

**Table of Contents**

<b><u>Section</u></b>		<b><u>Page</u></b>
19.1	ABUTMENTS.....	19.1(1)
19.1.1	<u>General</u> .....	19.1(1)
19.1.2	<u>Loads</u> .....	19.1(1)
19.1.3	<u>General Design and Detailing Criteria</u> .....	19.1(1)
19.1.4	<u>Semi-Integral Abutments</u> .....	19.1(3)
19.1.5	<u>Integral Abutments</u> .....	19.1(4)
	19.1.5.1    General.....	19.1(4)
	19.1.5.2    Design Criteria.....	19.1(4)
	19.1.5.3    Superstructure and Interior Substructure Design Criteria.....	19.1(4)
	19.1.5.4    Integral Abutment Details.....	19.1(4)
19.1.6	<u>Free-Standing Abutments</u> .....	19.1(5)
	19.1.6.1    Usage.....	19.1(5)
	19.1.6.2    Epoxy-Coated Steel.....	19.1(5)
	19.1.6.3    Seismic Shear Blocks.....	19.1(5)
19.1.7	<u>Pile Spacings and Loads</u> .....	19.1(5)
	19.1.7.1    General Design Criteria.....	19.1(5)
	19.1.7.2    Pile Design for Integral/Semi-Integral Abutments.....	19.1(5)
	19.1.7.3    Pile Design for Free-Standing Abutments.....	19.1(6)
19.1.8	<u>Wingwalls</u> .....	19.1(6)
19.1.9	<u>Drainage</u> .....	19.1(7)
19.1.10	<u>Joints</u> .....	19.1(7)
	19.1.10.1    Construction Joints.....	19.1(7)
19.1.11	<u>Concrete</u> .....	19.1(8)
19.2	INTERMEDIATE SUPPORTS.....	19.2(1)
19.2.1	<u>Types</u> .....	19.2(1)
	19.2.1.1    Pipe Pile Bents.....	19.2(1)
	19.2.1.2    Piers.....	19.2(1)
	19.2.1.3    Multi-Column Bents.....	19.2(1)
	19.2.1.4    Single-Column Piers.....	19.2(1)
19.2.2	<u>General Design Considerations</u> .....	19.2(1)
19.2.3	<u>Special Design Criteria</u> .....	19.2(2)
	19.2.3.1    Pipe Pile Bents.....	19.2(2)
	19.2.3.2    Hammerhead Piers.....	19.2(3)
19.2.4	<u>Pier and Bent in a Sloped Embankment</u> .....	19.2(3)
19.2.5	<u>Dynamic Load Allowance (IM) for Piers and Bents</u> .....	19.2(3)

**Table of Contents**

(Continued)

<b><u>Section</u></b>		<b><u>Page</u></b>
19.3	BEARINGS .....	19.3(1)
19.3.1	<u>General</u> .....	19.3(1)
19.3.2	<u>Fixed Steel Bearings</u> .....	19.3(3)
	19.3.2.1 General .....	19.3(3)
	19.3.2.2 Design .....	19.3(3)
19.3.3	<u>Steel Reinforced Elastomeric Bearings</u> .....	19.3(5)
	19.3.3.1 Elastomer.....	19.3(5)
	19.3.3.2 Behavior of Steel-Reinforced Elastomeric Bearing Pads..	19.3(6)
19.3.4	<u>Design of Steel-Reinforced Elastomeric Bearing Pads</u> .....	19.3(8)
19.3.5	<u>Seismic Design</u> .....	19.3(8)
	19.3.5.1 Application.....	19.3(9)
	19.3.5.2 Seismic Performance Zone 1 Criteria .....	19.3(9)
	19.3.5.3 Connections for Fixed Steel Shoes.....	19.3(10)
	19.3.5.4 Connections for Elastomeric Bearings and PTFE/ Elastomeric Bearings.....	19.3(10)
	19.3.5.5 Seismic Isolation Bearings .....	19.3(11)

## Chapter Nineteen

# SUBSTRUCTURES AND BEARINGS

Section 11 of the **LRFD Bridge Design Specifications** discusses the design requirements of abutments, piers and walls. Chapter Nineteen presents MDT supplementary information on the design of these structural components. Section 13.4 of the **MDT Structures Manual** presents Department criteria for the selection of substructure components within the context of structure type selection.

### 19.1 ABUTMENTS

#### 19.1.1 General

An abutment can include a backwall, a cap and wingwalls. The term “end bent” is often used interchangeably with “abutment.” A backwall is the portion of the abutment which functions as a wall providing lateral support for fill material on which the roadway rests immediately adjacent to the bridge.

Abutments can be classified as rigid or flexible abutments. Flexible abutments eliminate joints at the end of the superstructure by integrating the bridge deck with the backwall. Rigid abutments incorporate expansion joints at the end of the bridge between the deck and the backwall to accommodate thermal movements. Flexible abutments must be able to accommodate the movements through elastic behavior of the bridge and the surrounding soil because the deck is integral with the abutment.

An abutment may be designed as one of the following three types in descending order of preference:

1. Semi-integral Abutment. Flexible abutment with a pin joint between the backwall and cap to facilitate construction and subsequent maintenance.

2. Integral Abutment. Flexible abutment without a joint between the backwall and pile cap (in cross section, the backwall and pile cap may, in fact, appear as a monolithic rectangle with no apparent cap).
3. Free-standing Abutment. Rigid abutment with a joint between the bridge deck and the backwall.

Figure 19.1A presents schematics for the three basic types of abutments. Each of these is discussed in this Section.

Abutments shall generally be of the cast-in-place, reinforced concrete type. They shall be founded on spread footings, drilled shafts or driven pile footings.

A jointless flexible abutment, either integral or semi-integral, is preferred. Free-standing rigid abutments shall be used where the anticipated translational movements of the piles are too great, or settlement of the backwall is anticipated. The force effects of these displacements must be included in the design.

#### 19.1.2 Loads

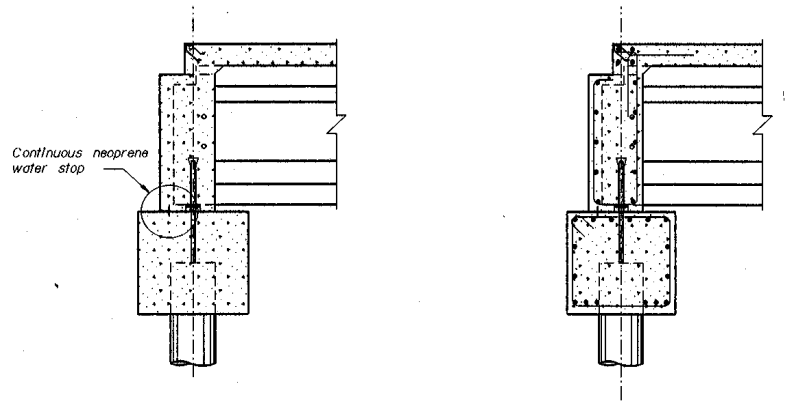
Reference: LRFD Articles 11.6.1.1 and 11.6.1.3

The static earth pressure shall be determined in accordance with Article 3.11 of the LRFD Specifications. Generally, no passive earth pressure shall be assumed to be generated by the prism of earth at the near face of the wall.

