phase 2 retrofit when it is not practical.
- For projects exempt from FHWA review, the Technical Services Branch Manager will approve exceptions to FHWA policy.

Consideration 9

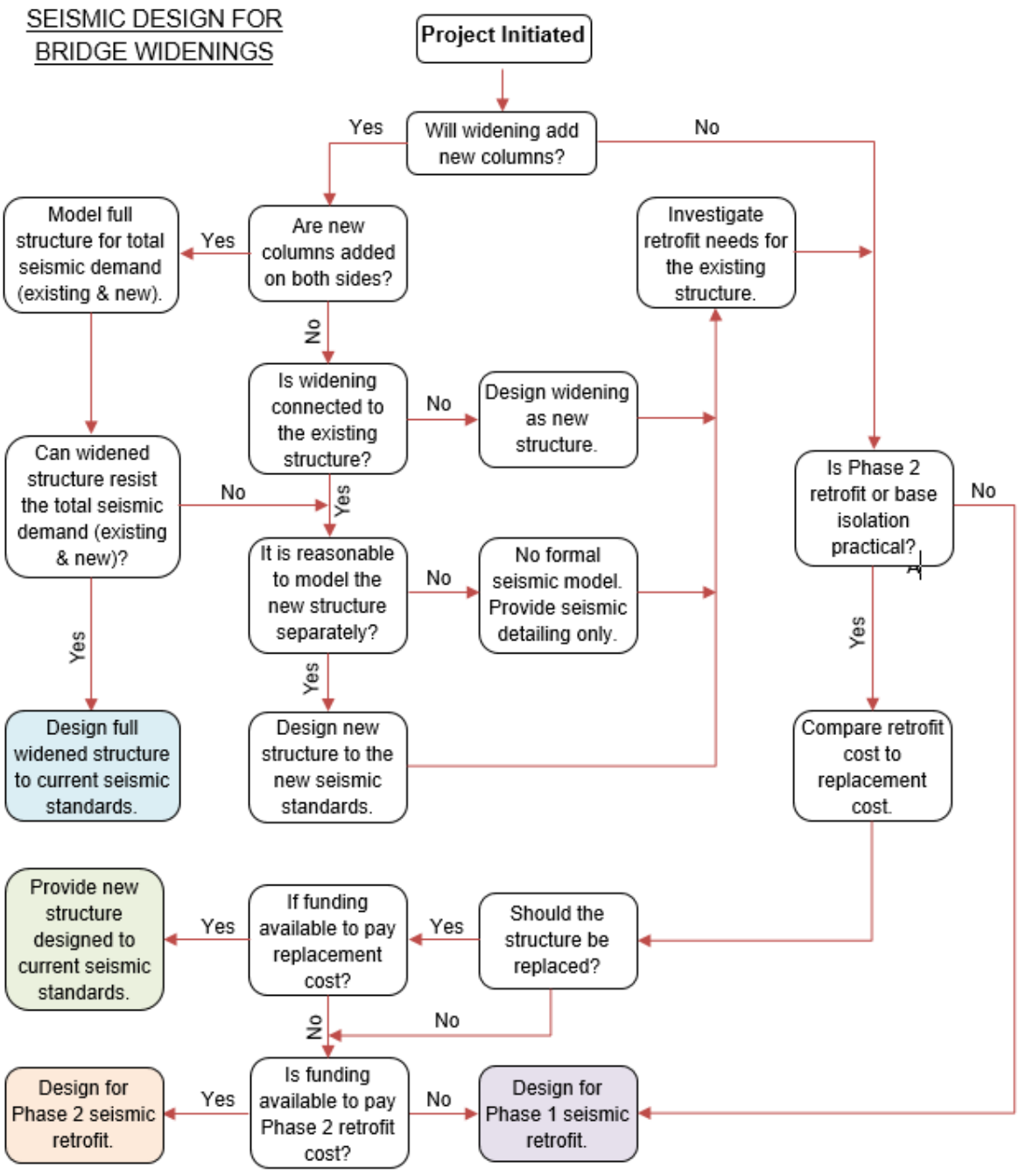
- Region holds the money. They may have factors/priorities we don't know about.
- Refusal by region to fund the needed retrofit and refusal by FHWA to grant an exception (if federal funding) could lead to cancellation of the project.
- It would be desirable to calculate a cost-benefit ratio.

SEISMIC DETAILING FOR BRIDGE WIDENINGS: *Figure 1.17.2.7-1* provides a framework for evaluating the level of seismic design required for bridge widenings. When detailed seismic modeling and analysis is not required, provide seismic detailing of all new bridge components. Seismic detailing consists of providing the reinforcement of the Earthquake Resisting Elements within the limits recommended in the design specifications, including:

- Maximum and minimum shear reinforcement (per *AASHTO Guide Specifications 8.6.4 & 8.6.5*).

- Maximum and minimum longitudinal reinforcement (per *AASHTO Guide Specifications 8.8.1 & 8.8.2*).
- Maximum spacing of longitudinal reinforcement (6 or 8 inch, depending on the shape of the column).
- Minimum size of lateral reinforcement (per *AASHTO Guide Specifications 8.8.9*).
- Maximum spacing of lateral reinforcement in plastic region (4 inch), where the plastic hinge region is determined according to the *AASHTO Guide Specifications 4.11.7.*
- Joint proportioning (per *AASHTO Guide Specifications 8.13.2)* and minimum joint shear reinforcing (per *AASHTO Guide Specifications 8.13.3*).

Figure 1.17.2.7-1 Seismic Design and Detailing Flowchart for Bridge Widenings



Notes:

1. Document all seismic-related decisions with a memo to the file.
2. The Bridge Section Seismic Committee will be available to assist in the decision process when requested by the designer.
3. The designer should strive for higher levels of seismic design/retrofit whenever practical.
4. See preceding pages for potential factors affecting seismic design decisions.

1.17.2.8 Seismic Retrofit

The *ODOT's Seismic Implementation: Policies and Design Guidelines* provides guidance for planning and executing ODOT's Seismic Program. It also provides recommendations regarding the design criteria and other considerations that need be evaluated during the early design phase of the project. When the seismic retrofit design is included in a project, use a phased approach for establishing a practical and economical retrofit strategy. The publication "Seismic Retrofitting Manual for Highway Structures" (*FHWA-HRT-06-032*) is recommended as a reference source to supplement the *Bridge Design Manual* and *Bridge CAD Manual*.

The following steps are provided to help designers initiating the design process:

- Most Oregon bridges fall under importance category of "**standard**", based on the Bridge Importance Category definitions provided on *FHWA-HRT-06-032*. Contact Bridge HQ when this category becomes questionable for a given structure.
- Contact Bridge HQ for information on the Anticipated Service Life (ASL) of the bridge.
- Revise the top-half of the *Table 1-2* of *FHWA-HRT-06-032* as shown in *Table 1.17.2.8-1*.

Table 1.17.2.8-1 Minimum Performance Levels for Retrofitted Bridges

EARTHQUAKE GROUND MOTION	**BRIDGE IMPORTANCE and SERVICE LIFE CATEGORY**					
	Standard			**Essential**		
	ASL 1	**ASL 2**	**ASL 3**	**ASL 1**	**ASL 2**	**ASL 3**
Lower Level Ground Motion Cascadia Subduction Zone Earthquake – Full Rupture	PL0[4]	~~PL3~~PL2	~~PL3~~PL2	PL0[4]	~~PL3~~PL2	PL3
Upper Level Ground Motion 7 percent probability of exceedance in 75 years; return period is about 1,000 years.	PL0[4]	PL1	PL1	PL0[4]	PL1	PL2

Replace *Equation 7-49* of *FHWA-HRT-06-032* with the following equation:

$$\phi_p = \left[2 - 5\left(\frac{V_m - V_i}{V_i - V_f}\right)\right]\phi_y$$

Replace *Equation 7-51* of *FHWA-HRT-06-032* with the following equation:

$$\phi_p = \left[2 - 4\left(\frac{V_{jh} - V_{ji}}{V_{ji} - V_{jf}}\right)\right]\phi_y$$

The equations above will produce results that are consistent with the narrative in *Article 7.8.2.7(b)* and *7.8.2.8(b)* of *FHWA-HRT-06-032*, given the following definition of curvature ductility, μ_ϕ:

$$\mu_\phi = \frac{\phi_y + \phi_p}{\phi_y}$$

For assessing the seismic performance of existing bridges use concrete strain limits provided in *Table 1.17.2.8-2* for existing bridge columns with poor confinement detailing.

Table 1.17.2.8-2 Concrete Strain Limits for Existing Bridge Columns

Confinement / Design	Inadequate Hoops and Hoop Spacing[1]	
	Inadequate Lap Splice[2]	Adequate Lap Splice
Operational	0.002	0.002
Life Safety	0.003	0.004
1. As adequate hoops are qualified those that meet the definition of "seismic hooks" in *Article 8.8.9* of *AASHTO Guide Specifications for Seismic Bridge Design* and are spaced no more than 6 inches apart.		
2. As adequate lap splices are qualified those that meet the requirements of *Article 5.10.8.4.3a* of *AASHTO LRFD Bridge Design Specifications* for Class B splice.		

PHASE 1

The Phase 1 Seismic Retrofit is considered to achieve "Life Safety" performance of Oregon bridges under seismic induced loading. Work during this phase is intended to prevent superstructure pull-off and bearing failure. Phase 1 Retrofit is not required on bridge painting projects or any projects where the scope of work does not extend below the deck.

Incorporate Phase 1 Seismic Retrofit on bridge rehabilitation projects when bridges are located in Seismic Zone 3 or 4. Bridges located in Seismic Zone 2 may be considered for Seismic Retrofit

if situated between bridges (on the same route) that have received or are receiving Seismic Retrofit, or between new bridges built to current seismic design standards.

As a minimum, for a Phase 1 Retrofit ensure that the girders will not pull off longitudinally or slide off laterally from the bents. This will normally involve addition of cable restraints, shear blocks, or beam seat lengthening and widening.

Identify a seismic design concept which will accomplish the intent to preclude span pull off or collapse of the superstructure. Depending on the concept selected, some strengthening of the superstructure may be required to ensure loads generated at the restraints or shear blocks can be transmitted without exceeding design stresses in the superstructure. For steel truss bridges, ensure all truss elements and connections provide sufficient resistance to failure or plastic deformation under seismic induced loading. Short pedestals or secondary columns above the main bent cap level must also be investigated for seismic induced loading and strengthened or braced, if necessary.

Upgrade existing bearings to elastomeric bearings, if needed to assure the designer's concept will work. Upgrading bearings to elastomeric should, also, be considered to improve seismic performance when existing bearings are known to have poor seismic performance, such as steel rocker bearings. Analysis for Phase 1 Retrofit will normally consist of a single degree of freedom model, which may be sufficient for normal bridges. However, a higher level analysis may be required, if needed to fully develop the designer's concept, or for bridges with irregular column lengths of multi-column bents or if the bents have significantly different stiffness. Use full column sections (uncracked) for this level of analysis to develop connection design loads. This is the minimum level of work that must be included. A cracked section analysis may be used to investigate the maximum anticipated movements.

PHASE 2

Work during this phase involves substructure (columns, footings and foundations) ductility enhancement and strengthening. Any additional or deferred Phase 1 Retrofit work would also be included. The end product is a retrofitted bridge with as much seismic loading resistance as a new bridge would have for the site. The Phase 2 Seismic Retrofit is considered to achieve the "Operational" performance of Oregon bridges under the seismic loading induced by the full rupture of Cascadia Subduction Zone Earthquake.

Evaluate the structure to investigate the level of effort and scope of work needed to do Phase 2 Retrofit. Phase 2 involves a complete seismic analysis of the widened or rehabilitated bridge for full seismic loading, including consideration of strengthening or restraints to the superstructure, substructure and foundations. The work may involve column and footing strengthening or enlargement, or the use of isolation bearings, and soil improvement, if there is potential for liquefaction. The decision about whether to actually do Phase 2 Retrofit in the project will be made after developing a retrofit concept, rough cost estimate and evaluation of the relative importance of the bridge to the transportation network, in comparison to the estimated cost and available funding for the project. The remaining service life, existing condition, and retrofit cost

versus replacement cost are also important factors and must be evaluated. Consult with the Bridge Section before proceeding with any Phase 2 seismic retrofit. The flowchart for seismic design of widenings in *BDM 1.17.2.7* (*Figure 1.17.2.7-1*) can be used as a guide to make the decision.

A seismic retrofit analysis typically requires the use of a "Site Factor" to develop the response spectrum used in the analysis. Site factors are based on the soil conditions at the site, (categorized as Site Classes A - F) as described in the *FHWA Seismic Retrofitting Manual for Highway Structures, Table 1-3*. For most normal bridges requiring Phase 1 retrofit work the site class can be determined using either existing soils data or a general knowledge of the site geology and soil conditions. If limited knowledge is available the default designation of Site Class D is acceptable. However, for Phase 2 level retrofit analysis more detailed soils information is required to better determine the design response spectrum and also to adequately characterize and model the foundations in the analysis. Additional exploration work may be required to obtain this information. This additional work is justified due to the increased cost of Phase 2 retrofit work and the need for a more refined analysis.

RAIL UPGRADE, DECK OVERLAYS, PRESERVATIONS, REPAIR, STRENGTHENING, AND OTHERS:

These projects should include seismic retrofit as described previously for "Seismic Retrofit".

1.17.2.9 Temporary Detour Bridges, Pedestrian Bridges, and Buried Structures

TEMPORARY DETOUR BRIDGES:

Design all temporary detour bridges meeting one of the following criteria according to *LRFD 3.10.10*:

- Bridge is expected to be in service for more than one year and ADT > 10,000.
- Bridge is expected to be in service for more than two years.

For all other temporary detour bridges, provide the minimum support length requirement according to *LRFD 4.7.4.4*.

PEDESTRIAN BRIDGES:

Design new pedestrian bridges that cross over roads carrying vehicular traffic according to requirements for Ordinary bridges, as specified in *BDM 1.17.2.3*.

BURIED STRUCTURES:

According to *AASHTO Guide Specifications 3.1*, buried structures, generally, do not need to be designed for seismic loads. However, for all buried structures supported on piling or drilled shafts type foundations, design the structure for seismic loading according to *BDM 1.17*.

1.17.2.10 Structure Modeling

(1) STRUCTURE MODELING, GENERAL:

- Use a "first cut" analysis with fixed supports. These results will be easier to interpret than a spring supported model and will give a baseline for comparison with additional analyses. With these results, make a rough substructure design. Now a new analysis can be performed with footing springs and the substructure design checked and refined. Additional cycles of redesign, analysis, and force comparison to previous analyses could be used in some cases but generally would not be required or warranted.
- A reasonable target for a seismic design check is 20 percent. Designer and checker should resolve differences greater than 20 percent, but it is impractical to try to refine the design beyond that.

(2) FOOTING SPRINGS:

See *BDM 1.10.4*.

(3) PROGRAMS:

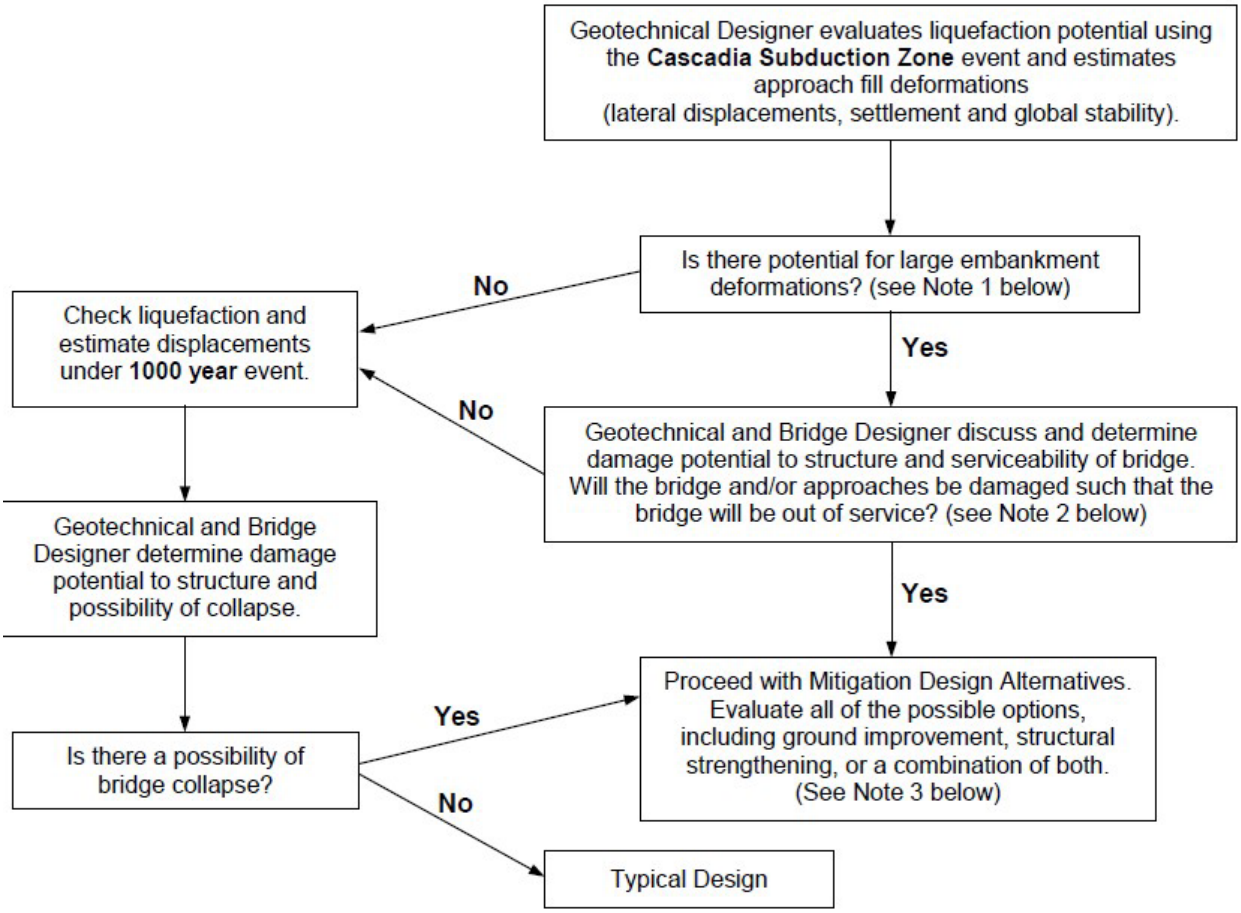
- The Uniform Load and single mode dynamic analysis methods are acceptable for many structures (see the code limitations) but multi-mode dynamic analysis by computer may be easier. The result of any analysis method must be judged for correctness. Is the result reasonable? Reviewing the calculated periods, modal participation factors and mode shapes can greatly aid this judgment. A high level of engineering judgment will be required at all times.
- MIDAS is ODOT's primary in-house static and dynamic analysis programs, and is available for bridge designers working at Bridge HQ or Region Tech Centers. Many design firms have adopted the use of SAP2000 or STAAD for seismic design of bridges. Other programs are also acceptable, provided the programs satisfy the analysis requirements and have been previously verified.

