Chapter 9 Bearings and Expansion Joints

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9.1 Expansion Joints

9.1.1 General Considerations

All bridges must accommodate, in some manner, environmentally and self-imposed phenomena that tend to make structures move in various ways. These movements come from several primary sources: thermal variations, concrete shrinkage, creep effects from prestressing, and elastic post-tensioning shortening. Except for elastic post- tensioning shortening, which generally occurs before expansion devices are installed, movements from these primary phenomena are explicitly calculated for expansion joint selection and design. Other movement-inducing phenomena include live loading (vertical and horizontal braking), wind, seismic events, and foundation settlement. Movements associated with these phenomena are generally either not calculated or not included in total movement calculations for purposes of determining expansion joint movement capacity.

With respect to seismic movements, it is assumed that some expansion joint damage may occur, that this damage is tolerable, and that it will be subsequently repaired. In cases where seismic isolation bearings are used, the expansion joints must accommodate seismic movements to allow the isolation bearings to function properly.

Expansion joints must accommodate cyclic and long-term structure movements in such a way as to minimize imposition of secondary stresses in the structure. Expansion joint devices must prevent water, salt, and debris infiltration to substructure elements below. Additionally, an expansion joint device must provide a relatively smooth riding surface over a long service life.

Expansion joint devices are highly susceptible to vehicular impact that results from their inherent discontinuity. Additionally, expansion joints have often been relegated a lower level of importance by both designers and contractors. Many of the maintenance problems associated with in-service bridges relate to expansion joints.

One solution to potential maintenance problems associated with expansion joints is to use construction procedures that eliminate the joints from the bridge deck. The two most commonly used methods are called integral and semi-integral construction. These two terms are sometimes collectively referred to as jointless bridge construction.

In integral construction, concrete end diaphragms are cast monolithically with both the bridge deck and supporting pile substructure. To minimize secondary stresses induced in the superstructure, steel piles are generally used in their weak axis orientation relative to the direction of bridge movement. In semi-integral construction, concrete end diaphragms are cast monolithically with the bridge deck. Supporting girders rest on elastomeric bearings within an L-type abutment. Longer semi-integral bridges generally have reinforced concrete approach slabs at their ends. Approach slab anchors, in conjunction with a compression seal device, connect the monolithic end diaphragm to the approach slab. Longitudinal movements are accommodated by diaphragm movement relative to the approach slab, but at the same time resisted by soil passive pressure against the end diaphragm.

Obviously, bridges cannot be built incrementally longer without eventually requiring expansion joint devices. The incidence of approach pavement distress problems increases markedly with increased movement that must be accommodated by the end diaphragm pressing against the backfill. Approach pavement distress includes pavement and backfill settlement and broken approach slab anchors.

Washington State Department of Transportation (WSDOT) has implemented jointless bridge design by using semi-integral construction. Office policy for concrete and steel bridge design is as follows:

9.1.1.A Concrete Bridges

Semi-integral design is used for prestressed concrete girder bridges under 450 feet long and for post-tensioned spliced concrete girder and cast-in-place post-tensioned concrete box girder bridges under 400 feet long. Use L-type abutments with expansion joints at the bridge ends where bridge length exceeds these values. In situations where bridge skew angles exceed 30 degrees, consult the Bearing and Expansion Joint Specialist and the State Bridge Design Engineer for recommendations and approval.

9.1.1.B Steel Bridges

Use L-type abutments with expansion joints at the ends for multiple-span bridges. Semi-integral construction may be used in lieu of expansion joints for single span bridges under 300 feet with the approval of the State Bridge Design Engineer. In situations where the bridge skew exceeds 30 degrees, consult the Bearing and Expansion Joint Specialist and the State Bridge Design Engineer for recommendations and approval.

In all instances, the use of intermediate expansion joints should be avoided wherever possible. The following table provides guidance regarding maximum bridge superstructure length beyond which the use of either intermediate expansion joints or modular expansion joints at the ends is required.

Superstructure Type	Maximum Length (Western WA)		Maximum Length (Eastern WA)	
	Semi-Integral	L-Abutment	Semi-Integral	L-Abutment
Concrete Superstructure				
Prestressed Girder*	450 ft	900 ft	450 ft	900 ft
P.T. Spliced Girder**	400 ft	700 ft***	400 ft	700 ft***
C.I.P.–P.T. box girder	400 ft	700 ft ***	400 ft	700 ft***
Steel Superstructure				
Plate Girder Box girder	300 ft	900 ft	300 ft	700 ft

* Based upon 0.16 in. creep shortening per 100 feet of superstructure length and 0.12 in. shrinkage shortening per 100 feet of superstructure length

** Based upon 0.31 in. creep shortening per 100 feet of superstructure length and 0.19 in. shrinkage shortening per 100 feet of superstructure length

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

Because the movement restriction imposed by a bearing must be compatible with the movements allowed by the adjacent expansion joint, expansion joints and bearings must be designed interdependently and in conjunction with the anticipated behavior of the overall structure.

A plethora of manufactured devices exists to accommodate a wide range of expansion joint total movements. Expansion joints can be broadly classified into three categories based upon their total movement range as follows:

Small Movement Joints Total Movement Range < 1¾ in.
Medium Movement Joints 1¾ in. < Total Movement Range < 5 in.
Large Movement Joints Total Movement Range > 5 in.

9.1.2 General Design Criteria

Expansion joints must be sized to accommodate the movements of several primary phenomena imposed upon the bridge following installation of its expansion joint devices. Concrete shrinkage, thermal variation, and long-term creep are the three most common primary sources of movement. Calculation of the movements associated with each of these phenomena must include the effects of superstructure type, tributary length, fixity condition between superstructure and substructure, and pier flexibilities.