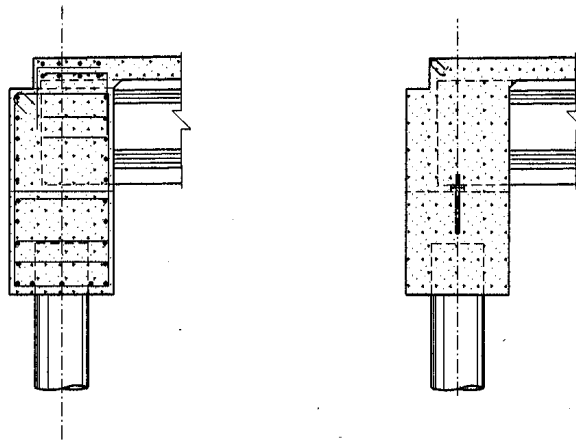
#### 19.1.3 General Design and Detailing Criteria

The following applies to the design and detailing of backwalls and wingwalls:

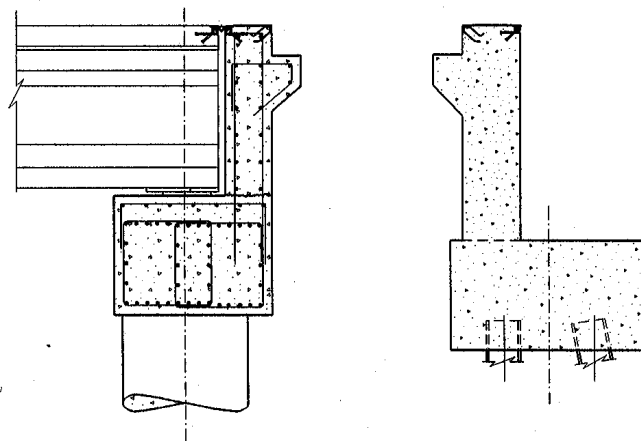
**Semi-Integral Abutment**



**Integral Abutment**



**Free Standing Abutment**



**TYPICAL ABUTMENT TYPES**

**Figure 19.1A**

1. **Bridge Approach.** Typical MDT practice is to design for the future possibility of a bridge approach slab but to not build the slab in the initial construction. When reinforced concrete bridge approach slabs are used, live load surcharge will not be considered on the end bent; however, the vehicular loads on the appropriate slabs shall be considered. Anchor the appropriate slab to the abutments if in a high seismic zone. A rigid approach slab helps to prevent compaction of the backfill behind the abutment.

Provide a paving notch on all on-system structures and off-system structures that have approach roadways that are paved or likely to be paved. If an approach slab will be constructed, show it on the General Layout.

2. **Bridge Approach Joints.** Provide a terminal joint or pavement relief joint at the end of the roadway of the bridge approach slab if the roadway pavement is concrete. A joint is not required if the entire adjacent pavement is asphalt.
3. **Wingwall Connection.** In general, U-shaped wingwalls should not extend more than 3 m behind the rear face of the abutment. If wingwalls longer than 3 m are needed, then an auxiliary footing must be provided. Also, if longer extensions are necessary, force effects in the connection between the wingwall and abutment, and in the wingwall itself, shall be investigated, and adequate reinforcing steel shall be provided.
4. **Thickness.** The minimum wall thickness for an abutment is 350 mm. Walls may be of constant thickness or with a battered fill face as required. Typically, the near face shall be vertical but, if conditions warrant (e.g., high walls, anticipated tilting), it may be slightly battered.
5. **Expansion Joints.** Vertical expansion joints should be considered for wall lengths exceeding 30 m.

6. **Backwall/Wingwall.** The junction of the abutment and wingwall is a critical design element, requiring special considerations. If the wingwall is tied to the backwall (i.e., there is no joint), design for at-rest pressure. All reinforcement must be developed into both elements such that full moment resistance can be obtained.
7. **Backwall Batter.** Vertical backwalls are preferred (i.e., no batter). For tall, free-standing walls, batter may be considered. Where used, the batter should be between 1:10 and 1:15 (H:V).
8. **Backfill.** Abutments and wingwalls shall be backfilled with Select Backfill specified by the Geotechnical Section. The neat line limits of the Select Backfill shall be shown on the plans or described in the special provisions. Show the Select Backfill quantity on the road plans.
9. **Reinforcing Steel.** If an expansion joint is located directly over the abutment cap, all reinforcement in the abutment wall shall be epoxy coated.

#### 19.1.4 **Semi-Integral Abutments**

The semi-integral abutment, or stub abutment, is MDT's typical end-bent configuration. Transverse and longitudinal superstructure forces are transmitted to the substructure through radius plate steel shoes with anchor bolts that allow rotation. Typically, the backwall and wingwalls are cast around the girder ends, attached to the slab and isolated from the pile cap. When U-shaped wingwalls are used, the wings can either be monolithic with the backwall and isolated from the pile cap or attached to the pile cap with the backwall left free to rotate. The joint between the backwall and the pile cap facilitates raising the superstructure if settlement occurs.

### 19.1.5 Integral Abutments

Reference: LRFD Article 11.6.2.1

#### 19.1.5.1 General

Traditionally, bridges have been designed with expansion joints and other structural releases that allow the superstructure to expand and contract relatively freely with changing temperatures and other geometric effects. Integral abutments eliminate expansion joints in the bridge decks, which reduce both the initial construction costs and subsequent maintenance costs.

Using integral abutments is effective in accommodating the horizontal seismic forces. Minimum beam seat length requirements need not be investigated for integral abutment bridges.

#### 19.1.5.2 Design Criteria

The following requirements must be satisfied in all cases where integral abutments are used:

1. Backfill. All integral abutments for girder type superstructures shall be back filled with Select Backfill.
2. Steel Girder Stability. Where steel girders are used, an analysis of the non-composite girder stability should be made to locate the first intermediate diaphragm to provide stability prior to and during the deck pour. In lieu of the analysis, an intermediate diaphragm should be placed within 3 m of the end support. An analysis will most likely yield a more economical, larger diaphragm spacing.

#### 19.1.5.3 Superstructure and Interior Sub-Structure Design Criteria

Although the ends of the superstructure are monolithically attached to integral abutments,

the rotation permitted by the piles is sufficiently high, and the attendant end moment sufficiently low, to justify the assumption of a pinned-end condition for girder design. The ends of the structures are also assumed to be free to translate longitudinally.

#### 19.1.5.4 Integral Abutment Details

Integral abutments are typically constructed using the following preferred method. The superstructure girders are set in place and anchored to the previously cast-in-place abutment cap. Typically, the concrete above the previously cast-in-place cap is poured at the same time as the superstructure deck. To address steel girder stability, refer to Comment #2 in Section 19.1.5.2.

Optional construction joints may be placed in the cap to facilitate construction. The optional joint below the bottom of beam may be used on all integral abutment bridges regardless of bridge length.

The abutment details shall meet the following requirements:

1. Width. The backwall width shall not be less than 750 mm.
2. Cap Embedment. The piling shall extend a minimum of 500 mm into the cap.
3. Concrete Cover. Concrete cover beyond the farthest most edge of the girder at the rear face of the abutment shall be at least 100 mm. The minimum cover shall also apply to the paving notch area. The top flange of steel girders and prestressed I-girders may be coped to meet this requirement.
4. Girder Anchorage. A minimum of three holes shall be provided through the webs of steel girders and through prestressed I-girders to allow #19 bars to be inserted to further anchor the girder to the cap. Position the holes so that, when the bars are inserted, they will be within the backwall cage.

5. **Reinforcement.** The minimum size of stirrups shall be #13 spaced at a maximum of 300 mm. Longitudinal backwall reinforcing steel be #22 @ 300-mm maximum spacing along both faces of the abutment.
6. **Corner Bars.** Use L-bars extending from the rear face of the backwall into the top of the slab at 300-mm spacing or less.