1.17.3 Reserved

1.17.4 Liquefaction Evaluation and Mitigation Procedures

The liquefaction potential of foundation soils will be determined by the geotechnical designer. If foundation soils are predicted to liquefy, the effects of liquefaction on foundation design and performance will be provided as described in *BDM 1.10.5*. The need for liquefaction mitigation will be in accordance with the following ODOT Liquefaction Mitigation Policy.

Figure 1.17.4-1 ODOT Liquefaction Mitigation Policy



Note 1: For meeting the performance requirements of the Cascadia Subduction Zone event (Operational), lateral deformation of approach fills of up to 12 inches are generally considered acceptable under most circumstances pending an evaluation of this amount of lateral deformation on abutment piling and abutment/cap. Larger structural lateral deformations and settlements may be acceptable under the 1000 year event as long as the "Life Safety" criteria are met.

Note 2: The bridge should be open to emergency vehicles after the Cascadia Subduction Zone event, following a thorough inspection. If the estimated embankment deformations (vertical or horizontal or both) are sufficient enough to cause concerns regarding the serviceability of the bridge, mitigation is recommended.

Note 3: Geotechnical and bridge designer to submit all liquefaction mitigation designs and cost estimates to Bridge Standards for review and approval.

A continuous discussion between the Geotechnical and bridge designers is to be expected for determining the most cost-effective mitigation alternative. The iteration for both geotechnical and structural analyses start with the geotechnical designer providing the bridge designer the following information:

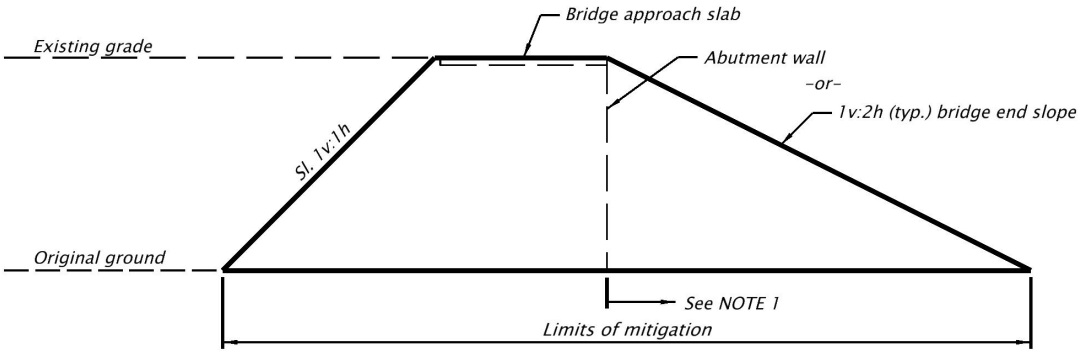
- lateral soil displacement.
- vertical soil displacements.
- vertical and horizontal loading on piling and end bent due to the soil displacement.

Bridge designer determines the lateral and vertical deformation demand and capacity of the bridge foundation and follows up with the geotechnical designer on the need for soil mitigation. Evaluation of the mitigation alternatives should consider both structural improvements and soil mitigation by maintaining a cost balance and equal risk between these two forms of mitigation. Final cost estimates should reflect this evaluation.

Refer to *ODOT research report SPR Project 361: "Assessment and Mitigation of Liquefaction Hazards to Bridge Approach Embankments in Oregon", Nov. 2002* and *"Ground Modification Methods Reference Manual, Volume I & Volume II", (Pub. No. FHWA-NHI-16-027 & NHI-16-028)* for mitigation alternatives and design procedures.

As a general guideline, the foundation mitigation should extend from the toe of the bridge end slope (or face of abutment wall) to a point that is located at the base of a 1:1 slope which starts at the end of the bridge approach slab, as shown in *Figure 1.17.4-2*.

Figure 1.17.4-2 Foundation Mitigation



NOTE 1: Extend ground improvement beyond the abutment face as needed for design.

1.17.5 Costs

(1) CONSTRUCTION COSTS: Apply the following factors to TS&L (preliminary) structure cost estimates to approximate the additional cost of seismic criteria (excluding liquefaction):

- Single Spans: 1.00
- Multiple Spans: 1.30 Irregular (widely varying columns lengths or support materials; unusual geometry or curvature)
  1.10 Other

(2) DESIGN COSTS: Apply the following factors to TS&L (preliminary) design cost estimates to approximate the additional cost of seismic design criteria (excluding liquefaction):

- Single Spans: 1.00
- Multiple Spans: 1.20 Trestles
  1.50 Irregular (widely varying columns lengths or support materials; unusual geometry or curvature)
  1.35 Other

1.17.6 Instrumentation

Consider placement of accelerometers on the ground and on structure portions for large or unusual structures. Consult with the State Bridge Engineer to determine if this is appropriate and fits with the ODOT Strong Motion Program.

1.17.7 Dynamic Isolators

Isolators may be useful for either new construction or retrofit work. Isolators change structure response by lengthening the periods of primary vibration. This tunes the structure response away from the typical earthquake's maximum response frequencies. This effect, along with added damping, works to reduce the system response. The result is reduced substructure forces.

Typical steps to model an isolated structure include:

1. Compute service loads (D, L, LF, CF, W, WL, R, S and T) for the worst single girder.
2. Use these loads, and the applicable seismic loading, in the Dynamic Isolation System, Inc. (DIS) program PC-LEADER to get a preliminary isolator size and its properties. DIS has given us permission to use the program even though we will not specify only their bearing.
3. Develop a full structural model (superstructure, substructure, and bearings/isolators). Normally this will be done on a per girder basis so the substructure should be proportioned to fit this basis. The model can often be a two dimensional model.

4. In the structural model use the equivalent isolator stiffness (Keff). This stiffness should be further modified to fit modeling assumptions of a bearing cantilevered from the substructure at interior supports.
5. Load the structural model with dynamic loading through a modified response spectrum. The response spectrum can be taken from the PC-LEADER output or developed from the *Guide Specification for Seismic Isolation Design*.
6. Develop another full structural model to represent the "as-is" structure. Dynamically load this model with a normal response spectrum. This gives a basis to evaluate the isolation effectiveness.
7. It may be necessary or desirable to adjust the relative isolator stiffness to better distribute the dynamic forces. It is important the final isolator properties function adequately for service loads. The isolator characteristics must also be realistic and achievable.

An example isolator modeling is given in the *Bridge Example Design notebook*.

Other computer programs are acceptable, provided the programs satisfy the analysis requirements and have been previously verified.

1.17.8 Seismic Restrainer Design (New Designs and Retrofits)

1.17.8.1 Seismic Restrainer Design, General

The intent is to prevent superstructure pull-off and bearing failure. Work restrainers only in the elastic range. Design the restrainer connection for 125 percent of the restrainer design force.

Note that *LRFD 3.10.9.5* requires “sufficient slack” so that the restrainer does not start to act until the design displacement is exceeded.

Restrainers may be omitted where the available seat width meets or exceeds “N” of the Design Specifications and 4 times the calculated design earthquake elastic deflection. Seat widths meeting these criteria are presumed to accommodate the large elasto-plastic movements of a real structure under seismic loading.

Design restrainers for a minimum force equal to the peak site bedrock acceleration coefficient “A” times the weight of the lighter portion being connected.

In all instances it is necessary to design or check the transfer mechanism for force transfer from superstructure to substructure (bearings, diaphragms).

1.17.8.2 Information for Restrainer Design

(1) CONCRETE:

Concrete bearing strength based on $0.85f''_c$ ($\Phi = 1.0$).

Maximum increase for supporting surface wider than loaded area = 2.0.

Multiply by 0.75 when loaded area is subject to high edge stresses.

For concrete shear lugs, use *LRFD equation 5.8.4.1-1* for shear friction as outlined in *LRFD 5.8.4*.

(2) STRUCTURAL STEEL:

Design structural steel members using the *AASHTO LRFD Bridge Design Specifications*.

(3) FASTENERS:

Table 1.17.8.2-1 Steel to Steel Restrainer Design Parameters

		A 307		A 325	
Diameter (inch)	**Nominal Area (in^2)**	**Tension (0.76 x 60 ksi)**	**Shear (0.38 x 60 ksi)**	**Tension (0.76 x 120 ksi)**	**Shear (0.38 x 120 ksi)**
0.75	0.4418	20.1 k	10.1 k	40.3 k	20.1 k
0.875	0.6013	27.4 k	13.7 k	54.8 k	27.4 k
1.0	0.7854	35.8 k	17.9 k	71.6 k	35.8 k

Notes:

Tension loads are based on *LRFD equation 6.13.2.10.2-1*.

Shear loads are based on *LRFD equation 6.13.2.7-2* assuming one shear plane per bolt and with threads included in the shear plane.

Shear loads may be increased 25 percent if the threads are excluded from the shear plane.

(4) STEEL RODS:

Table 1.17.8.2-2 Steel Rods Restrainer Design Parameters

		A307 Fu = 58 ksi	A449 Fu varies	
Diameter (Inch)	**Stress Area (in^2)**	**Tension (kips) Ft=Fy=36 ksi**	**Tension (kips) Ft = Fy**	**Fy (ksi)**
0.750 0.875 1.00	0.334 0.462 0.606	12.0 16.6 21.8	30.7 42.5 55.8	92
1.125 1.250 1.375 1.500	0.763 0.969 1.155 1.405	27.5 34.9 41.6 50.6	61.8 78.5 93.9 114.0	81
1.750 2.250	1.900 2.500	68.4 90.0	110.0 145.0	58
Notes: Tensioning of A 449 steel rods must be specified, if required by the design. Tensioning requirements are not part of the specification as they are with A 325. Use nominal area for elongation calculations.				

(5) WIRE ROPE:

See *BDM 1.17.8.6* for a complete discussion of Structural Wire Rope, Wire Rope Connections & Turnbuckles.

F_t = (0.95)(176.1 ksi)(area) = 0.95(minimum breaking strength).

Note: Yield strength is approximately equal to minimum breaking strength.

Table 1.17.8.2-3 Wire Rope Restrainer Design Parameters

Diameter (in)	Area (in^2)	Minimum Breaking Strength (kips)	Design Load (kips)
1/2	0.119	23.9	22.7
3/4	0.268	52.9	50.2
7/8	0.361	71.6	68.0
1	0.471	93.0	88.3
1 3/8	0.906	173.0	164.0

The area values in *Table 1.17.8.2-3* are based on ASTM A603. The minimum breaking strength above is based on ASTM A1023. The design load in *Table 1.17.8.2-3* is based on 0.95 x the minimum breaking strength. For sizes other than 7/8 inch diameter, ASTM A1023 is likely to be used.

E for wire rope = 10,000 ksi

f_y for wire rope = 176.1 ksi

ASTM A603 lists the E for structural wire rope as 20,000 ksi for "prestretched" wire rope. Wire rope used for seismic applications will not be prestretched, however, so an E of 10,000 ksi should be used.

(6) RESIN BONDED ANCHORS:

See *BDM 1.5.11.2*, "Post-Installed Anchors".

(7) CONCRETE INSERTS:

Use hot-dip galvanized expanded coil concrete inserts with closed-back ferrule threaded to receive UNC threaded bolts.

Inserts are readily available in ¼ inch sizes. Other sizes are only available in very large quantities. Therefore, only the standard sizes listed in *Table 1.17.8.2-4* are recommended.

Table 1.17.8.2-4 Concrete Inserts Restrainer Design Parameters

Diameter (in)	**Tension (kips) A307 or A325**	**Shear (kips)**	
		A307	**A325**
3/4	12.6	7.4	12.6
1	19.3	13.4	19.3
1-1/4	34.4	21.4	34.4
1-1/2	54.3	31.0	54.3

Tension and shear capacity for concrete failure is based on *equation 6.5.2* from the *PCI Design Handbook (3rd Edition)* with $\Phi = 1.0$ and with a factor of safety of 1.5. *Equation 6.5.2* controls both shear and tension for shallow embedment depths. See the *PCI Design Handbook* for group effects, edge distance effects and combined tension and shear.

Tension capacity of the insert cannot exceed the tension capacity of the bolt. Shear capacity of the insert cannot exceed the shear capacity of the bolt or the insert tension capacity.

Tension capacity of the bolt = $0.76A_bf_{ub}$, where A_b = bolt stress area (*LRFD equation 6.13.2.10.2-1*).

Shear capacity of the bolt = $0.38A_bf_{ub}$ (*LRFD equation 6.13.2.7-2*).

1.17.8.3 Longitudinal Restrainer Design

(1) IN-SPAN HINGES: Use the following general procedure (a modified CALTRANS method):

- Estimate restrainers to use (with elongation) and gapping desired/allowed.
- Determine joint openings (including approximate temperature movement (fall) and creep and shrinkage if appropriate).
- Determine frame stiffness and capacity.
- Determine adjacent frame stiffness and capacity.
- Plot force/deflection relationship considering component stiffnesses, joint openings (including temperature, creep, and shrinkage openings), and restrainer gapping.
- Assume a final force and deflection under single-mode response to get equivalent stiffness.
- Calculate period and resulting response coefficient.
- Calculate dynamic force and locate on the force/deflection curve.
- Review that the force capacity of the system is not exceeded, the assumed/acceptable deflection is not exceeded, and the equivalent stiffness and period are approximately as before.
- If checks are not okay modify system and recycle through as needed.

(2) BENTS WITH SUPERSTRUCTURE CONTINUOUS OVER THE BENT:

- Connect superstructure to substructure with capacity to form plastic hinging in the column(s).

(3) BENTS WITH ONLY THE DECK CONTINUOUS OVER THE BENT:

- Connect each span to substructure to form plastic hinging in the column(s).

(4) BENTS WITH NO SUPERSTRUCTURE CONTINUITY OVER THE BENT:

- With frames each side of bent:
  Connect each span to substructure to form plastic hinging in the column(s). Also connect adjacent superstructure portions by the same techniques as "in-span hinges." The adjacent super-structure portions may be connected by span to substructure connections of adequate capacity to function for both portions.
- With simple spans each side of bent:
  Connect each span to the substructure to form plastic hinging in the column(s).

Note: The plastic hinging capacity should be determined from column interaction curves with axial and moment Φ values of 1.0. Enter the curve with the unfactored dead load axial force (plus any redundancy induced axial force due to lateral seismic loading), determine the accompanying moment capacity and multiply this value by 1.3. This is the plastic moment capacity.

1.17.8.4 Transverse Restrainer Design

(1) IN-SPAN HINGES:

- Design for force transfer of 2.5(A)(supported dead load).

(2) BENTS WITH SUPERSTRUCTURE CONTINUOUS OVER THE BENT:

- Connect supported spans with force to form a failure mechanism (plastic hinging at the top of frame (column or crossbeam) and plastic hinging at bottom of column.

(3) BENTS WITH ONLY THE DECK CONTINUOUS OVER THE BENT:

- Connect supported spans with force to form a failure mechanism (plastic hinging at the top of frame (column or crossbeam) and plastic hinging at bottom of column.
- Prorate design force to ahead and back side of bent by dead load ratio.

(4) BENTS WITH NO SUPERSTRUCTURE CONTINUITY OVER THE BENT:

- Connect supported spans with a force equal to 2.5(A)(supported dead load).

Note: The plastic hinging capacity should be determined from column interaction curves with axial and moment Φ values of 1.0. Enter the curve with the unfactored dead load axial force (plus any redundancy induced axial force due to lateral seismic loading), determine the accompanying moment capacity and multiply this value by 1.3. This is the plastic moment capacity.

1.17.8.5 Hold-downs

Hold-downs or bearing replacement may be needed at vulnerable bearings such as fixed or rocker type steel bearings.

1.17.8.6 Structural Wire Rope (Cables) and Turnbuckles

Structural wire rope (cable) may be used in seismic retrofit and safety cable applications. For these applications, structural wire rope must have zinc coating for corrosion protection. *ASTM A603* structural wire rope with a Class C coating is the preferred wire rope specification. This wire rope has large wires and significant zinc coating. However, *A603* wire rope is only available by special order at a minimum of 10,000 feet.

Use A603 wire rope for all coastal area seismic retrofit applications. When there is not sufficient quantity of wire rope available in the stockpile, a new order of 10,000 feet should be purchased using project funds. Such a purchase will require preapproval from FHWA since the excess wire rope will be stockpiled for use on future projects.

For non-coastal applications, A603 wire rope is still preferred. However, *ASTM A1023* wire rope can be used where less corrosion protection is considered acceptable. A1023 wire rope uses smaller wires and has approximately one-third the zinc coating compared to A603. However,

A1023 wire rope is readily available on the market and so does not need to be stockpiled. Optional sizes of A1023 wire rope are also readily available. Those sizes are listed in *BDM 1.17.8.6*.

A603 and A1023 are the only wire rope specifications recommended for seismic retrofit applications. Other types of wire rope investigated are *ASTM A586* and *ASTM A741*. A586 wire rope is used for high-strength structural tension members, but is not readily available on the market. A741 wire rope is used for safety barrier applications (such as I-5 median between Portland and Salem). A741 has less strength compared to A603 and A1023, is difficult to make swaged connections, and is also not readily available.

7/8 inch diameter wire rope is recommended for most seismic retrofit applications. 1/2 inch diameter wire rope is recommended for safety cable applications and seismic retrofit applications where the wire rope must be wrapped around tight corners. Bending radius for A603 and A1023 wire rope are shown in *Table 1.21.1-1 and Table 1.21.1-2*.