9.1.2.A Shrinkage Effects

Accurate calculation of shrinkage as a function of time requires that average ambient humidity, volume-to-surface ratios, and curing methods be taken in consideration as summarized in LRFD-BDS Article 5.4.2.3.3. Because expansion joint devices are generally installed in their respective blockouts at least 30 to 60 days following concrete deck placement, they must accommodate only the shrinkage that occurs from that time onward. For most situations, that shrinkage strain can be assumed to be 0.0002 for normal weight concrete in an unrestrained condition. This value must be corrected for restraint conditions imposed by various superstructure types.

$$\Delta L_{shrink} = \beta \cdot \mu \cdot L_{trib} \qquad (9.1.2\text{-}1)$$

Where:

L_{trib} = Tributary length of the structure subject to shrinkage
β = Ultimate shrinkage strain after expansion joint installation; estimated as 0.0002 in lieu of more refined calculations
μ = Restraint factor accounting for the restraining effect imposed by superstructure elements installed before the concrete slab is cast
= 0.0 for steel girders, 0.5 for precast prestressed concrete girders, 0.8 for concrete box girders and T-beams, 1.0 for concrete flat slabs

9.1.2.B Thermal Effects

Bridges are subject to all modes of heat transfer: radiation, convection, and conduction. Each mode affects the thermal gradients induced and deflection patterns generated in a bridge superstructure differently. Climatic influences vary geographically resulting in different seasonal and diurnal average temperature variations. Additionally, different types of construction have different thermal “inertia” properties. For example, a massive concrete box girder bridge will be much slower to respond to an imposed thermal stimulus than would a steel plate girder bridge composed of many relatively thin steel elements.

Variation in the superstructure average temperature produces uniform elongation or shortening. Uniform thermal movement range is calculated using the maximum and minimum anticipated bridge superstructure average temperatures in accordance with LRFD-BDS Article 3.12.2.1 Procedure A. For establishing the maximum and minimum

design temperatures using Procedure A, most of western Washington is classified as a moderate climate. Eastern Washington and higher elevation areas of western Washington having more than 14 days per year with an average temperature below 32˚F are classified as a cold climate. The maximum and minimum design temperatures, T_U and T_L, respectively, used for uniform thermal movement effects, taken from LRFD-BDS Table 3.12.2.1-1 are:

Moderate Climate:

Concrete Bridges:	10˚F to 80˚F
Steel Bridges:	0˚F to 120˚F

Cold Climate:

Concrete Bridges:	0˚F to 80˚F
Steel Bridges:	-30˚F to 120˚F

Total unfactored thermal movement range is then calculated as:

$$\Delta L_{temp} = \alpha \cdot L_{trib} \cdot \delta T \qquad (9.1.2\text{-}2)$$

Where:

L_{trib} = Tributary length of the structure subject to thermal variation

α = Coefficient of thermal expansion; 0.000006 in./in./˚F for concrete and 0.0000065 in./in./˚F for steel

δT = Bridge superstructure average temperature range as a function of bridge type and location

As noted in LRFD-BDS Article 3.4.1, the larger of the two load factors for uniform temperature, γ_{TU}, provided in LRFD-BDS Table 3.4.1-1 shall be used to calculate factored uniform thermal movements. Design of expansion joints requires calculation of the maximum and minimum temperatures associated with the factored temperature range. Assuming that the unfactored and factored temperature range are centered upon each other, the factored minimum and maximum design temperatures are:

$$T_{min} = .5 \cdot (T_L + T_U) - .5 \cdot \gamma_{TU} \cdot (T_U - T_L)$$
$$T_{max} = .5 \cdot (T_L + T_U) + .5 \cdot \gamma_{TU} \cdot (T_U - T_L)$$

Where

T_{min} = Minimum factored design temperature

T_{max} = Maximum factored design temperature

T_L = Minimum (unfactored) design temperature

T_U = Maximum (unfactored) design temperature

γ_{TU} = Load Factor as specified in LRFD-BDS

In accordance with the *Standard Specifications*, contract drawings state dimensions at the normal temperature of 64˚F unless specifically noted otherwise. Construction and fabrication activities at average temperatures other than 64˚F require the Contractor or fabricator to adjust lengths of structural elements and concrete forms accordingly.

Some expansion joint devices are installed in pre-formed concrete blockouts sometime after the completion of the bridge deck. The expansion joint device must be cast into its respective blockout with a gap setting corresponding to the ambient superstructure average temperature at the time the blockouts are filled with concrete. To accomplish this, expansion device gap settings must be specified on the contract drawings as a function of superstructure ambient average temperature. Generally, these settings are specified for temperatures of 40˚F, 64˚F, and 80˚F.

9.1.3 Small Movement Range Joints

Elastomeric compression seals, poured sealants, asphaltic plugs, pre-formed closed cell foam, epoxy-bonded elastomeric glands, steel sliding plates, and bolt-down elastomeric panels have all been used in the past for accommodating small movement ranges. The current policy is to use compression seals and rapid-cure silicone sealants almost exclusively.

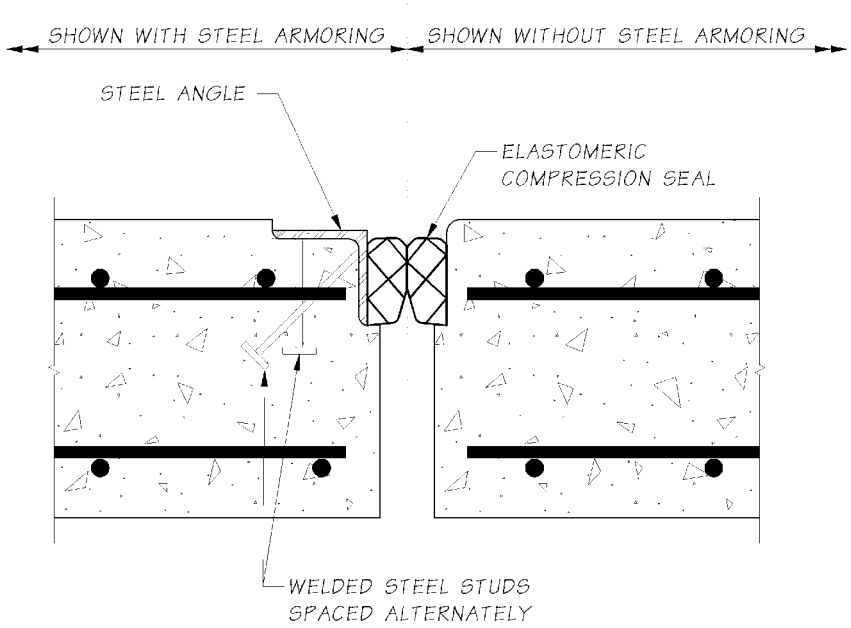
9.1.3.A Compression Seals

Compression seals are continuous manufactured elastomeric elements, typically with extruded internal web systems, installed within an expansion joint gap to effectively seal the joint against water and debris infiltration. Compression seals are held in place by mobilizing friction against adjacent vertical joint faces. Design philosophy requires that they be sized and installed to always be in a state of compression.

