CHAPTER 6
SUPERSTRUCTURES

6-1 Guidelines for Selecting Type of Superstructure

Bridges shall be designed as continuous or continuous for live load, whenever possible. Regardless of superstructure type, a concerted effort shall be made to minimize the number of joints. All bridges shall be designed in accordance with the AASHTO Standard Specifications criteria for Seismic Performance Category A or B. Refer to Figure 2-4 to determine whether a bridge is located in Seismic Performance Category A or B.

Many details in this manual are for simple spans and may not be applicable for continuous spans, unless specifically indicated.

When it is necessary to haul very long or heavy prestressed concrete or steel girders into remote areas, access routes should be checked to make reasonably certain that limited load capacities of existing bridges or sharp curves do not prevent the shipment of these girders to the bridge site. Since it is not feasible to transport AASHTO Type V and VI prestressed concrete girders over land due to weight limitations, these girders should only be used for ~~larger~~ coastal structures that are accessible to barge traffic. If restrictions exist, place a note on the plans to draw the Contractor's attention to the existing conditions. Section 105-15 of the Standard Specifications addresses restrictions of load limits in the vicinity of the project.

If steel is selected for the superstructure, the use of AASHTO M270 Grade 50W (345W) or HPS 70W (HPS 485W) weathering steel is preferred to painted structural steel when atmospheric corrosion is not a problem. For restrictions on the use of weathering steel, see Section 12-13.

AASHTO M270 Grade 50W shall typically be used for plate girders. However, for continuous structures, a hybrid combination of HPS 70W in the flanges of the negative moment region and Grade 50W steel in other areas results in the optimum use of HPS and should be considered if girder spacing can be increased in order to eliminate a girder line.

In general, design two span bridges over divided highways and one span bridges in lieu of three span bridges over non-divided highways. Early coordination with Roadway Design is necessary to ensure that vertical alignments provide adequate clearance for economical superstructure depths. For estimated superstructure depths, as provided to both the Roadway Design and Hydraulics Units, see Figure 6-1.

Generally, for stream crossings, the use of prestressed concrete girders is preferred. However, since the use of prestressed concrete is often limited by the span lengths and free board, consideration should be given at each site for the most feasible span arrangement and type.

For short span stream crossings, prestressed concrete cored slab bridges are more economical than continuous cast-in-place deck slab bridges. Only when conditions are contrary to the general guidelines described in Section 6-4 should consideration be given to the use of continuous cast-in-place deck slab bridges.

6-2 Decks

General The clear width for new bridges on streets with curb and gutter approaches shall be the same as the curb to curb approach width except where sidewalk or bikeways are carried across the structure. The 2'-0" (610 mm) gutter widths are based on the use of the standard 2'-6" (760 mm) curb and gutter. If other curb and gutter widths are used, bridge widths shall be adjusted accordingly.

Follow the Roadway plans and Structure Recommendations for crown drops for all bridges, superelevated or non-superelevated, except for special cases such as wide roadways and curb and gutter approaches. For superelevated sections with curb and gutter approaches, continue the superelevation to the gutter on both sides. When the roadway crown of dual lanes is sloped from the inside edge of pavement, the bridge crown should also be sloped from this point.

The pay item for deck slab shall be "Reinforced Concrete Deck Slab" on a square foot (square meter) basis. Compute the area to the nearest square foot (tenth of a square meter). The quantities for Class AA concrete and the reinforcing steel for the deck slab shall be shown in the Superstructure Bill of Material.

For all bridges except cored slabs greater than 1500 feet in length, a profilograph test on the final deck surface is required. Place the following note on the plans:

For Bridge Deck Rideability and Grooving, see Special Provisions.

The riding surface of reinforced concrete bridge floors not within 18 inches (460 mm) of the gutter lines and 2 inches (50 mm) of expansion joints shall be grooved. Approach slabs that do not contain an asphalt overlay shall be grooved to the same limits as the bridge floors. The pay item for this work shall be "Grooving Bridge Floors" on a square foot (square meter) basis.

Steel superstructure plans shall be detailed for metal stay-in-place forms. Prestressed concrete girder plans shall be detailed for precast prestressed concrete panels except as noted in this section under "Cast-in-Place Concrete Decks, Precast Prestressed Concrete Panels". The ~~Special Provisions allow the~~

Contractor may~~, at his~~ opt~~ion,~~ to use removable forms for steel span structures and removable forms or metal stay-in-place forms for prestressed girder spans. For the note to be placed on the General Drawing, see Section 5-2 “General”.

For corrosion protection of bridge decks, see Section 12-13.

Cast-in-Place Concrete Decks

Slabs Supported on Beams or Girders

Use the office standard slab design tables as shown in Figures 6-2 though 6-5 for designing slabs to carry a HS20 (MS18) live load. Limit the overhang widths from the centerline of girder to edge of superstructure to the suggested maximums shown in Figure 6-6. Figures 6-7 and 6-8 may be used to summarize the slab design and determine the required beam bolster heights.

For a specified beam or girder spacing, the slab design tables provide the total slab thickness, main reinforcement (top and bottom ‘A’ bars), longitudinal reinforcement (bottom ‘B’ bars) and the size of beam bolsters upper (BBU). The tables are based on Grade 60 (Grade 420) reinforcing steel and a concrete compressive strength of 3500 psi (24.1 MPa). The top ‘A’ bars in the slab have been designed for continuity over several supports and have been analyzed for cantilever action in overhangs consistent with Figure 6-6. If plan details are not consistent with these conditions, the designer must check to determine whether the overhang loads control the design of top ‘A’ bars. There will be some conditions, such as superelevated sections with large horizontal curve offsets, bridges on sag vertical curves, or increased girder camber that will require an increase in the slab thickness or buildup.

Longitudinal steel in the top of slab for prestressed concrete girder superstructures shall be as follows:

- Simple Spans - #4 ~~(#13)~~ bars at 1'-6" (#13 bars at~~#13 bars at~~ 450 mm) centers with metal stay-in-place forms or #4 ~~(#13)~~ bars at 9" (#13 bars at~~#13 bars at~~ 220 mm) centers with prestressed concrete deck panels
- Continuous Spans - See Section 6-3 “Continuous for Live Load Deck Slabs”

In prestressed concrete girder spans, place the following note on plans:

Longitudinal steel may be shifted slightly, as necessary, to avoid interference with stirrups in prestressed concrete girders.

Longitudinal steel in the top of slab for structural steel superstructures shall be as follows:

- Simple Spans - #4 ~~(#13)~~ bars at 1'-6" (#13 bars at~~#13 bars at~~ 450 mm) centers

- Continuous Spans - Follow the AASHTO Standard Specifications

The main reinforcement should be set to provide 2 ½ inches (65 mm) clear from top of slab and 1 ¼ inches (32 mm) clear from bottom of slab or the top of the metal stay-in-place forms.

The main reinforcing steel is to be placed perpendicular to the chords for all horizontally curved bridges regardless of the skew.

For skews less than 60º or greater than 120º, detail three #6 (#19) 'A' bars in the top of the slab for the acute corners of deck slabs. These bars shall be placed parallel to the joint, spaced at 6 inches (150 mm), and extended beyond the centerline of the first interior girder.

If beam or girder spacings are closer than usual, thereby resulting in a thin slab and light slab steel, a check shall be made to determine if slab steel is adequate to the resist the load from the railing.

Slabs as Main Supporting Members (Cast-in-Place Deck Slab Bridges)

Design these spans in accordance with the AASHTO Standard Specifications. The main reinforcement should be set to provide 2 ½ inches (65 mm) clear from top of slab and 1 ¼ inches (~~30~~ 32 mm) clear from bottom of slab and the beam bolster spacing shall be 1'-6" (450 mm).

Metal Stay-In-Place Forms

Metal stay-in-place forms shall be used for all structural steel spans and ~~for~~ prestressed concrete girder spans ~~outside the limits for precast panels~~in non-corrosive sites.

For continuous steel beam or girder spans, place the following note on the plans:

Metal Stay-in-Place Forms shall not be welded to beam or girder flanges in the zones requiring Charpy V-~~-~~Notch test. See Structural Steel Detail Sheets.

The approval for metal stay-in-place forms shall conform to the ~~Special Provisions of the project~~Standard Specifications. The standard procedure as outlined in Figures 6-9 and 6-10 should be used for checking the forms. No overstress or excessive deflection of the form or support angle shall be permitted.

Precast Prestressed Concrete Panels

When precast prestressed concrete panels are used, the Contractor is responsible for the design and details of the panels and the submittal of the plans for approval.

Prestressed concrete deck panels shall be used only on prestressed concrete girders and only within the following limits:

- Skew limits as shown in Figure 6-11. Spacings greater than 8'-6" (2.59 m) should be checked for skew allowance.
- Girder build-ups less than 5" (125 mm)
- Structures with girder lines less than 2 inches (50 mm) out of parallel from bent to bent.
- Maximum superelevation of 0.05.
- When total length of structure exceeds 250 feet (75 m) for stream crossing, check for floating water access. If floating water access in unavailable, do not use prestressed panels.
- For projects requiring staged construction and a positively connected temporary bridge rail, detail metal stay-in-place forms.
- For projects with sidewalks requiring deck drains, detail metal stay-in-place forms.

If the 4 foot (1.22 m) wide panel skew limit as given in Figure 6-11 is the only limitation exceeded, place the following note on the plans:

> ***The skewed end conditions of Span ___ at Bent No. ___ are such that the use of 4' (****1.22 m)* ***wide prestressed concrete deck panels is not possible; however, the use of 8' (****2.44 m)* ***wide prestressed concrete deck panels is possible.***

~~If the above conditions are not met, and the structure is not at a Corrosive Site, detail the plans for metal stay-in-place forms.~~ If the structure is at a Corrosive Site and the above conditions are not met, see Section 12-13 for guidance.

The general guidelines for plan preparation incorporating prestressed concrete deck panels are as follows:

- The Standard PDP1~~SM~~, "Precast Prestressed Concrete Deck Panels", shall be used. The Contractor has the option of using either a grout bed or a polystyrene support system.
- The longitudinal steel in the cast-in-place portion of the slab shall be #4 bars at 9" (#13 bars at 220 mm) centers with simple span girders. For longitudinal reinforcing in continuous deck slabs, see Section 6-3 "Continuous for Live Load Deck Slabs".
- The top bars shall be supported above the top of the precast panels by beam bolsters at 3'-0" (1.0 m) centers. See Figure 6-76 for illustration.

- In the overhang of the slab, specify #4 bars at 1'-6" (#13 bars at 450 mm) centers for the bottom layer of transverse reinforcement detailed with two bar supports.

~~Place the following note on the plans:~~

> ~~***For removal of falsework on bent diaphragms, see Special Provisions for Prestressed Concrete Panels.***~~

- When prestressed concrete panels are used at a Corrosive Site, see Section 12-13.

Steel Grid Floors

For design guidance, see the AASHTO Standard Specifications. Structures shall not be designed with open steel grid floors. When steel grid floors are designed to be concrete filled, specify "Concrete Filled Steel Grid Floor" on the plans. Drains, where needed, are to be welded to the floor.

Timber Floors

For the design of transverse planks, use a wheel load distribution over 12 inches (305 mm) in width. See Section 2-1 "Article 3.25.1". For other types, design per the AASHTO Standard Specifications.

Use 1 ½ inch (40 mm) nominal or 1 ¼ inch (30 mm) finished, retaining strips with the 1 ½ inch (40 mm) wearing surface.

Drainage shall be provided for by blocks under the wheel guard.

Hardware weights shall be computed from the tables of Figures 6-12 and 6-13. The hardware shall be galvanized for bridges over saltwater.

Deck Drains

General

Drains shall not be located over unprotected fill slopes, traffic lanes, or shoulders. If locating drains over slope protection is unavoidable, disallow the use of the stone slope protection option (Alternate "B" on Standard SP1~~SM~~).

PVC pipes, 6 inch (152 mm) nominal diameter, shall not be used adjacent to an unprotected sidewalk. As an alternative, detail a 4 inch (102 mm) nominal diameter PVC pipe at a spacing determined by the Hydraulics Unit, or at a minimum of 6 feet (1.8 m) on center.

For drains to be used with prestressed concrete girder bridges, see Figure 6-14. For drains to be used with rolled beam or plate girder bridges, see Figure 6-15.

In some circumstances, the Hydraulics Unit may require scuppers to be placed on the bridge. Use Standards BS1~~SM~~ and BS2~~SM~~ "Bridge Scupper Details". When a collection system will not be attached to the structure, see Figures

6-16 and 6-17 for additional details. Detail the location of the inlet on the Typical Section and Plan of Span sheets.

Stream Crossings

Except as noted elsewhere in this section, deck drains are required on all stream crossings.