Table 1.17.8.6-1 A603 Wire Rope Bending Radius

Wire Rope Diameter (inch)	Suggested (inch)	Minimum (inch)
1/2	18	11
7/8	32	18

Table 1.17.8.6-2 ASTM A1023 Wire Rope Bending Radius

Wire Rope Diameter (inch)	Suggested (inch)	Minimum (inch)
1/2	13	9
3/4	19	13
7/8	23	15
1	26	17
1-3/8	35	24

The bending radius values above are based on a *1997 Bethlehem Wire Rope product catalog* from Williamsport Wirerope Works, Inc.

1.17.8.6.1 General Notes for Structural Wire Rope, Turnbuckles and Connections

Use the following general notes on the plans for structural wire rope in seismic retrofit applications using the 7/8 inch diameter wire rope from the ODOT stockpile:

> Zinc-coated 7/8 inch diameter structural wire rope for seismic restraint devices will be provided by the Agency.

Use the following general notes on the plans for structural wire rope in seismic retrofit and safety cable applications using ASTM A1023 wire rope:

> Provide zinc-coated X″ (1/2″, 3/4″, 7/8″, 1″ or 1 3/8″) structural wire rope for seismic restraint devices (and safety cables) according to ASTM A1023.

Use the following general notes on the plans for turnbuckles and wire rope connections in seismic retrofit and safety cable applications:

> Provide hot-dip galvanized turnbuckles according to ASTM F1145.
>
> Provide hot-dip galvanized socket connections. Ensure socket connections can develop the minimum breaking strength of the connecting wire rope.

1.17.8.6.2 Special Provisions for Wire Rope

Under the heading **Structural Wire Rope for Seismic Restraints & Safety Cables** use the following:

[When using 7/8 inch wire rope from the ODOT stockpile for seismic retrofit:]

> Zinc-coated 7/8″ diameter structural wire rope for seismic restraint devices will be provided by the Agency. Agency provided wire rope was manufactured according to ASTM A603 with Class C coating. Wire rope construction is 6 x 7 with a Wire Strand Core (WSC). Agency provided wire rope has been previously certified to meet a minimum breaking strength of 71,600 pounds. Wire rope is stored on spools with up to 2500 feet on each spool.
>
> Agency provided wire rope is stored at the following location:
>
> > c/o District 2B Manager
> >
> > Oregon Department of Transportation
> >
> > 9200 SE Lawnfield Rd
> >
> > Clackamas, OR 97015
> >
> > Phone: 971-673-6200
>
> Notify Bridge Engineering Headquarters of the quantity of wire rope removed within 24 hours. Follow up this notification with a written memo documenting the time of removal, quantity removed (to the nearest foot), and the project for which it will be used. Send the memo to:
>
> > State Bridge Operations Engineer

Bridge Engineering Headquarters

4040 Fairview Industrial Drive SE, MS #4

Salem, OR 97302-1142

Phone: 503-986-3323

FAX: 503-986-3407

The quantity of wire rope included for use in this project, including both testing and installation, is (____) linear feet. This quantity of wire rope will be provided at no cost to the contractor. Additional wire rope required by the contractor due to fabrication errors or waste must be purchased from the Department at the Department's cost as established by the Engineer.

[When using ASTM A1023 wire rope for seismic retrofit:]

Provide zinc-coated X" (1/2", 3/4", 7/8", 1" or 1 3/8") diameter wire rope for seismic restraint devices according to ASTM A1023. Provide 6 x 19 wire rope construction with a steel core. Manufacture wire rope from extra improved plow steel. Ensure a minimum breaking strength of XX,XXX pounds (insert appropriate strength from design properties in *BDM 1.17.8.6*).

[When using 1/2 inch wire rope for safety cable:]

Provide zinc-coated 1/2" diameter structural wire rope for safety cable according to ASTM A1023. Provide 6 x 19 wire rope construction with a steel core. Manufacture wire rope from extra improved plow steel. Ensure a minimum breaking strength of 23,900 pounds.

1.17.8.6.3 Special Provisions for Turnbuckles and Socket Connections

Use the following special provisions for turnbuckles and socket connections in seismic retrofit and safety cable applications:

Provide Type 1 hot-dip galvanized turnbuckles according to *ASTM F1145*.

- Ensure turnbuckles develop the minimum breaking strength of the connecting wire rope.
- Provide turnbuckles with a 24 inch take-up unless shown otherwise.
- Test turnbuckles according to the requirements outlined in ASTM A1023.
- For seismic restraint devices, provide either a jam nut or lock wire at each end of each turnbuckle. For safety cables, provide lock wire at each end of each turnbuckle. Provide 14 gage or heavier lock wire that is either hot-dip galvanized or plastic coated.

Testing for Socket Connections – Select an independent laboratory to test three sets of wire rope assemblies. Provide approximately 3 foot segments of wire rope with galvanized stud attachments at each end. Provide stud attachments of similar size and material as to be used on the project. Test each wire rope assembly to failure in tension. Ensure the tested wire rope

assembly develops the minimum breaking strength of the wire rope and ensure that failure does not occur in the connecting parts. Ensure all three wire rope segments meet the minimum breaking strength requirement. However, when the wire rope breaks at a load less than the minimum breaking strength of the wire rope **and** at a location at least 6 inches from a connection, that test will be disregarded. When any wire rope assembly fails to meet these requirements, except as noted above, revise the connection details and prepare and test three new wire rope assemblies.

1.17.8.6.4 Design Properties

Modulus of elasticity for wire rope (non-prestretched) = 10,000 ksi.

Table 1.17.8.6.4-1 Approximate Gross Metallic Area and Minimum Breaking Strength for Wire Rope

Wire Rope Diameter (inch)	Area (in^2)	Strength (lb)	Weight (lb/ft)
1/2	0.119	23,900	0.46
3/4	0.268	52,900	1.04
7/8	0.361	71,600	1.41
1	0.471	93,000	1.85
1-3/8	0.906	173,000	3.49

Area values above are approximate and are based on *ASTM A603*. Minimum breaking strength and weight values above are based on *ASTM A1023*. Note that A1023 does not provide area values. Weight values for A603 are slightly smaller.

The sizes of *ASTM A1023* zinc-coated wire rope shown above are readily available from northwest suppliers.

The Agency is no longer stockpiling wire rope.

1.17.9 Tsunami Design

Design bridges within the tsunami inundation zone for tsunami loads according to the *AASHTO Guide Specifications for Bridges Subject to Tsunami Effects*. The Statewide Geohazards Viewer can be used for the initial evaluation of the bridge being located within the tsunami inundation zone.

Wave parameters for estimating tsunami loads may be obtained from the Tsunami Inundation Portal by entering the latitude and longitude of the bridge site.

The *AASHTO Guide Specifications for Bridges Subject to Tsunami Effects* are intended for the design and construction of new conventional bridges to resist the effects of tsunami waves. For tsunami design of non-conventional bridges, develop a Project Specific Tsunami Design Criteria

(PSDC). Obtain approval by the State Bridge Engineer for PSDC no later than Design Verification Package submittals.

Tsunami loads need not be combined with seismic loads.

Obtain approval by the State Bridge Engineer for incorporating provisions outlined in *Section 7* of the *AASHTO Guide Specifications for Bridges Subject to Tsunami Effects* for reduction of tsunami loading, such as venting and sacrificial elements.

APPENDIX – SECTION 1 – Glossary

A

Abutment - Supports at the end of the bridge used to retain the approach embankment and carry the vertical and horizontal loads from the superstructure. Current terminology is bent or end bent.

Access Control - The condition where the legal right of owners or occupants of abutting land to access a highway is fully or partially controlled by the Department of Transportation.

Advance Plans – 95-100% complete plans including special provisions, normally sent at 15 weeks.

Advertisement - The period of time between the written public announcement inviting proposals for projects and the opening of the proposals (bid or letting date).

Aggregate - Inert material such as sand, gravel, broken stone, or combinations thereof.

Aggregate, Coarse - Aggregates predominantly retained on the No. 4 sieve for portland cement concrete and those predominantly retained on the 1/4” for asphalt concrete.

Aggregate, Fine - Those aggregates which entirely pass the 3/8” sieve.

Aggregate, Dense Graded - A well-graded aggregate so proportioned as to contain a relatively small percentage of voids.

Aggregate, Open Graded - A well-graded aggregate containing little or no fines, with a relatively large percentage of voids.

Aggregate, Well-Graded - An aggregate possessing proportionate distribution of successive particle sizes.

Air-Entraining Agent - A substance used in concrete to increase the amount of entrained air in the mixture. Entrained air is present in the form of minute bubbles and improves the workability and frost resistance.

Allowable Headwater - The maximum elevation to which water may be ponded upstream of a culvert or structure as specified by law or design.

Allowable Span – The greatest horizontal distance permitted between supports.

Anchor Bolts - Bolts that are embedded in concrete which are used to attach an object to the concrete such as rail posts, bearings, steel girder-to-crossbeam connections, etc.

Annual Average Daily Traffic (AADT) – The average 24-hour traffic volume at a given location over a full 365 day year.

Anode - The positively charged pole of a corrosion cell at which oxidations occur.

Apron - The paved area between wingwalls at the end of a culvert.

Arch - A curved structure element primarily in compression, producing at its support reactions having both vertical and horizontal components.

Arch Pipe - A conduit in the form of a broad arch without a bottom.

Average Daily Traffic (ADT) - The average 24-hour traffic volume at a given location for some period of time, being the total volume during a stated period divided by the number of days in that period. Unless otherwise stated, the period is a year.

Axle Load - The load borne by one axle of a traffic vehicle.

Award - Written notification to the bidder that the bidder has been awarded a contract.

B

Backfill - Material used to replace or the act of replacing material removed during construction; also may denote material placed or the act of placing material adjacent to structures.

Backwater - The water upstream from an obstruction in which the free surface is elevation above the normal water surface profile.

Bar Chair - A device used to support horizontal reinforcing bars above the base of the form before the concrete is poured.

Bar Cutting Diagram - A diagram used in the detailing of bar steel reinforcement where the bar lengths vary as a straight line.

Base Course - The layer of specified material of designed thickness placed on a subbase or a subgrade to support a surface course.

Bascule Bridge - A bridge over a waterway with one or two leaves which rotate from a horizontal to a near-vertical position, providing unlimited clear headway.

Base Flood - Flood having 1% chance of being exceeded in any given year.

Battered Pile - A pile driven in an inclined position to resist horizontal forces as well as vertical forces.

Beam - Main longitudinal load carrying member in a structure, designed to span from one support to another (girder).

Bearings - Device to transfer girder reactions without overstressing the supports.

Bearing Capacity - The load per unit area which a structural material, rock, or soil can safely carry.

Bearing Failure - A crushing of material under extreme compressive load.

Bearing Seat - A prepared horizontal surface at or near the top of a substructure unit upon which the bearings are placed.

Bearing Stiffener - A stiffener used at points of support on a steel beam to transmit the load from the top of the beam to the support point.

Bedrock - The solid rock underlying soils or other superficial formation.

Bench Mark - A relatively permanent material object bearing a marked point whose elevation above or below an adopted datum is known.

Bent - Supports at the ends or intermediate points of a bridge used to retain approach embankments and vertical and horizontal loads from the superstructure.

Bicycle Lane - A lane in the traveled way designated for use by bicyclists.

Bicycle Path - A public way physically separated from the roadway, that is designated for use by bicyclists.

Bid Schedule - The list of bid items, their units of measurement, and estimated quantities, bound in the proposal booklet. (When a contract is awarded, the Bid Schedule becomes the Schedule of Contract Prices.)

Bidder - Any qualified individual or legal entity submitting a proposal in response to an advertisement.

Biennium - For the State of Oregon, a two-year period, always odd numbered years, starting July 1 and ending two years later on June 30.

Bleeding (Concrete) - The movement of mixing water to the surface of freshly placed concrete.

Bowstring Truss - A general term applied to a truss of any type having a polygonal arrangement of its top chord members conforming to or nearly conforming to the arrangement required for a parabolic truss.

Box Beam - A hollow structural beam with a square, rectangular, or trapezoidal cross-section.

Box Culvert - A culvert of rectangular or square cross-section.

Breakaway - A design feature that allows a device such as a sign, luminaire, or traffic signal support to yield or separate upon impact. The release mechanism may be a slip plane, plastic hinges, fracture elements, or a combination of these.

Bridge - A structure spanning and providing passage over a river, chasm, road, or the like, having a length of 20 feet or more from face to face of abutments or end bents, measured along the roadway centerline.

Bridge Approach - Includes the embankment materials and surface pavements that provide the transition between bridges and roadways.

Bridge Approach Slab - A reinforced concrete slab placed on the approach embankment adjacent to and usually resting upon the abutment back wall; the function of the approach slab is to carry wheel loads on the approaches directly to the abutment, thereby eliminating any approach roadway misalignment due to approach embankment settlement.

Bridging - A carpentry term applied to the cross-bracing fastened between timber beams to increase the rigidity of the floor construction, distribute more uniformly the live load and minimize the effects of impact and vibration.

Bridge Railing - A longitudinal barrier whose primary function is to prevent an errant vehicle from going over the side of the bridge structure.

Brush Curb - A curb 10” or less in width, which prevents a vehicle from brushing against the railing or parapet.

Buckle - To fail by an inelastic change in alignment as a result of compression.

Built-Up Member - A column or beam composed of plates and angles or other structural shapes united by bolting, riveting or welding.

Bulkhead – A partition built into wall forms to terminate each placement of concrete.

Buoyancy - Upward force exerted by the fluid in which an object is immersed.

Bushings - A lining used to reduce friction and insulate mating surfaces usually on steel hanger plate bearings.

Butt Splice - A splice where the ends of two adjoining pieces of metal in the same plane are fastened together by welding.

Butt Weld - A weld joining two abutting surfaces by combining weld metal and base metal within an intervening space.

C

Cable-Stayed Bridge - A bridge in which the superstructure is directly supported by cables, or stays, passing over or attached to towers located at the main piers.

Caisson - A watertight box of wood or steel sheeting; or a cylinder of steel and concrete, used for the purpose of making an excavation. Caissons may be either open (open to free air) or pneumatic (under compressed air).

Camber - A predetermined vertical curvature built into a structural member, to allow for deflection and vertical grade.

Cast-in-Place - The act of placing and curing concrete within formwork to construct a concrete element in its final position.

Catch Basin - A receptacle, commonly box shaped and fitted with a grilled inlet and a pipe outlet drain, designed to collect the rain water and floating debris from the roadway surface and retain the solid material so that it may be periodically removed.

Catenary - The curve obtained by suspending a uniform rope or cable between two points.

Cathode - The negatively charged pole of a corrosion cell that accepts electrons and does not corrode.

Cathodic Protection - A means of preventing metal from corroding; this is done by making the metal a cathode through the use of impressed direct current and by attaching a sacrificial anode.

Catwalk - A narrow walkway to provide access to some part of a structure.

Chain Drag - A series of short medium weight chains attached to a T-shaped handle; used as a preliminary technique for inspecting a large deck area for delamination.

Chamfer – A beveled edge formed in concrete by a triangular strip of wood (chamfer strip) placed in a form corner.

Change Order - A written order issued by the Engineer to the contractor modifying work required by the contract and establishing the basis of payment for the modified work.

Chord - A generally horizontal member of a truss.

Clay - Soil passing a No. 200 sieve that can be made to exhibit plasticity (putty-like properties) within a range of water contents.

Clear Zone - Roadside border area, starting at the edge of the traveled way, that is available for safe use by errant vehicles. Establishing a minimum width clear zone implies that rigid objects and certain other hazards with clearances less than the minimum width should be removed and relocated outside the minimum clear zone, or remodeled to make breakaway, shielded, or safely traversable.

Closed Spandrel Arch - A stone or reinforced concrete arch span having spandrel walls to retain the spandrel fill or to support either entirely or in part the floor system of the structure when the spandrel is not filled.

Cobbles - Particles of rock, rounded or not, that will pass a 12” square opening and be retained on a 3” sieve.

Cofferdam - A barrier built in the water so as to form an enclosure from which the water is pumped to permit free access to the area within.

Cohesionless Soil - A soil that when unconfined has little or no strength when air-dried and that has little or no cohesion when submerged.

Cohesive Soil - A soil that when unconfined has considerable strength when air-dried and that has significant cohesion when submerged. Clay is a cohesive soil.

Commission - The Oregon Transportation Commission.

Composite Section - Two sections made of the same or different materials together to act as one integral section; such as a concrete slab on a steel or prestressed girder.

Compression Seals - A preformed, compartmented, elastomeric (neoprene) device, which is capable of constantly maintaining a compressive force against the joint interfaces in which it is inserted.

Concept Plans – plans to determine the basic features of a project including alignments, typical sections, slopes, preliminary drainage and TS&L bridge plans.

Concrete Overlay – 1.5” to 2” of concrete placed on top of the deck, used to extend the life of the deck and provide a good riding surface.

Contract - The written agreement between the Division and the contractor describing the work to be done and defining the obligations of the Division and the contractor.

Contract Plans - Detailed drawings and diagrams usually made to scale showing the structure or arrangement, worked out beforehand, to accomplish the construction of a project and object(s).

Contract Time - The number of calendar days shown in the proposal which is allowed for completion of the work.

Contraction Joint - A joint in concrete that does not provide for expansion but allows for contraction or

shrinkage by the opening up of a crack or joint.

Contractor - The individual or legal entity that has entered into a contract with the Division.

Coordinates - Linear or angular dimensions designating the position of a point in relation to a given reference frame. It normally refers to the State Plane Coordinate System.

Core - A cylindrical sample of concrete removed from a bridge component for the purpose of destructive testing.

Counterfort Wall - A reinforced concrete retaining wall whose vertical stem has triangular-shaped ribs projecting into the soil and spaced at regular intervals to provide strength and stability.

Crash Cushion - An impact attenuator device that prevents an errant vehicle from impacting fixed object hazards by gradually decelerating the vehicle to a safe stop or by redirecting the vehicle away from the hazard.

Crash Tests - Vehicular impact tests by which the structural and safety performance of roadside barriers and other highway appurtenances may be determined. Three evaluation criteria are considered, namely (1) structural adequacy, (2) impact severity, and (3) vehicular post-impact trajectory.

Creep - Time dependent inelastic deformation under elastic loading of concrete or steel resulting solely from the presence of stress.