### 19.1.6 **Free-Standing Abutments**

#### 19.1.6.1 Usage

Use free-standing abutments where integral and semi-integral abutments cannot accommodate the magnitude of the longitudinal movements. Free-standing abutments can be founded on piles, drilled shafts or spread footings.

#### 19.1.6.2 Epoxy-Coated Steel

For abutments that have a bridge deck expansion joint located between the end of the deck and the face of the backwall, all reinforcing steel in the abutment shall be epoxy coated. This includes all cap, backwall and, if present, wingwall reinforcing.

#### 19.1.6.3 Seismic Shear Blocks

In seismic areas, shear blocks may be formed into the top of the abutment cap to provide lateral restraint for beams that do not have side restraint provided by the bearings or other means.

### 19.1.7 **Pile Spacings and Loads**

#### 19.1.7.1 General Design Criteria

The following criteria applies to piling for both integral and semi-integral abutments:

1. **Pile Spacing.** Use a single row of piles for an integral or semi-integral abutment. Pile spacing should not normally exceed 3 m; however, if the cap is properly analyzed and designed as a continuous beam, this restriction need not apply. If practical, one pile may be placed beneath each girder. To reduce force effects for a large beam spacing, consideration may be given to twin piles under the beam, spaced at not less than 750 mm. See Chapter 20 for minimum pile spacings. The piles are considered to be free ended and capable of resisting only horizontal and vertical forces.
2. **Number.** The number of piles shall not be less than four, unless otherwise approved by the Bridge Area Engineer.
3. **Overhang.** The minimum cap overhang shall be 450 mm.

#### 19.1.7.2 Pile Design for Integral/Semi-Integral Abutments

The following criteria apply specifically to piles and loads at integral and semi-integral abutments:

1. **Loads/Forces.** For structures satisfying the requirements provided in Section 13.4.4, force effects in the abutment piles due to temperature, shrinkage, creep and horizontal earth pressures may be neglected.

An alternative analysis must be used if the criteria in Section 13.4.4 are not met. The following steps should be considered in this analysis:

- a. The point of zero superstructure movement should be established by considering the elastic resistance of all substructures and bearing devices.
- b. The effects of creep, shrinkage and temperature should be considered.

- c. Any movement at any point on the superstructure should be taken as being proportional to its distance to the point of zero deflection.
  - d. Lateral curvature of the superstructure may be neglected if it satisfies the provisions of Article 4.6.1.2 of the LRFD Specifications.
  - e. Vertical force effects in the abutment piles should be distributed linearly with load eccentricities properly accounted for.
  - f. Lateral soil resistance should be considered in establishing force effects and buckling resistance of piles.
  - g. Force effects should be combined in accordance with the provisions of Article 3.4.1 of the LRFD Specifications.
2. **Pile Type.** Only steel H-piles or steel pipe piles are permitted at integral abutments. For semi-integral abutments, steel H-piles, steel pipe piles or fluted steel piles are permitted. The orientation of steel H-piles (strong versus weak axis) is a design consideration, and it is preferable that all piles be oriented the same. All abutment piling shall be driven vertically and only one row of piling is permitted.
  3. **Pile Driving.** Piles shall be driven a minimum of 3 m into natural ground. If piles cannot be driven to this depth due to an existing cohesive earth stratum, with a standard penetration resistance (N) exceeding 35 blows per 305 mm located with the 3 m interval below the bottom of the cap, the piles shall be placed in oversized, predrilled holes before driving. The diameter of the oversized holes should be 100 mm greater than the maximum cross sectional dimension of the pile. The holes shall be backfilled with uncrushed base course aggregate size 17 mm (pea gravel) following the pile driving operation.

If piles cannot be driven a minimum of 3 m into natural ground due to a rock stratum, socket the piles into undersized holes drilled into the rock. The diameter of the undersized holes shall equal the inside diameter of the pipe pile, if pipe piles are used, or 25 mm less than the maximum pile dimension for steel H-piles. Socket the pile a minimum of 1 m into the rock formation; the pile should extend at least 3 m below the cap.

### 19.1.7.3 Pile Design for Free-Standing Abutments

The following criteria apply to piles at free-standing abutments:

1. **Pile Spacing.** At least two rows of piles or battered piles must be provided to provide the necessary longitudinal stiffness. The minimum pile spacing is 750 mm parallel to the centerline of the abutment.
2. **Batter.** Up to one-half of the piles may be battered to increase the overturning stability of the structure.
3. **Movement.** The effects of the movements due to overturning pressures or lateral pressures shall be investigated (e.g., ensure that the closing of joints does not occur).

### 19.1.8 Wingwalls

Reference: LRFD Article 11.6.1.4

Wingwalls shall be of sufficient length and depth to prevent the roadway embankment from encroaching onto the stream channel or the defined clear opening. Design the wingwall lengths to keep the embankment at least 300 mm below the beam seat or the top of the cap. Generally, the slope of the fill will not be steeper than 2:1 (H:V), and wingwall lengths will be established on this basis.



With respect to abutments, the following applies to wingwalls:

1. Pile Supported. If turnback wingwalls on rigid abutments have a total length of more than 3 m, auxiliary pile footings for wingwall support should be investigated. Pile-supported wings shall be avoided for integral backwalls.
2. Connections. In general, U-shaped wingwalls should not extend more than 3 m behind the rear face of the abutment. If wingwalls longer than 3 m are needed, force effects in the connection between the wingwall and abutment, and in the wingwall itself, shall be investigated and adequate reinforcing steel be provided. For rigid free-standing abutments, the forces are merely due to permanent loads and live-load surcharge. For flexible abutments, other transient loads must be considered in addition to the permanent loads.
3. Thickness. The minimum thickness of any wingwall with an abutment shall be 350 mm.
4. Design. Unattached wingwalls shall be designed as retaining walls.
5. Concrete. For wingwalls, use Class DD concrete.

### 19.1.9 Drainage

Provide positive drainage as needed in the embankment behind the abutment and wingwalls by using select backfill, weep holes, perforated drain pipe, a manufactured backwall drainage system or a combination of these options. Include provisions for select backfill in all abutment designs in accordance with the geotechnical recommendations in the Geotechnical Report.

Provide details of the selected drainage system on the bridge plans. Generally, the cost of

furnishing and installing most systems can be absorbed in the cost of select backfill.

Static ground water levels should always be considered while evaluating an appropriate drainage system. Drainage systems should not be installed to allow pressurized backwater to saturate the abutment backfill during highwater events.

Generally, for relatively shallow girders supported on integral or semi-integral abutments with straight wings or turnback wings less than 3 m long, select backfill will be all that is needed to promote good drainage.

For bridges with taller abutment walls, girders deeper than 1.5 m or abutments with a total height of more than 2.5 m from the bottom of pile cap or footings to the top of the backwall should be given consideration for additional drainage features. If a drainage system is determined necessary, a perforated drainage pipe placed at the base of the abutment wall or footing is preferred. The pipe should be placed inside a free draining gravel media, wrapped in drainage fabric and sloped to drain to a point outside the abutment walls.

The other systems identified may be used to address site-specific needs with approval by the Bridge Area Engineer.

### 19.1.10 Joints

#### 19.1.10.1 Construction Joints

To accommodate normal construction practices, the designer should indicate the following horizontal construction joints on the plans. MDT does not use shear keys for horizontal construction joints:

1. In semi-integral abutments, a horizontal construction joint shall be indicated between the bottom of slab fillet and the top of the backwall.