- For prestressed girder and cast-in-place deck slab bridges, the drains shall be 6" (152 mm) ϕ PVC pipes spaced at 12'-0" (3.6 m) centers.
- For cored slab bridges, use 4" (102 mm) ϕ PVC drains spaced at 6'-0" (1.8 m) centers. These drains shall be placed on top of the cored slab units and extended horizontally through the rail with a 4 inch (100 mm) overhang.
- For structural steel bridges, the drains shall be 6" (152 mm) ϕ PVC pipes spaced at 12'-0" (3.6 m) centers unless the grade is greater than 2% on a normal crown deck 40 feet (12.2 m) or less in clear width. For this situation, work with the Hydraulics Unit to see if the drain spacing can be increased. Where deck drains have a significant impact on bridge aesthetics, the deck drains shall be painted. Place the same note on the plans that is used when deck drains are required on weathering steel grade separations ~~for grade separations on the plans~~.

Bridges identified by the Hydraulic Report as being in close proximity to particularly sensitive waters shall be designed to eliminate direct discharge from the deck into the receiving water. Contact the Structure Utilities group in the Design Services Unit for assistance in plan preparation for the collection system.

Grade Separations

Drains are not required unless the bridge deck has a clear width greater than 40 feet (12.2 m), the superelevation is greater than 0.03, the bridge is longer than 350 feet (105 m), or there are other unusual drainage conditions. If any of these conditions exist, work with the Hydraulics Unit to develop the drainage system.

When deck drains are required on weathering steel grade separations, place the following note on the plans:

> ***PVC deck drains shall be painted with two coats of brown primer meeting the requirements of Article 1080-12 of the Standard Specifications. Each coat shall be 2 dry mils** (0.050 mm) **thick. Deck drains shall be roughened prior to painting. No separate payment shall be made for painting PVC deck drains as this is considered incidental to***

the pay item for Reinforced Concrete Deck Slab [~~(~~Sand Lightweight Concrete]~~)~~.

The above note shall be modified and placed on the plans when deck drains are required for painted structural steel superstructures.

Railroad Overheads

Drains are not required except in very unusual circumstances. In these instances, approval must be obtained from the railroad for all drainage systems.

Sidewalks and Bikeways

When a sidewalk is required by the Structure Recommendations, it shall be 5'-0" (1500 mm) or 5'-6" (1650 mm) wide and 6 inches (150 mm) high. See Figure 6-18.

Cover for the reinforcing steel shall be 2 ½ inches (65 mm) minimum clear to the top bar and 1 ¼ inches (~~30~~ 32 mm) clear to the bottom 'B' bar. The transverse reinforcing steel shall be #4 ~~(#13)~~ bars at 1'-0" (#13 bars at~~#13 bars at 180~~ 300 mm) centers in top of the sidewalks. Also detail 4 - #4 (#13) dowels ~~spaced at 1'-1" (340 mm) or 1'-2" (370 mm)~~ in the transverse direction ~~and~~ at ~~5~~7'-0" (~~1~~2.~~5~~1 m) in the longitudinal direction. The longitudinal reinforcing steel shall be as detailed in Figure 6-18. Figure 6-18a details reinforcing steel for the sidewalk that is cast with the cored slabs units.

Concrete Median Strips

Where a permanent median strip is required on the bridge, the reinforcing steel shall be epoxy coated and detailed as shown in Figure 6-19.

Provide the same opening for expansion joint in the median strip as that in the deck opening. See Figure 6-19 for details.

Bridge Rails

Railing, sidewalks and guardrail anchorage shall conform to the current AASHTO Standard Specifications. All bridge railing shall be successfully crash tested in accordance with NCHRP Report ~~230~~ 350~~Criteria~~.

Concrete Rails

Bridges with no sidewalks and with reinforced concrete decks shall typically have concrete barrier rail as detailed in Figures 6-20 through 6-24.

Standard CBR1~~SM~~ "Concrete Barrier Rail" should be used in the plan development of reinforced concrete decks. Standard CBR1~~SM~~ is drawn to show general details. Modification may be needed to match a particular structure. The plan view of the end of rail detail and the plan of spans showing reinforcing steel in barrier rail shall be shown on the Standard CBR1~~SM~~. When unarmored evazote ~~or compression~~ joint seals are used in the deck joint, #5 (#16) ‘S3’ ~~through~~ and ‘S4~~6~~’ bars shall be installed using an adhesive bonding system near the joint as shown in Figure 6-25. When an armored evazote joint with elastomeric concrete is used, do not adhesively anchor these bars. Use ‘S1’ and ‘S2’ bars, or comparable, in lieu of the ‘S3’ ~~through~~ and ‘S4~~6~~’ bars. For an example of the use of Standard CBR1~~SM~~, see Figure 6-26.

Use ½ inch (13 mm) expansion joint material at 30 foot (9 m) maximum centers when using New Jersey type concrete barrier rail or concrete median barrier rail. Provide an expansion joint in the rail over all continuous bents. All reinforcing steel in concrete barrier and median barrier rails shall be epoxy coated. For median barrier rail details, see Figures 6-27 through 6-31.

For permanent concrete median barrier rails, the width and height will be as required by the roadway typical section at the bridge. When using New Jersey type median barrier, extend the barrier a minimum of 3 inches (75 mm) beyond the approach slab and square off the end.

There have been several instances when a metal barrier, either box beam or double guardrail, has been used in lieu of the permanent concrete median. In the future, whenever this type installation is used, require the use of expansion anchors and drilled holes in the cast-in-place concrete or preset anchor assemblies.

Barrier rail details for cored slab structures are shown on the Standard PCS3~~SM~~ “Prestressed Cored Slab Unit”. The plan view showing the reinforcing steel in the end of the barrier rail should be shown on the Standard PCS3~~SM~~. The reinforcing steel and stirrups for the barrier rail shall be shown on the Plan of Spans.

Metal Rails

~~Nine~~ Eight Structure Standard drawings are available and should be used for plan development:

- BMR1~~SM~~ - “1 Bar Metal Rail”
- BMR2~~SM~~ - “Rail Post Spacings and End of Rail Details for One or Two Bar Metal Rails”
- BMR3~~SM~~ - “2 Bar Metal Rail”
- BMR4~~SM~~ - “2 Bar Metal Rail”
- ~~BMR5SM - “Rail Post Spacings and End of Rail Details for Two Bar Metal Rails”~~
- BMR5~~6SM~~ - “3 Bar Metal Rail”

- BMR6~~7SM~~ - "3 Bar Metal Rail"
- BMR7~~8SM~~ - "3 Bar Metal Rail~~Rail Post Spacings and End of Rail Details for Three Bar Metal Rails~~"
- BMR8~~9SM~~ - "Guardrail Anchorage Details for Metal Rails~~Rail Post Spacings and End of Rail Details for Three Bar Metal Rails~~"

Metal rails shall be as shown on the Standards. The post spacing shall be a maximum of 6'-6" (1980 mm) on center.

For Standard Metal Rails, provide the same movement capability in the rail's expansion joint as that in the deck opening. Show the rail opening on the appropriate Metal Rail Standard. Provide an expansion joint in the rail over all continuous bents.

Three Bar Metal Rails are used for structures with sidewalks. Use Standards BMR5, BMR6~~SM~~, BMR7~~SM~~, and BMR8~~SM and BMR9SM~~. The post closest to the end post shall be placed as shown on Standard BMR5~~6SM~~. The next two posts shall be spaced at a distance of one-half the normal post spacing not to exceed 3'-3" (990 mm). ~~The details showing p~~Post spacing and end post details should be drawn on an additional plan sheet. ~~g~~Guardrail attachments~~, if required,~~ should be ~~drawn~~ shown on Standard BMR8~~9SM~~. See Figure 6-32.

Two Bar Metal Rails are used for structures carrying bicycle routes. Use Standards BMR2~~SM~~, BMR3~~SM~~, BMR4~~SM~~, and BMR8~~5SM~~. The post closest to the end post shall be placed as shown on ~~s~~Standard BMR3~~SM~~. ~~The details showing p~~Post spacing ~~and guardrail attachment, if required,~~ should be drawn on ~~s~~Standard~~s~~ BMR2~~SM~~ and guardrail attachments should be shown on Standard BMR8~~5SM~~. Also include the end post and parapet details shown in Figures 6-33 and 6-35~~4~~ on an additional plan sheet. Figure 6-35~~4~~a details barrier rail steel for Two Bar Metal Rails on cored slabs.

Other types of rail are to be used in special cases only. If One Bar Metal Rail is used, use Standards BMR1~~SM and~~, BMR2~~SM~~ and BMR8. The post closest to the end post shall be placed as shown on Standard BMR1~~SM~~. The next two posts shall be spaced at a distance of one-half the normal post spacing not to exceed 3'-3" (990 mm). ~~The details showing p~~Post spacing ~~and guardrail attachment, if required,~~ should be drawn on Standard BMR2~~SM~~ and ~~BMR8~~ guardrail attachments should be ~~shown~~drawn on Standard BMR8. Also include the end post and parapet details shown in Figures 6-34 and 6-35 on an additional ~~the~~ plan sheet.

The pay item for parapets for one and two bar metal rails shall be "1'-___ x ___" Concrete Parapet" ("___ x ___ mm Concrete Parapet") and paid for per linear foot (meter).

Temporary Bridge Rail

For staged construction the Traffic Control, Pavement Marking, and Delineation Section of the Traffic Congestion and Engineering Operations Unit (Traffic Control) may require a temporary bridge rail. The pay item for temporary bridge rail will be a Traffic Control item and a Roadway detail or standard. Close coordination between Structure Design, Roadway Design and Traffic Control is extremely important. The following procedure shall be followed:

The Project Engineer shall contact the Roadway Project Engineer and the Traffic Control Section Head to determine the width of the bridge deck needed to maintain traffic during construction. This will determine the location of the temporary barrier. The offset distance shall then be the distance from the back of the barrier to the edge of the slab.

If the offset distance is less than 6'-0" (1830 mm), the portable concrete barrier [Roadway Standard 1170.01] shall be anchored to the slab. The same anchorage is required when a temporary barrier divides opposing traffic and is 2'-0" (600 mm) or less from the edge of any traffic lane. Traffic Control will be responsible for determining pay limits and estimating pay item quantities. The Project Engineer should include a sketch of the barrier including the offset distance and the following note should be added to the plans:

> ***See Traffic Control Plans for location and pay limits of the anchored portable concrete barrier.***

The Project Engineer shall furnish the beginning and ending approach slab stations to the Traffic Control Section Head and the Roadway Design Engineer.

If the offset distance is 6'-0" (1830 mm) or greater, the portable concrete barrier [Roadway Standard 1170.01] shall be used but attachment to the bridge deck is not required.

Project Engineers shall submit the Preliminary General Drawing in addition to the requirements of the above paragraphs to the Traffic Control Section as soon as they are developed.

Guardrail Anchorage

General

Guardrail transition and attachment details shall satisfy the requirements of NCHRP Report 350. Roadway Design will recommend the ~~type~~ location of guardrail attachments ~~for~~ to ~~each corner of~~ the bridge on the Structure Recommendations or the Roadway plans. ~~Typically,~~ Guardrail Anchor Unit

Type III, used to attach a thrie-beam guardrail to a vertical face parapet, will typically be specified at all four corners of the bridge. However, the trailing ends of dual structures in the median may not require guardrail if certain conditions are met. ~~The four types of Anchor Units recommended by Roadway Design and their general usage are:~~

- ~~For approach ends of bridges with barrier rail, Guardrail Anchor Unit Type XI with anchor assembly as shown in Fig. 6-36.~~
- ~~For trailing ends of bridges with barrier rail, Guardrail Anchor Unit Type XIII with anchor assembly as shown in Fig. 6-37.~~
- ~~For approach ends of bridges with sidewalks and three bar metal rail, Guardrail Anchor Unit Type VI Modified with anchor assemblies as shown on Standard BMR9SM.~~
- ~~For approach ends of bridges using either one bar or two bar metal rails - Guardrail Anchor Unit Type XV with anchor assembly as shown on Standard BMR5SM.~~
- ~~For the trailing ends of bridges with metal rails, either Type XIII, Type VI Modified or Type XV may be specified depending on the width of roadway or two way traffic.~~

Concrete Barrier Rails ~~with Cast-in-Place Decks~~

Concrete barrier rail transitions are required in order to provide a vertical face for the guardrail attachment. This transition section and the guardrail attachment will occur on the approach slab. See Section 12-1 "Barrier Rail Transitions".~~Where guardrail anchorage is required, Structure Standard GRA1SM "Guardrail Anchorage for Barrier Rail" shall be used. The Standard GRA1SM is drawn to show the condition of Guardrail Anchor Unit Types XI or XIII. Plan views showing Type XI or XIII Assemblies should be shown on the standard. See Figure 6-36 for the location of Type XI assemblies with different skew conditions.~~

~~When Guardrail Anchor Unit Type XIII is recommended, additional elevation and plan views will be required on the Standard GRA1SM. See Figure 6-37 for the location of Type XIII assemblies with different skew conditions.~~

~~A sketch showing the points of attachment should also be provided on the Standard GRA1SM.~~

~~Barrier Rail with Cored Slab~~

~~Where guardrail anchorage is required, Structure Standard GRA1SM "Guardrail Anchorage for Barrier Rail" shall be used with the necessary modifications. The Standard GRA1SM is drawn to show the condition of Guardrail Anchor Unit Type XI. Plan views showing Type XI assemblies should be shown on the standard. See Figure 6-36 for the location of Type XI~~

~~assemblies with different skew conditions. Stirrups in the barrier rail should be placed carefully to miss the Guardrail Anchor assemblies.~~

~~When Guardrail Anchor Unit Type XIII is recommended for cored slab structures, additional elevation and plan views will be required along with the necessary modifications on the Standard GRA1SM. See Figure 6-37 for the location of Type XIII assemblies with different skew conditions.~~

~~A sketch showing points of attachment should also be provided on Standard GRA1SM.~~

Metal Rails

The end posts for each metal rail are located on the bridge and have a vertical face to which the guardrail will attach.~~; therefore, the guardrail attachment will occur on the bridge at the end post.~~ A sketch showing points of guardrail anchor assembly attachments should be drawn on the Standard BMR8. See Figures 6-32, 6-33, and 6-3~~5~~4 for location of the guardrail anchor assembly. ~~"Rail Post Spacings and End of Rail Details for One or Two Bar Metal Rails". If there are no guardrail attachments, the details and notes regarding guardrail anchor assembly should be deleted from the standard drawing.~~

~~In general, structures with Three Bar Metal Rail or flow through rail shall have a guardrail and rubrail attached to the end post at the approach ends. The Project Engineer should work closely with the Roadway Design Unit for the location and details of the anchor assembly and other related modifications. A blockout shall be detailed in the sidewalk parapet when a rubrail anchor assembly is required. See Figures 6-32 and 6-38 for details.~~

Construction Joints

General

All continuous or continuous for live load bridges shall contain at least one transverse construction joint, regardless of pour quantities.