Cross-bracing - Bracing used between stringers and girders to hold them in place and stiffen the structure.

Cross-section - The exact image formed by a plane cutting through an object usually at right angles to a central axis.

Crown Section - Roadway section with the height of the center of the roadway surface above its gutters.

Culvert - Federal Highway Administration definition: “A structure not classified as a bridge having a span of 20 feet or less spanning a watercourse or other opening on a public highway”; a conduit to convey water through an embankment.

Curb - A vertical or sloping member along the edge of a pavement or shoulder forming part of a gutter, strengthening or protecting the edge, and clearly defining the edge of vehicle operators. A curb is a horizontal offset varying from 10” to less than 18”. The surface of the curb facing the general direction of the pavement is called the “face”.

Curing - The preparation of a material by chemical or physical processing for keeping or use; treating concrete by covering its surface with some material to prevent the rapid evaporation of water.

Cut-Off-Wall - A wall built at the end of a culvert apron to prevent the undermining of the apron.

D

Dead End - End of post-tensioned bridge where tendons are anchored but no jacking takes place (opposite of jacking end).

Dead Load - Structure weight including future wearing surface on deck and attachments.

Deadman - A concrete mass, buried in the earth behind a structure, that is used as an anchor for a rod or cable to resist horizontal forces that act on the structure.

Deck – Structural element of the bridge superstructure that directly supports vehicular and pedestrian traffic, distributing load to supporting elements.

Deformed Bars - Concrete reinforcement consisting of steel bars with projections or indentations to increase the mechanical bond between the steel and concrete.

Delamination - Subsurface separation of concrete into layers.

Department - The Department of Transportation of the State of Oregon.

Design Volume or Design Hourly Volume - A volume determined for use in design representing traffic expected to use the highway. Unless otherwise stated, it is an hourly volume.

Diaphragm - Structural: A structural member used to tie adjoining girders together and stiffen them in a lateral direction as well as to distribute loads.

Diamond Grinding - Process to abrade or remove a surface, such as concrete, by the cutting action of rotating circular blade with diamond-tipped teeth.

Direct Tension Indicator - Load-indicating washer for bolts.

Doby - A precast block of concrete of various sizes used to support or provide clearances between reinforcing bars and formwork.

Dolphins - A group of piles or sheet piling driven adjacent to a pier. Their purpose is to prevent extensive damage or possible collapse of a pier from a collision with a ship or barge.

Draped Strands - Strand pattern for prestressing strands, where strands are draped to decrease the prestressing stress at the ends of the girder where the applied moments are small.

Drift Pin - A metal pin, tapered at both ends, used to draw members of a steel structure together by being driven through the corresponding bolt holes.

Drip Groove - A groove formed into the underside of a projecting concrete sill or coping to prevent water from following around the projection.

E

E - modulus of elasticity of a material; the stiffness of a material.

E&C – Engineering & Contingencies. Engineering costs are ODOT's costs to administer the construction contract. Contingencies are unforeseen costs due to construction extra work price agreements or types of problems caused by weather, accidents, etc. by the contract pay item.

Elastomeric Bearing Pads - Pads ½" and less in thickness made of all rubber-like material that supports girders and concrete slabs; pads over ½" in thickness consist of alternate laminations of elastomer and metal.

End-Bearing Pile - A pile which provides support primarily due to reaction at its tip.

Environmental Classes – Classes (1, 2 or 3)

Environmental Class I Environmental Impact Statement: Projects that normally involve significant changes in traffic capacities and patterns. These projects generally involve major right-of-way acquisitions. Both draft and final Environmental Impact Statements are required.

Environmental Class II Categorical Exclusions: Projects that normally involve the improvement of pavement conditions on traffic safety, but little, if any, change in traffic capacities or patterns. Right-of-way requirements must be minor. These projects are categorically excluded from further environmental documentation, unless permit requirements indicate otherwise.

Environmental Class III Environmental Assessment: Projects that do not clearly fall within Class I or Class II. These projects require assessments to determine their environmental significance.

Epoxy - A synthetic resin which cures or hardens by chemical reaction between components which are mixed together shortly before use.

Epoxy Coated Rebar - Steel reinforcement coated with a powdered epoxy resin, to prevent corrosion of the bar steel.

Expansion Bearings - Bearings that allow longitudinal movement of the superstructure relative to the substructure and rotation of the superstructure relative to the substructure.

Expansion Device - A device placed at expansion points in bridge superstructures to carry the vertical bridge loads without preventing longitudinal movement.

Expansion Joint - A joint in concrete that allows expansion due to temperature changes, thereby preventing damage to the structure.

Extra Work - Work not included in any of the contract items as awarded but determined by the Engineer necessary to complete the project according to the intent of the contract. This may be paid on a negotiated price, force account, or established price basis.

Extrados - The curved edge of an arch rib or barrel formed by the intersection of the top and side arch surfaces.

F

Falsework - A temporary construction work on which a main or permanent work is wholly or partially supported until it becomes self-supporting. For cast-in-place concrete or steel construction, it is a structural system to support the vertical and horizontal loads from forms, reinforcing steel, plastic concrete, structural steel, and placement operations.

Fatigue - The tendency of a member to fail at a lower stress when subjected to cyclical loading that when subjected to static loading.

Fatigue Crack - Any crack caused by repeated cyclic loading.

Federal-Aid System of Highways - The national system of interstate highways, Federal-aid highway system, system of secondary and feeder roads, Federal-aid grade crossing projects, federal forest highway systems and projects and other highway and related projects, all within the meaning of the Federal-Aid Road Act

(1916), and all acts amendatory thereof and supplementary thereto, and the federal regulations issued under such acts.

Fender - A structure that acts as a buffer to protect the portions of a bridge exposed to floating debris and water-borne traffic from collision damage.

Fiscal Year - For the State of Oregon, July 1 through June 30 of the next year; for the Federal government, October 1 through September 30 of the next year. The Federal fiscal year (FY) is broken into quarters:
F1Q (October, November, December)
F2Q (January, February, March)
F3Q (April, May, June)
F4Q (July, August, September)

Felloe Guard - Timber curb, usually 10” x 12”, bolted to timber deck and timber rail post. Sometimes called wheel guard.

Filler Plate - A steel plate or shim used for filling in space between compression members.

Fit Condition – Deflected girder geometry associated with a targeted dead load condition for which the cross-frames are detailed to connect to the girders.

Fixed Bearings - Bearings that do not provide for any longitudinal movement of the superstructure relative to the substructure, but allows for rotation of the superstructure relative to the substructure.

Flat Slab - A reinforced concrete superstructure that has a uniform depth throughout.

Flood Plain - An area that would be inundated by a flood.

Floodway - A stream channel plus any adjacent flood plain areas that must be kept free of encroachment so that the 100-year flood can be conveyed without substantial increases in flood heights.

Floor Beam - A transverse structural member that extends from truss to truss or from girder to girder across the bridge.

Flux - A material that protects the weld from oxidation during the fusion process.

Force Account Work - Items of extra work ordered by the Engineer that are to be paid for by material, equipment, and labor.

Forms - A structural system constructed of wood or metal used to contain the horizontal pressures exerted by plastic concrete and retain it in its desired shape until it has hardened.

Fracture Critical – See Nonredundant Steel Tension Members

Fracture Mechanics - Study of crack growth in materials.

Free-Standing Retaining Wall – A retaining wall that is not part of the bridge abutment walls.

Friction Pile - A pile that provides support through friction resistance along the surface area of the pile.

Functionally Obsolete Bridges - Those bridges which have deck geometry, load carrying capacity (comparison of the original design load to the current state legal load), clearance, or approach roadway alignment which no longer meet the usual criteria for the system of which they are a part as defined by the Federal Highway Administration.

G

Gabions - Rock-filled wire baskets used to retain earth and provide erosion control.

Galvanic Action - Electrical current between two unlike metals.

Galvanize - To coat with zinc.

Geotextiles - Sheets of woven or non-woven synthetic polymers or nylon used for drainage and soil stabilization.

Girder - Main longitudinal load carrying member in a structure (beam).

Glare Screen - A device used to shield a driver's eye from the headlights of an oncoming vehicle.

Grade Separation - A crossing of two highways or a highway and a railroad at different levels.

Gravity Wall - A retaining wall that is prevented from overturning by its weight alone.

Green Concrete - Concrete that has set but not appreciably hardened.

Grid Flooring - A steel floor system comprising a lattice pattern which may or may not be filled with concrete.

Grout - A mixture of cementitious material and water having a sufficient water content to render it a free-flowing mass, used for filling (grouting) the joints in masonry, for fixing anchor bolts and for filling post-tensioning ducts.

H

Hammerhead Pier - A pier that has only one column with a cantilever cap and is somewhat similar to the shape of a hammer.

Hanger Plate - A steel plate that connects the pins at hinge points thus transmitting the load through the hinge.

Haunch - An increase in depth of a structural member usually at points of intermediate support.

Haunched Slab - A reinforced concrete superstructure that is haunched (has an increased depth) at the intermediate supports.

Headwall - A concrete structure at the ends of a culvert to retain and protect the embankment slopes, anchor the culvert, and prevent undercutting.

High Performance Concrete (HPC) – Concrete with enhanced properties including higher strength, greater durability and decreased permeability.

High Performance Steel (HPS) - Steel with enhanced properties including increased durability and weldability.

Hinge - A device used to hold the ends of two adjoining girders together, but does not allow for longitudinal movement of the superstructure. A point in a structure where a member is free to rotate.

Holddown Device - A device used on bridge abutments to prevent girders from lifting off their bearings as a result of the passage of live load over the bridge.

Honeycomb - A surface or interior defect in a concrete mass characterized by the lack of mortar between the coarse aggregate particles.

Howe truss - A truss of the parallel chord type with a web system composed of vertical (tension) rods at the panel points with an X pattern of diagonals.

Hydration - The process by which cement combines with water to form a hard binding substance.

Hybrid Girder - A steel plate girder with the web steel having a lower yield strength than the steel in one or both flanges.

Hydrodemolition - Process to abrade or remove a surface, such as concrete, by streams of water ejected from a nozzle at high velocity.

I

Incidental Work - Work necessary for fulfillment of the contract but which is not listed as a pay item in the contract and for which no separate or additional payment will be made.

Initial Set (Concrete) - Initial stiffening of concrete, with time based upon penetration of a weighted test needle. In the field, it is commonly assumed to be the time when the dead weight of vibrator does not penetrate into the concrete.

Inlet Control - The case where the discharge capacity of a culvert is controlled at the culvert entrance by the depth of headwater and the entrance geometry, including barrel shape, cross-sectional area, and inlet edge.

Intermediate Stiffener - A vertical transverse steel member used to stiffen the webs of plate girders between points of support.

Internal File Number - Number assigned by the Bridge Front Office as part of office automation (computerized files) and used to track all files.

Invert - The bottom or lowest point of the internal surface of the transverse cross-section of a pipe.

Inventory Rating (Design Load) - Load level that produces normal design stresses in the structures. The inventory rated load is the load that can safely utilize an existing structure for an indefinite period of time.

International System of Units (SI) - The modernized metric system.

Intrados - The curved edge of an arch rib or barrel formed by the intersection of the bottom and side arch surfaces.

Isotropic - Have the same material properties in all directions, e.g., steel.

J

Jacking End - End of post-tensioned bridge where jacking takes place (opposite of dead end).

Jetting - Forcing water into holes in an embankment to settle or to compact the earth. Forcing water through holes in piles to install the piles to a specified depth before driving.

K

Key Number - Number assigned to a project by Program Section to identify it in the Project Control System (PCS). All structures in a project have the same key number.

Kilogram (kg) - The base unit for mass in the International System of Units (metric).

King Post Truss - Two triangular panels with a common center vertical; the simplest of triangular trusses.

L

Lacing - Small flat plates used to connect individual sections of built up members.

Laitance - A weak mortar that collects at the surface of freshly placed concrete, usually caused by an excess of mixing water or by excessive finishing.

Lamellar Tear - Incipient cracking between the layers of the base material (steel).

Lateral Bracing - Bracing placed in a horizontal plane between steel girders near the bottom or top flanges or both.

Latex Modified Concrete (LMC) - Emulsion of synthetic rubber or plastic obtained by polymerization used as a concrete additive to decrease permeability.

Leaf - The movable portion of a bascule bridge which forms the span of the structure.

Lenticular Truss - A truss having parabolic top and bottom chords curved in opposite directions with their ends meeting at a common joint; also known as a fish belly truss.

Level of Performance - The degree to which a longitudinal barrier, including bridge railing, is designed for containment and redirection of different types of vehicles.

Liquid Penetrant Inspection - Nondestructive inspection process for testing for continuities that are open to the surface, by using a liquid dye.

Live Load - Force of the applied moving load of vehicles and pedestrians.

Load Rating - Evaluation of the safe live load capacity of the weakest member of a bridge.

LRFD - Load Resistance Factor Design.

Longitudinal Stiffener - A longitudinal steel plate (parallel to girder flanges) used to stiffen the webs of welded plate girders. Normally thicker webs are used to eliminate longitudinal stiffeners.

Low Relaxation Strands - Prestressing tendons that are manufactured by subjecting the strands to heat treatment and tensioning causing a permanent elongation. This increases the strand yield strength and reduces strand relaxation under constant tensile stress.

M

Magnetic Particle Inspection (MT) - Nondestructive inspection process for testing for the location of surface cracks or surface discontinuities, by applying dry magnetic particles to a weld area or surface area that has been suitably magnetized.

Microsilica (Silica Fume) (MC) (SFC) - Very fine non-crystalline silica used as an admixture in concrete to improve the strength, permeability and abrasion resistance.

Minor Structure Concrete (MSC) - Nonstructural concrete furnished according to contractor proportioning, placed in minor structures and finished as specified. Previously called commercial concrete.

Modular Expansion Joints - Multiple, watertight joint assemblies for bridges requiring expansion movements greater than 4 inches.

Mud Sill - A timber platform laid on earth as a support for vertical members or bridge falsework.

Mylars - Full-size drawings on mylar. The final “legal” drawing used for signatures and printing contract plans.

N

NDT - Nondestructive testing, a method of checking the structural quality of materials that does not damage them.

Negative Moment - The moment causing tension in the top fibers and compression in the bottom fibers of a structural member.

Negative Reinforcement - Reinforcement placed in concrete to resist negative bending moments.

Newton (N) - The derived unit for force (mass times acceleration or kg times m/s^2) in the International System of Units (metric).

Nominal - Used to designate a theoretical dimension, size, or slope that may vary from the actual by a very small or negligible amount. Example: a 1” nominal diameter steel pipe has an actual 0.957” inside diameter.

Nominal Pile Resistance – LRFD term for the maximum axial pile bearing resistance. Equivalent to the ultimate pile capacity term used in allowable stress design.

Non-Redundant Structure - Type of structure with single load path, where a single fracture in a member can lead to the collapse of the structure.

Nonredundant Steel Tension Members (Previously Fracture Critical) - Members of a bridge where a single

fracture in a member can lead to collapse.

Nosing – A bulkhead at the ends of bridges or at expansion joints made of a durable material to protect and reinforce the slab edge. It also provides a smooth edge or surface at expansion joints to facilitate installation and provide a better seal.

O

Operating Rating (Permit Loads) - The absolute maximum permissible stress level to which a structure may be subjected. It is that stress level that may not be exceeded by the heaviest loads allowed on the structure. Issue special permits for heavier than normal vehicles only if such loads are distributed so as not to produce stress in excess of the operating stress.

Outlet Control - The case where the discharge capacity of a culvert is controlled by the elevation of the tail water in the outlet channel and the slope, roughness, and length of the culvert barrel, in addition to the cross-sectional area and inlet geometry.

Orthotropic - A description of the physical properties of a material that has pronounced differences in two or more directions at right angles to each other.

P

Parapet - A low concrete rail designed and placed to prevent traffic from passing over the edge of a bridge deck or end of box culvert.

Pascal (Pa) - The derived unit for pressure or stress ($Pa=N/m^2$) in the International System of Units (metric).

Paving Dam – (see Nosing) - – A bulkhead at the ends of bridges or at expansion joints made of a durable material to protect and reinforce the slab edge and provide a stopping place for the wearing surface.

Paving Ledge – A ledge or corbel attached to the end beam of a bridge, to provide support for a current or future approach slab.

Performance Level - See Level of Performance.

Pier - Intermediate substructure unit of a bridge. Current terminology is bent.

Pile - A long, slender piece of wood, concrete, or metal to be driven, jetted, or cast-in-place into the earth or river bed to serve as a support or protection.

Pile Bent - A pier where the piles are extended to the pier cap to support the structure.

Pile Cap - A member, usually of reinforced concrete, covering the tops of a group of piles for the purpose of tying them together and transmitting to them as a group the load of the structure that they support.

Pipe Arch - A conduit in the form of a broad arch with a slightly curved integral bottom.

Plastic Deformation - Deformation of material beyond the elastic range.

Positive Moment - In a girder the moment causing compression in the top flange and tension in the bottom flange.

Post-Tensioning - Method of prestressing in which the tendon is tensioned after the concrete has cured.

Pot Bearing - A bearing type that allows for multi-directional rotation by using a neoprene or spherical bearing element.

Prestress Camber - The deflection in prestressed girders (usually upward) due to the application of the prestressing force.

Pratt Truss - A truss with parallel chords and a web system composed of vertical posts with diagonal ties inclined outward and upward from the bottom chord panel points toward the ends of the truss; also known as N-truss.

Preliminary Plans – 85-90% complete plans, normally sent at 20 weeks.

Prestressed Concrete - Concrete in which there have been introduced internal stresses (normally pretensioned steel) of such magnitude and distribution that the stresses resulting from given external loadings are counteracted to a desired degree

Pretensioned - Any method of prestressing in which the strands are tensioned before the concrete is placed.

Project Manager - The Engineer's representative who directly supervises the engineering and administration of a contract.

Proposal - A written offer by a bidder on forms furnished by the Division to do stated work at the prices quoted.

PS&E - Literally, Plans, Specifications, and Estimates. Usually it refers to the time when the plans, specifications, and estimates on a project have been completed and referred to FHWA for approval. When the PS&E has been approved, the project goes from the preliminary engineering phase to the construction phase.