2. In integral abutments, in addition to the construction joint indicated between the bottom of slab fillet and the top of the backwall, a horizontal construction joint shall also be indicated at beam seat.
3. In free-standing abutments, a horizontal construction joint shall be indicated on the drawings between the top of the cap or footing and the bottom of the backwall. Some expansion joint types may require another construction joint at the bottom of the paving notch.
4. In turnback wings, a horizontal construction joint shall be indicated at an elevation the same as the top of the cap.

Planned vertical construction joints are normally associated with phase construction issues or perhaps close proximity to an existing structure. Provision needs to be made for splicing or mechanical rebar couplers on horizontal reinforcing steel. Vertical reinforcing steel should be at least 75 mm from the construction joint.

#### **19.1.11 Concrete**

Use Class DD concrete for all substructure components.

## 19.2 INTERMEDIATE SUPPORTS

Reference: LRFD Article 11.7

### 19.2.1 Types

MDT uses four basic types of intermediate supports for bridges, which are discussed in the following sections. Also, see Section 13.4.7 for more information.

#### 19.2.1.1 Pipe Pile Bents

Under the right conditions, pipe pile bents may provide the most economical substructure. Do not use this type of bent in the presence of large horizontal forces. Note that debris accumulation can increase stream and ice forces significantly.

#### 19.2.1.2 Piers

MDT uses two types of piers:

1. Single Wall. This is a wall set on a spread footing or a pile cap with multiple rows of piles.
2. Hammerhead. For larger structural heights and pier widths, a hammerhead pier (either with rectangular or rounded stem) is often more suitable. The strut-and-tie model of LRFD Article 5.6.3 should be considered where the length of the cantilever is less than twice the depth of the cantilever.

#### 19.2.1.3 Multi-Column Bents

Concrete frame bents may be used to support a variety of superstructures. The columns of the bent may be either circular or rectangular in cross section. The columns may be directly supported by the footing or by a partial height wall. If the columns rest directly on the footing, the footing shall be designed as a two-way slab.

#### 19.2.1.4 Single-Column Piers

The round column is commonly used because of its ease of design, its concrete confinement for seismic and its multi-directional flow characteristics.

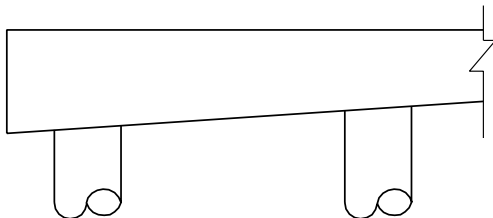
### 19.2.2 General Design Considerations

In general, the following design criteria apply to intermediate supports, where applicable:

1. Piers in Waterways. Wall piers should have a solid wall to an elevation of 300 mm above the  $Q_{100}$  high-water level. Depending on aesthetics and economics, the remainder of the wall may be either solid or multiple columns. The dimensions of the wall may be reduced by providing cantilevers to form a hammerhead pier. River piers shall have ice protection. The steel protectors may be in the form of angles, casings or plates. The nose plates or angles shall extend from the channel bottom to 300 mm above the  $Q_{100}$  high-water elevation on the upstream end of the pier only.
2. Footings. Bents founded on spread footings have typically been designed with separate footings under each column. Existing analytical techniques provide tools for the analysis of a common footing for all columns, and this configuration may result in a more economical footing.
3. Highway Bridge Over Railroad. See Chapter Twenty-one for more information.
4. Column Reinforcement. Column vertical bars shall extend into the cap beam to within 50 mm of the top reinforcement. The vertical column bars must be fully developed when they exit the cap beam and the spread footing or pile cap.
5. Size. For spread footings of piers or bents in rivers, the least ratio of footing width to bent or pier height shall be 1:4. For pile footings of piers or bents in rivers, the least ratio of

pile-group width to bent or pier height shall be 1:4. For dry-land structures, the least ratio of spread-footing or pile-group width to bent or pier height shall be 1:5. Columns are typically rectangular, square or round, with a minimum diameter or thickness of 600 mm. Diameter increments shall be in multiples of 150 mm. Solid pier walls shall have a minimum thickness of 600 mm. If conditions warrant, caps up to 300 mm wider than the thickness or diameter of columns may be used. Caps shall be at least 80 mm wider than the thickness or diameter of the columns.

6. Cap Extension. The width of caps shall project beyond the sides of columns. The added width of the cap shall be a minimum of 40 mm on the outside of the column. This width will reduce the reinforcement interference between the column and cap. The cap should preferably have cantilevered ends to balance positive and negative moments in the cap.
7. Step Caps. Where one end of the cap is on a considerably different elevation than the other, the difference shall be accommodated by increasing the column heights as shown below:



The bottom of the cap shall be sloped at the same rate as the cross slope of the top of the bridge deck.

8. Epoxy-Coated Steel Under Expansion Joints. All reinforcing steel in cap beams at intermediate piers where an expansion joint is located directly over the cap shall be

epoxy coated. Note that this does not apply to all piers. It applies only to those substructures which support the ends of two superstructure units with an expansion joint located directly over the cap. Because most structures are single continuous units, this type of substructure is relatively uncommon and will generally occur only on long structures with multiple continuous units.

9. Concrete. For intermediate supports, use Class DD concrete.
10. Steel Splices. If a pier is less than 3 m in height, do not splice the steel extending out of the footing. For small columns with a high percentage of vertical steel and for columns in seismically active regions, mechanical connectors should be used for splicing the vertical steel. No splices may be located within the plastic regions of the column and, where used elsewhere, they should be staggered.
11. Compressive Steel. Compressive steel tends to buckle when the cover is gone or when the concrete around the steel is weakened by compression. The criteria in the LRFD Specifications, Article 5.7.4.6 or 5.10.11, for ties and spirals, should be rigidly adhered to.
12. Minimum Edge Distance for Anchor Bolts. The edge distance from the center of the anchor bolt to the edge of the cap shall be 250 mm.

### 19.2.3 Specific Design Criteria

This Section presents design criteria which applies to the specific type of intermediate support.

#### 19.2.3.1 Pipe Pile Bents

The following applies to the design of pipe pile bents:

1. Limitations. This type of support has a relatively low resistance to longitudinal forces. This support should also not be used if the stream carries large debris or heavy ice flow. Scour should be considered in establishing design pile lengths and for the structural design of the pile.
2. Cap Beam. Pile bents always need a cap beam for structural soundness, which may be an integral part of the superstructure.
3. Loads. Because the piles are relatively flexible compared to the abutments, the force effects induced in the piles by lateral displacement is small. Where practical, one pile should be placed beneath each girder line. The vertical load carried by the piles shall be the girder reaction and the appropriate portion of the pile cap dead load. Assuming the bent acts as a rigid frame in a direction parallel to the bent, force effects due to lateral displacement and lateral loads may be uniformly distributed among the piles.

#### **19.2.3.2 Hammerhead Piers**

The following applies to the design of hammerhead piers:

1. The bottom of a hammerhead cap should preferably be a minimum of 2 m above the finished ground line on stream crossings to help prevent debris accumulation.
2. The design of the cantilever is affected by the cantilever depth versus length geometry. The strut-and-tie model of LRFD Article 5.6.3 should be considered where the length of the cantilever is less than twice the depth of the cantilever. Otherwise, the sectional models for moment and shear are appropriate.
3. Non-contact splices should not be used at the connection of the bottom of the cap beam to the column.