For continuous steel bridges, regardless of pour quantities, indicate the required pour sequence and location of joints. Determine a pour sequence that will minimize the residual dead load tensile stress in the deck. In general, the Wisconsin DOT Pouring Sequence, as shown in Figures 6-39 and 6-40, should be used to determine the joint locations as measured along the survey line.

For continuous for live load prestressed girder bridges, regardless of pour quantities, detail construction joints approximately 5 feet (1.5 m) to 10 feet (3.0 m) from the edge of the bent diaphragms. A range is provided to allow for optimization of the pour quantities. See Figure 6-41 for details.

Additional joints shall be provided, if necessary, to limit the above pour quantities as follows:

- For prestressed concrete girders with precast deck panels, detail a permitted transverse construction joint for pours between 100 and 200 yd^3 (76 and 153 m^3) and a construction joint for pours over 200 yd^3 (153 m^3).
- For all other superstructure types, detail a permitted construction joint in the deck for pours between 250 and 300 yd^3 (190 and 230 m^3) and a construction joint for pours greater than 300 yd^3 (230 m^3).

Transverse construction joints shall be placed along the skew. See Figure 6-42 for details. For all skewed bridges, extend full length transverse reinforcing steel through transverse construction joints.

Longitudinal reinforcing steel should extend a minimum of a development length beyond all transverse joints.

In cast-in-place deck slab bridges where the slab is to be cast monolithically with the bent caps, detail a permitted construction joint between the bottom of the slab and the top of the bent cap. In addition, detail a permitted transverse construction joint in the slab along the centerline of each bent within the continuous unit. Longitudinal reinforcing steel must be extended through these joints as required by design. Transverse reinforcing steel shall not be extended through the skewed transverse construction joints.

Longitudinal Joints

Longitudinal joints are necessary for staged construction. To facilitate form placement and removal and to eliminate the possibility of water leaking through the joint on-to the flanges, longitudinal joints for staged construction shall be located 1 foot (300 mm) from the centerline of the beam or girder. For AASHTO Types V and VI and Modified Bulb Tee prestressed concrete girders, this joint shall be located 2 feet (600 mm) from the centerline of the girder.

AASHTO Types V and VI, and Modified Bulb Tee prestressed concrete girders shall typically be spaced at 6 feet (1830 mm) surrounding the longitudinal joint. All other beams or girders shall typically be spaced at 4 feet (1220 mm) surrounding the longitudinal joint.

~~For all steel superstructures, do not detail intermediate diaphragms in the staging bay if at least three beams or girders lie on both sides of the longitudinal joint. For continuous steel superstructures, do not detail interior bent diaphragms in the staging bay. For prestressed concrete girder superstructures, do not detail intermediate diaphragms in the staging bay.~~

Transverse reinforcing steel should not extend through longitudinal joints. Use dowels here in the top of the slab only. The dowels are placed through the formwork prior to casting the concrete for the deck. Place the following note on the plans:

> ***Dowels shall be placed in the same horizontal plane as the top slab reinforcing steel.***

Closure Pours

~~On~~ For prestressed concrete superstructures with staged construction~~ bridges~~, detail a closure pour the entire bridge length if any span ~~pour on the bridge~~ exceeds 100 feet (30.5~~26~~ m) in length. Always detail a closure pour for structural steel superstructures with staged construction, regardless of the span length. Locate the longitudinal joints and space beams or girders according to the requirements of “Longitudinal Joints” above.

Expansion Joints

General

The type of expansion joint or seal to be used at a deck joint is generally determined by the length of expansion for which the joint is provided and the skew angle of the joint.

The maximum and minimum design temperatures for expansion joints shall be~~The range of temperature for thermal movement shall be from~~ 10° to 1~~2~~10°F (-~~18~~12° to ~~49~~43°C) for steel beams with a concrete slab and ~~from~~ 20° to 10~~0~~5°F (-7° to ~~38~~41°C) for concrete beams with a concrete slab respectively. The total movement shall be computed ~~by multiplying the thermal movement by the appropriate factor~~ as follows:

Total Movement, $M_{TOT} = \alpha L(T_{MAXDESIGN} - T_{MINDESIGN} + 30°F(-1°C))$

~~For steel beams and girders,~~
~~Total Movement = 1.25 x Thermal Movement~~

- ~~For concrete girders,~~
  ~~Total Movement = 1.50 x Thermal Movement~~

Where L is the expansion length and α is the coefficient of thermal expansion.

Provide #5 (#16) ‘G’ bars parallel to the joint and extending the full width of the bridge. The ‘G’ bar shall be located as close to the joint edge as possible. Care should be taken to ensure that the ‘G’ bar can be tied to other reinforcing steel. Place the following note on the plans:

> ***#5 (#16) G__ bar may be shifted slightly, as necessary, to clear reinforcing steel and stirrups.***

When a prestressed girder extends across a skewed joint and under the adjacent span, $^{3}/_{8}$ inch (10 mm) expansion joint material shall be placed on the portion of the top flange extending under the adjacent span. See Figure 6-77.

Provide 1 ½ inch (38 mm) expansion joint material between adjacent ends of cored slab units and between cored slabs and cast-in-place concrete. See Figure 6-84.

~~Where the expansion occurs at the end bent, the appropriate joint shall be used between the approach slab and the end of deck slab. If modular, strip seal or expansion joint seals are required, refer to Section 12-1 "Full Width Approach Slabs" for instructions on extending the barrier rail onto the approach slabs.~~

Evazote Joint Seals

For a maximum joint opening of 3 ½ inches (89 mm) normal to the centerline of the joint at 20°F (-7°C) for concrete superstructures or at ~~0~~10°F (-~~18~~12°C) for steel superstructures, use an evazote joint seal at both interior bents and end bents.

The joint shall be sawed prior to the casting of the barrier rail or sidewalk. Figures 6-43 and 6-44 are to be used as a guideline for selecting the joint.

For joints located at interior bents, see Figure 6-47 for typical details to show on the plans.

For joints located at end bents, the joint seal details are provided on the BAS standard drawings. For cover plate details at sidewalks, see Figures 6-48 through 6-50.

For projects with a design year ADTT of 2500 or more and all bridges on the NHS, regardless of ADTT, the evazote joint seal shall be armored from gutterline to gutterline. Elastomeric concrete shall be used surrounding the armor and shall also be detailed from gutterline to gutterline. In the sidewalk or barrier rail, a formed opening shall be detailed, in lieu of a sawed joint, with the seal turned up into the sidewalk or barrier rail. ~~The width of the formed opening shall match the computed sawed opening.~~ Use Standard AEJ1~~SM~~ and modify CBR1 and the BAS standard drawings, as applicable, to show the formed opening and delete all references to the sawed opening.

Payment for the evazote joint seals shall be at the lump sum price for "Evazote Joint Seals". Place the following note on the plans:

> ***The nominal uncompressed seal width of the evazote joint seal shall be _____ at Bent No ___. For Evazote Joint Seals, see Special Provisions.***

~~**Optional Compression Joint Seals**~~

~~At joints where the skew is between 70° and 110°, the Contractor shall have the option to use a compression joint seal. A maximum joint opening of 3 ½ inches (89 mm) normal to the centerline of the joint should not be exceeded. The joint shall be sawed prior to the casting of the barrier rail or sidewalk. Figures 6-45 and 6-46 are to be used as guidelines for selecting the joint.~~

~~For joints located at interior bents, see Figure 6-47 for typical details to show on the plans. For joints located at end bents, the joint seals are detailed on BAS standard drawings. For cover plate details at sidewalks, see Figures 6_48 through 6-50.~~

~~Optional compression joint seals shall be armored consistent with the criteria for evazote joint seals.~~

~~Payment for the optional compression joint seals shall be at the lump sum price for "Evazote Joint Seals". Place the following notes on the plans:~~

> ***~~The Contractor may at his option use a Compression Joint Seal in lieu of the Evazote Joint Seal.~~***
>
> ***~~The nominal uncompressed seal width of the optional compression joint seal shall be ______ at Bent No.___. For Optional Preformed Compression Joint Seals, see Special Provisions.~~***

Expansion Joint Seals

Where ~~compression or~~ evazote joint seals cannot be utilized, use the standard expansion joint seal or the strip seal expansion joint. For total movements exceeding ~~over~~ 2 ½ inches (65 mm), use the modular expansion joint seal. Maintain a 1 inch (25 mm) minimum joint opening normal to the centerline of joint when fully expanded.

- **Standard Expansion Joint Seals**

  Four Structure Standard drawings are available and should be used for plan development:

  ◊ EJS1~~SM~~ - "Expansion Joint Seal Details"
  ◊ EJS2~~SM~~ - "Expansion Joint Seal Details for Barrier Rail"
  ◊ EJS3~~SM~~ - "Expansion Joint Seal Details for Sidewalk"
  ◊ EJS4~~SM~~ - "Expansion Joint Seal Details for Sidewalk"

  In general, EJS1~~SM~~ and EJS2~~SM~~ are used for barrier rails and EJS1~~SM~~, EJS3~~SM~~ and EJS4~~SM~~ are used for sidewalks. Figures 6-51 through 6-53 show examples on the use of the standard drawings for a structure with a sidewalk.

On Standard EJS1~~SM~~, delete the "Expansion Joint Details" that do not apply. The 'J1' bar in the "Expansion Joint Details" should be detailed and included in the Superstructure Bill of Material. See Figure 6-54 for a detail of the 'J1' bar. The 'J1' bar shall be epoxy coated. Compute the total movement and show on the "Movement and Setting at Joint" table on Standard EJS1~~SM~~. See Figure 6-54 for example computations for the "Movement and Setting at Joint" table.

Standard EJS2~~SM~~ illustrates general details. The "Plan of Expansion Joint Seal", left and right sides, shall be detailed on the standard drawing. See Figure 6-55 for details. Show the pavement marking alignment sketch on the plans. This information can be obtained from the Traffic Control Engineer in accordance with the Policy and Procedure Manual. See Figure 6-58 for an example of the pavement marking alignment sketch.

Cover plates will be required over expansion joint seals. Care should be taken in orientation of the cover plates with respect to traffic. The bolts on the cover plate shall be on the side of the approaching traffic. The Type I cover plate has bolts on the left end of the plate when looking at the top of the plate, and the Type II cover plate has the bolts on the right end. In general, Type II will be used for two-way traffic, and Types I and II will be used for structures with one-way traffic. Calculate the length of the cover plate and place this dimension on the standard drawings. See Figures 6-55 through 6-57 and Figures 6-27 through 6-30 for details on calculating the cover plate length for barrier rails and median barrier rails respectively.

The "Plan of Expansion Joint Seal", left and right sides, should be drawn on Standard EJS3~~SM~~. See Figure 6-56 for the detail showing the "Plan of Expansion Joint Seal" for sidewalks.

Place the pavement marking alignment sketch and the plan view of the sidewalk cover plate on Standard EJS4~~SM~~. See Figure 6-57 for details of the sidewalk cover plate.

Payment for the expansion joint seals shall be at the lump sum price for "Expansion Joint Seals". Place the following note on the plans:

For Expansion Joint Seals, see Special Provisions.