Pumping - The ejection of mixtures of water, clay and silt along or through transverse or longitudinal joints, crack or payment edges, due to vertical movements of the roadway slab under traffic.

Q

Queen-post Truss - A parallel chord type of truss having three panels with the top chord occupying only the length of the center panel; unless center panel diagonals are provided, it is a trussed beam.

R

Radiographic Inspection - Nondestructive inspection process where gamma rays or X rays pass through the object and cast an image of the internal structure onto a sheet of film as the result of density changes.

Redundant Structure - Type of structure with multiple-load paths where a fracture in a single member cannot lead to the collapse of the structure.

Reflection Crack - A crack appearing in a resurfacing or overlay caused by movement at joints or cracks in

the underlying base or surface.

Rehabilitation – Work required to restore the structural integrity of a bridge, or bridge element, as well as work necessary to correct major safety defects. Rehabilitation activities are considered bridge preservation.

Reinforced Pile Tip - Metal reinforcement fastened to the pile tip to protect it during driving.

Replacement – Total reconstruction of a structurally deficient or functionally obsolete bridge, or bridge element, with a new one constructed in the same vicinity. The replacement structure, or element, must comply with current design codes, policies and practices.

Residual Camber - Camber due to the prestressing force minus the dead load deflection of the girder.

Retrofit – Work required to upgrade a bridge, or bridge element, beyond its original intended purpose and design capacity. This work often includes strengthening to add structural capacity.

Right of Way - Land, property, or property interest, usually in a strip, acquired for or devoted to transportation purposes.

Riprap - A facing of stone used to prevent erosion. It is usually dumped into place, but is occasionally placed by hand.

Roadside Barrier - A longitudinal barrier used to shield roadside obstacles or non-traversable terrain features. It may occasionally be used to protect pedestrians from vehicle traffic.

Roadway - The portion of a highway, including shoulders, for vehicular use.

Rubble - Irregularly shaped pieces of varying size stone in the undressed condition obtained from a quarry.

S

Sacrificial Anode - The anode in a cathodic protection system.

Sand - Particles of rock that will pass a No. 4 sieve and be retained on a No. 200 sieve.

Scaffolding - Temporary elevated walkway or platform to support workmen, materials and tools.

Scarify - To loosen, break up, tear up, and partially pulverize the surface of soil, or of a road.

Scour - Erosion of a river bed area caused by water flow.

Scour Protection - Protection of submerged material by steel sheet piling, riprap, mattress, or combination of such methods.

Screeding - The process of striking off excess material to bring the top surface to proper contour and elevation.

Seal - A concrete mass (usually not reinforced) poured under water in a cofferdam that is designed to resist hydrostatic uplift. The seal facilitates construction of the footing in dry conditions.

Shear Connector - A connector used to joint cast-in-place concrete to a steel section and to resist the shear at the connection.

Shear Lag - Nonuniform stress pattern due to ineffective transmission of shear.

Shed Roof - Roadway section with the height of one gutter greater than the centerline and other gutter.

Sheet Pile - A pile made of flat or arch cross-section to be driven into the ground or stream bed and meshed or interlocked with like members to form a wall, or bulkhead.

Sheet Pile Cofferdam - A wall-like barrier composed of driven piling constructed to surround the area to be occupied by a structure and permit dewatering of the enclosure so that the excavation may be produced in the open air.

Shoofly - Detour alignment of temporary railroad track and bridge around the site of a permanent railroad bridge replacement.

Shotcrete - Mortar or concrete pneumatically projected at high velocity onto a surface.

Shoulders - The portions of the roadway between the traveled way and the inside edges of slopes of ditches or fills, exclusive of auxiliary lanes, curbs, and gutters.

Shy Distance (E-Distance) - The distance from the edge of the traveled way beyond which a roadside object will not be perceived as an immediate hazard by the typical driver, to the extent that the vehicle's placement or speed will be changed.

Shrinkage - Contraction of concrete due to drying and chemical changes, dependent on time.

Silt - Soil passing a No. 200 sieve that is non-plastic or exhibits very low plasticity.

Simple Spans - Spans with the main stress carrying members non-continuous, or broken, at the intermediate supports.

Skew or Skew Angle - The acute angle formed by the intersection of a line normal to the centerline of the roadway with a line parallel to the face of the abutments or piers, or in the case of culverts with the centerline of the culverts. Left hand forward skew indicates that, look up station, the left side of the structure is further up station that the right hand side. Right hand skew indicates that the right side of structure is further up station that the left side.

Slab - A structural concrete element that provides a flat horizontal surface to distribute loads into a pressure that can be supported by the bearing capacity of the underlying soil substrate.

Sleeper slab - Used at the roadway end of the approach slab to support the reinforced approach slab and provide a smooth connection to the pavement.

Slip Base - A structural element at or near the bottom of a post or pole that will allow release of the post from its base upon impact while resisting wind loads.

Slope - The degree of inclination to the horizontal. It is sometimes described by such adjectives as steep, moderate, gentle, mild or flat.

Slope Paving - Pavement placed on the slope in front of abutment to prevent soil erosion.

Soffit - The bottom surface of a beam or an arch rib or barrel.

Spandrel - The area between the roadway and the arch in the side view of an arch bridge.

Special Provisions - The special directions, provisions, and requirements peculiar to the project that augment the standard specifications. They are commonly referred to as “specials”.

Specifications - The body of directions, provisions, and requirements, together with written agreements and all documents of any description, made or to be made, pertaining to the method or manner of performing the work, the quantities, and the quality of materials to be furnished under the contract.

Spread Footing - A footing that is supported directly by soil or rock.

Spur Dike - A wall or mound built or extended out from the upstream side of an abutment used for training the stream flow to prevent erosion of stream bank. May also be used where there is no bridge, but the stream flows along the side of highway embankment.

Stainless Steel Teflon Bearings - Incorporated stainless steel and teflon with steel to provide the necessary expansion movement.

St. Venant Torsion - Uniform torsion resulting in no deformation of the cross-section.

State Plane Coordinates - The plane-rectangular coordinate system established by the United States Coast and Geodetic Survey. Plane coordinates are used to locate geographic position.

Station - A distance of 100 feet measured horizontally.

Stirrup - Vertical U-shaped or rectangular shaped bars placed in concrete beams to resist the shearing stresses in the beam.

Strengthening – Work to add structural capacity to a bridge element or structure.

Stress Relieved Strands - Any prestressing tendons that are manufactured by relieving the high residual stresses that were introduced into the steel during the wire drawing and stranding operations. Stress relieving is not a heat treatment and does not change the strand yield strength.

Strip Seal Joint - Molded neoprene glands inserted and mechanically locked between armored interfaces of extruded steel sections.

Structurally Deficient Bridges - Those bridges which have been (1) restricted to light vehicles only, (2) closed, or (3) require immediate rehabilitation to remain open, as defined by the Federal Highway Administration.

Subgrade - The top surface of completed earthwork on which subbase, base, surfacing, pavement, or a course of other material is to be placed.

Substructure - Those parts of a structure which support the superstructure, including bents, piers, abutments, and integrally built wingwalls, up to the surfaces on which bearing devices rest. Substructure also includes portions above bearing surfaces when those portions are built integrally with a substructure unit (e.g. backwalls of abutments). When substructure and superstructure elements are built integrally, the division between substructure and superstructure is considered to be at the bottom soffit of the longitudinal or transverse beam, whichever is lower. Culverts and rigid frames are considered to be entirely substructure.

Sufficiency Rating (SR) - A method of evaluating data by calculating four separate factors to obtain a numeric value which is indicative of bridge sufficiency to remain in service. The result of this method is a percentage in which 100 percent would represent an entirely sufficient bridge and zero percent would

represent an entirely insufficient or deficient bridge.

Superelevation - The difference in elevation between the inside and outside edges of a roadway in a horizontal curve; required to counteract the effects of centrifugal force.

Superplasticizer - A high range water-reducing admixture that increases the slump of freshly mixed concrete without increasing the water content.

Superstructure - Those parts of a structure above the substructure, including bearing devices.

Surcharge - Any load that causes thrust on a retaining wall, other than backfill to the level of the top of the wall. Also preloading of an embankment to minimize the time for initial consolidation of the subsurface soils.

Suspension Bridge - A bridge in which the floor system is supported by catenary cables which are supported upon towers and are anchored at their extreme ends.

Suspender - A wire cable, metal rod or bar connected to a catenary cable of a suspension bridge at one end and the bridge floor system at the other, thus transferring loads from the roadway to the main suspension members.

T

Tack Welds - Small welds used for temporary connections.

Telltale (Tattletale) - Any device designed to indicate movement of formwork or falsework.

Tendon - A name for prestressed reinforcing element whether wires, bars, or strands.

Tenon - A constant diameter extension welded to the tip of the tapered metal arm of a luminaire support pole to receive the luminaire.

Thixotropy - Property of a material that enables it to stiffen in a short period on standing, but to acquire a lower viscosity again on mechanical agitation. A property desirable for post-tensioning duct grout.

Three-Dimensional Finite Element Analysis - Analysis in which a three-dimensional continuum is modeled as an assemblage of discrete elements in three-dimensional space.

Three-Hinged Arch - An arch which is hinged at each support and at the crown.

Through Structure - A structure that has its floor connected to the lower portion of the main stress-carrying members, so that the bracing goes over the traffic. A structure whose main supporting members project above the deck or surface.

Tining - Is used on finished concrete deck or slab surfaces to provide friction and reduce hydroplaning. Grooves are placed in the plastic concrete or cut into the hardened concrete.

Torsional Stress - Shear stress on a transverse cross-section resulting from a twisting action.

Transformed Section - A hypothetical section of one material so as to have the same elastic properties as a section of two materials.

Transition - A section of barrier between two different barriers or, more commonly, where a roadside barrier is connected to a bridge railing or to a rigid object such as a bridge pier. The transition should produce a gradual stiffening of the approach rail so vehicular pocketing, snagging, or penetration at the connection can be avoided.

Traveled Way - The portion of the roadway for the movement of vehicles, exclusive of shoulders and auxiliary lanes.

Tremie - A pipe or tube through which concrete is deposited underwater.

Trial Batch - A batch of concrete prepared to establish or check proportions of the constituents.

Turnbuckle - A long, cylindrical, internally threaded nut used to connect the elements of adjustable rod and bar members.

Turn-of-the-Nut - A bolt-tightening method.

Two-hinged Arch - A rigid frame which may be arch-shaped or rectangular but is hinged at both supports.

U

Ultrasonic Inspection - A non-destructive inspection process where by an ultra-high frequency sound wave induced into a material is picked up in reflection from any interface or boundary.

Unbonded Strands - Strands so coated as to prevent their forming a bond with surrounding concrete. Used to reduce stress at the ends of a member.

Underpinning - The addition of new permanent support to existing foundations to provide additional capacity.

Uplift - A force tending to raise a structure or part of a structure and usually caused by wind and eccentric loads, or the passage of live-load over the structure.

Utility - A line, facility, or system for producing, transmitting, or distributing communications, power, electricity, heat, gas, oil, water, steam, waste, storm water not connected with highway drainage, or any other similar commodity which directly or indirectly serves the public. The term utility shall also mean the utility company, district, or cooperative, including any wholly owned or controlled subsidiary.

V

Vierendeel Truss - A Pratt truss without diagonal members and with rigid joints between top and bottom chords and the verticals.

Vibrator - An oscillating device inserted at selected locations to consolidate fresh concrete.

W

Wales - Horizontal support members in close contact with a row of sheet piles in a cofferdam or shoring

wall. Sometimes called whalers.

Warrants - The criteria by which the need for a safety treatment or improvement can be determined.

Warren Truss - A triangular truss consisting of sloping members between the top and bottom chords and no verticals; members form the letter W.

Water/Cement Ratio - The weight of water divided by the weight of cement in a concrete; ratio controls the strength of the concrete.

Waterproofing Membranes - Impervious material overlaid with bituminous concrete to protect decks from the infiltration of chlorides and resulting deterioration.

Wearing Surface - The top layer of a pavement designed to provide structural values and a surface resistant to traffic abrasion.

Weep Hole - A drain hole through a wall to prevent the building up of hydraulic pressure behind the wall.

Weld Inspection - Covers the process, written procedure, and welding in process. Post weld heat maintenance if required, post weld visual inspection and non-destructive testing as specified in contract and Standard Specifications.

Welded Wire Reinforcing - A two-way reinforcing mat, fabricated from cold-drawn steel wire, having parallel longitudinal wires welded at regular intervals to parallel transverse wires.

Well-Graded - An aggregate possessing a proportionate distribution of successive particle sizes.

Wetlands - Areas that are inundated or saturated by surface or ground water at a frequency and duration sufficient to support, and that under normal circumstances do support, vegetation typically adapted for life in saturated soil conditions. Wetlands generally include swamps, marshes, bogs, and similar areas.

Wheel Load – Half of an axle load.

Wingwall - A wall attached to the abutments of bridges or box culverts retaining the roadway fill. The sloping retaining walls on each side of the center part of a bridge abutment.

X

Y

Yield - Permanent deformation (permanent set) which a metal piece takes when it is stressed beyond the elastic limit.

Young's Modulus - modulus of elasticity of a material (E); or the stiffness of a material.

Z

APPENDIX – SECTION 1 – Abbreviations (Initialisms and Acronyms)

A

A&E	Architectural and Engineering
AADT	Annual Average Daily Traffic
AASHTO	American Association of State Highway and Transportation Officials (since 1973)
ABC	Accelerated Bridge Construction
AC	Asphalt Concrete
ACI	American Concrete Institute
ACP	Asphalt Concrete Pavement
ACT	Area Commission on Transportation
ACWS	Asphalt concrete wearing surface
ADA	Americans with Disabilities Act
ADT	Average daily traffic (see Definitions)
ADTT	Average Daily Truck Traffic
AEE	Association of Engineering Employees
AHP	Analytic Hierarchy Process
AISC	American Institute of Steel Construction
a.k.a.	Also known as
ANSI	American National Standards Institute
API	Area of Potential Impact
APJ	Asphaltic Plug Joint
APM	Area Project Manager Agency Project Manager
APWA	American Public Works Association
AREA	American Railway Engineering Association
AREMA	Manual for Railway Engineering of the American Railway Engineering and Maintenance-of-way Association
ARS	Accident Records System (Accident Data Unit, Transportation Research Section)
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
AWS	American Welding Society

B

B-Team	Team of Bridge Engineering Section Supervisors & Engineers
BART	Biological Activity Reaction Test
BCM	Bridge CAD Manual
BDC	Bridge Design Coordinator
BDM	Bridge Design Manual
BDS	Bridge Design System (AASHTO software)
BDS	Bridge Data System (ODOT software)
BDWO	Bridge Design Work Order
BLT	Bridge Leadership Team
BMDM	Bridge Maintenance Design Manual
BMP	Best Management Practice
BNSF	Burlington Northern Sante Fe Railroad
BPR	Bureau of Public Roads (now FHWA)
BR	Bridge
BRASS	Bridge Rating and Analysis of Structural Systems (software)
Brg.	Bearing
BRSFUP	Bridge Rail Safety Features Upgrade Program
Bt.	Bent

C

C	Degrees Celsius Compression
CAD	Computer-aided drafting
CAFL	Constant Amplitude Fatigue Limit
CalTrans	California Department of Transportation
CBC	Conventional Bridge Construction
CBR	Major Steel Bridges
CF	Cubic feet
CFRP	Carbon Fiber Reinforced Polymer
CFS	Cubic Feet per Second
CG	Center of Gravity
CICS	Customer Information and Control System (Transportation inventory and Mapping Unit software on the mainframe)
CIP	Cast-in-place

CL	Centerline
	Clear
CMP	Construction Mitigation Plan
	Construction Management Plan
	Corrugated metal pipe
CMR	Change Management Request
CPM	Critical Path Method (method of scheduling)
	Consultant Project Manager
CQCR	Comments, Questions, Concerns and Requests
CQP	Consultant Quality Plan
CSL	Crosshole Sonic Log
CSZE	Cascadia Subduction Zone Earthquake
CTP	Continuous Trip Permit
Ctr.	Center
CUP	Conditional Use Permit
CWI	Certified Welding Inspector
CY	Cubic yard

D

D	Depth
DAP	Design Acceptance Plans
DAW	Design Acceptance Workshop
DBE	Disadvantaged Business Enterprises
DCSA	Bridge Design Criteria and Standards Assessment
DD	Bridge Design Deviation
DE	Design Exception
Dia.	Diameter
DIS	Dynamic Isolation System, Inc.
DL	Dead load
DOS	Disk Operating System for personal computers
DOT	Department of Transportation
DQP	Design Quality Plan

E

E&C	Engineering and Contingencies (used in cost estimates)

EAC	Emulsified Asphalt Concrete
EB	Eastbound
El.	Elevation
Elev.	Elevation
EOR	Engineer of Record
ESR	Evaluation Service Report
Extg.	Existing

F

F	Degrees Fahrenheit
FEMA	Federal Emergency Management Agency
FF	Far face (don't use for "fill face")
FFO	Full Federal Oversight
FHWA	Federal Highway Administration (formerly BPR)
FRP	Fiber Reinforced Polymer
ft-k	foot-kips
ft-lbs	foot-pounds
FWS	Future Wearing Surface

G

Galv.	Galvanized
GDM	Geotechnical Design Manual
GFRP	Glass Fiber Reinforced Polymer
GIS	Geographic Information System
GPR	Ground Penetrating Radar
GR	Grade
GSD	AASHTO Guide Specifications for Seismic Design

H

HDM	Highway Design Manual Hydraulics Design Manual
HPC	High Performance Concrete
HPS	High Performance Steel
HQ	Headquarters

I

IBC	International Building Code
IC	Internal Curing
ID	Inside diameter Identification
IGA	Inter-Governmental Agreement
ISA	Internal Shear Anchor
ITS	Intelligent Transportation Systems

J

Jt.	Joint

K

k	Kilo, one thousand
K	Kip (kilopound, 1000 pounds)
kg	Kilogram, metric mass unit
km	Kilometer (1000 meters)
kN	KiloNewton, metric force unit
KSF	Kips per Square Foot
KSI	Kips per Square Inch

L

LAG	Local Agency Guidelines
Lbs	Pounds
LF	Linear feet
LL	Live load
LMC	Latex Modified Concrete
LPA	Local Public Agency
LRFD	Load Resistance Factor Design
LRFD	*AASHTO LRFD Bridge Design Specifications*
L.S.	Lump Sum

M

m	Meter, metric length unit

	Milli, one thousandth
M	Mega, one million
MASH	Manual for Assessing Safety Hardware
Max.	Maximum
MBJS	Modular Bridge Joint System
MBM	Major Bridge Maintenance
MC	Microsilica concrete Miscellaneous Channels
MCTD	Motor Carrier Transportation Division
Min.	Minimum
MP	Milepoint, milepost (even milepoint)
MPCO	Multi-Layer Polymer Concrete Overlay
MOT	Maintenance of Traffic
MSC	Minor structure concrete
MSCS	Management Scheduling Control System (to replace PCS)
MSE	Mechanically Stabilized Earth (retaining walls)
MT	Magnetic Particle

N

N	Newton, metric force unit
NACE	National Association of Corrosion Engineers
NBI	National Bridge Inventory
NBIS	National Bridge Inspection Standards
NCHRP	National Cooperative Highway Research Program (from the Transportation Research Board)
NDE	Non-destructive Evaluation
NDT	Non-destructive Testing
NEPA	National Environmental Protection Act of 1969
NHI	National Highway Institute
NHS	National Highway System
NLF	No Load Fit
NRST	Nonredundant Steel Members
NSBA	National Steel Bridge Alliance
NSM	Near Surface Mount
NTS	Not to Scale

O

OC	On Center (center-to-center)
OD	Outside Diameter
ODOT	Oregon Department of Transportation
ODLAP	ODOT Delivery of Local Agency Projects
OHWM	Ordinary High Water Mark
ORS	Oregon Revised Statutes
OSHA	Occupational Safety and Health Administration (U.S.)