Compression seals can be installed against smooth vertical concrete faces or against steel armoring. When installed against concrete, special concrete nosing material having enhanced impact resistance may be used, particularly on rehabilitation projects. Polyester concrete and elastomeric concrete have been used successfully. Consult the Bearing and Expansion Joint Specialist for current policy.

Each elastomeric compression seal shall be furnished and installed as a single, continuous piece across the full width of the bridge deck. No field splices of the compression seal shall be allowed. For widening projects, a new compression seal shall be furnished and installed as a single, continuous piece across the full width of the original and widened portions of the roadway. Field splicing to the original elastomeric compression seal shall not be allowed.

Figure 9.1.3-1 Compression Seal Joint



In design calculations, the minimum and maximum compressed widths of the seal are generally set at 40 percent and 85 percent of the uncompressed width. These measurements are perpendicular to the joint axis. It is generally assumed that the compressed seal width at the normal construction temperature of 64˚F is 60 percent of

its uncompressed width. For skewed joints, bridge deck movement must be separated into components perpendicular to and parallel to the joint axis. Shear displacement of the compression seal should be limited to a specified percentage of its uncompressed width, usually set at about 22 percent. Additionally, the expansion gap width should be set so that the compression seal can be replaced over a reasonably wide range of construction temperatures. Manufacturers' catalogues generally specify the minimum expansion gap widths into which specific size compression seals can be installed. The expansion gap width should be specified on the contract drawings as a function of the superstructure average temperature.

Compression seal movement design relationships can be expressed as:

$\Delta L_{temp\text{-}normal} = \Delta L_{temp} \cdot \cos\theta$ [thermal movement normal to joint]

$\Delta L_{temp\text{-}parallel} = \Delta L_{temp} \cdot \sin\theta$ [thermal movement parallel to joint]

$\Delta L_{shrink\text{-}normal} = \Delta L_{shrink} \cdot \cos\theta$ [shrinkage movement normal to joint]

$\Delta L_{shrink\text{-}parallel} = \Delta L_{shrink} \cdot \sin\theta$ [shrinkage movement parallel to joint]

$W_{min} = W_{install} - [(T_{max} - T_{install})/(T_{max} - T_{min})] \cdot \Delta L_{temp\text{-}normal} > 0.40 \cdot W$

$W_{max} = W_{install} + [(T_{install} - T_{min})/(T_{max} - T_{min})] \cdot \Delta L_{temp\text{-}normal} + \Delta L_{shrink\text{-}normal} < 0.85 \cdot W$

Where:

θ = skew angle of the expansion joint, measured with respect to a line perpendicular to the bridge longitudinal axis

W = uncompressed width of the compression seal

$W_{install}$ = expansion gap width at installation

$T_{install}$ = superstructure temperature at installation

W_{min} = minimum expansion gap width

W_{max} = maximum expansion gap width

T_{min} = minimum superstructure average temperature

T_{max} = maximum superstructure average temperature

Algebraic manipulation yields:

$W > (\Delta L_{temp\text{-}normal} + \Delta L_{shrink\text{-}normal})/0.45$

$W > (\Delta L_{temp\text{-}parallel} + \Delta L_{shrink\text{-}parallel})/0.22$

Now, assuming $W_{install} = 0.6 \cdot W$,

$W_{max} = 0.6 \cdot W + [(T_{install} - T_{min})/(T_{max} - T_{min})] \cdot \Delta L_{temp\text{-}normal} + \Delta L_{shrink\text{-}normal} < 0.85 \cdot W$

Rearranging yields:

$W > 4 \cdot [(T_{install} - T_{min})/(T_{max} - T_{min}) \cdot \Delta L_{temp\text{-}normal} + \Delta L_{shrink\text{-}normal}]$