- **Strip Seal Expansion Joints**

Where the total movement is within the range for the standard expansion joint seal but the ADTT is less than 500, use a strip seal expansion joint. Two Structure Standard drawings are available and should be used for plan development:

◊ SSJ1~~SM~~ - "Strip Seal Expansion Joint Details"

◊ SSJ2~~SM~~ - “Strip Seal Expansion Joint Details for Barrier Rail”

On Standard SSJ1~~SM~~, delete the “Strip Seal Details” that do not apply. Compute the total movement and show on the “Movement and Setting at Joint” table on Standard SSJ1~~SM~~. See Figure 6-54 for example computations for the “Movement and Setting at Joint” table.

On Standard SSJ2~~SM~~, provide a plan detail of the expansion joint at the barrier rail at both sides of the bridge. This detail must show the length of the cover plate and whether it is a Type I or II, as described in “Standard Expansion Joint Seals”. As with an expansion joint seal, the cover plate must be oriented so the bolts are on the side of the approaching traffic. See Figure 6-59 to determine the required length of the cover plate. A pavement marking alignment sketch is not necessary on Strip Seal Expansion Joint Standards.

Payment for the strip seal expansion joints shall be at the lump sum price for “Strip Seal Expansion Joints”. Place the following note on the plans:

For Strip Seals, see Special Provisions.

- **Modular Expansion Joint Seals**

For modular expansion joint seals use Structure Standards MEJS1~~SM~~ or MEJS2~~SM~~ for plan development. Do not detail the joint. The contractor will submit detailed drawings and specifications for the proposed modular expansion joint seal. Compute the total movement as described above and show on the standard drawing. Also show cover plate details, the pavement marking alignment sketch and the “Plan of Modular Expansion Joint Seal”, left and right sides. See Figures 6-60 through 6-63 for these and other details to be included in the plans.

For modular expansion joints, no separate quantity is to be shown on the plans for the closure pours adjacent to the joint. Provide header elevations at the transverse construction joints.

For modular expansion joints located at end bents, the backwall and the approach slab details shall be modified as shown in Figure 6-60. Modify the BAS Standard to detail a 5 inch (125 mm) Class A concrete base course and allow an option for a 4 inch (100 mm) Type ~~HB~~ B-25.0B asphalt concrete base course. Replace the appropriate existing notes with the following:

The 5" (**125 mm**) Class A concrete base course shall be finished to a smooth surface and a layer of 30 lb (**13.6 kg**) roofing felt shall be placed between concrete base and the approach slab to prevent bond. The ~~width of the~~ concrete base course shall extend 1'-0" (300 mm) beyond the end of the approach slab and the width shall be the same

~~width~~ as that of the approach slab ~~and shall extend 3 m) beyond the slab as shown~~. The approach slab shall not be cast until the concrete base has reached an age of 3 curing days.

The Contractor may ~~at his option~~ use 4" (100 mm) Type ~~HB~~ B-25.0B asphalt concrete base course in lieu of 5" (125 mm) Class A concrete. The 4" (100 mm) Type ~~HB~~ B-25.0B asphalt concrete base shall extend 1'-0" (300 mm) beyond the end of the approach slab ~~as shown~~ and the width shall be the same as that of the ~~1 foot (300 mm) outside each edge of~~ approach slab.

Special snowplow protection of modular expansion joint seals will be necessary on bridges meeting the following criteria:

◊ the bridge is located in Divisions 7, 9, 11, 12, 13 or 14, Wake County, Durham County, Cabarrus County, or Mecklenburg County

◊ the skew angle is between 50° and 70° or between 110° and 130°

When both of the above conditions exist, place the following note on plans:

Special snowplow protection is required. See Special Provision for Modular Expansion Joint Seals.

Otherwise, use the following plan note:

For Modular Expansion Joint Seals, see Special Provisions.

Payment for the modular expansion joint seals shall be at the lump sum price for “Modular Expansion Joint Seals”.

Construction Elevations

Construction elevations shall be computed during the plan preparation stage. Three copies for each bridge shall be turned in with the project. One copy shall be retained for the office file and two copies shall be forwarded to the Construction Unit.

A computer program is available for computing these elevations. The output from this program shall be sent to the field along with illustrative sketches of the output.

See the Policy and Procedure Manual for additional information on the Construction Elevations file.

Construction Elevations for Bridge Decks

Furnish construction elevations for all bridges except cored slabs for the purpose of setting deck forms and screeds and include the following information:

- Crown elevations along the centerline of roadway and overhang elevations at the bottom of the slab along the outside edges of superstructure. These elevations are to be furnished at 4 foot (1.2 m) spacing with an elevation point located on the midspan of each of the three lines specified. At longitudinal construction joints, provide the overhang elevations on the top of the slab at 4 foot (1.2 m) intervals.
- Header elevations along the centerline of each joint at the end bents and interior bents ~~and all transverse construction joints~~. All header elevations should be provided at 2 foot (0.6 m) intervals normal to the centerline of roadway.

If a longitudinal screed is required, header elevations shall also be provided at all transverse construction joints.

See Figure 6-64 for an example of the sketches required for a skewed span. These sketches shall show the beginning, midpoint and ending stations of Span A for lines 1 and 3 and the identification stations at bents for line 2 as printed on the computer output sheet.

The appropriate sketch for Span A only shall be completed and attached to each of the three copies of construction elevations that are run for each bridge. The sketch of Figure 6-65 is detailed for a tangent bridge but may be used for a curved bridge by designating the degree of left or right curve on the centerline. Figure 6-65 provides blank spaces to be filled in with the appropriate stations. Space is also available in the figure to show a small cross-section of the bridge roadway.

Bottom of Slab Elevations

Bottom of slab elevations shall be furnished for all beams and girders for the purpose of setting the forms for the buildups. These elevations shall be provided at 10th points between the centerline of bearings for each line of prestressed girders, rolled beams, ~~or~~and plate girders with spans less than 100 feet (30.5 m). For plate girders with any span longer than 100 feet (30.5 m), provide bottom of slab elevations at 20th points. If any plate girder span exceeds 200 feet (61 m), provide bottom of slab elevations at 30th points. Separate vertical curve and superelevation ordinates are not needed by the Construction Unit and should not be included in the construction elevations package.

The appropriate sketch is to be completed and attached to each of the three copies of construction elevations. See Figures 6-66 and 6-67.

Construction Elevations for Approach Slabs

Construction elevations are to be computed for left edge, centerline, and right edge of the approach slabs. Use the same criteria for approach slab construction elevations as for bridge deck construction elevations, except all elevations shall be computed for the top of the approach slab.

For those approach slabs with an asphalt overlay, calculate the construction elevations at the top of the concrete surface. Provide header elevations along ~~the~~ a line parallel to and 2'-6" (75~~6~~0 mm), measured perpendicular to the end bent, from the centerline of the joint ~~at the end bent~~. For approach slabs with ~~special drainage (i.e., shoulder berm gutter)~~flexible pavement, do not provide construction elevations along the longitudinal asphalt-concrete interface parallel to the survey line~~gutterline~~.

Utility Supports on Bridges

All details and notes concerning utilities that are to be placed on the plans will be furnished by Design Services.

See the Policy and Procedure Manual for additional guidance.

6-3 Prestressed Concrete Girders

Design

Girders shall be AASHTO Type II, Type III, Type IV, Type V, Type VI, 63" (1600 mm) Modified Bulb Tee or 72" (1829 mm) ~~(72 in)~~ Modified Bulb Tee as shown in Figures 6-68 and 6-69. Design for the pretensioning method of prestressing with straight or straight and draped strands as required.

Prestressed girder spans are to be designed for live and dead loads to be carried by the composite action of the slab and girders.

For continuous for live load deck slabs, use the same depth girders at continuous bent diaphragms.

Frequently, girders of the same size and similar length in the same bridge or within bridges of the same project require only slightly different number of strands. In this situation, consideration should be given to using the same number of strands for these girders.

Concrete strengths up to 8000 psi (55.1 MPa) may be used routinely. Specify high strength concrete (> 6000 psi (41.4 MPa)) only in those spans where required by design. For use of concrete strengths greater than 8000 psi (55.1 Mpa), consult with the Engineering Development Squad for approval. ~~When a concrete strength greater than 6000 psi (41.4 MPa) is specified, place the following note on the prestressed girder standard drawing:~~

~~For High Strength Prestressed Concrete Members, see Special Provisions.~~

The release strength of the concrete shall be no higher than required by design.

High strength seven-wire, low-relaxation (LR) strands shall be used for prestressing. The properties and applied prestressing for the strands shall be as listed below:

Type	Grade	Area	Ultimate Strength	Applied Prestressing
0.50" ϕ LR (12.70 mm)	270	0.153 in^2 (98.71 mm)	41,300 lbs /strand (183.7 kN /strand)	30,980 lbs /strand (137.8 kN /strand)
0.60" ϕ LR (15.24 mm)	270	0.217 in^2 (140.00 mm)	58,600 lbs /strand (260.7 kN /strand)	43,950 lbs /strand (195.5 kN /strand)

~~[INSERT POLICY ON 0.6" STRANDS HERE]~~All p~~P~~restressed girder~~s~~ types may be designed with 0.50” (12.70 mm) ϕ straight or draped strand patterns~~.~~ If a straight strand design can be achieved by adding up to 6 strands to the total number of strands required for a draped design, then detail the straight strand pattern on the plans. If the straight strand design requires the addition of more than 6 strands, detail the draped strand design. Draped strand hold down points shall be located 5’0” (1.500m) on each side of the centerline of the prestressed girder. However, s~~S~~ince steeply draped strands exert a considerable load on hold-down bolts in the bottom of the girder form, the slope on draped strands shall not exceed 12.5%. When the uplift force due to draped strands exceeds 20 kips (89 kN ~~(20 kips~~), place the following note on the plans:

The uplift force due to draped strands is ______ ~~kN~~ kips (KN~~kips~~).

AASHTO Type V and VI girders, and 63" (1600mm) and 72" (1829mm) Modified Bulb Tee girders designed using a ½"ϕ draped strand pattern shall also be designed and detailed with an optional 0.6"ϕ debonded straight strand pattern. The shear shall be investigated and detailed separately for both type strand patterns.

When designing debonded strand patterns, the following criteria shall apply:

- The number of debonded strands shall preferably not exceed 25% but never more than 30% of the total number of strands.
- The number of debonded strands in any row shall not exceed 40% of the total number of strands in that row.
- The exterior strands in each horizontal row shall be fully bonded.
- Debonded strands and corresponding debond lengths shall be symmetrically distributed about the centerline of the member.
- Debonded strands in a given row shall be separated by at least one fully bonded strand.

- The number of debonded strands terminated at a given section shall not exceed four.
- The minimum debond length shall be four feet and subsequent lengths shall vary in two feet increments.

When extending a girder type with ½”ϕ draped strands to its full capacity, a 0.6” ϕ debonded straight strand pattern may not be adequate for the same capacity. In this case, design the girder with 0.6”ϕ draped strands in order to reduce the total number of strands.

The pattern for the release of the prestressing strands shall not be shown on the plans.

For ~~the AASHTO Type II, Type III and Type Iv~~all girder~~s~~ types with a straight strand pattern, detail at least one pair of strands between the neutral axis and 6 inches (150 mm) from the bottom of the girder to facilitate the tying of stirrups.

Bevel the ends of the girders only when the grade, skew, or horizontal curve of the structure creates interference at end bents and joint locations. The ends of girders should not be beveled at the bents in continuous for live load spans. The tolerance on girder lengths should be considered when determining the necessity for bevel. Girder length tolerances are provided in the Standard Specifications. Use the sloped bearing-bearing length of girders when the sloped distance exceeds the horizontal distance by more than ¼ inch (6 mm).

Maintain a minimum of 3” (75mm) clearance between the end of the girder and the end bent backwall.

Notches in the top flange at the end of the Type II and Type III girders are detailed in Standards PCG1~~SM~~ and PCG2~~SM~~. These notches will accommodate most skew conditions. For a 90° skew, eliminate the notch. Modify the ‘S3’ and ‘S4’ bars on the Type II girder standard drawing and the ‘S3’ and ‘S6’ bars on the Type III girder standard drawing to ‘S2’ bars. Add two horizontal ‘U’ shaped ‘S3’ stirrups in the top flange. For details of these modifications, see Figures 6-70 and 6-71.

Notches in the top flange at the end of the Type IV girder should be detailed on each structure as dictated by skew conditions. Modify the ‘S2’ bars to straight bars in pairs in the region of the notch. Move the ‘S3’ bars to clear the notch.

Include a girder layout sheet in the plans. See Figure 6-72 for example.

For the use of prestressed concrete girders at Corrosive Sites, see Section 12-13.

Continuous In lieu of a more rational design procedure, prestressed girders with continuous

for Live Load Deck Slabs

deck slabs may be designed for simple span dead plus live loads.

For continuous for live load deck slabs with precast deck panels, detail the top mat of reinforcement as shown in Figure 6-73.