4. Architectural treatments should be discussed at the Design Parameters Meeting.

#### **19.2.4 Pier and Bent in a Sloped Embankment**

For piers or bents located in the sloped portion of an embankment, the earth pressure against the back of the footing and column shall be increased 100% to include the effect of adjacent embankment. The effect of the embankment in front of the pier or bent shall be neglected. Piers and bents located in the embankment shall be investigated for stability not considering the superstructure loads.

#### **19.2.5 Dynamic Load Allowance (IM) for Piers and Bents**

Reference: LRFD Article 3.6.2.1

Dynamic Load Allowance (IM), traditionally called impact, shall be included in the design of piers and bent columns, but shall not be applied to the design of their footings.

## 19.3 BEARINGS

### 19.3.1 General

Reference: LRFD Articles 14.4 and 14.6

Bearings ensure the functionality of a bridge by allowing translation and rotation to occur while supporting the vertical loads. For most normal applications, MDT uses two bearing types. They are steel rocker plates and elastomeric bearing pads.

Steel rocker plates are commonly used for fixed bearings on both prestressed concrete and steel girder bridges where no longitudinal movement of the bearing is required. Standard steel bearing details for each standard prestressed girder are shown in the standard girder drawings.

Elastomeric bearings are typically used for steel girder bridges or for special conditions on prestressed concrete girder bridges. Elastomeric bearings need to be designed for each location within a structure and can be designed as either fixed bearings or expansion bearings to provide for longitudinal movement at the beam end. MDT's current design practice generally results in the use of steel reinforced elastomeric bearings. Although plain elastomeric bearings may be considered for special situations, steel reinforced bearings are more common.

Both steel rocker plates and elastomeric bearings provide for girder end rotations about an axis perpendicular to the girder centerline. When selecting and designing bearings for a bridge, the designer must consider the type of superstructure, span lengths, span arrangement, substructure and foundation conditions. Bearings will be designed to accommodate needed girder end rotations and movements in the longitudinal direction. MDT's typical bearing designs do not account for rotation or translation in the transverse direction.

The following will apply:

1. Movements. Consideration of movement is important for bearing design. Movements include both translations and rotations. The sources of movement include bridge skew and horizontal curvature effects, initial camber or curvature, construction loads, misalignment or construction tolerances, settlement of supports, thermal effects, creep, shrinkage and traffic loading. Bearing pads on skewed structures should be oriented parallel to the principal rotation axis. Where insufficient seat width exists, the bearing pads may be oriented normal to the support.
2. Effect of Bridge Skew and Horizontal Curvature. Skewed bridges move both longitudinally and transversely. The transverse movement becomes significant on bridges with skew angles greater than 20 degrees that have bearings not oriented parallel to the movement of the structure.  
  
Curved bridges move both radially and tangentially. These complex movements are predominant in curved bridges with small radii and with expansion lengths that are longer than 60 m.  
  
MDT does not typically consider the effects of skew. For large bridges with unusual geometry, these movements may need consideration.  
  
The effect of curvature is normally addressed in expansion bearings by orienting the slots in the sole plates parallel to the span chord.
3. Effect of Camber and Construction Procedures. The initial camber of bridge girders and out-of-level support surfaces induce bearing rotation. Initial camber may cause a larger initial rotation on the bearing, but this rotation may grow smaller as the construction of the bridge progresses. Rotation due to camber and the initial construction tolerances are sometimes the largest component of the total bearing rotation. Due to the short duration of the

initial rotation from application of the dead load of the slab, it is MDT's design practice to not account for dead load rotations in the design of the bearings and to assume that the pads are equally stressed across their full width after application of full dead load. Pads will be designed for rotations of 0.005 radians to account for construction irregularities. In addition, include live load rotation in the pad design. Longitudinal girder slope is accounted for by beveling the sole plate for slopes greater than 2% or where the thickness of the sole plate varies more than 2 mm across the width of the plate. The curved surfaces on the steel rocker plate bearings will typically account for dead load and live load rotations without additional consideration.

4. Thermal Effects. Thermal translation,  $\Delta L$ , is estimated by:

$$\Delta L = \alpha (L_E) (\Delta T)$$

where  $L_E$  is the expansion length and  $\alpha$  is the coefficient of thermal expansion, use  $10.8 \times 10^{-6} / ^\circ\text{C}$  for normal density concrete and  $11.7 \times 10^{-6} / ^\circ\text{C}$  for steel, and  $\Delta T$  is the change in the average bridge temperature. A change in the bridge temperature causes thermal translation. Maximum and minimum bridge temperatures for bearing design are defined the same as for expansion joint design in bridge decks (see Section 15.3.7) as  $-40^\circ\text{C}$  to  $45^\circ\text{C}$ . The change in bridge temperature ( $\Delta T$ ) between the installation temperature and the design extreme temperatures is used to compute the positive and negative movements. To reduce extreme movements in one direction or the other, it is desirable to lock down the fixed bearings near the mean temperature. To reduce thermal stresses in the bridge or bearing movements, it may be desirable in some situations to specify welding the bearings at a temperature close to mean. It should be further noted that a given temperature change causes thermal movement in all directions of the bridge;

however, this is rarely accounted for in design.

5. Loads and Restraint. Restraint forces occur when any part of a movement is prevented. Forces due to direct loads include the dead load of the bridge and loads due to traffic, earthquakes, water and wind. Temporary loads due to construction equipment and staging also occur. The majority of the direct design loads are reactions of the bridge superstructure on the bearing, and they can be estimated from the structural analysis. The applicable AASHTO load combinations specified in LRFD Article 3.4.1 must be considered.

6. Serviceability, Maintenance and Protection Requirements. Bearings under deck joints collect large amounts of dirt and moisture and promote problems of corrosion and deterioration. As a result, these bearings should be designed and installed to have the maximum possible protection against the environment and to allow easy access for inspection.

The service demands on bridge bearings are very severe and result in a service life that is typically shorter than that of other bridge elements. Therefore, thought should be given in the design process to bearing maintenance and replacement. The primary requirements are to allow space suitable for lifting jacks during the original design and to employ details that permit quick removal and replacement of the bearing.

7. Clear Distance. The minimum clear distance between the bottom shoe of a steel bearing and the edge of the bearing seat or cap shall be 75 mm. For elastomeric pads resting directly on the concrete bridge seat, the minimum edge distance shall be 75 mm as well, except under deck expansion joints where 150 mm is required. The required distance from the center of anchor bolts to the nearest edge of concrete is 250 mm. Seismic support lengths must be checked and Code requirements met.

8. **Bearing Selection.** Bearing selection is influenced by many factors such as loads, geometry, maintenance, available clearance, displacement, rotation, deflection, availability, policy, designer preference, construction tolerances and cost.

In general, vertical displacements are prevented, rotations are allowed to occur as freely as possible, and horizontal displacements may be either accommodated or prevented. The loads should be distributed among the bearings in accordance with the superstructure analysis.

Unless conditions dictate otherwise, conventional steel radius plate bearings should be used for fixed shoes of prestressed girder bridges and small steel girder structures. All expansion bearings of both steel and prestressed girder bridges and fixed bearings of larger steel bridges will be designed using elastomeric bearings. Plain elastomeric bearings will accommodate small amounts of movement; however, when the practical limits of the plain bearing pads are exceeded, the designer must consider using Polytetrafluorethylene (PTFE) sliding bearings, commonly referred to as Teflon or TFE bearings, in conjunction with a stainless steel sliding surface and a steel-reinforced elastomeric bearing pad. See Figure 19.3A for a general summary of expansion bearing capabilities. The values shown in the table are for general guidance only. For large or unusual structures not commonly constructed in Montana, more elaborate bearing systems may be required.