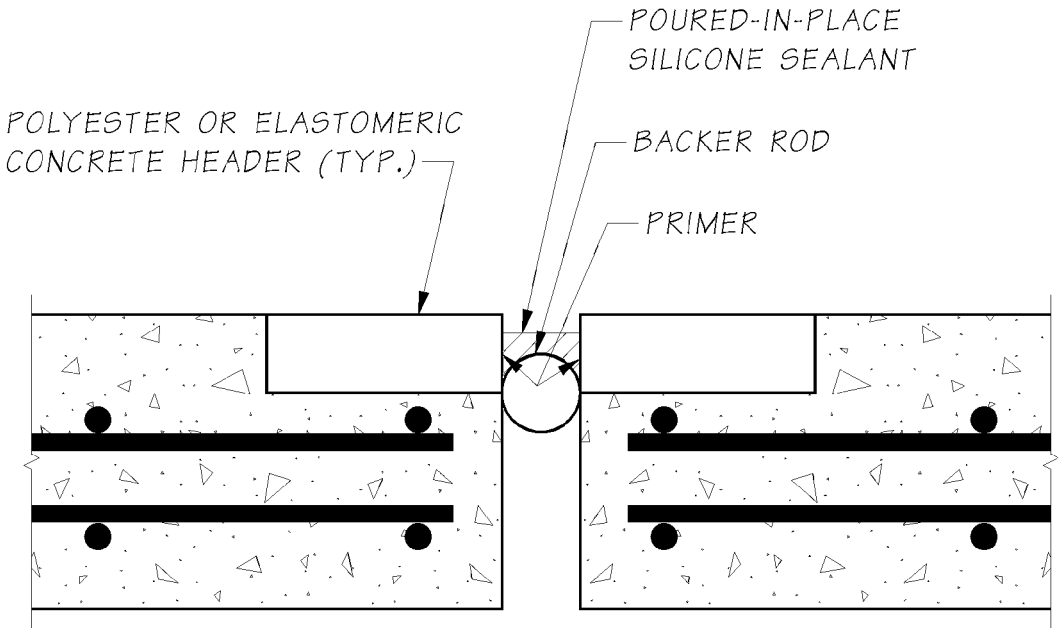
9.1.3.B Rapid-Cure Silicone Sealants

Durable low-modulus poured sealants provide watertight expansion joint seals in both new construction and rehabilitation projects. Most silicone sealants possess good elastic performance over a wide range of temperatures while demonstrating high levels of resistance to ultraviolet and ozone degradation. Other desirable properties include self-leveling and self-bonding characteristics.

Rapid-cure silicone sealants are particularly good candidates for rehabilitation in situations where significant traffic disruption consequential to extended traffic lane closure is unacceptable. Additionally, unlike compression seals, rapid-cure silicone sealants do not require straight, parallel substrate surfaces in order to create a watertight seal.

Rapid-cure silicone sealants can be installed against either concrete or steel. It is extremely critical that concrete or steel substrates be thoroughly cleaned before the sealant is installed. Some manufacturers require application of specific primers onto substrate surfaces prior to sealant installation to enhance bonding. Consult the Bearing and Expansion Joint Specialist for specifics.

Figure 9.1.3-2 Rapid-Cure Silicone Sealant Joint



Rapid-cure silicone sealants should be designed based upon the manufacturer's recommendations. Maximum and minimum working widths of the poured sealant joint are generally recommended as a percentage of the sealant width at installation. Depending upon the manufacturer, these joints can accommodate tensile movements of up to 100 percent and compressive movements of up to 50 percent of the sealant width at installation. A minimum recess is typically required between the top of the roadway surface and the top of the sealant surface. This recess is critical in assuring that tires will not contact the top surface of the sealant and initiate its debonding from substrate material.

It is important to recognize that the maximum allowable opening and closing design movements are a function of the joint width at the time the sealant is poured. As such, consideration should be given that the poured sealant can be installed over a reasonably wide range of construction temperatures and still perform, over the full factored uniform temperature range, within the allowable movement limits recommended by the manufacturer. In some instances, it may be necessary to stipulate that the Contractor install the sealant within a more restrictive range of bridge temperatures than the commonly assumed 40˚F to 80˚F.

For an existing expansion joint having a known expansion gap width at a known bridge uniform temperature, maximum and minimum installation temperatures can be calculated. Assuming that all shrinkage and long-term creep effects have already occurred, it can be shown, algebraically, that

$$T_{install,min} = \frac{1}{1-\zeta_c}\left[T_{max} - \zeta_c \cdot T - \frac{\zeta_c \cdot W_T}{\alpha \cdot L_{trib}}\right]$$

$$T_{install,max} = \frac{1}{1+\zeta_e}\left[T_{min} + \zeta_e \cdot T + \frac{\zeta_e \cdot W_T}{\alpha \cdot L_{trib}}\right]$$

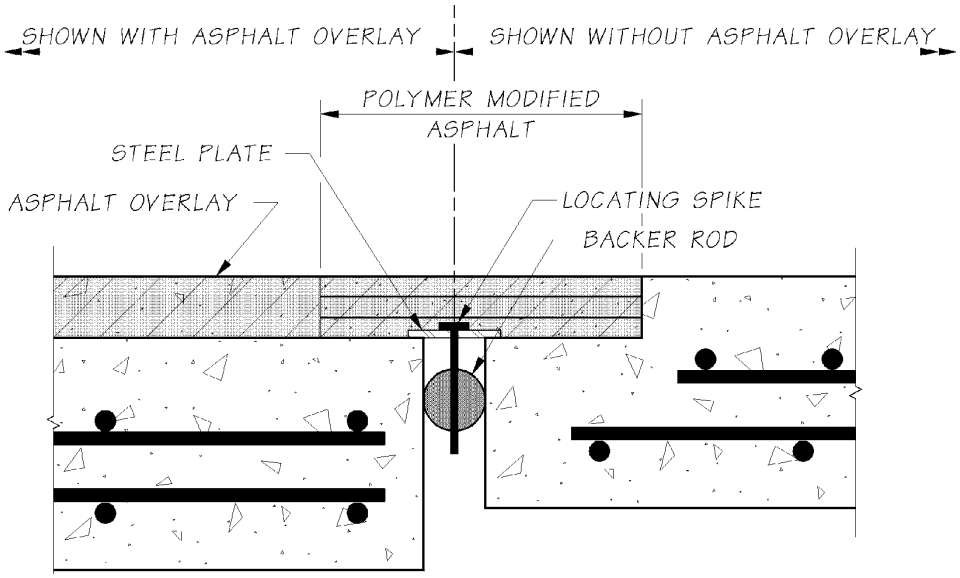
Where

W_T = existing expansion gap width
T = bridge uniform temperature at time W_T is measured
ζ_c = maximum allowable closure (contraction) of the joint as a fraction of its width at the time the sealant is installed, as recommended by the manufacturer for design purposes
ζ_e = maximum allowable opening (expansion) of the joint as a fraction of its width at the time the sealant is installed, as recommended by the manufacturer for design purposes
α = coefficient of thermal expansion of the bridge superstructure
L_{trib} = tributary length contributing to uniform thermal movement
T_{min} = minimum factored design temperature
T_{max} = maximum factored design temperature

9.1.3.C Asphaltic Plug Joints

Asphaltic plug joints consist of a flexible polymer modified asphalt installed in a preformed blockout atop a steel plate and backer rod. In theory, asphaltic plug joints provided a seamless smooth riding surface. However, when subjected to high traffic counts, heavy trucks, or substantial acceleration/deceleration traction, the polymer modified asphalt tends to creep, migrating out of the blockouts. As a consequence, WSDOT no longer specifies the use of asphaltic plug joints.