For continuous for live load deck slabs with metal stay-in-place forms, provide slab reinforcement to satisfy composite dead plus live load moments. Comparable to the AASHTO Standard Specifications for continuous steel girders, provide at least 1% of the cross sectional area of the concrete slab for the longitudinal reinforcement. Two-thirds of this required reinforcement shall be placed in the top layer of slab reinforcement and the remaining one-third shall be placed in the bottom layer. See Figure 6-74 for details.

Stirrups

Stirrup requirements shall be as prescribed in the AASHTO Standard Specifications. Stirrups are to extend 6 inches (150 mm) above the top of the girder. Consideration shall be given to adjusting this extension when an increased buildup is required.

Slab Thickness

The slab thickness for composite design is to be the thickness of the slab less ¼ inch (6 mm) monolithic wearing surface. The slab shall be constructed with a buildup over the girders between the bottom of the slab and the top of the girder. Provide a minimum 2 ½ inch (65 mm) buildup at the centerline of bearing to accommodate the support system for the panels and 1 inch (25 mm) of final camber in the girder. See Figure 6-75. When metal stay-in-place forms are used, the minimum buildup at the centerline of the bearing may be reduced to 2 inches (50 mm). Regardless of the forming system used, when the final camber of the girder exceeds 1 inch (25 mm), the buildup shall be increased accordingly.

Whenever possible, use a constant buildup at the centerline of all bearings of a bridge to avoid steps in the bottom of the slab across bents.

The dimension at the centerline of bearing may be decreased for spans with crest vertical curves but should be increased for spans with sag vertical curves, large cambers, or superelevated spans on sharp horizontal curves.

The buildup over the girders shall be neglected in the composite design.

Diaphragms

Bent and End Bent Diaphragms

Bent diaphragms for simple span girders shall be cast-in-place concrete with a uniform depth of 1'-6" (460 mm) or 2'-0" (610 mm) below the bottom of the slab as shown in Figure 6-76. See Figures 6-76 and 6-77 for typical details of diaphragms at the interior bents. Show the #8 (#25) ‘K’ bars going over the girder. For a 90º skew, the 10 inch (260 mm) diaphragms shall be located at the end of the girder.

When the face of the bent diaphragm is offset from the end of the girder, provide additional reinforcement in the concrete between the diaphragm and the centerline of the joint as follows, see Figure 6-76:

- For an offset distance of 5 inches (130 mm) to less than 7 inches (180 mm), use one 'K' bar and #4 (#13) 'S' bars spaced at 12 inches (300 mm).
- For an offset distance of 7 inches (180 mm) to less than 11 inches (280 mm), use two 'K' bars and #4 (#13) 'S' bars spaced at 12 inches (300 mm).
- For an offset distance greater than 11 inches (280 mm), use three 'K' bars equally spaced and #4 (#13) 'S' bars spaced at 12 inches (300 mm).

Bent diaphragms for simple span girders with a continuous for live load deck slab shall be detailed as shown in Figures 6-78 and 6-79. The #4 (#13) 'U' and 'S' bars shall be spaced at 12 inch (300 mm) centers along the diaphragm.

Intermediate Diaphragms

Intermediate diaphragms shall be cast-in-place concrete with 1 ¼" (31.75 mm) ϕ tie rods tightened before casting the concrete. See Figures 6-80 and 6-81. The length of the tie rods shall not exceed 40 feet (12 m). Diaphragms may be staggered in order to keep the length of the tie rod below 40 feet (12 m). Diaphragms shall be placed at right angles to the centerline of the roadway and required as follows:

- None for spans 40 feet (12 m) or less
- One for spans greater than 40 feet (12 m)

Place the following notes on the plans:

> ***Temporary struts shall be placed between prestressed girders adjacent to the diaphragms and the nuts on the 1 ¼" (****31.75 mm****)*** ϕ ***tie rods shall be fully tightened before diaphragms are cast. Struts shall remain in place 3 days after concrete is placed. The tie rods shall be re-tightened after the struts have been removed.***
>
> ***Concrete in bent and intermediate diaphragms may be Class A in lieu of Class AA. Payment shall be made under the unit contract price for Reinforced Concrete Deck Slab.*** (Simple spans)
>
> ***Concrete in intermediate diaphragms may be Class A in lieu of Class AA. Payment shall be made under the unit contract price for Reinforced Concrete Deck Slab.*** (Continuous for live load spans)

For prestressed concrete girder superstructures with a closure pour, do not detail intermediate diaphragms in the staging bay.

When the bridge is located at a highly Corrosive Site, use a grouted recess for the tie rod ends on the exterior girder. See Figure 6-82.

When utilities are attached to a bridge and are in conflict with an intermediate diaphragm, raise the bottom of the conflicting diaphragm to the bottom of the web. If the diaphragm is still in conflict, then eliminate the diaphragm in that bay only.

Camber and Dead Load Deflection

Camber and dead load deflections at 10th points shall be shown for both interior and exterior girders on all prestressed concrete girder spans in the following manner:

Camber (girder alone in place) =________________ ↑
Deflection due to Superimposed D.L.* =________________ ↓
Final camber (or deflection) =________________ ↑ or ↓

* Includes future wearing surface in superimposed dead load.

Deflections and cambers shall be shown in feet (meters) to three decimal places, except the Final Camber which shall be shown to the nearest sixteenth of an inch (millimeter).

The camber and deflection at the time of erection is calculated based on "A Rational Method for Estimating Camber and Deflection in Precast Prestressed Members" as published in the PCI Journal, Volume 22, No. 1. This method applies multipliers to the initial camber and deflection to arrive at the camber at the time of erection. For this method, an average erection time of 28 days after casting is assumed and 73% of the camber is achieved by erection time. The unit weight of the concrete for the camber and deflection computations is assumed to be 142 lbs/ft^3 (22.3 kN/m^3).

6-4 Prestressed Concrete Cored Slabs

Design

Cored slabs are to be of the AASHTO standard shape, Type SIII or Type SIV as shown in Figure 6-83 and are to be designed for the pre-tensioning method of prestressing with straight strands. ~~When a concrete strength greater than 6000 psi (41.4 MPa) is specified, place the note on the prestressed cored slab standard drawing:~~

~~***For High Strength Prestressed Concrete Members, see Special Provisions.***~~

Specify high strength concrete only in spans where required by design. The prestressing strands shall be seven-wire, high strength Grade 270, 0.50" (12.70 mm) ϕ low-relaxation strands.

Generally, cored slabs may be used for skews between 60° and 120° where the grade is 4% or less. Cored slabs are permitted on vertical curves as long as a 2'-8" (813 mm) minimum dimension from the top of the barrier rail to the top of the wearing surface is maintained.

~~In order to accommodate the crane loads when top-down construction is required~~For those projects requiring top-down construction, design the cored slab units for an HS25 (MS 22.4) live load and limit the bearing-to-bearing spans lengths to 50 feet (15.2 m).

In most cases, the used of cored slabs should be limited to tangent alignments. However, on slight curves, it may be economical to design a cored slab structure detailed with curved pavement markings. If this option is used, the Project Engineer shall coordinate with the Roadway Design Unit as described below.

If the design is known to be a cored slab bridge with barrier rails at the time of the Structure Recommendations, the Roadway Project Engineer should recommend a clear roadway width that is in an even 3'-0" (914 mm) increment. Otherwise, the Structure Project Engineer shall increase the recommended clear roadway width to the next even 3'-0" (914 mm) increment and inform the Roadway Project Engineer of the necessary adjustment. See the Policy and Procedure Manual for an example form letter.

The barrier rail shall be placed such that there is no offset from the edge of the exterior unit to the exterior face of the barrier rail~~ is 1 inch (25 mm)~~. The barrier rail shall be attached to the exterior units by casting reinforcing steel into the exterior units and pouring the barrier rail after the units are post-tensioned.

When required, a minimum sidewalk width of 5'-0" (1500 mm) or 5'-6" (1650 mm) shall be used unless otherwise recommended. Place the sidewalk and parapet so the offset from the edge of the exterior unit to the exterior face of the parapet is 1 inch (25 mm). See Figure 6-18a. If the overall width is not in an even 3'-0" (914 mm) increment, increase the sidewalk width as necessary and inform the Roadway Project Engineer of any adjustments so the guardrail location, where necessary, can be adjusted accordingly.

Three standard drawings are available and should be used in plan development:

- PCS1~~SM~~ - "3'-0"~~914 mm~~ x 1'-6"~~457 mm~~ Prestressed Concrete Cored Slab Unit"
- PCS2~~SM~~ - "3'-0"~~914 mm~~ x 1'-9"~~533 mm~~ Prestressed Concrete Cored Slab Unit"

- PCS3~~SM~~ - “3'-0"~~914 mm~~ x 1'-__"~~mm~~ Prestressed Concrete Cored Slab Unit”

Standard PCS1~~SM~~ or PCS2~~SM~~ shall be used in combination with Standard PCS3~~SM~~.

The standard drawings provide general details; therefore, some modification or adjustment may be needed to suit a particular structure. The barrier rail details are drawn for a 2 inch (50 mm) asphalt wearing surface measured at the centerline of the bearing at the gutterline. To accommodate large cambers, this wearing surface thickness may exceed 2 inches (50 mm). In this case, the reinforcing details for the barrier rail should be modified accordingly. See Figures 6-84 through 6-86 for an example use of the standard drawings.

Where debonded strands are required, indicate the strands to be debonded on the standard drawing as illustrated in Figure 6-84. Place the following note on the plans:

> ***Bond shall be broken on these strands for a distance of ______ feet (**meters)* ***from end of cored slab unit. See Standard Specifications Article 1078-7.***

The offset dimension for the ‘S3’ bar is based on 1 inch (25 mm) minimum clear distance to the voids. Detail the spacing for the ‘S3’ bars and the ‘U’ shaped stirrups to coincide in exterior cored slab units. For cored slab structures with skews less than 75° or greater than 105°, provide additional skewed stirrups between the ‘S1’ and the first ‘S2’ stirrup such that the spacing between stirrups does not exceed 1'-0" (300 mm). See Figure 6-87.

For the use of cored slabs at a Corrosive Site, see Section 12-13.

Diaphragms

Diaphragms shall be located at the center of spans up to 40 feet (12 m). For spans over 40 feet (12 m), the diaphragms shall be located at third points. If the bridge is on a skew between 60° and 120°, skew the diaphragms also. Through the center of the diaphragm, a 2" (50 mm) ϕ hole shall be formed for the post-tensioned strand. The strand shall be 0.50" (12.70 mm) ϕ seven wire, high strength low-relaxation. The anchorage recess for the strand shall be grouted. See Figures 6-88 and 6-89.

Camber and Dead Load Deflections

The camber and dead load deflection shall be shown for all cored slab spans in the following manner:

Camber (Girder alone in place)	=________________	↑
Deflection due to Superimposed D.L.*	=________________	↓
Final camber (or deflection)	=________________	↑ or ↓

* Includes future wearing surface.

All deflections and cambers shall be shown to the nearest sixteenth of an inch (millimeter).

The camber and deflection at the time of erection is calculated based on "A Rational Method for Estimating Camber and Deflection in Precast Prestressed Members" as published in the PCI Journal, Volume 22, No. 1. This method applies multipliers to the initial camber and deflection to arrive at the camber at the time of erection. For this method, an average erection time of 28 days after casting is assumed and 65% of the camber is achieved by erection time.

6-5 Steel Plate Girders and Rolled Beams

Design

For all steel beam and girder spans, both simple and continuous, use the Load Factor Design Method.

When atmospheric corrosion is not a problem, the use of AASHTO M270 Grade 50W (345W) or HPS 70W (HPS 485W) steel is more economical and preferred. When it is necessary to use painted structural steel, AASHTO M270 Grade 50 (345) should be specified.

Use the fewest number of beams or girders consistent with a reasonable deck design. Use buildups over all beams and girders. When metal stay-in-place forms are used, the buildups shall be the same width as the beam or girder top flange. If metal stay-in-place forms are not used, the buildups shall be detailed approximately 6 inches (150 mm) wider than the beam flange. Indicate on the plans that a chamfer is not required on the corners of these buildups. Buildups should not be provided on the outside of exterior girders. Instead, concrete should be sloped downward from the bottom of the top flange to the outside of the overhang. See Figure 6-90.

For grade separations, use a constant depth for all exterior steel beams or girders. Interior beams or girders shall be designed for the most economical depths, but in no case shall they exceed the depth of the exterior beams or girders. Where the use of short end spans with shoulder piers is unavoidable, tapered plate girders for both interior and exterior girders shall be used in lieu of haunched rolled beams.

Typically, design all beams and girders for composite action. The slab thickness for composite design shall be the slab thickness less ¼ inch (6 mm) monolithic wearing surface.

In the negative moment region of continuous spans, use a consistent number of studs per row as that used in the positive moment region and space the studs at 2'-0" (600 mm). This spacing may be modified at locations of high stress in the tension flange as per the AASHTO Standard Specifications.

For economical and fatigue reasons, do not design rolled beams with cover plates except to match existing beams for rehabilitation and widening projects.