The final step in the selection process consists of completing a design of the bearing in accordance with the LRFD Specifications. The resulting design will provide the geometry and other pertinent specifications for the bearing.

On structure widenings, the designer is cautioned against mismatching bearing types. Yielding type bearings, such as

elastomeric, should not be used in conjunction with non-yielding type bearings.

Girder bridges without integral abutments must have at least one fixed bearing line. If integral abutments meeting the empirical design limits outlined in Chapter 19 are used, interior fixed bearings are not required.

9. **Anchor Bolts.** Use swedged anchor bolts to connect all steel and elastomeric bearing assemblies to the concrete beam seat. Bolts will be sized to accommodate anticipated longitudinal and transverse design forces. Where anchor bolts lie within the confines of the backwall on semi-integral abutments, use smooth dowel rods with expansion caps to allow for future grade adjustments.

### 19.3.2 **Fixed Steel Bearings**

#### 19.3.2.1 **General**

The top plate of steel bearings shall be at least as wide as the bottom girder flange plus sufficient added width to accommodate the anchor bolts and nuts.

When the flexibility of tall, slender piers is sufficient to absorb the horizontal movement at the bearings due to temperature change without developing undue force in the superstructure, bearings or pier, then two or more piers may be fixed to distribute the longitudinal force among the piers.

#### 19.3.2.2 **Design**

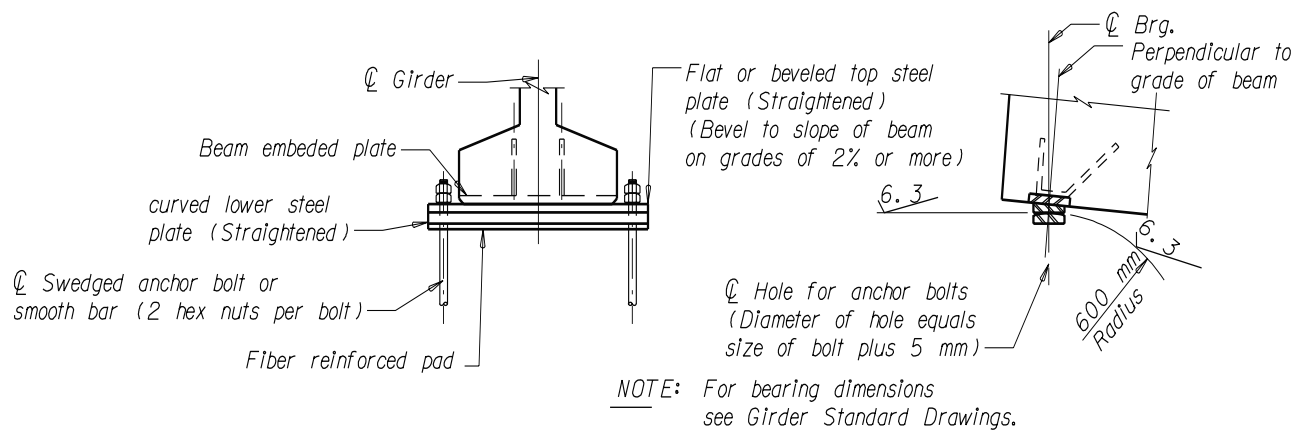
Figure 19.3B illustrates representations of the steel rock plate bearings used by MDT.



Bearing Type	Load		Translation		Rotation	Costs	
	Min. (kN)	Max. (kN)	Min. (mm)	Max. (mm)		Initial	Maintenance
Elastomeric Pads							
Plain (PEP)	0	450	0	15	0.01	Low	Low
Cotton Duck-Reinforced(CDP)	0	1400	0	5	0.003	Low	Low
Fiberglass-Reinforced(FGP)	0	600	0	25	0.015	Low	Low
Steel-Reinforced Elastomeric Bearing	225	3500	0	100	0.04	Low	Low
Flat PTFE Slider (Polytetrafluorethylene)	0	>10,000	25	>100	0	Low	Moderate
Curved Sliding Cylindrical	0	7000	0	0	> 0.04	Moderate	Moderate
Pot Bearing	1200	10,000	0	0	0.02	Moderate	High
Curved PTFE	1200	7000	0	0	> 0.04	High	Moderate

SUMMARY OF EXPANSION BEARING CAPABILITIES

Figure 19.3A



### STEEL ROCKER PLATE SHOE DETAILS

Figure 19.3B

The fixed shoe details are representative of typical steel rocker plate bearings discussed previously in this Chapter. Standard fixed shoe details are included on each standard prestressed girder drawing and need not be designed or covered elsewhere in the contract plans and specifications. Fixed shoe details for steel bridges or other non-standard applications will need to be designed and shown on the plans. Design requirements are simply to size the bearing such that concrete and steel stresses remain within an acceptable range throughout the controlling service and extreme event load conditions. Typical design checks would be for compression of the concrete under the bearing plate and for bearing plate bending about the bottom flange of the beam. Bearing anchor bolts will be designed to resist the resulting stresses from the combined transverse and longitudinal forces applied at the bearings.

#### 19.3.3 Steel-Reinforced Elastomeric Bearings

Reference: LRFD Articles 14.7.5 and 14.7.6

The behavior of steel-reinforced elastomeric bearings is influenced by the shape factor (S) where:

$$S = \frac{\text{Plan Area}}{\text{Area of Perimeter Free to Bulge}}$$

It is usually desirable to orient elastomeric bearings so that the long side is parallel to the principal axis of rotation, because this facilitates the accommodation of rotation. If holes are placed in a steel-reinforced bearing, the steel reinforcement thickness should be increased in accordance with LRFD Article 14.7.5.3.7.

Steel-reinforced elastomeric bearings have many desirable attributes. They are usually a low-cost option, and they require minimal maintenance. Further, these components are relatively forgiving if subjected to loads, movements or rotations that are slightly larger than those considered in their design. This is not to encourage the engineer to underdesign elastomeric bearings, but it simply notes that extreme events, which have a low probability of occurrence, will have far less serious consequences with these elastomeric components than with other bearing systems.

##### 19.3.3.1 Elastomer

Reference: LRFD Articles 14.7.5.2 and 14.7.6.2

Both natural rubber and neoprene are used in the construction of bridge bearings. The differences between the two are usually not very significant. Neoprene has greater resistance than natural rubber to ozone and a wide range of chemicals, and so it is more suitable for some harsh chemical environments. However, natural rubber generally stiffens less than neoprene at low temperatures.

All elastomers are visco-elastic, nonlinear materials and, therefore, their properties vary with strain level, rate of loading and temperature. Bearing manufacturers evaluate the materials on the basis of Shore A Durometer hardness, but this parameter is not a good indicator of the shear modulus,  $G$ . A Shore A Durometer hardness of 50 to 60 will be used in Montana, and this leads to shear modulus values in the range of 0.78 to 1.14 (use least favorable value for design) MPa @23°C. The shear stiffness of the bearing is its most important property because it affects the forces transmitted between the superstructure and substructure.

Elastomers are flexible under shear and uniaxial deformation, but they are very stiff against volume changes. This feature makes possible the design of a bearing that is stiff in compression but flexible in shear.