P

Pa	Pascal, metric stress or pressure unit
PA	Price Agreement
PCC	Portland Cement Concrete
	Point on compound curve
PCF	Pounds per Cubic Foot
PCI	Prestressed Concrete Institute
PCS	Project Control System (to be replaced by MSCS)
PD	Project Delivery
PDG	Project Delivery Guide
PDLT	Project Delivery Leadership Team
PE	Professional Engineer (registered)
PFSJ	Precompressed Foam Silicone Joint
PL	Project Leader
	Plate
PM	Project Manager
POR	Professional of Record
PPC	Premixed Polymer Concrete
	Precast Prestressed Concrete
PQR	Pre-Qualification Request
	Public Relations
P/S	Prestressed Concrete
PS&E	Plans, Specifications & Estimate
PSBS	Project Specifications Bid System
PSDC	Project Specific Design Criteria
PSF	Pounds per Square Foot

	Precast Spread Footing
PSI	Pounds per Square Inch
P/T	Post-tensioned concrete
PVC	Polyvinyl chloride
PWS	Present Wearing Surface

Q

QA	Quality Assurance
QC	Quality Control
QPL	Qualified Products Listing

R

R	Radius
R, 1R	Resurfacing
R/W	Right of Way
RBI	Region Bridge Inspector
RBLE	Regional Bridge Lead Engineer
RCBC	Reinforced Concrete Box Culvert
RCDG	Reinforced Concrete Deck Girder
Rdwy.	Roadway
RE-CP	Resident Engineer – Consultant Projects
RFI	Request for Information
RFP	Request for Proposals
RR	Railroad
RRR, 3R	Resurfacing, Restoration and Rehabilitation
RRRR, 4R	Resurfacing, Restoration, Rehabilitation and Reconstruction
RT	Radiographic

S

SAFETEA-LU	Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy of Users of 2005
SDC	Seismic Design Category
SDCL	Simple for Dead and Continuous for Live Load
SDLF	Steel Dead Load Fit
SF	Square feet

SFC	Silica Fume Concrete
SFLMC	Silica Fume Latex-Modified Concrete
SHPO	State Historic Preservation Office
SIDL	Superimposed Dead Load
Sl.	Slope
SOW	Statement of Work
SP	*ODOT Construction Specifications*
Spcg.	Spacing
Spcs.	Spaces
SPMT	Self-Propelled Modular Transporters
SPT	Standard Penetration Test for soils
SR	Sufficiency Rating
Std.	Standard
STIP	State Transportation Improvement Program
STP	Single Trip Permit
STRUDL	Structural Design Language
SY	Square Yard

T

T&E	Threatened and Endangered
TCP	Traffic Control Plan
TcP	Time cost per person
TDLF	Total Dead Load Fit
TiAB	Titanium Alloy Bars
TIP	Thermal Integrity Profile
TL	Test Level
TP&DT	Temporary Protection and Direction of Traffic
TPAR	Temporary Pedestrian Accessible Routes
TPM	Transportation Project Manager
TS&L	Type, Size and Location (formerly called preliminary)
TSF	Tons per Square Foot (don't use)
Typ.	Typical

U

UBIT	Under Bridge Inspection Truck

UHPC	Ultra High Performance Concrete
UPRR	Union Pacific Railroad
USCG	United States Coast Guard
USGS	United States Geological Survey
UST	Underground Storage Tank
UT	Ultrasonic

V

Var.	Varies
VE	Value Engineering

W

WOC	Work Order Contract
WPS	Welding Procedure Specifications
WS	Wearing surface
WSDOT	Washington State Department of Transportation
WSC	Wire Strand Core (cables)
Wt.	Weight

X

Y

Z

APPENDIX – Section 1.2 – AASHTO/BDM Cross-Reference

Table A1.2-1 AASHTO/BDM Cross Reference

Bridge Design Manual		AASHTO LRFD Cross-References		
BDM Section	**Title**	**LRFD Section**	**Title**	**Comments**
2.14.7.2 2.18.3	Bridge Length	2.6.4.3	Bridge Waterway	BDM adds specific design floods and minimum freeboard to AASHTO specs.
2.18.2(2)	Structure Depth	2.5.2.6.3-1	Span to Depth Ratios	BDM gives span-to-depth ratios for concrete bridges but leaves span-to-depth ratios for steel bridges to AASHTO.
2.18.2(2)	Structure Depth	2.5.2.6.2	Criteria For Deflection	BDM states that AASHTO optional live load deflection criteria is not required for bridges that satisfy the span-to-depth ratios in BDM ~~2.5.2.6.3-1~~ 2.18.2(4)
2.17.1(1)	Bridge Types and Economics	4.6.2.1.4, 5.14.4.1, 9.7.1.4	Slab Edge Beam Requirements	BDM Yields to AASHTO Requirements. AASHTO requirements also apply to CIP voided slabs if design deviation is approved.
2.14.7.1	Hydraulics, General	2.6.4.4.2	Bridge Scour	BDM states that bottom of spread footings should be 6 feet below normal streambed. AASHTO states that the bottom of footing should be below the scour depth.
1.10.5.3	Spread Footing Foundation Design	10.6.1.2	Bearing Depth	BDM states that spread footings should be at least 6 feet below streambed and also below the scour depth for the 500-year flood event. AASHTO states that the footings should be located to bear below the maximum anticipated depth of scour.

Bridge Design Manual		AASHTO LRFD Cross-References		
BDM Section	**Title**	**LRFD Section**	**Title**	**Comments**
1.10.5.3(2)	Nominal and Factored Bearing Resistances	10.5.5.2.2	Spread Footings	BDM resistance factors for bearing of spread footings are higher than those shown in Table 10.5.5.2.2 in the AASHTO Specs for extreme event conditions of scour and earthquake loading.
1.10.5.3(4)	Spread Footing Stability	11.6.2.3	Overall Stability	BDM specifies a factor of safety 1.5 for overall stability. AASHTO specifies phi factors for stability = 0.75 or 0.65 depending on geotechnical information.
1.10.5.4.1(1)	Pile Resistance	10.7.3.8	Nominal Axial Pile Resistance	BDM refers specifically to AASHTO specs for determining axial pile capacity.
1.10.5.5.1(1)	Drilled Shaft Diameters and Requirements	10.8.1.3	Shaft Diameter	BDM states that smallest shaft diameter is 12 inches. AASHTO adds that if the shaft is to be manually inspected, the diameter should not be less than 30 inches.
1.10.5.5.1(1)	Column Diameter	10.8.1.3	Maximum Column Size	AASHTO states that columns on top of drilled shafts can be the same size as the drilled shaft, but BDM requires that columns be smaller than shafts by 6 inch or 1 foot depending on shaft diameter.
1.10.5.5.1(6)	Shaft Settlement	10.8.2.2	Shaft Settlement	BDM refers to AASHTO for the determination of drilled shaft settlement.
1.10.5.5.1(11)	Shaft Reinforcement	5.13.4.6.3	Volumetric Reinforcement Ratio and transverse rebar spacing	BDM overrides LRFD 5.13.4.6.3 because the shaft diameter is always larger than the column diameter.
1.10.5.5.1(11)	Shaft Reinforcement	5.13.4.6.3	Volumetric Reinforcement Ratio and transverse rebar spacing	BDM adds a formula for computing transverse reinforcement required in non-contact splice region.

Bridge Design Manual		AASHTO LRFD Cross-References		
BDM Section	**Title**	**LRFD Section**	**Title**	**Comments**
1.10.5.5.1(1)	Shaft Reinforcement Cover	5.12.3-1	Cover for Main Reinforcing Steel	BDM provides specific reinforcement cover requirements for drilled shafts.
1.3.2	Dead Loads	3.5.1-1	Unit Weights	BDM assumes 150 pcf for ACP, but AASHTO assumes 140 pcf.
1.3.4	Construction and Temporary Loads	3.4.3.1	Jacking Forces	BDM modified permanent load factor to 1.5 from AASHTO's 1.3.
1.3.8	Thermal Forces	3.12.2.1-1	Temperature Ranges	Temperature ranges vary slightly between BDM and AASHTO for Climates (Sections)
1.3.1	Ductility, Redundancy, and Operational Importance	1.3.3, 1.3.4, 1.3.5	Ductility, Redundancy, and Operational Importance	BDM requires State Bridge Engineer approval for Redundancy Factor less than 1.0, and it states that for the Operational Importance Factor, use a value of 1.0 for all bridges, assuming all bridges to be "typical".
1.3.5	Sidewalk Loading	3.6.1.6	Pedestrian Loads	BDM adds specific details for applying vehicular live load to curb mountable sidewalks.
1.11.2.3	Wingwall Design and Construction	11.6.1.5.2	Wingwalls	BDM adds bar extension requirements for abutments on stiff footings to distribute flexure. AASHTO does state that bar lengths should vary to avoid "planes of weakness"
1.17	Seismic Design	3.10, 5.10.11, 5.13.4.6, 11.6.5, 11.8.6, 11.10.7	Earthquake Effects	BDM specifies the use of AASHTO Guide Specifications for LRFD Seismic Bridge Design for projects initiated after May 1st 2009. For projects initiated before May 1st 2010, BDM specifies the use of AASHTO LRFD Bridge Design Specifications. Additional requirements and guidelines for

Bridge Design Manual		AASHTO LRFD Cross-References		
BDM Section	**Title**	**LRFD Section**	**Title**	**Comments**
				both AASHTO documents are included in the BDM.
1.5.1	Concrete, General	5.4.2.1	Compressive Strength	BDM has independent concrete classes.
1.5.1	Concrete, General	5.4.2.4	Modulus of Elasticity	BDM and AASHTO have same formula
1.5.5-2	Minimum Bar Covering	5.12.3	Concrete Cover	BDM and AASHTO have separate tables
1.5.5.1	Reinforcement for Shrinkage and Temperature	5.10.8	Shrinkage and Temperature Reinforcement	BDM bases reinforcement area on concrete thickness only, but AASHTO bases reinforcement area on ratio of volume of section to perimeter of section. AASHTO also has additional required minimum spacing for specific elements.
1.5.5.4	Minimum Bar Spacing	5.10.3	Spacing of Reinforcement	BDM specifies 2.5d for the bar spacing, but AASHTO specifies 1.5d for clear distance between bars. Both state 1.5 inches minimum clear between bars and 1.5(maximum aggregate size) for minimum clear between bars.
1.5.5.5	Compression Development Length	5.11.2.2.1	Compression Development Length	AASHTO gives two equations with the one that results in the lowest value controlling. BDM specifies the largest value from the equations.
1.5.6.1	Design of Precast Prestressed Elements	5.9.4.1.2-1, 5.9.4.2.2-1	Tensile Stress Limits	AASHTO allows 0.19sqrt(f'c) for certain situations but BDM allows only 0.0948sqrt(f'c)
1.5.8.6	Post-Tension Strand Duct Placement	5.10.3.3.2	Post-Tensioning Ducts C-C Spacing	AASHTO specifies spacing requirements; BDM doesn't call out spacing requirements.
1.14.1.2	Elastomeric Bearing Pads	14.7.5, 14.7.6	Elastomeric Pads and Steel Reinforced	BDM specifies that AASHTO Method A should be used to design bearing pads unless there is

Bridge Design Manual		AASHTO LRFD Cross-References		
BDM Section	**Title**	**LRFD Section**	**Title**	**Comments**
			Elastomeric Pads	a specific need to use AASHTO Method B.
1.14.1.3	Proprietary Pot, Disc, Slide, Radial, or Spherical Bearings	5.4.2.3	Shrinkage and Creep	BDM provides a simplified approach for determining creep and shrinkage coefficients.
1.9.2.1.2	Reinforcing	9.7.2	Empirical Design of Decks	BDM excludes the use of the Empirical Method in AASHTO for deck design
1.9.2.1.2	Reinforcing	4.6.2.1	Decks	BDM STD details with deck design tables utilize AASHTO specifications from LRFD 4.6.2.1 to develop reinforcement values.
1.14.2.2	Deck Expansion Joint Seals	14.5.6.6	Compression and Cellular Seals	AASHTO specifies a maximum skew angle for compression joint seals equal to 20 degrees, but this limitation is not stated in the BDM or standard drawing.
1.13.1	Rail Selection	13.7.2	Test Level Selection Criteria	AASHTO Defines Test Level Criteria
1.13.1	Rail Selection	A13.2	Traffic Rail Design Forces	AASHTO provides design forces in rails in Table A13.2-1
1.6.2.8.3	Steel Girders	C6.13.6.1.4a	Flexural Members	BDM modifies this AASHTO comment to reflect moment of inertia of the smaller section rather than the smaller flange.
1.8.5	(Timber) Preservative Treatment	8.4.3.2	(Wood) Treatment Chemicals	AASHTO has specific requirements for the use of wood preservative chemicals on pedestrian bridges.
1.12.3	Sound walls, General	1-2.1.2 (AASHTO Guide Specs for Sound Barriers)	Wind Load	AASHTO Guide Specs for Sound Barriers provides wind load equations and exposure categories. Example designs are also provided

Bridge Design Manual		AASHTO LRFD Cross-References		
BDM Section	**Title**	**LRFD Section**	**Title**	**Comments**
1.12.3	Sound wall Seismic Load	1-2.1.3 (AASHTO Guide Specs for Sound Barriers)	Seismic Load	AASHTO Guide Specs for Sound Barriers provides equations and factors for seismic loads on sound barriers.
1.12.3	Sound wall Overturning Factor of Safety	1-8.2 (AASHTO Guide Specs for Sound Barriers)	Spread Footings	BDM uses AASHTO factors of safety with slight modifications. (Ice and snow load not included)
2.14.4.2	Roadway Clearances	2.3.3.3	Highway Horizontal Clearances	AASHTO calls out horizontal clearance requirements that are consistent with the values shown in BDM Figure 2.14.4.2A-B
1.16.3.3	Falsework	Figure 16 (AASHTO Construction Handbook for Bridge Temporary Works)	Bridge Deck Falsework	See AASHTO Figure 16 for conceptual layout of deck overhang falsework.
1.16.3.5.2	Cofferdams Seals	Page 71 (AASHTO Construction Handbook for Bridge Temporary Works)	Sealing and Buoyancy Control	AASHTO confirms that force from sheet pile friction should not be included in uplift resistance.
1.3.6	Vehicular Collision Forces	3.6.5	Vehicular Collision Force	BDM adds specific requirements for barriers in front of obstacle components.

APPENDIX – Section 1.5.11.2.2 – Resin Anchor Design

This appendix contains the legacy equations used by designers prior to 2019, which were replaced by the equations according to *ACI 318 Chapter 17* with ODOT modifications. Use the legacy equations only when it is required.

GENERAL EQUATION FOR RESIN TENSION CAPACITY

Ultimate tension capacity = $R_0 * R_1 * R_2 * \pi * D * E * [U(max) - (35\ lb/in^3 * E)]$
where:

π = Pi = 3.14159
D = anchor diameter (inches)
E = anchor embedment (inches)
U(max) = 1400 psi for "low strength" resin
= 2300 psi for "high strength" resin

R_0 = reduction factor for non-redundant applications. This applies when anchors are used with only two anchors per attachment.
R_0 = 0.85 for non-redundant horizontal applications
R_0 = 1.0 for all other applications

R_1 = reduction factor due to edge distance
R_1 = 1.0 - (1.5-A)/2.5 when edge distance < 1.5 * E
R_1 = 1.0 when edge distance ≥ 1.5 * E
where A = edge distance/E

R_2 = reduction factor due to anchor spacing
R_2 = 1.0 - (1.0-B)/1.7 when anchor spacing < 1.0 * E
R_2 = 1.0 when anchor spacing ≥ 1.0 * E
where B = anchor spacing/E

Specify edge distance and anchor spacing greater than $6 * D$ or $0.5 * E$, whichever is greater.

When rebar is anchor material, add two times the anchor diameter to the required anchor embedment. This extra embedment is necessary for rebar since the exact location of rebar deformations cannot be known. Most of the tension load in a rebar anchor is transferred to the concrete at the deformation location. For this reason fully-threaded anchors are generally preferred for most resin-bonded anchor applications.

For horizontal applications, add 20 percent to the required anchor embedment. This extra embedment is necessary since full resin coverage cannot be assured for horizontal applications. Horizontal applications angled down a minimum of 15 degrees do not require the additional 20 percent.