The minimum W-section used as a primary member shall be a W 27x84~~690x125~~ (W 690x125~~W 27x84~~). The overhang widths for these rolled beams shall not exceed 27 in (690 mm) ~~(27 in)~~. When a W27 (W690) steel section is required, p~~P~~lace the following note on the plans:

Needle beam type supports are required for the overhang falsework in the spans with 27" *(690 mm)* ***beams.***

The end of beams and girders at expansion joints skewed at 90° should be 1 ½ inches (40 mm) from the formed opening of the joint. The end of beams and girders for skewed bridges should be located further from the edge of the expansion joints so that the top flange, which would otherwise project into the joint, can be clipped ½ inch (13 mm) from the formed opening of the joint. See Figure 6-91.

The minimum clear distance between the end bent backwall and the end of the girder is 3" (75 mm).

When designing economical welded plate girders, observe the following rules:

- Maintain a constant web depth and vary the areas of the flange plates. Flange widths in field sections shall be kept uniform where practical. It is far more economical to design a field section with a uniform flange width and a varying flange thickness than vice versa. ~~If~~ When a constant flange width is used in a given field section, the fabricator can order wide plates of varying thickness and make transverse butt splices. The fabricator can then cut the pre-welded pieces longitudinally to the specified constant flange width.
- Limit the flange thickness change ratio to 2:1. For example, if using a 2 inch (50 mm) flange plate, do not transition to less than a 1 inch (25 mm) flange plate. For flange and web butt joint welding details, see Figure 6-92.
- When practical, limit the flange plate thickness to between ¾ in (20 mm) ~~(¾ in)~~ and 3 inches (~~50~~ 70 mm).
- Limit the number of welded flange geometry transitions. Approximately 600 lbs (~~315~~ 270 kg ~~(700 lbs~~) of flange material must be saved to justify the introduction of a welded flange transition. For spans less than 100 feet (30 m) ~~(100 ft~~) in length, a savings of ~~6~~500 lbs (23~~6~~0 kg ~~(600 lbs~~) of flange material will generally offset the cost of a welded flange transition. Use a maximum of two flange transitions or three plate sizes in a particular field section. This case usually applies in the negative moment region. In positive moment areas, one flange size can often be carried through the field section. Bolted field

splices in continuous girders are good locations for changing flange geometry as this eliminates a welded butt splice.

- Limit the number of different plate thickness used in a particular bridge or group of bridges within a project. The amount of a particular plate thickness that the fabricator can order is directly related to the unit cost of the material. The lightest steel bridge is not necessarily the most economical. Consideration must be given to the cost of fabrication processes in order to realize an economical design. For metric projects, refer to the Metric Structural Steel Special Provision for typical plate thicknesses.
- If the girder length exceeds 1~~20~~35 feet (~~37~~ 41.1 m), detail the plans for a bolted field splice. When transitioning the web plate thickness at a field splice, increment the web thickness at least $^{1}/_{8}$ inch (3 mm) so that $^{1}/_{16}$ inch (1.5 mm) fill plates may be used.

Diaphragms **General**

For staged construction in all steel superstructures, with the exception of horizontally curved girder superstructures, do not detail intermediate diaphragms in the staging bay if at least three beams or girders lie on both sides of the longitudinal joint. For continuous steel superstructures to not detail interior bent diaphragms in the staging bay. When required for staged construction, diaphragms in the bay containing a longitudinal construction joint shall be detailed with a bolted connection between the diaphragm and the connector plates. Provide vertical slots in one connector plate and horizontal slots in the opposing connector plate to allow for field adjustment. Make the slots 1 inch (25 mm) by 1 ½ inch (40 mm) and detail structural plate washers. Place the following note on the plans:

> ***Nuts on bolts for connecting diaphragm to connector plate shall be left loose for purpose of adjustment until both sides of slab have been poured.***

For both normal crown and superelevated bridges, detail the diaphragm parallel to the bridge deck.

For economical reasons, provide uniformity in the diaphragm member sizes and types used on a bridge or throughout a project, whenever practical.

Bent and End Bent Diaphragms

At the end bents and interior bents of simple spans, use steel diaphragms with ¾" (19.05 mm) ϕ shear studs anchored into a concrete end beam. See Figures 6-93 and 6-94. In the bent diaphragms show the 'K' bars going over the beams or girders. If the concrete diaphragm is wider than 2 feet (610 mm), use three #16 'K' bars equally spaced at the bottom of the concrete diaphragm.

For rolled beams, use C 12x20.7 (C 310x31) channels for 27 inch (690 mm) beams, C 15x33.9 (C 380x50) channels for beams 30 inches (760 mm) through 33 inches (840 mm), and MC 18x42.7 (MC 460x64) channels for beams 36 inches (920 mm) deep. For details see Figures 6-95 through 6-97.

For plate girders less than 36 inches (920 mm) deep, Figure 6-97 may be used if the connector plate is detailed as in Figure 6-101, with the connector plate welded to the top and bottom flange. For plate girders 36 inches (920 mm) through 48 inches (1220 mm) deep, end bent and interior bent diaphragms shall be as shown in Figure 6-98. For plate girders greater than 48 inches (1220 mm) deep, diaphragms must be designed on an individual basis. These should be detailed similar to Figures 6-99 and 6-100. The dimension between the bottom flange and the diaphragm or bracing member must be determined by the detailer. Show the minimum length and the weld size required for gusset plate attachments.

For continuous spans, detail either a cross-frame or K-frame type interior bent diaphragm similar to those of Figures 6-106 through 6-109. For skews between 70º and 110º, place the interior bent diaphragms along the skew. For all other skews, place the interior bent diaphragms perpendicular to the girders and use one~~a~~ bearing stiffener as a connector plate.

Intermediate Diaphragms

Place the intermediate diaphragms normal to the beams or girders for all skews. A maximum spacing of 25 feet (7.6 m) shall be used for all intermediate diaphragms.

For rolled beam simple spans, use C 12x20.7 (C 310x31) channels for ~~beams through~~ 27 inch (690 mm) beams, C 15x33.9 (C 380x50) channels for beams 30 inches (760 mm) through 33 inches (840 mm), and MC 18x42.7 (MC 460x64) channels for beams 36 inches (920 mm) deep. For details see Figures 6-102 through 6-104.

For rolled beam continuous spans, use C 15x33.9 (C 380x50) channels for all beams less than 36 inches (920 mm) and MC 18x42.7 (MC 460x64) for beams 36 inches (920 mm) deep as shown in Figures 6-103 and 6-104.

For plate girders less than 36 inches (920 mm) deep, Figure 6-104 may be used if the connector plate is detailed as in Figure 6-101, with the connector plate welded to the top and bottom flange. For plate girders 36 inches (920 mm) through 48 inches (1220 mm) deep, diaphragms shall be detailed as shown in Figure 6-105. Intermediate diaphragms for girders 49 inches (1245 mm) through 60 inches (1525 mm) deep shall be as shown in Figure 6-106. Cross-frames for girders greater than 60 inches (1525 mm) deep must be designed on an individual basis. These should be detailed similar to Figures 6-107 through 6-109. The dimension between the bottom flange and the cross-frame bracing member must be determined by the detailer. Show the minimum length and the weld size required for the cross-frame, gusset plate or lateral bracing attachments.

When traffic must be maintained during construction beneath a bridge with plate girders greater than 60 inches (1525 mm) in depth, provide both cross-frames of Figures 6-107 and 6-108 in the plans. Label the cross-frame with the welded gusset plates, Figure 6-107, as an optional intermediate diaphragm. Place the following note on the plans:

At the Contractor's option, the diaphragm with the welded gusset plates may be used in lieu of the diaphragm with bolted angles at no additional cost to the Department.

Lateral Bracing

When required by the AASHTO Standard Specifications, lateral bracing should be detailed similar to the details in Figures 6-100 or 6-109 through 6-111. Lateral bracing shall be designed on an individual basis.

Stiffeners and Connector Plates

For simply supported ends of rolled beam spans, end stiffeners shall be provided on both sides of the web for interior beams and the inside of the web for exterior beams. Place the following note on the plans:

Stiffeners are not required on the outside of exterior beams.

These end stiffeners shall serve as connector plates for the diaphragms and shall be detailed parallel to the end of the beam. Therefore, when the ends of the beam are beveled for grade, the end stiffeners will be vertical. If the ends of the beam are not beveled, the end stiffeners shall be normal to the beam flange. Typically, these stiffeners shall have widths such to provide approximately ½ inch (13 mm) distance to the edge of flange. The stiffener thickness shall not be less than $^{1}/_{12}$ its width, nor less than $^{3}/_{8}$ inch (9 mm).

For plate girders, the bearing stiffeners shall be designed according to the AASHTO Standard Specifications. For the details of bearing stiffeners, see Figure 6-112. For skews between 70° and 110°, the bearing stiffener may be placed along the skew and used as a connector plate for bent diaphragms. In this case, detail the bearing stiffener mill to bear at the bottom and provide fillet welds at the top and bottom of the stiffener. At continuous bents, check the fatigue stress range for a Category C fatigue detail as per the AASHTO Standard Specifications. For other skews, detail the diaphragms approximately 1'-0" (300 mm) from the center of the bearing to clear the bearing stiffener and detail a separate connector plate, see Figure 6-101.

When detailing connector plates, do not provide a width dimension as the fabricator will determine the width. Stiffener or connector plate details shall include the weld termination details -of Figure 6-113. The welded connections for stiffeners or connector plates to beam or girder webs shall be in accordance with Figure 6-114.

When the skew is less than 60° or greater than 120°, a bent gusset plate shall be used to join the diaphragm member with the connector plate welded perpendicular to the web. The gusset plate shall be the same thickness as the connector plate. The number of bolts used to connect the gusset plate to the connector plate shall be consistent with the connections of Figures 6-95 through 6-100 or as required by design. The height of the gusset plate and welds shall be detailed as shown in the example of Figure 6-114a. Do not detail the gusset plate width or bend radius.

Intermed-iate Stiffeners

Intermediate stiffeners for plate girders shall be designed according to the AASHTO Standard Specifications. The use of transversely stiffened webs shall be based on the depth of the web plate. When designing girder webs less than 50 inches (1270 mm) in depth, unstiffened webs are economical while for depths 50 inches (1270 mm) or greater, partially stiffened webs are more cost effective. The determination of how much of the web is to be stiffened must be made by considering the labor cost of the stiffener versus the cost of the web material saved. For relative cost analyses, assume that the cost of the stiffener steel is four times greater than that of the web. It is suggested that designers select a web thickness such that a minimum number of intermediate stiffeners is required.

For interior girders, intermediate stiffeners may be placed on alternating sides of the web. For exterior girders the intermediate stiffeners shall be placed on the inside of the web only. For intermediate stiffener details, see Figure 6-112.

Longitudinal stiffeners, due to their high fabrication cost and poor fatigue performance, shall be considered for only those spans greater than 250 feet (76.2 m) in length and shall not be used unless approved by the State Bridge Design Engineer. If required, longitudinal stiffeners shall be uninterrupted by placing the longitudinal stiffener on the side of the web opposite the transverse stiffeners.

Shear Connectors for Composite Action

For all beams and girders designed for composite action, use ¾" (19.05 mm) ϕ by 5 inch (127 mm) minimum length studs. For proper slab penetration and concrete cover, the shear connectors shall be detailed to satisfy the AASHTO Standard Specifications. Therefore, consideration shall be given to increasing the length of the connectors when an increased buildup is required.

For shear connectors attached to the channel bent diaphragms, use ¾" (19.05 mm) ϕ by 4 inch (102 mm) stud length. See Figure 6-115 for details.

High Strength Bolts

High strength bolts shall be shown on the plans for all field bolted connections including diaphragms and beam or girder splices.

When AASHTO M270 Grade 50W (345W)~~ type~~ steel is specified, the high strength bolts, nuts and washers shall conform to AASHTO M164 ~~(M164)~~ Type 3.

Place the following note on the plans:

> ***Tension on the AASHTO M164 bolts shall be calibrated using direct tension indicator washers in accordance with Article 440-10 of the Standard Specifications. ~~For Direct Tension Indicators, see Special Provisions.~~***

Bolted Field Splices

Bolted field splices shall be designed as per the AASHTO Standard Specifications. Flange and web splices shall be symmetrical about the centerline of the splice.

Bolted field splices should only be detailed when required to limit the girder field section lengths to 1~~20~~35 feet (~~37~~41.1 m) or when known shipping limitations exist. Do not detail an additional bolted field splice if girder symmetry about the bents is the sole consideration. In this case, a permitted bolted field splice may be beneficial to the Contractor and the following note should be placed on the plans:

> ***A bolted field splice will be permitted in the girders in Span __. If a field splice is used, it shall be made entirely at the Contractor's expense and no additional measurement or payment will be made for the additional materials required. The location, details, and splice material will be subject to the approval of the Engineer.***

The contact surface of bolted parts to be used in the slip-critical connections shall be Class C for AASHTO M270 Grade 50 (345) steel or Class B for AASHTO M270 Grade 50W (345W) steel. Design these connections with a minimum of $^{1}/_{8}$ inch (3 mm), preferably ¼ inch (6 mm), additional edge distance beyond the AASHTO Standard Specification requirements to provide greater tolerance for punching, drilling and reaming. ~~See Figure 6-116.~~ Use a 3 inch (75 mm) minimum ~~edge~~ distance from the centerline of the web splice to the first row of bolts~~ in the web splice~~. See Figure 6-116.