Elastomers stiffen at low temperatures. The low-temperature stiffening effect is very sensitive to the elastomer compound and the increase in shear resistance can be controlled by selection of an elastomer compound that is appropriate for the climatic conditions. For Montana, the minimum low-temperature elastomer shall be Grade 4, unless Special Provisions are used, in which case Grade 3 is acceptable. The designer shall indicate the elastomer grade in the contract documents.

### 19.3.3.2 Behavior of Steel-Reinforced Elastomeric Bearing Pads

Steel-reinforced elastomeric bearings are often categorized with elastomeric bearing pads, but the steel reinforcement makes their behavior

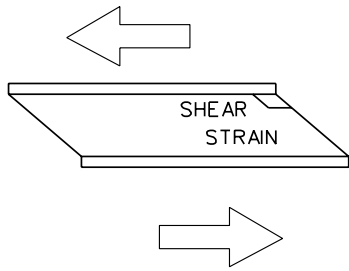
quite different. Steel-reinforced elastomeric bearings have uniformly spaced layers of steel and elastomer. The bearing accommodates translation and rotation by deformation of the elastomer. The elastomer is flexible under shear stress but stiff against volumetric changes. Under uniaxial compression, the flexible elastomer would shorten significantly and sustain large increases in its plan dimension, but the stiff steel layers restrain this lateral expansion. This restraint induces a bulging pattern as shown in Figure 19.3C and provides a large increase in stiffness under compressive load. This permits a steel-reinforced elastomeric bearing to support relatively large compressive loads while accommodating large translations and rotations.

The design of a steel-reinforced elastomeric bearing pad requires an appropriate balance of compressive, shear and rotational stiffnesses. The shape factor affects the compressive and rotation stiffness, but it has no impact on the translational stiffness or deformation capacity.

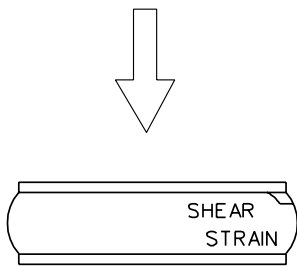
A bearing pad must be designed to control the stress in the steel reinforcement and the strain in the elastomer. This is done by controlling the elastomer layer thickness and the shape factor of the bearing. Fatigue, stability, delamination, yield and rupture of the steel reinforcement, stiffness of the elastomer, and geometric constraints must be satisfied.

Large rotations and translations require taller bearings. Translations and rotations may occur about the longitudinal or transverse axis of a steel-reinforced elastomeric bearing.

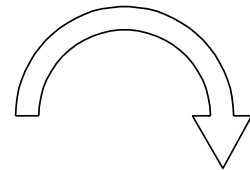
Steel-reinforced elastomeric bearings become large if they are designed for loads greater than about 3000 kN. Uniform heating and curing during vulcanization of such a large mass of elastomer becomes difficult, because elastomers are poor heat conductors. Manufacturing constraints thus impose a practical upper limit on the size of most steel-reinforced elastomeric bearings. If the design loads exceed 3000 kN, the designer should check with the manufacturer for availability.



A) SHEAR DEFORMATION



C) COMPRESSION



B) ROTATIONAL DEFORMATION

**STRAINS IN A STEEL REINFORCED ELASTOMERIC BEARING**

**Figure 19.3C**

#### **19.3.4 Design of Steel-Reinforced Elastomeric Bearing Pads**

Reference: LRFD Articles 14.7.5 and 14.7.6

Steel-reinforced elastomeric bearings may be designed using either of two methods, commonly referred to as Method A and Method B. The Method A procedure, which is typically used in Montana, found in the LRFD Specifications, Article 14.7.6 shall be used for conventional elastomeric bearings. The Method B procedure found in the LRFD Specifications, Article 14.7.5 shall be used for high-capacity bearings, which are not typically used by MDT.

Design criteria for both Methods are based upon satisfying fatigue, stability, delamination, steel-reinforcement yield/rupture, and elastomer stiffness requirements. The design of a steel-reinforced elastomeric bearing requires an appropriate balance of compressive, shear and rotational stiffnesses. The shape factor, as defined by the steel shim spacing, significantly affects the compressive and rotational stiffness of the bearing. However, it has no impact on the translational stiffness of the bearing or its translational deformation capacity.

The minimum elastomeric bearing length or width shall be 150 mm. All pads shall be 50 to 60 durometer hardness. For overall bearing heights less than 90 mm, a minimum of 3 mm of side clearance shall be provided beyond the edges of the steel shims. For overall heights over 90 mm, a minimum of 6 mm of side clearance shall be provided. The top and bottom cover layers shall be no more than 70 percent of the thickness of the interior layers.

In determining bearing pad thicknesses, it should be assumed that slippage will not occur. The total elastomer thickness shall be no less than twice the maximum longitudinal or transverse deflection. If the factored shear force sustained by the deformed pad at the strength limit state exceeds one-fifth of the minimum vertical force due to permanent loads, the bearing shall be secured against horizontal movement.

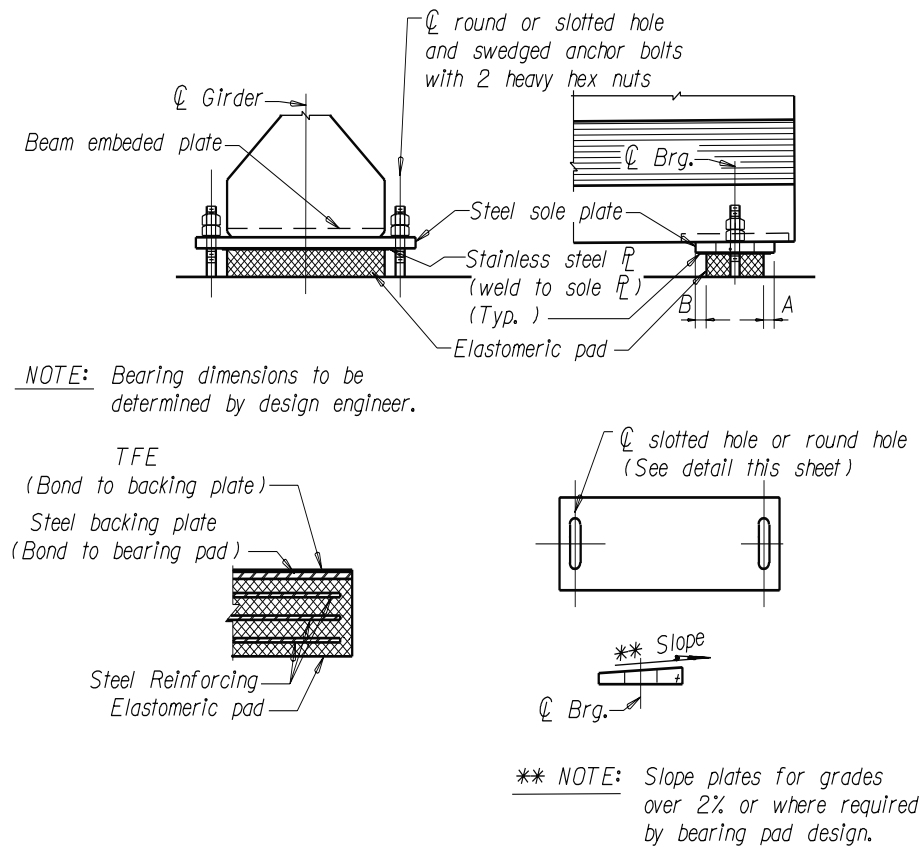
Figure 19.3D illustrates representations of the elastomeric bearings used by MDT.