RESIN TENSION EQUATION: SERVICE LOADS

Ultimate tension capacity ≥ 3 * design tension load

RESIN TENSION EQUATION: SEISMIC LOADS

Ultimate tension capacity:

≥1.9 * design seismic load for "low strength" resin

≥1.6 * design seismic load for "high strength" resin

Note: for seismic loading, maximum rod loading ≤ 0.9 Fy

Fy = Yield strength of the anchor rod

RESIN TENSION EQUATION: LRFD LOADS

0.5 * Ultimate tension capacity ≥ factored load

GENERAL EQUATION FOR RESIN SHEAR CAPACITY

Ultimate Shear Capacity = $R1 * R2 * \lambda * D * E * f'c$ where:

- D = anchor diameter (inches)
- E = anchor embedment (inches)
- f'c = compressive strength of concrete
- λ = 0.75 for "low strength" resin
- λ = 1.0 for "high strength" resin
- R_1 = reduction factor due to edge distance
  - $R1 = 1.0 - (1.5 - A)/2.0$ when edge distance < 1.5 * E
  - $R1 = 1.0$ when edge distance ≥ 1.5 * E
- R_2 = reduction factor due to anchor spacing
  - $R2 = 1.0 - (1.0 - B)/1.7$ when anchor spacing < 1.0 * E
  - $R2 = 1.0$ when anchor spacing ≥ 1.0 * E

If concrete for an existing structure appears to be in good condition, use f'c = 1.2 times the concrete strength shown on the existing plans.

RESIN SHEAR EQUATION: SERVICE LOADS

Ultimate shear capacity ≥ 3 * design shear load

RESIN SHEAR EQUATION: SEISMIC LOADS

Ultimate shear capacity ≥ 1.7 * design seismic shear load

RESIN SHEAR EQUATION: LRFD LOADS

0.5 * Ultimate shear capacity ≥ factored load

COMBINED RESIN TENSION AND SHEAR

Combined Stress Ratio (CSR) ≤ 1.0

$CSR = (ft / Ft) + (fv / Fv)2$

f_t , f_v = factored loads (i.e., the right side of service load, seismic, or LRFD equations)

F_t , F_v = capacities (i.e., the left side of service load, seismic, or LRFD equations)

APPENDIX – Section 1.11 – Substructures

Figure A1.11.2.9-1 End Bent Details for Prestressed Slabs and Boxes

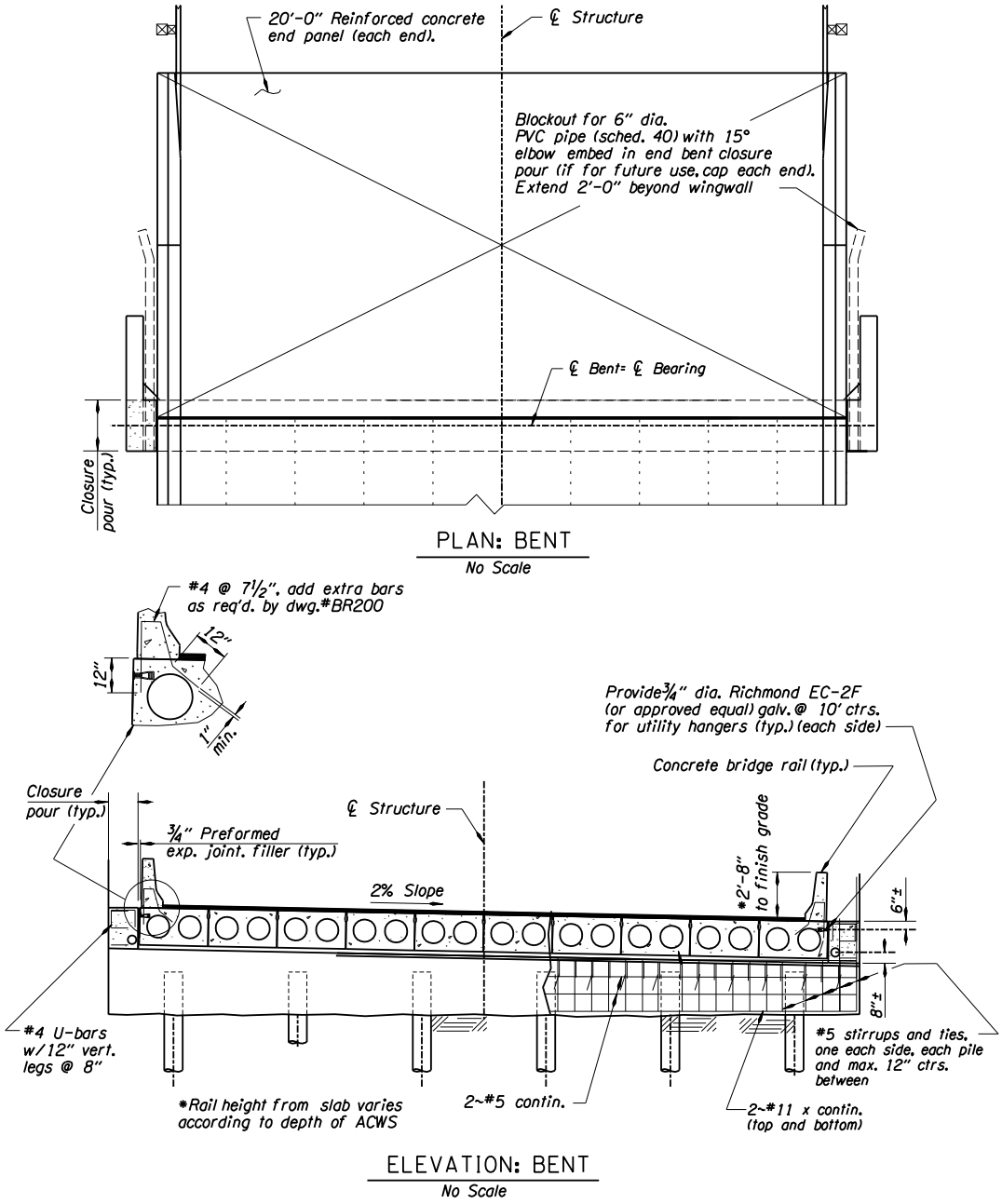
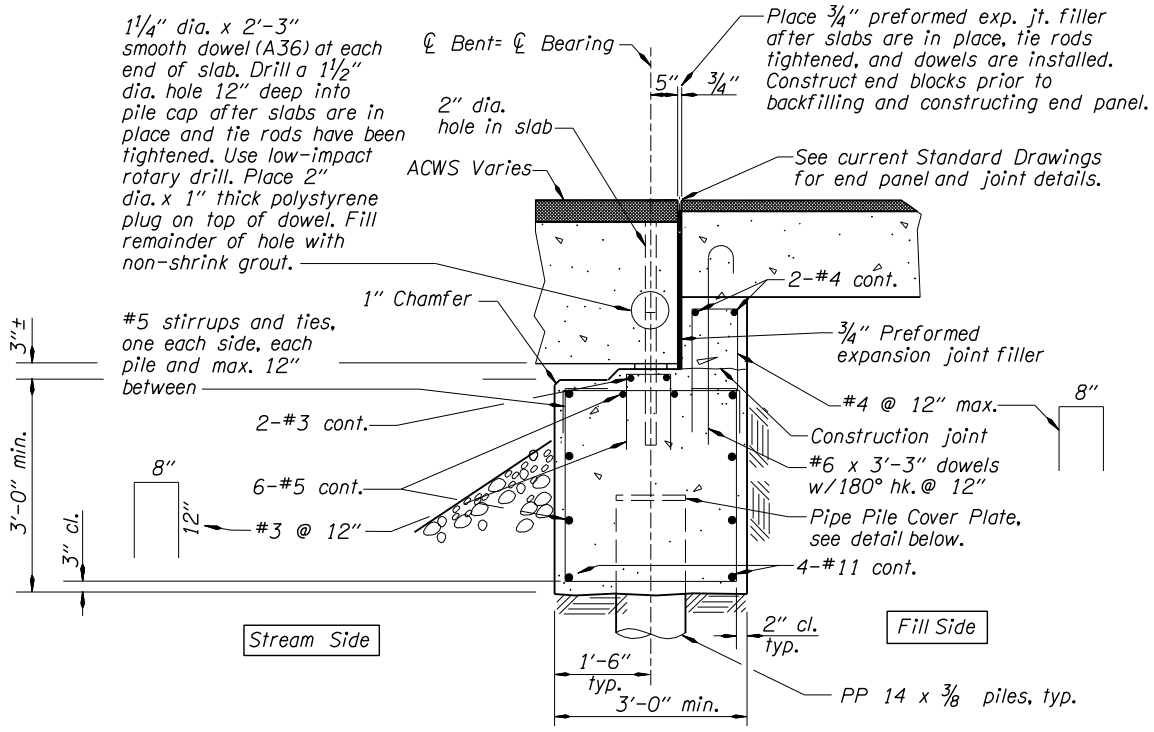


Figure A1.11.2.9-2



TYPICAL BENT SECTION

Scale ¾" = 1'-0"

Figure A1.11.2.9-3

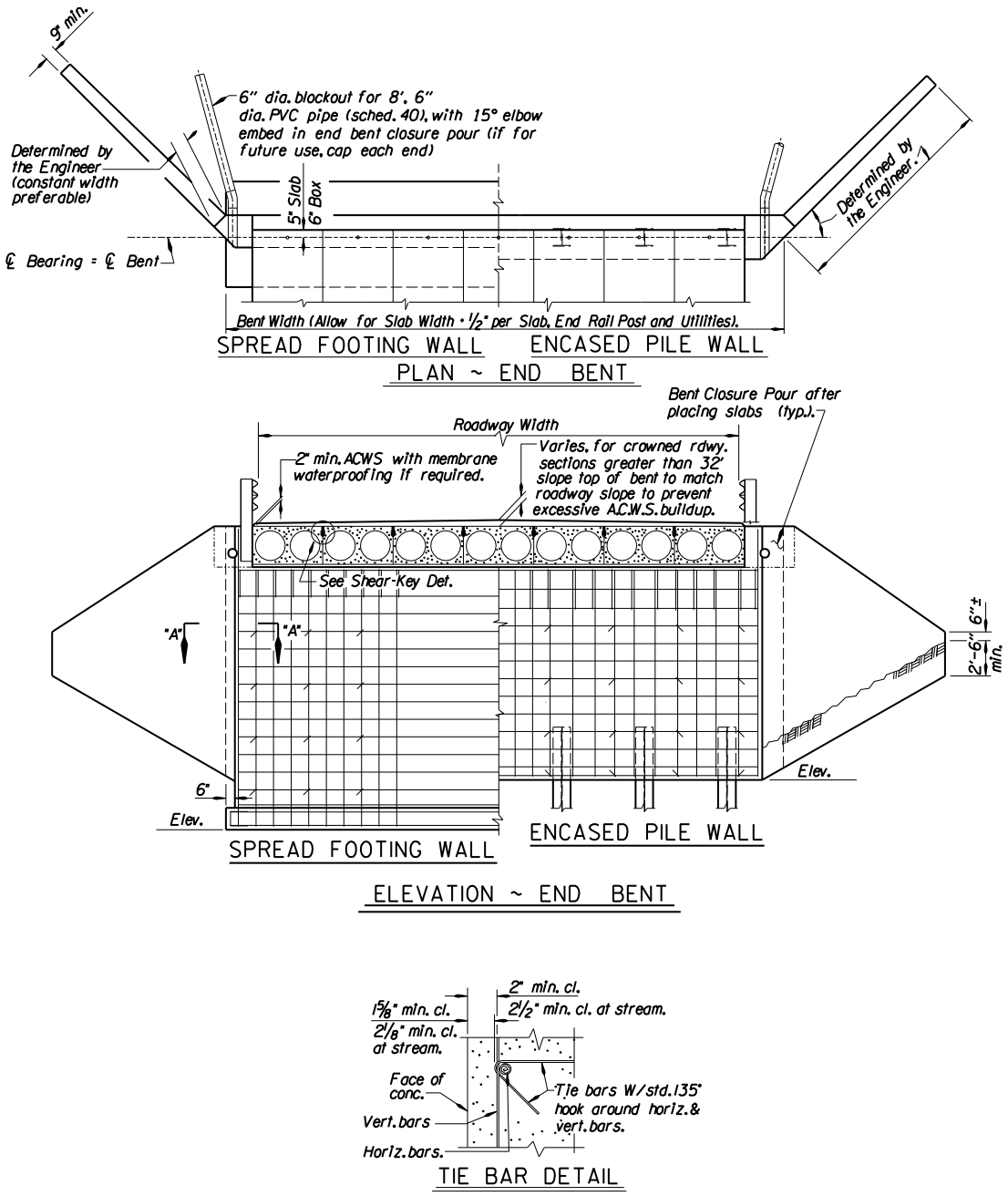
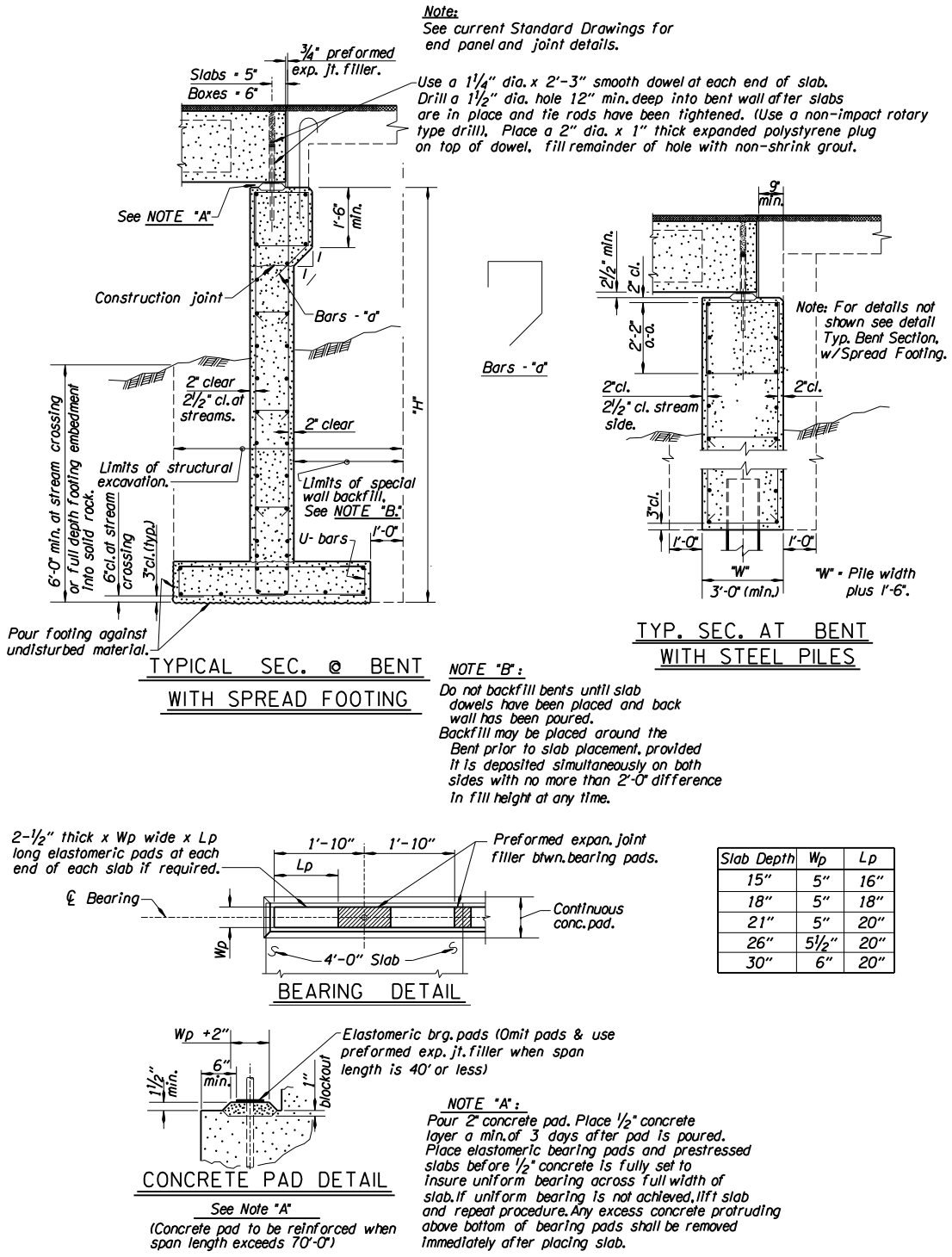


Figure A1.11.2.9-4



NOTE "B":
Do not backfill bents until slab dowels have been placed and back wall has been poured.
Backfill may be placed around the Bent prior to slab placement, provided it is deposited simultaneously on both sides with no more than 2'-0" difference in fill height at any time.

Slab Depth	Wp	Lp
15"	5"	16"
18"	5"	18"
21"	5"	20"
26"	5 1/2"	20"
30"	6"	20"

NOTE "A":
Pour 2" concrete pad. Place 1/2" concrete layer a min. of 3 days after pad is poured. Place elastomeric bearing pads and prestressed slabs before 1/2" concrete is fully set to insure uniform bearing across full width of slab. If uniform bearing is not achieved, lift slab and repeat procedure. Any excess concrete protruding above bottom of bearing pads shall be removed immediately after placing slab.