Charpy V-Notch Test

All structural steel furnished for main beam and girder components subject to tensile stresses shall meet requirements of the longitudinal Charpy V-Notch Test.

For rolled beams, place the following note on the plans:

> ***A Charpy V-Notch Test is required on all beam sections, cover plates and splice plates as shown on the plans and in accordance with Article 1072-9 of***

the Standard Specifications*.****~~See Special Provision for Charpy V-Notch Tests.~~***

For simple span plate girders, place the following note on the plans:

> ***A Charpy V-Notch Test is required for web plates, bottom flange plates, bottom flange splice plates and web splice plates*** *(if used)* ***for all girders and in accordance with Article 1072-9 of the Standard Specifications.~~See Special Provision for Charpy V-Notch Tests.~~***

For continuous plate girders, see Figure 6-117 for Charpy V-Notch Test notes and the girder components that require testing.

For horizontally curved girders, place the following note on the plans:

> ***For Charpy V-Notch Test, See Special Provisions.***

Design Details

For steel beams on grade, the ends of the beams or girders should be beveled to maintain concrete cover. A correction should be made to the length between the bearings of beams and girders on a grade when the sloped distance exceeds the horizontal distance by more than ¼ inch (6 mm). Show the sloped length in parentheses on the bottom flange detail or over the tapered girder elevation. Place the following note on the plans:

> ***End of beams and girders shall be plumb.***

For continuous rolled beam spans, include in the plans a designation of the regions where the top flange is in tension and include the following note:

> ***No welding of forms or falsework to the top flange will be permitted in this region.***

When detailing welded steel girders, show the flange and web butt joint welding details in accordance with Figure 6-92. Shop web splices should not be located within 2'-0" (600 mm) of a shop flange splice. In negative moment regions of continuous girders, provide transverse stiffeners in lieu of detailing a web shop splice to transition to a thicker web. Indicate where the additional shop web and flange splices will be allowed by placing the following note on the plans:

> ***Shop splices are permitted to limit the maximum required flange piece lengths to 60 feet (****18 m****) and web piece lengths to ~~(~~45 feet (****14 m****). Permitted flange and web shop splices shall not be located within 15 feet (****4.5 m****) of maximum dead load deflection*** *(nor within 15 feet (4.5 m) of intermediate bearings of continuous units)****. Keep 2 feet (****600 mm****) minimum between web and flange shop splices. Keep 6" (****150 mm****) minimum between connector plate or transverse stiffener welds and web or flange shop splices.***

Deflections and Cambers

Dead Load Deflections

Deflections and cambers for all rolled beams and plate girders with spans less than 100 feet (30.5 m)~~, simple and continuous,~~ should be shown at the 10^{th} points. For plate girders with any span longer than 100 feet (30.5 m), provide deflections and cambers at 20^{th} points. If any plate girder span exceeds 200 feet (61 m), deflections and cambers shall be shown at 30^{th} points.

Show the deflections for these points in feet (meters) to three decimal places. The deflections shall be shown for both interior and exterior beams and girders. Tabulate the deflections, vertical curve ordinates, and superelevation ordinate as follows:

Deflection due to weight of steel	= _____________
Deflection due to weight of slab	= _____________
Deflection due to weight of rail	= _____________
Total Dead Load Deflection	= _____________
Vertical Curve Ordinate	= _____________
Superelevation Ordinate	= _____________
Camber due to dissipation resulting from heat curving (curved girders only)	= _____________
Required Camber	= _____________

Deflections, ordinates and cambers shall be shown in feet (meters) to three decimal places, except the Required Camber, which shall be shown to the nearest sixteenth of an inch (millimeter).

When a slab contains several pours, additional diagrams should provide the deflections at the ~~10^{th}~~ appropriate points due to each individual pour. These diagrams are used by the Contractor to determine ordinates for grading with a longitudinal screed and are required for the interior beams or girders only. Since longitudinal screeds are disallowed for pours exceeding 85 feet (26 m) in length, it is not necessary to provide pour deflection diagrams for pours exceeding this limit.

The superelevation ordinate is required when a bridge is on a horizontal curve or spiral alignment. It is also required on the spans of tangent bridges that have a variable superelevation. The superelevation ordinate is generally deducted from the total dead load deflection but must, in special cases, be added to the total dead load deflection. The superelevation ordinate should not be combined with the vertical curve ordinate but shown separately in the table of dead load deflections. The superelevation ordinates may be obtained from the computer program "Construction Elevations - Bottom of Slab Elevations Along Beams".

The camber due to dissipation resulting from heat curving is required for horizontally curved steel girders only, and shall be determined in accordance with the AASHTO Standard Specifications.

Differential Deflections of Simple Span Plate Girders

~~Since~~T~~ t~~he tributary area method of computing deflections due to the weight of the slab is adequate if differential deflections between adjacent girders are less than 1 inch (25 mm). However, ~~not always an accurate predictor of the deflections that occur in the field. Therefore, ~~a more refined method of computing deflections due to the weight of the slab is required when the deflections between adjacent girders differ by more than 1 inch (25 mm).~~ If differential deflections between adjacent girders are less than 1 inch (25 mm), the use of the tributary area method remains sufficient.~~

For staged construction, ~~compare adjacent girders within the same stage. D~~differential deflections greater than 1 inch (25 mm) will most likely occur between the two adjacent girders closest to the closure pour but within the same stage.~~;~~ ~~however, they may also occur between an interior and exterior girder in any long span whether or not the bridge is staged.~~

The analysis method chosen must take into account the effect of the diaphragms on the stiffness and relative deflection of the girders. For bridges between 45°and 135°, a two-dimensional model is typically accurate in predicting the actual deflections. For bridges outside this skew range, ~~other appropriate analysis methods, including ~~a three-dimensional model, ~~will be~~is necessary.

Camber of Continuous Spans

In addition to the deflection curves for continuous spans, camber curves should be shown and labeled as “Schematic Camber Ordinates”. On vertically curved bridges use the following note:

Slope for the zero camber base line varies.

Beam Cambers (Rolled Beams)

If the total dead load deflection plus vertical curve and superelevation ordinates is less than ¾ inch (19 mm), do not show a “Required Camber”. Place the following note on the plans:

No shop camber required, turn natural mill camber up.

Otherwise, detail simple span beams to be cambered to the nearest $^{1}/_{16}$ inch (1 mm). When one beam in a span requires camber, detail all of the beams in that span with camber.

Beam Camber on Sag Vertical Curve Bridges (Rolled Beams)

In preparing the table of dead load deflections and camber, careful consideration should be given to insure that no thinning of the slab occurs in a sag vertical curve. When the net deflection (dead load deflection minus any superelevation ordinate) exceeds the sag vertical curve ordinate by more than ¼ inch (6 mm), the natural mill camber shall be turned up in the usual manner. However, if the net deflection equals or exceeds the sag vertical curve ordinate by less than ¼ inch (6 mm), call for the natural mill camber to be turned downward. If the sag vertical curve ordinate is greater than the net deflection, the bridge seats should be adjusted accordingly and the plans should call for the natural mill camber to be turned downward.

Live Load Deflection

For the purpose of computing live load deflections, all beams or girders in a typical section may be assumed to act together and therefore deflect equally.

Construction Practices

Place the following applicable notes on the plans:

Structural steel erection in a continuous unit shall be complete before falsework or forms are placed on the unit.

Previously cast concrete in a continuous unit shall have attained a minimum compressive strength of 3000 psi (*20.7 MPa)* ***before additional concrete is cast in the unit.*** (This note should be reworded when simple spans have multiple pours.)

Barrier rail in a continuous unit shall not be cast until all slab concrete in the unit has been cast and has reached a minimum compressive strength of 3000 psi (*20.7 MPa)****.***

Barrier rail in each span shall not be cast until all slab concrete in that span has been cast and has reached a minimum compressive strength of 3000 psi (*20.7 MPa)****.*** (This note should be used for all simple spans.)

Direction of casting deck concrete shall be from the fixed bearing end toward the expansion bearing end of the span. (For simple span steel girders with a total expansion length of 150 feet (46 m) or greater)

The Contractor may, when necessary, propose a scheme for avoiding interference between metal stay-in-place form supports or forms and beam/girder stiffeners or connector plates. The proposal shall be indicated, as appropriate, on either the steel working drawings or the metal stay-in-place form working drawings.

Horizontally

General

Curved Plate Girders

Curved girder bridges shall be used on special instructions only when the combination of degree of curvature and length of span make it impractical to utilize straight chord girders on a curved bridge alignment.

The effects of curvature must be accounted for in the design of steel superstructures where the girders are horizontally curved. The magnitude of the effect of curving girders is primarily a function of radius, span, diaphragm spacing, and to a lesser degree, girder depth and flange proportions. Two effects of curvature develop in these bridges that are either nonexistent or insignificant in straight girder bridges. First, the general tendency is for each girder to overturn, thereby transferring both dead and live load from one girder to another in the cross section. The net result of this load transfer is that some girders carry significantly more load than others. This load transfer is carried through the diaphragms. The second effect of the curvature is the concept of lateral flange bending. This bending is caused by torsion in the curved members that is almost completely resisted by horizontal shear in the girder flanges. These bending stresses either compound or reduce the vertical bending stresses.

Follow the AASHTO Guide Specifications for Horizontally Curved Highway Bridges when designing horizontally curved girders.

Details

All curved girder bridges shall be continuous and designed for composite action. All diaphragms shall be placed radially and spaced so as to limit the flange edge stresses due to lateral flange bending.

Diaphragms, including their connections to the girders, must be designed to carry the total load transferred at each diaphragm location. For sharply curved structures, full depth diaphragms shall have connections to the girder webs and flanges that transfer the flange shears to the diaphragm without over stressing the girder web to flange weld. Transverse welds on the girder flanges will be permitted if the allowable stresses are reduced as per the fatigue criteria pertaining to the connection details.

Special consideration must be given to the expansion and girder end rotation characteristics of curved steel member bridges. On a curved steel member bridge, expansion between the fixed and expansion bearings will occur along a chord between the two bearing points. It is necessary to provide expansion bearings that will permit horizontal movement along this chord. Both the fixed and expansion bearings must provide for end rotation about a radial line.

The splices in flanges of curved girders must be designed to carry both the lateral bending stresses as well as vertical bending stresses in the flanges.

Follow the guide specifications for the allowable flange tip stress and fatigue stress.

6-6 Bearings and Anchorage

General

The use of 50 durometer elastomeric bearings for all bridge types is preferred. For those instances where the use of elastomeric bearings is impractical, consideration shall be given to the use of pot or TFE bearings.

The allowable bearing stress on concrete shall be in accordance with the AASHTO Standard Specifications. The bearing pressure on the TFE sliding surface shall not exceed 3000 psi (20.7 MPa) for TFE expansion bearing assemblies. Pot bearings are designed by the supplier according to the loads and movement specified on the structure plans.

With the exception of pot bearings, steel bearing plates used with steel beams or plate girders shall be AASHTO M270 Grade 50W (345W) or 50 (345), or at the designers option Grade 36 (250). In accordance with the Standard Specifications, steel bearing plates for prestressed girders shall be AASHTO M270 Grade 36 (250) and all bearing plates, bolts, nuts and washers used with prestressed girders shall be galvanized.

For TFE expansion bearing assemblies, all bearing plates shall be galvanized except the plates receiving the TFE pad or stainless steel sheet. The plates receiving the TFE pad or stainless steel sheet shall be commercially blast cleaned and, except for the areas with special facing, shall be painted in accordance with the Special Provisions.

All steel in pot bearings shall be AASHTO M270 Grade 50W (345W). The plates in the pot bearing assemblies shall be commercially blast cleaned and, except for the areas with special facing, shall be metallized in accordance with the Special Provisions.

Detail a $^{3}/_{16}$ inch (5 mm) preformed bearing pad under steel masonry plates.

Elastomeric Bearing Pads

Prestressed Concrete Cored Slabs

The use of level, unreinforced pads is preferred. The pads shall be designed in accordance with the AASHTO Standard Specifications. In general, use 6 inch (150 mm) by $^{5}/_{8}$ inch (16 mm) pads as a minimum. Place the details on Standard PCS3~~SM~~. It may be necessary to slope the cap to allow the use of level pads, see Section 7-1 “Sloped Caps”.