Plain or reinforced elastomeric bearings, whether fixed or expansion, are to be custom designed for each required location within the structure. It is the project design engineer's responsibility to size the pads, plates and anchorage and to provide the design information to the detailer to be placed on the plans. General required plan information is shown in Figure 19.3D. Modify this information as needed for specific situations. Include the size, thickness and layering information of the pad, the size, thickness hole, dimensions and beveling of the sole plate and the anchor bolt size. Also required is a Table of Expansion Shoe Dimensions with the shoe adjustment per 1 degree of temperature change. The temperature range, total design movement and the bearing pad design load must be documented in the plans.

Design the pads using Method A, Article 14.7.6 of the LRFD Specifications or MDT's internal design software documented in Chapter 25 of this **Manual**. For normal situations, calculations for design movements can be limited strictly to temperature change. As a general rule, shrinkage and creep calculations need not be included in the design movement for bearing pads. For fixed shoes, the holes in the plate will typically be 5 mm larger than the bolt diameter. For expansion bearings slotted holes should be 5 mm wider than the bolt and sized in length to accommodate the full design movement plus two times the bolt diameter. Sliding surfaces will be TFE on stainless steel. TFE is bonded to the elastomeric pad and the stainless steel is welded to the steel sole plate. Sole plate dimensions are to be larger than the pad and TFE so that the TFE is fully protected from dirt and debris during the full range of shoe movement.

#### **19.3.5 Seismic Design**

This Section discusses seismic design for bearing assemblies.



**ELASTOMERIC SHOE DETAIL**

**Figure 19.3D**

**19.3.5.1 Application**

All bridges shall be designed in accordance with the LRFD Specifications. Most of Montana is classified by AASHTO as being in Seismic Performance Zone 1. The Missoula and Butte Districts, however, are characterized as being of high seismic risk with acceleration coefficients high enough to significantly affect the bridge designs.

**19.3.5.2 Seismic Performance Zone 1 Criteria**

Reference: LRFD Articles 3.10.9 and 4.7.4.4

All bridges shall comply with the following LRFD Specifications criteria for Zone 1:

1. Minimum Support Length. Adequate support length is probably the most important contributor to satisfactory performance of a bridge during a seismic event. The support length required by Article 4.7.4.4 of the LRFD Specifications shall be provided at the expansion ends of all structures unless longitudinal restrainers are provided.
2. Minimum Bearing Force Demands. The connection of the superstructure to the substructure shall be designed to resist a horizontal seismic force equal to 0.20 times the tributary dead load force in the restrained directions. No additional adjustment factors, loading cases or friction forces shall be applied to increase or decrease this minimum horizontal seismic force. This force shall extend into the

substructure design as an extreme event load case.

Fixed bearings, such as steel shoes, shall be attached to the pier cap with anchor bolts. Some examples of acceptable means of restraint at semi-fixed or expansion bearings in Zone 1 include concrete shear keys, beams resting in concrete channels and steel side retainers bolted to the cap.

In designing the bearing connections for Zones 2, 3 and 4, the actual calculated seismic design forces, as adjusted by Article 3.10.7 of the LRFD Specifications, shall be used. The longitudinal seismic forces at expansion bearings may be resisted either by using seismic restraining devices (positive horizontal linkage), or they may be transferred to the bearing connections at the nearest fixed pier. Positive linkage shall be provided by ties, cables, dampers or other equivalent mechanism. Friction shall not be considered a positive linkage.

See Article 3.10.9.6 of the LRFD Specifications to determine if hold-down devices are required.

#### **19.3.5.3 Connections for Fixed Steel Shoes**

The connection between a fixed steel shoe and the pier cap shall be made with anchor bolts. The anchor bolts, the pintles and the high-strength bolts in the top shoe shall be verified that their ultimate shear resistance is adequate to resist the calculated seismic forces. See Article 6.13.2.7 of the LRFD Specifications for determining the nominal shear resistance of anchor bolts and pintles.

The masonry anchor bolts shall extend into the concrete a minimum of 380 mm, and anchor bolts used in seismic performance Zone 2, 3 and 4 shall meet the requirements of Article 14.8.3 of the LRFD Specifications.

Anchor bolts should be located beyond the limits of the bottom flange and avoid conflict with interior diaphragms. Provide adequate clearance

for installation of the nuts. The grade of structural steel used for the anchor bolts or pintles shall be clearly indicated in the plans.

#### **19.3.5.4 Connections for Elastomeric Bearings and PTFE/Elastomeric Bearings**

All elastomeric PTFE/elastomeric bearings shall be provided with adequate seismic-resistant anchorage to resist the transverse horizontal forces in excess of those accommodated by shear in the bearing. The restraint may be provided by one of the following methods:

1. steel side retainers with anchor bolts;
2. concrete shear keys placed in the top of the pier cap, or channel slots formed into the top of cap at the abutments (see Section 19.3.5.5); or
3. concrete channels formed in the top of abutment caps or expansion pier caps.

Steel side retainers and the anchor bolts shall be designed to resist the minimum transverse seismic force for the zone in which the bridge is located. The number of side retainers shall be as required to resist the seismic forces and shall be placed symmetrically with respect to the cross section of the bridge. Many times, side retainers will be required on each side of the girder flange of each beam line. The strength of the beams and diaphragms shall be sufficient to transmit the seismic forces from the superstructure to the bearings.

Concrete channels or shear blocks formed around each beam in the top of abutment caps or expansion pier caps represent an acceptable alternative to steel side retainers. The top of the top shoe plate shall be set a minimum of 50 mm below the top of the concrete channel. The minimum depth of the channel shall be 150 mm. The horizontal clearance from the side of the top shoe or edge of beam to the side wall of the channel shall be 15 mm or less. Adequately reinforce all shear blocks and channels to resist the applied loads.

Integral abutments are a very effective way of accommodating the horizontal seismic forces of Zones 1 and 2. An integrally designed abutment will inherently resist the transverse seismic forces. Minimum support length requirements need not be checked for this type of substructure, provided that the beams are adequately connected to the wall. See Section 19.1.5 for integral abutment requirements.

which includes the testing requirements that will be the responsibility of the bearing supplier.

#### 19.3.5.5 Seismic Isolation Bearings

The use of seismic isolation bearings should be considered for seismic retrofit of continuous steel bridges in Seismic Zones 2, 3 and 4. MDT's experience indicates that the savings in substructure rehabilitation cost, resulting from an isolation bearing design, roughly offsets the substantial cost of the isolation bearings. The use of seismic isolation bearings should be based on performing a cost analysis comparing other alternatives, such as elastomeric bearings with suitable retainers or longitudinal restraining devices. The use of seismic isolation bearings in Seismic Zone 1 is not cost effective.

The minimum bearing support length requirements of the LRFD Specifications for seismic design shall be satisfied at the expansion ends of bridges with seismic isolation bearings. The minimum bearing force demands should be assumed to be the actual calculated seismic forces.

Seismic isolation bearings significantly reduce the seismic forces on the substructure, possibly to the point where a non-seismic load case may control the pier design. This, however, does not relieve the designer of the need to provide pile anchorage, confinement steel in plastic hinge regions and proper location of lap splices. The design of seismic isolation bearings shall be in accordance with the **AASHTO Guide Specifications for Seismic Isolation Design, 1999**. The LRFD Specifications requires that all bearing systems shall be tested under both static and cyclic loading prior to acceptance. The designer shall prepare a Special Provision,