Figure 9.1.3-3 Asphaltic Plug Joint



9.1.3.D Headers

Expansion joint headers for new construction are generally the same Class 4000D structural concrete as used for the bridge deck and cast integrally with the deck.

Expansion joint headers installed as part of a rehabilitative and/or overlay project are constructed differently.

Being a flexible material, hot mix asphalt (HMA) cannot provide rigid lateral support to an elastomeric compression seal or a rapid cure silicone sealant bead. Therefore, rigid concrete headers must be constructed on each side of such an expansion joint when an HMA overlay is installed atop an existing concrete deck. These headers provide a rigid lateral support to the expansion joint device and serve as a transition between the HMA overlay material and the expansion joint itself.

WSDOT allows either polyester concrete or elastomeric concrete for expansion joint headers. These two materials, which provide enhanced durability to impact in regard to other concrete mixes, shall be specified as alternates in the contract documents. General Special Provisions specify the material and construction requirements for polyester and elastomeric concrete.

Because HMA typically experiences additional compaction after placement and is more susceptible to wheel line rutting than concrete, the finished header surface shall be set slightly below the adjacent HMA. This vertical recess shall be dimensioned on plan sheets as ¼", +/- 1/8".

Modified concrete overlay (MCO) material can provide rigid side support for an elastomeric compression seal or a rapid cure silicone sealant bead without the need for separately constructed elastomeric concrete or polyester concrete headers. This alternative approach requires the approval of the Bearing and Expansion Joint Specialist. Such modified concrete overlay headers may utilize welded wire fabric as reinforcement. Contract 7108 which includes Bridges No. 90/565N&S and 90/566N&S is an example.

9.1.3.E Bridge Paving Joint Seals

Sawcut joints with poured rubber are allowed on the bridge deck provided that the tributary length for calculating movement demand at the joint location does not exceed 50 feet and that the adverse consequence of leakage through the joint is minimal. This is applicable to both concrete bridge decks and to HMA overlays. Sawcut joints with poured rubber are not recommended where leakage through the deck could cause accelerated corrosion to structural steel elements, including steel bearing assemblies.

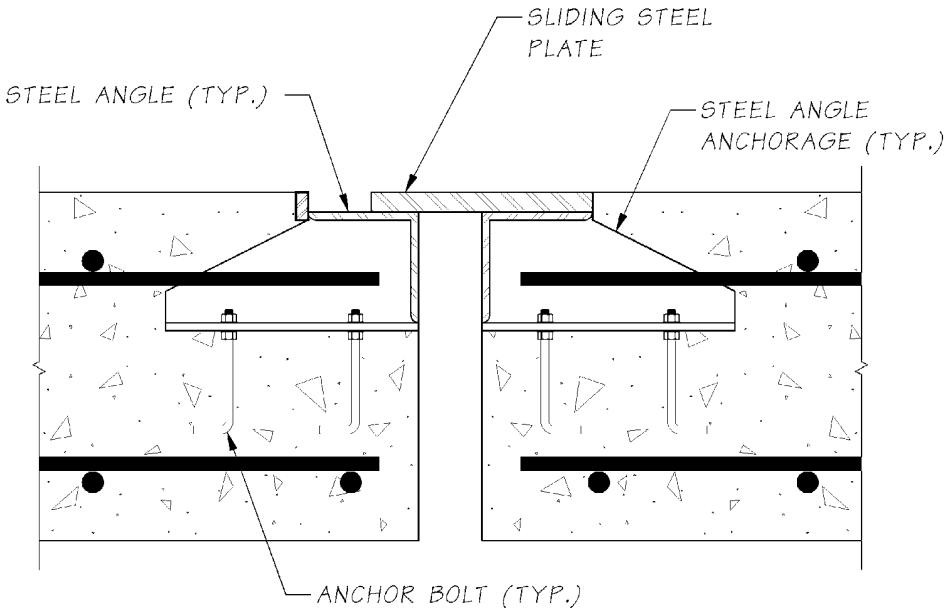
9.1.4 Medium Movement Range Joints

Steel sliding plates, strip seals, and bolt-down panel joints have all been used in the past for accommodating medium movement ranges. The current policy is to use strip seal joints almost exclusively.

9.1.4.A Steel Sliding Plate Joints

Two overlapping steel plates, one attached to the superstructure on each side of the joint, can be used to provide a smooth riding surface across an expansion joint. Unfortunately, steel sliding plates do not generally provide an effective barrier against intrusion of water and deicing chemicals into the joint and onto substructure elements. Consequently, these joints have been supplanted by newer systems, such as strip seals, with improved resistance to water penetration.

Figure 9.1.4-1 Steel Sliding Plate Joint



Before the advent of more modern systems, steel sliding plates were specified extensively. Their limited use today includes the following specific applications:

1. High pedestrian use sidewalks
2. Modular expansion joint upturns at traffic barriers
3. Roadway applications involving unusual movements (translation and large rotations) not readily accommodated by modular expansion joints.

In these applications, the sliding plates are generally galvanized or painted to provide corrosion resistance.

Repeated impact and corrosion have deteriorated many existing roadway sliding steel plate systems. In many instances, the anchorages connecting the sliding plate to the concrete deck have broken. When the integrity of the anchorages has been compromised, the steel sliding plates must generally be removed in their entirety and replaced with a new, watertight system. Where the integrity of the anchorages has not been compromised, sliding plates can often be retrofitted with poured sealants or elastomeric strip seals. In some applications, steel sliding plate expansion joints can be retrofitted to accommodate an ACP overlay by cutting the steel sliding plate and welding a strip seal steel shape to the existing steel anchorage on each side of the expansion gap. The steel shapes retain the ACP and anchor the strip seal elastomeric seal. In such cases, a vertical recess between the top of the ACP and the top of the steel shape shall be detailed and dimensioned on plan sheets as ¼", +/- 1/8". This recess mitigates additional compaction of the ACP under traffic and a minimal amount of wheel line rutting.