Slab on Beams or Girders

The use of steel reinforced elastomeric pads in combination with steel sole plates is preferred. Use a sole plate thickness of 1 ¼ inches (32 mm), unless the sole plate is beveled or fill plates are required. Incorporate any required fill plate thickness up to 1 inch (25 mm) into the sole plate - do not use separate fill plates. When the grade plus final in-place camber exceeds 1%, ~~or if the maximum in service rotation for the bearing pad is exceeded,~~ bevel the sole plate to match the grade plus final camber. Use 1 inch (25 mm) minimum clearance between the edge of the elastomeric bearing and the edge of the sole plate in the direction parallel to the beam or girder. For steel beams or girders, use ½ inch (13 mm) minimum clearance between the edge of the elastomeric bearing pad and the steel sole plate in the direction perpendicular to the beam or girder.

For steel beams or girders, refer to Standards EB1~~SM~~ and EB2~~SM~~ for standard pad Types I through VI. These standard pads ~~meet~~satisfy the allowable rotation criteria for the following ~~S~~span capacities~~,~~ and load ratings ~~and allowable rotations for these pads are as follows~~:

Steel Beams or Girders		
Pad Type	Maximum Length of Superstructure Expanding at the Bearing	Maximum DL plus LL (Service Load, No Impact)
I	85 feet (27 m)	91 kips (396 kN)
II	125 feet (37 m)	119 kips (557 kN)
III	150 feet (44 m)	144 kips (674 kN)
IV	175 feet (52 m)	184 kips (809 kN)
V	197 feet (56 m)	200 kips (943 kN)
VI	210 feet (65 m)	262 kips (1134 kN)

If the design values shown in the above table are exceeded either by movement or~~,~~ load ~~or rotation~~, increase the hardness to 60 durometer and check the elastomeric bearings in accordance with the AASHTO Standard Specifications. If 60 durometer hardness is acceptable, place the following note on plans:

Elastomer in all bearings shall be 60 durometer hardness.

If the design values for 60 durometer hardness are exceeded by either movement, load or rotation, pot bearings or TFE bearings shall be used. If the design values are exceeded at the fixed location only, a fixed bearing assembly may be used here in conjunction with elastomeric bearings at the expansion location. See Figure 6-124 for details.

Taper the bottom flange to 1~~5~~2 inches (~~380~~ 300 mm) at the ends of plate girders as required to accommodate the anchor bolt gage for Elastomeric Pad Type I and II. For Elastomeric Pad Type III-VI, taper the bottom flange to 15 inches (380 mm) at the end of the plate girder.

For prestressed concrete girders, refer to Standards EB3~~SM~~ and EB4~~SM~~ for standard pad Types II through VII. These standard pads ~~meet~~satisfy the allowable rotation criteria for the following ~~S~~span capacities~~,~~ and load ratings ~~and allowable rotations for these pads are as follows~~:

Prestressed Concrete Girders		
Pad Type	Maximum Length of Superstructure Expanding at the Bearing	Maximum DL plus LL (Service Load, No Impact)
II	145 feet (44 m)	82 kips (366 kN)
III	145 feet (44 m)	115 kips (512 kN)
IV	175 feet (53 m)	137 kips (611 kN)
V	200 feet (61 m)	180 kips (801 kN)
VI	229 feet (70 m)	211 kips (936 kN)
VII	249 feet (76 m)	264 kips (1174 kN)

If the design values shown in the above table are exceeded either by movement or~~,~~ load~~ or rotation~~, individual designs and details in accordance with the AASHTO Standard Specifications shall be used. It is more economical to maintain the plan view dimensions of the standard pads and adjust the pad thickness or durometer hardness of the elastomer. If 60 durometer hardness is used, place the following note on plans:

Elastomer in all bearings shall be 60 durometer hardness.

When elastomeric bearings are used on continuous for live load deck slabs, both bearings at the continuous bents shall be fixed.

Payment

Payment for elastomeric bearings shall be shown on the Total Bill of Material at the lump sum price for "Elastomeric Bearings". Payment for steel sole plates used with plate girders or rolled beams is included in the pay item for

"Structural Steel". Payment for steel sole plates used with prestressed girders is considered incidental to the cost of the girder.

Pot Bearings

When pot bearings are used, place the vertical and horizontal service load requirements on the plans, see Figure 6-118.

- In Seismic Performance Category A, the horizontal load for pot bearings shall be 10% of the total load or 20% of the dead load, whichever is greater.
- For Seismic Performance Category B, the horizontal load for pot bearings shall be the lateral load obtained from a seismic analysis (SEISAB) or 10% of the total load, whichever is greater.

Refer to Standard PB1~~SM~~ and Figures 6-118 through 6-121 for plan details such as masonry plate size, anchor bolt gage, and overall bearing height. Show the plan dimensions and thickness of the masonry plate and the width of the sole plate in the direction perpendicular to the beam or girder. The sole plate shall extend 1 inch (25 mm) beyond both sides of the bottom flange. Do not detail the width of the sole plate in the direction parallel to the girder or the thickness of the sole plate ~~[DO WE NEED A POLICY ON SOLE PLATE WIDTH FOR POT BEARINGS NUMEROUS QUESTIONS ON HOW TO SET SUBSTRUCTURE UP].~~ Use the anchor bolt gage from Figure 6-119 to check for conflicts with reinforcing steel in the bent cap.

Align the masonry plate so that the centerline of the plate is normal to the bent cap. Bevel the sole plates to match the final grade of the bottom flange at the location of the bearing and show the slope percentage above the sole plate details. For expansion bearings, use 4 inch (102 mm) grout cans for anchorage. See Figures 6-118 through 6-121 for typical details.

On curved girder bridges, expansion occurs along a chord drawn between the fixed and expansion bearings. This angle shall be shown on the plans so the pot bearings can be set correctly in the field.

For an example of pot bearings, see Figure 6-122.

Place the appropriate notes on the plans:

The Contractor shall adjust the girder buildups as necessary to incorporate a maximum permissible variation in pot bearing depth of ½ in (13 mm) ~~(½ in)~~, see Special Provision for Pot Bearings.

Sole plates should be welded to beam flanges and anchor bolts should be grouted before falsework is placed.

At all points of support in Spans _____, nuts for anchor bolts shall be tightened finger tight and given an additional 1/4 turn. The thread of the nut and bolt shall then be burred with a sharp pointed tool.

> ***When welding the sole plate to the girder, use temperature indicating wax pens, or other suitable means, to ensure that the temperature of the bearing does not exceed 250°F (**121°C**). Temperatures above this may damage the TFE or elastomer.***

Disc Bearings shall be permitted as an option to pot bearings. When pot bearings are required, place the following note on the plans:

> ***The Contractor may substitute Disc Bearings for the Pot Bearings shown. For Optional Disc Bearings, see Special Provisions.***

Payment for pot bearings shall be shown on the Total Bill of Material at the lump sum price for "Pot Bearings".

TFE Bearings

When TFE bearings are used, refer to Standard TFE1~~SM~~ and Figure 6-123 for typical details. Use 4 inch (102 mm) grout cans at expansion assembly locations. At fixed locations, use a curved sole plate with a 2'-0" (610 mm) radius and a flat masonry plate with a thickness of 1 ¼ in (32 mm), unless the sole plate is beveled or fill plates are required. See Figure 6-124.

Size the TFE pad based on the bearing loads. Limit the compressive stress to 3000 psi (20.7 MPa) including any stress due to eccentric loading. Use a ½ inch (13 mm) minimum clearance between the edge of the TFE pad and the edge of the stainless steel sheet in all directions. The length of the stainless steel sheet in the direction parallel to the girder shall also be based on the anticipated movement due to thermal effects and end rotation, rounded up to the next inch (20 mm). For the temperature setting table and details to be shown on the plans, see Figure 6-118.

When the grade of the girder at the location of the bearing due to roadway grade and final camber is between 4% and 8%, bevel the top of the curved sole plate 1 inch (25 mm) in 24 inches (610 mm). When the grade of the girder at the location of the bearing is greater than 8%, bevel the top of the curved sole plate to match the grade of the girder. When fill plates are required, place the following note on the plans:

> ***At the Contractors option, fill plates** *(where used)* **may be combined with masonry plates.***

Place the appropriate notes on the plans:

> ***For TFE Expansion Bearing Assemblies, see Special Provisions.***
>
> ***All bearing plates shall be AASHTO M270 Grade _______.***

At fixed points of support, nuts for anchor bolts shall be tightened finger tight and then backed off 1/2 turn. The thread of the nut and bolt shall then be burred with a sharp pointed tool.

Anchor bolts should be grouted before falsework is placed.

The 1 ½" (38.10 mm) ϕ pipe sleeve shall be cut from Schedule 40 PVC plastic pipe. The PVC pipe shall meet the requirements of ASTM D1785.

No separate payment will be made for the pipe sleeves. Payment shall be included in the lump sum contract price bid for "TFE Expansion Bearing Assemblies".

Cambered girder lengths shall be adjusted and bearings are to be placed on the cambered girder so as to be aligned with the anchors after the dead load deflection has occurred. Shop drawings shall be prepared accordingly.

The last note shall be modified and placed on rolled beam spans where the dead load deflection and slope produces a change in length of more than ¼ inch (6 mm).

Payment for TFE bearing assemblies shall be shown on the Total Bill of Material at the lump sum price for "TFE Expansion Bearing Assemblies". Payment for fixed bearing assemblies used in conjunction with TFE expansion bearings shall be included in the pay item for "Structural Steel".

Anchorage

For prestressed girder spans, use 2" (50.80 mm) ϕ anchor bolts set 18 inches (460 mm) into the concrete cap. The anchor bolt gage for sole plates shall be computed as the bottom flange width plus 6 inches (150 mm).

For cored slab spans, provide 1" (25 mm) ϕ holes in fixed end bearing pads and 2 ½" (64 mm) ϕ holes in expansion end bearing pads for #6 (#19) dowels. Dowels shall be 1'-6" (460 mm) long set 9 inches (230 mm) into the concrete cap.

For rolled beam and plate girder spans with elastomeric bearings, use 1 ¾" (44.45 mm) ϕ anchor bolts, set 18 inches (460 mm) into the concrete cap. The anchor bolt gage for elastomeric bearings shall be as shown on Standards EB1~~SM~~ and EB2~~SM~~.

For TFE expansion bearing assemblies, use 1 ½" (38.10 mm) and 1 ¾" (44.45 mm) ϕ anchor bolts set 15 inches (380 mm) into the concrete cap for the expansion and fixed ends, respectively. The anchor bolt gage for sole plates shall be computed as the bottom flange width plus 5 inches (130 mm). This may be varied to suit special conditions.

For pot bearings, use 1 ½" (38.10 mm) ϕ anchor bolts set 15 inches (380 mm) into the concrete cap.

The required length of the anchor bolt shall be the required projection plus the embedment length in the concrete cap. Compute the amount of projection of anchor bolts required by adding the thickness of all materials through which the bolt must project plus:

- 2 $^{1}/_{8}$ inches (54 mm) for 1 ½" (38.10 mm) ϕ bolts used with pot bearings, rounded to the next $^{1}/_{8}$ inch (~~3~~1 mm).
- 2 ¼ inches (60 mm) for 1 ½" (38.10 mm) ϕ bolts, except when used with pot bearings, and 1 ¾" (44.45 mm) ϕ bolts rounded up to next ½ inch (10 mm).
- 2 ½ inches (65 mm) for 2" (50.80 mm) ϕ bolts rounded up to next ½ inch (10 mm).

For elastomeric bearings, detail the anchor bolt length on both the applicable EB Standard Drawing and the substructure unit sheet.

Except when detailing pot bearings, if the required projections on a given substructure unit vary by 1 inch (30 mm) or less, show the projection for all bolts as the maximum required on that substructure unit.

Bearing Plate Details

For surface finish details, see Figure 6-125.

At the fixed end of prestressed girder spans, use 2 $^{7}/_{16}$" (62 mm) ϕ holes in the sole plates.

At the fixed end of rolled beam spans, use 1 $^{15}/_{16}$" (49 mm) ϕ holes in the sole plates and the elastomeric bearing pads.

At the fixed end of plate girder spans, use 1 $^{15}/_{16}$" (49 mm) ϕ holes in the masonry plate and elastomeric pad and 1 $^{15}/_{16}$" (49 mm) by 2 ¼ inch (57 mm) slots at the top tapered to a 1 $^{15}/_{16}$" (49 mm) ϕ hole at the bottom of the sole plate.

At the expansion end, the slot size should be determined according to the amount of expansion and end rotation anticipated. See Figure 6-126 for the required slot size.

Sole Plate Welds

Show the weld size for the connection between the sole plate and the bottom flange for all bearing types.

The end of prestressed girders, rolled beams or plate girders should extend at least 1 inch (25 mm) beyond the edge of the sole plate.

The sole plate shall be field welded to the embedded plate in the prestressed girder with a $^{5}/_{16}$ inch (8 mm) minimum groove weld. For the expansion ends of steel beams or girders on elastomeric bearings, detail a field weld between the sole plates and the flanges. Place the following note on the plans:

> ***When field welding the sole plate to the girder flange, use temperature indicating wax pens, or other suitable means, to ensure that the temperature of the sole plate does not exceed 300ºF (**149ºC**). Temperatures above this may damage the elastomer.***

For pot bearings, detail a field weld between the sole plate and the bottom flange.