Figure A1.11.2.9-5

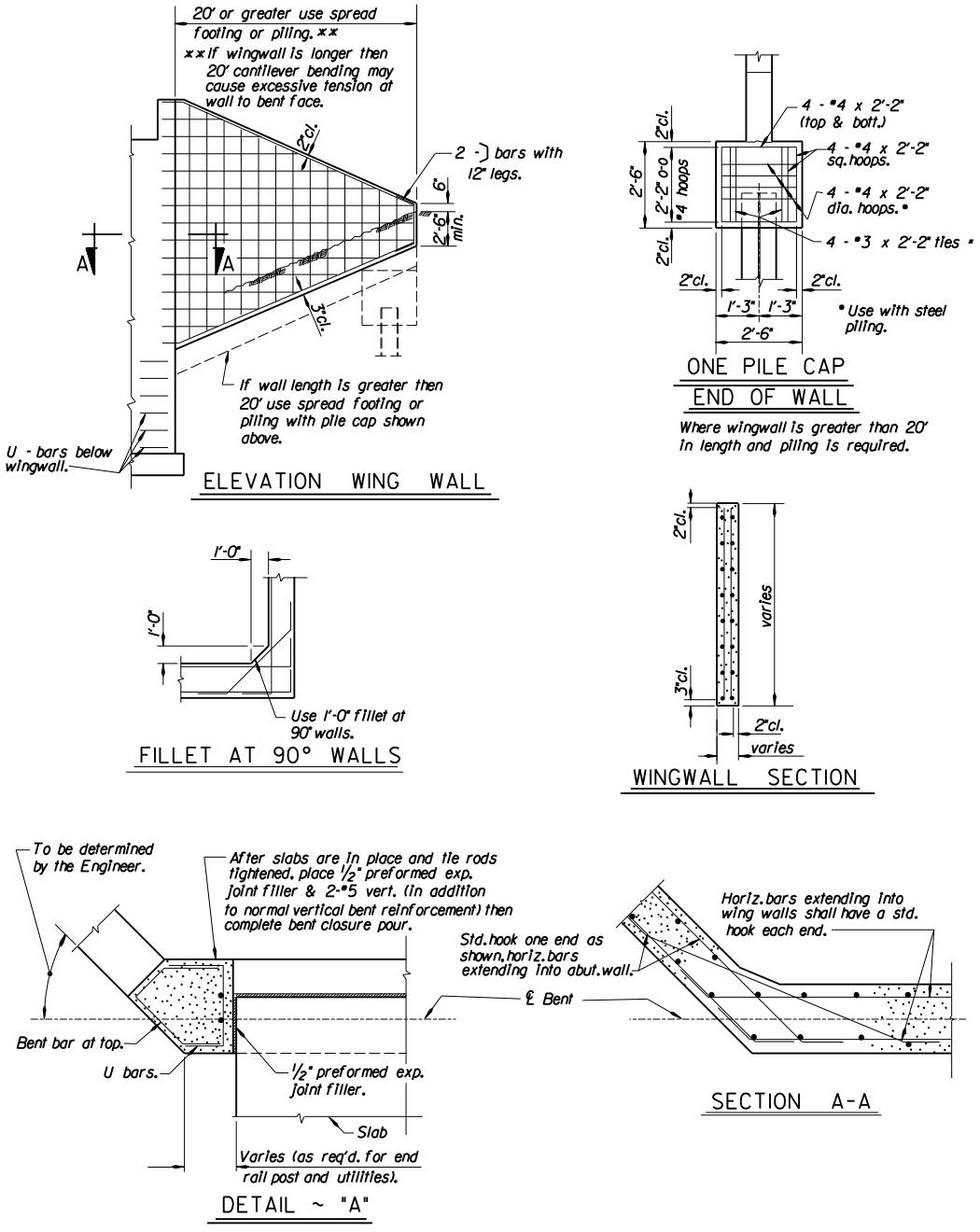
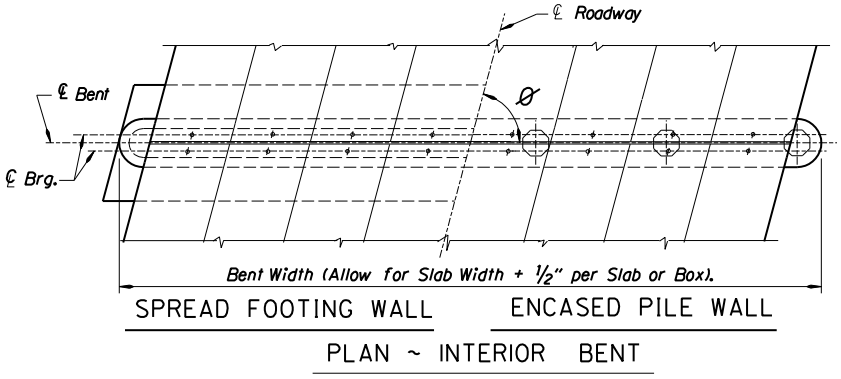
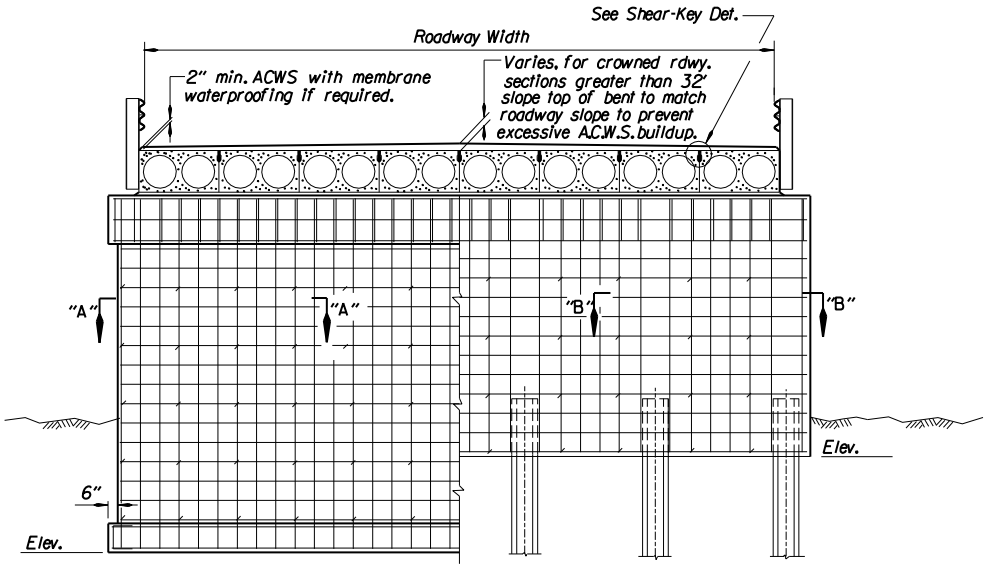


Figure A1.11.3.2-1 Interior Bent Details for Prestressed Slabs and Boxes



PLAN ~ INTERIOR BENT



ELEVATION ~ INTERIOR BENT

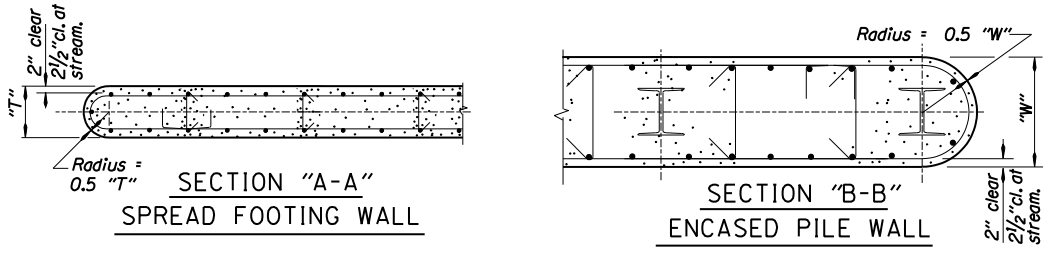


Figure A1.11.3.2-2

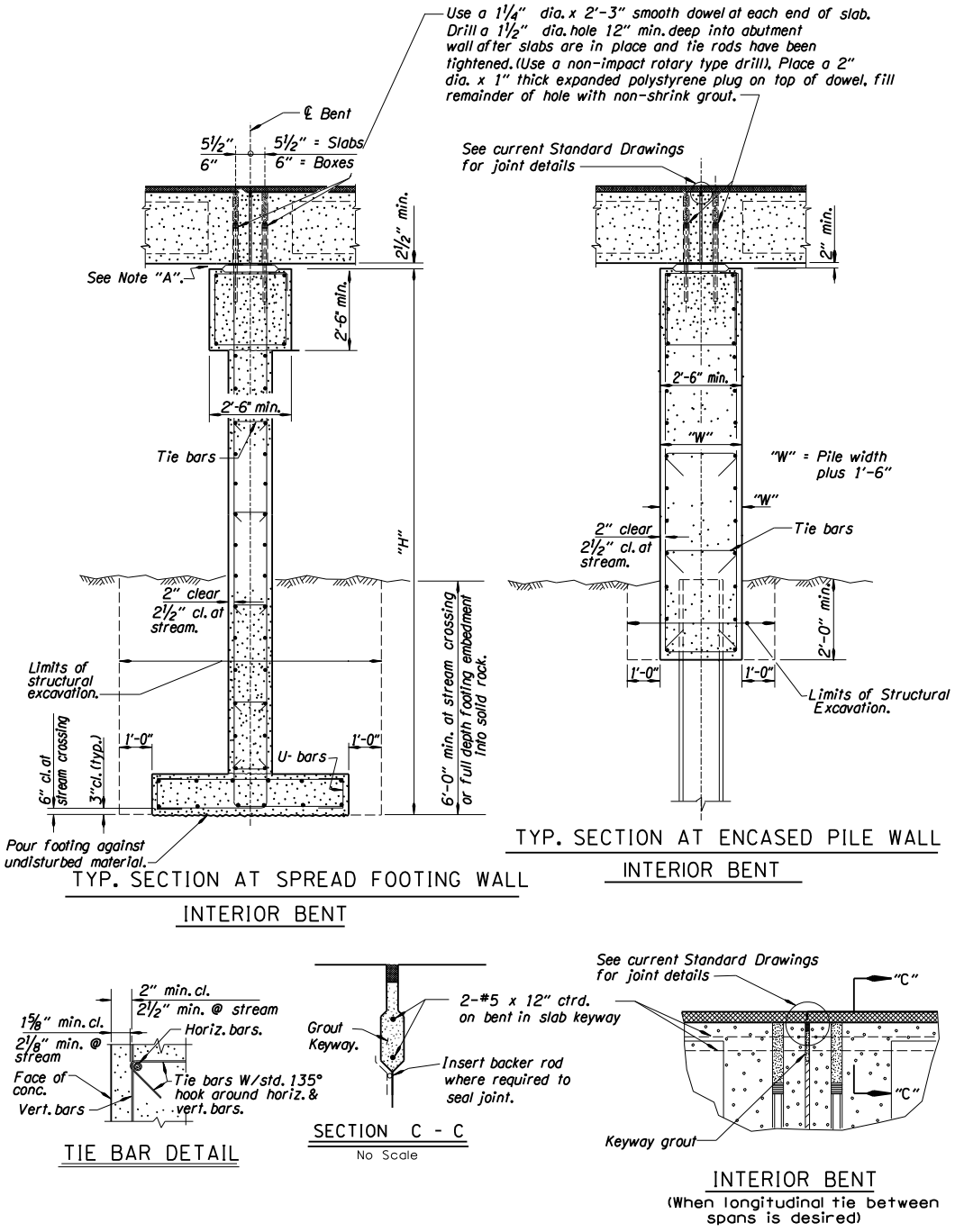
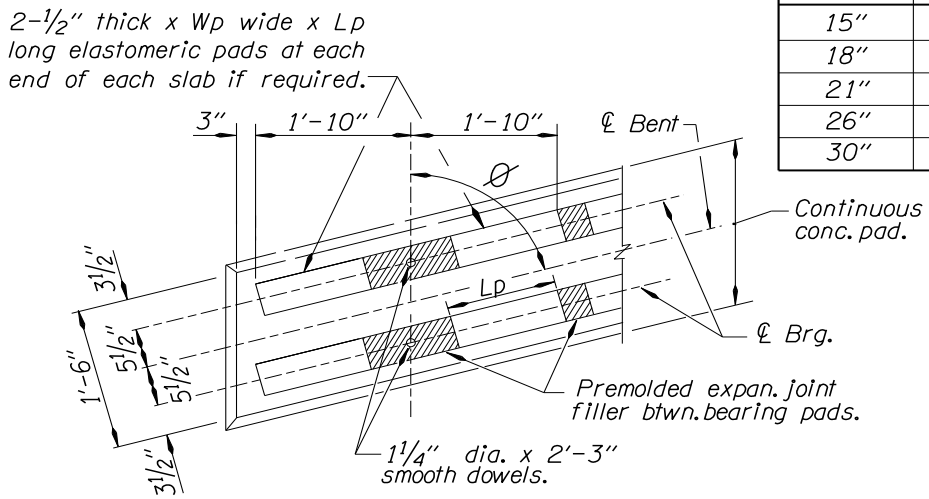
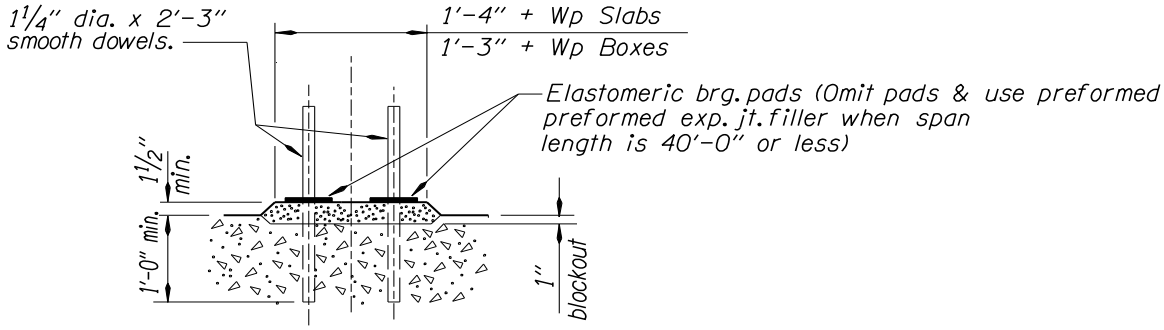


Figure A1.11.3.2-3

Slab Depth	Wp	Lp
15″	5″	16″
18″	5″	18″
21″	5″	20″
26″	5½″	20″
30″	6″	20″



BEARING DETAIL
(PRESTRESSED SLABS)



CONCRETE PAD DETAIL

See Note "A" below

(Concrete pad to be reinforced when length exceeds 70′)

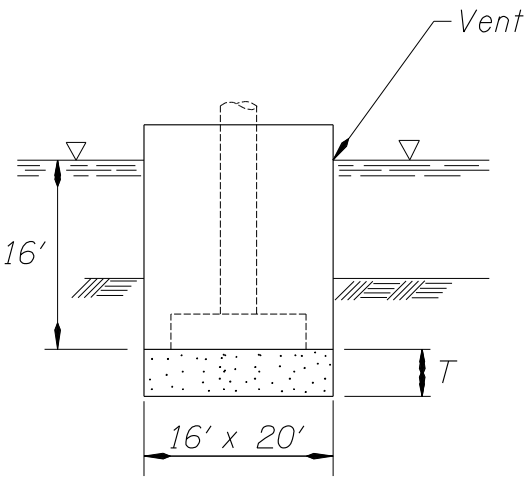
NOTE "A":

Pour 2″ concrete pad. Place ½″ concrete layer a min. of 3 days after pad is poured. Place elastomeric bearing pads and prestressed slabs before ½″ concrete is fully set to insure uniform bearing across full width of slab. If uniform bearing is not achieved, lift slab and repeat procedure. Remove any excess concrete protruding above bottom of bearing pads immediately after placing slab.

APPENDIX – Section 1.16.3 – Bridge Temporary Works

SPREAD FOOTING EXAMPLE (USING METHOD 1):

Figure A1.16.3.5.2-1 Method 1 Spread Footing



Determine the seal thickness for a 16′ x 20′ cofferdam. Water depth is 16 feet from the vent to the top of the seal.

$$Estimated\ T = 0.4 * (16' + 10' est.thickness) = 10.4'$$

Summing vertical forces:

$$Uplift\ force = weight\ of\ water\ displaced$$

$$Uplift\ force = (Area) * (Depth\ of\ Water) * (Unit\ force\ of\ water)$$

$$Uplift\ force = (16') * (20') * (16' water\ depth) * (0.0624\ k/ft^3)$$

$$Force\ of\ seal = buoyant\ force\ of\ the\ seal$$

$$Force\ of\ seal = (16') * (20') * (T' seal\ thickness) * (0.15\ k/ft^3 - 0.0624\ k/ft^3)$$

Uplift force = Force of seal

Solving for T:

$T = 11.4'$

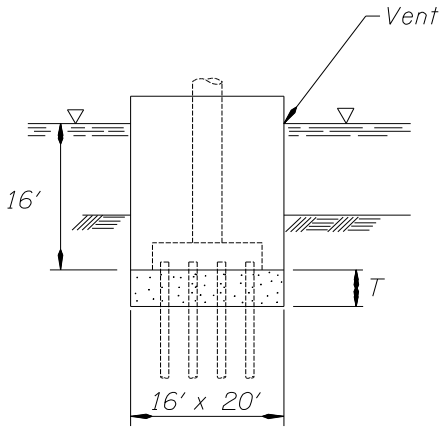
Use 11.5′ seal thickness.

Note: F.S = 1.0 for T = 11.4′

PILE-SUPPORTED EXAMPLE (USING METHOD 1):

Determine the seal thickness for a 16′ x 20′ cofferdam, with 12 – 12″ diameter steel piles. Uplift capacity is 10 kips per pile. Water depth is 16 feet from the vent to the top of the seal.

Figure A1.16.3.5.2-2 Method 1 Pile Supported



$Estimated\ T = 0.25 * (16' + 10' est.thickness) = 6.5'$

Summing vertical forces:

$Uplift\ force = weight\ of\ water\ displaced$

$Uplift\ force = (16') * (20') * (16' water\ depth) * (0.0624\ k/ft^3)$

$Uplift\ force = 319.49k$

$Weight\ of\ seal = buoyant\ force\ of\ the\ seal$

$Weight\ of\ seal = (16') * (20') * (T' seal\ thickness) * (0.15\ k/ft^3 - 0.0624\ k/ft^3)$

$Weight\ of\ seal = 28.03 * (T)\ k/ft$

$Pile\ displaced\ concrete = (12\ pile) * (0.785\ ft^2) * (T') * (0.15 - 0.0624)\ k/ft^3$

$Pile\ displaced\ concrete = 0.825 * (T)\ k/ft$

$Bond\ on\ piles = (12\ pile) * \pi * 1' * 6.5' * 0.010\ ksi * 144\ in^2/ft^2$

$Bond\ on\ piles = 352.86\ k$

$Pile\ uplift\ capacity = (12\ pile) * (10\ k/pile)$

$Pile\ uplift\ capacity = 120\ k < 352.86\ k$

Use 120 k

$Uplift\ force = (Seal\ weight) - (Pile\ disp.conc.) + (Pile\ uplift\ capacity)$

$319.49\ k = 28.03\ (T) - 0.825\ (T) + 120$

Solving for T:

$T = 7.33'$

Use 7.5′ seal thickness.

Note: F.S. = 1.0 for T = 7.33′