9.1.4.B Strip Seal Joints

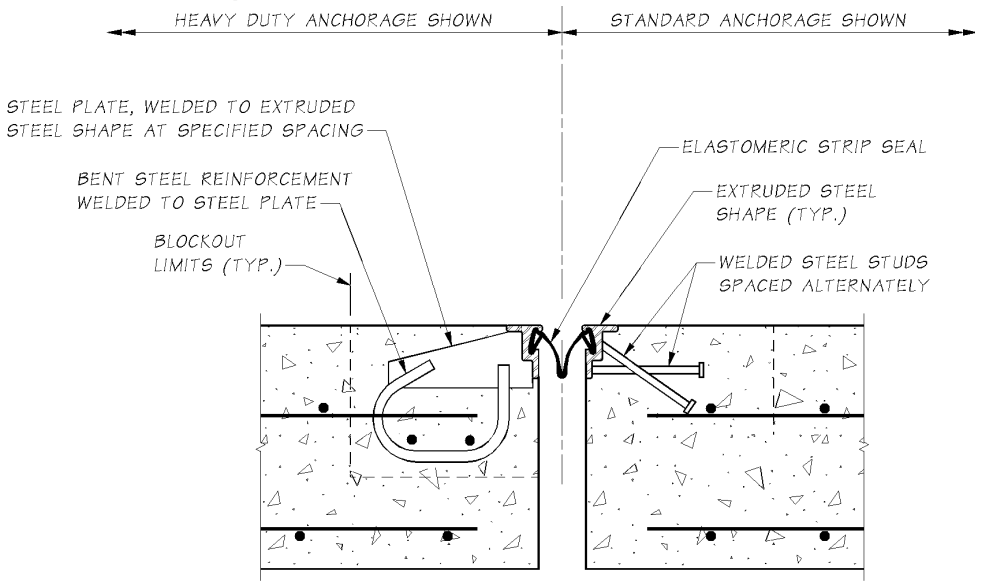
An elastomeric strip seal system consists of a preformed elastomeric gland mechanically locked into metallic edge rails generally embedded into the concrete deck on each side of an expansion joint gap. Unfolding of the elastomeric gland accommodates movement. Steel studs are generally welded to the steel extrusions constituting the edge rails to facilitate anchorage to the concrete deck. Damaged or worn glands can be replaced with minimal traffic disruption.

The metal edge rails effectively armor the edges of the expansion joint, obviating the need for a special impact resistant concrete, usually required at compression seal and poured sealant joints. The designer must select either the standard or special anchorage. The special anchorage incorporates steel reinforcement bar loops welded to intermittent steel plates, which in turn are welded to the extrusion. The special anchorage is generally used for very high traffic volumes or in applications subject to snowplow hits. In applications subject to snowplow hits and concomitant damage, the intermittent steel plates can be detailed to protrude slightly above the roadway surface in order to launch the snowplow blade and prevent it from catching on the forward extrusion.

The special anchorage requires a 9-inch deep blockout, as opposed to a 7-inch deep blockout for the standard anchorage. The standard anchorage is acceptable for high traffic volume expansion joint replacement projects where blockout depth limitations exist.

Metal edge rails may be field spliced using weld procedures provided by the strip seal expansion joint manufacturer. However, elastomeric strip seal elements shall not be field spliced. Each elastomeric strip seal element shall be furnished and installed as a single, continuous piece across the full width of the bridge deck.

Figure 9.1.4-2 Strip Seal Joint

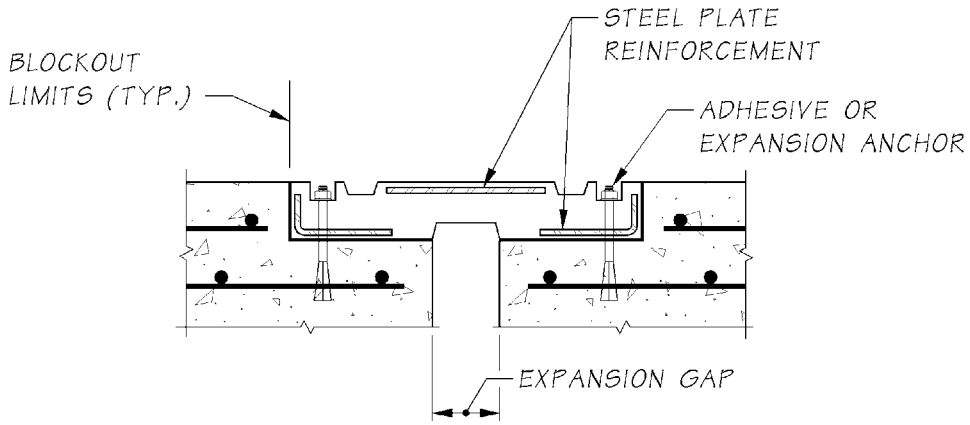


Preformed silicone strip seal systems are an alternative to conventional armored elastomeric strip seal systems. A preformed silicone strip seal comprises an inverted V-shaped, preformed, extruded silicone seal that is installed with a single component silicone locking adhesive and primer. Preformed silicone strip seals can be cut and bonded in the field to effect bends and upturns. Preformed silicone strip seals are being marketed for opening and closing movement ranges between 1½ inches and 5 inches. While steel plate armoring details have been developed for use with preformed silicone strip seals, this armoring is not considered as robust, under traffic, as conventionally armored strip seal joints, particularly when subjected to high traffic volumes and heavy truck loading. Preformed silicone strip seal expansion joints shall be used under traffic only in applications in which the expansion gap associated with the minimum factored design temperature does not exceed 4 inches. For use outside of this range, approval from the State Bridge Design Engineer is required.

9.1.4.C Bolt-down Panel Joints

Bolt-down panel joints, sometimes referred to as expansion dams, are preformed elastomeric panels internally reinforced with steel plates. Bridging across expansion gaps, these panels are bolted into formed blockouts in the concrete deck with either adhesive or expansive anchors. Expansion is accompanied by stress and strain across the width of the bolt-down panel between anchor bolts.

Figure 9.1.4-3 Bolt-down Panel Joint



Because of durability concerns, we no longer specify bolt-down panel joints. On bridge overlay and expansion joint rehabilitation projects, bolt-down panels are being replaced with rapid-cure silicone sealant joints or strip seal joints. For rehabilitation of bridges having low speed or low volume traffic, existing bolt-down panel joints may be retained and/or selective damaged panels replaced.

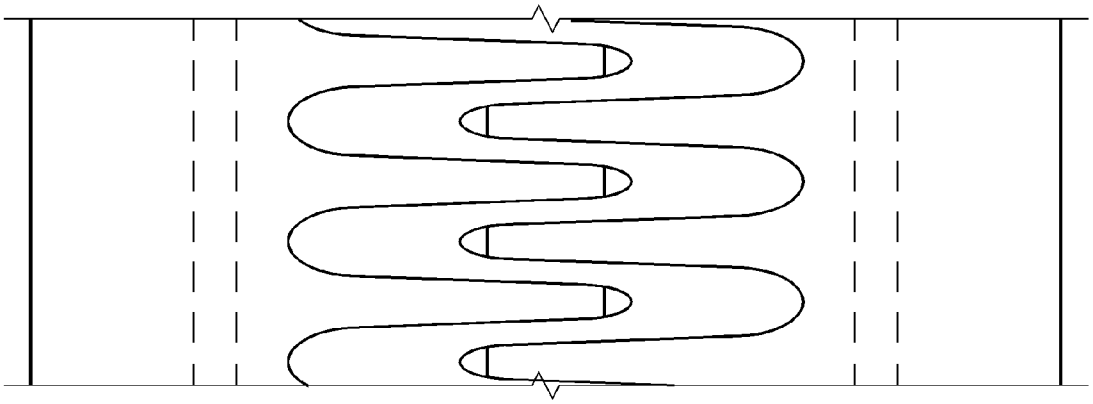
9.1.5 Large Movement Range Joints

Steel finger and modular joints have all been used in the past for accommodating large movement ranges.

9.1.5.A Steel Finger Joints

Finger joints have been successfully used to accommodate medium and large movement ranges. They are generally fabricated from steel plate and are installed in cantilevered configurations. The steel fingers must be designed to support traffic loads with sufficient stiffness to preclude excessive vibration. In addition to longitudinal movement, finger joints must also accommodate any rotations or differential vertical deflection across the joint. Finger joints may be fabricated with a slight downward taper toward the ends of the fingers in order to minimize potential for snowplow blade damage. Unfortunately, finger joints do not provide an effective seal against water infiltration. Elastomeric and metal troughs have been installed beneath steel finger joints to catch and redirect runoff water. However, in the absence of routine maintenance, these troughs clog and become ineffective.

Figure 9.1.5-1 Steel Finger Joint



9.1.5.B Modular Expansion Joints

Modular expansion joints are complex structural assemblies designed to provide watertight wheel load transfer across expansion joint openings. These systems were developed in Europe and introduced into the U.S. in the 1960s. To date, modular expansion joints have been designed and fabricated to accommodate movements of up to 85 inches. In Washington State, the largest modular expansion joints are those on the newest Tacoma Narrows Bridge. These joints accommodate 48 inches of service movement and 60 inches of seismic movement. Modular expansion joints are generally shipped in a completely assembled configuration. Although center beam field splices are not preferable, smaller motion range modular expansion joints longer than 40 feet may be shipped in segments to accommodate construction staging and/or shipping constraints.

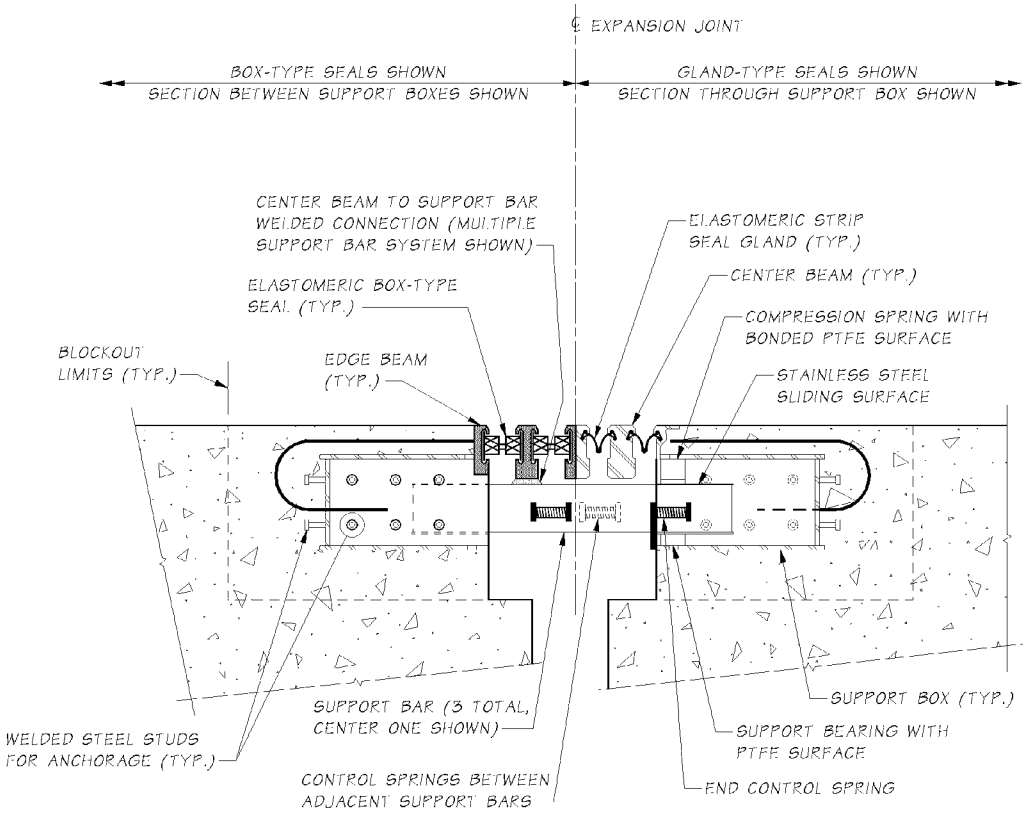
9.1.5.B.1 *Operational Characteristics*

Modular expansion joints comprise a series of steel center beams oriented parallel to the expansion joint axis. Elastomeric strip seals or box-type seals attach to adjacent center beams, preventing infiltration of water and debris. The center beams are supported on support bars, which span in the primary direction of anticipated movement. The support bars are supported on sliding bearings mounted within support boxes. Polytetrafluoroethylene (PTFE) – stainless steel interfaces between elastomeric support bearings and support bars facilitate the unimpeded translation of the support bars as the expansion gap opens and closes. The support boxes generally rest on either cast-in-place concrete or grout pads installed into a preformed blockout.

Modular expansion joints can be classified as either single support bar or multiple support bar systems. In multiple support bar systems, a separate support bar supports each center beam. In the more complex single support bar system, one support bar supports all center beams at each support box location. This design concept requires that each center beam be free to translate along the longitudinal axis of the support bar as the expansion gap varies. This is accomplished by attaching steel yokes to the underside of the center beams. The yoke engages the support bar to facilitate load transfer. Precompressed elastomeric springs and PTFE – stainless steel interfaces between the underside of each center beam and the top of the support bar and between the bottom of the support bar and bottom of the yoke support each center beam and allow it to translate along the longitudinal axis of the support bar. Practical center beam span lengths limit the use of multiple support bar systems for larger movement range modular expansion joints. Multiple support bar systems typically become impractical for more than nine seals, which

corresponds to movement ranges exceeding 27 inches. Hence, the single support bar concept typifies these larger movement range modular expansion joints.

Figure 9.1.5-2 Modular Expansion Joint



The highly repetitive nature of axle loads predisposes modular expansion joint components and connections to fatigue susceptibility, particularly at center beam-to-support bar connections and center beam field splices. Until recently, bolted connections of center beams to support bars have demonstrated poor fatigue endurance. Welded connections have been preferred, but must be carefully designed, fatigue tested, fabricated, and inspected to assure satisfactory fatigue resistance. WSDOT'S current provisions for modular expansion joint systems, contained in *Standard Specifications* Section 6-02.3(13)C, require stringent fatigue-based design and test criteria for modular expansion joints. These provisions also specify criteria for manufacturing, shipping, storing, and installing modular expansion joints.

Modular expansion joints may need to be shipped and/or installed in two or more pieces and subsequently spliced together in order to accommodate project staging and or practical shipping limitations. Splicing generally occurs after concrete is cast into the blockouts. The center beams are the elements that must be connected. These field connections have historically been either welded, bolted, or a hybrid combination of both.

Center beam field splices have historically been the weak link of modular expansion joints because of their high fatigue susceptibility and their tendency to initiate progressive zipper-type failure. The reduced level of quality control achievable with a field operation in regard to a shop operation contributes to this susceptibility. A full penetration weld is the only type of field splice recommended for modular expansion joint center beams. Specific recommendations regarding center beam field splices will be subsequently discussed as they relate to shop drawing review and construction.

9.1.5.B.2 *Movement Design*

Calculated total movement range establishes modular expansion joint size. WSDOT policy has been to provide a 15 percent factor of safety on these calculated service movements. Current systems permit approximately 3 inches of service load movement per elastomeric seal element; hence total service load movement rating provided will be a multiple of 3 inches.

Modular expansion joints must be fully serviceable and maintainable at their full range of factored design temperatures throughout their operational lifetimes. This includes the time before and after all long-term creep and shrinkage have occurred. To minimize impact and wear on bearing elements, the maximum gap between adjacent center beams under service load conditions should be limited to about 3½ inches.

Modular expansion joints are also subject to bridge movements associated with extreme events. Extreme event load combinations include earthquakes and, in the case of floating structures, extreme wind and wave loading. Because the fatigue limit state almost always controls center beam and support bar design, a larger movement capacity per cell is acceptable to accommodate extreme event movements provided that 1) support bars and boxes are detailed to accommodate the increased movement, and 2) detachment of elastomeric seals is acceptable. This is discussed further in Section 9.3 Seismic Isolation Bearings.

To facilitate the installation of modular expansion joints at temperatures other than the 64˚F normal temperature, the contract drawings shall specify expansion gap distance face-to-face of edge beams as a function of the superstructure temperature at the time of installation.

Modular expansion joint movement design relationships can be expressed as:

$$n = MR / mr$$
$$G_{min} = (n - 1) \cdot w + n \cdot g$$
$$G_{max} = G_{min} + MR$$

Where MR = total movement range of the modular expansion joint

mr = movement range per elastomeric seal
n = number of seals
$n - 1$ = number of center beams
w = width of each center beam
g = minimum gap per strip seal element at full closure
G_{min} = minimum distance face-to-face of edge beams
G_{max} = maximum distance face-to-face of edge beams

9.1.5.B.3 *Review of Shop Drawings and Structural Design Calculations*

The manufacturer's engineer generally performs structural design of modular expansion joints. Modular expansion joint provisions, contained in *Standard Specifications* Section 6-02.3(13)C, require that the manufacturer submit structural calculations, detailed fabrication drawings, and applicable fatigue tests for approval by the Engineer. All structural elements must be designed and detailed for both strength and fatigue. Additionally, modular expansion joints should be detailed to provide access for inspection and periodic maintenance activities, including replacement of seals, control springs, and bearing components.

WSDOT's modular expansion joint provisions delineate explicit requirements for their design, fabrication, and installation. These comprehensive provisions build upon WSDOT's past experience specifying modular expansion joints and incorporate the NCHRP Report 402 *Fatigue Design of Modular Bridge Expansion Joints*. The provisions include requirements for the shop drawings, calculations, material certifications, general fabrication methods, corrosion protection, shipping and handling, storage, installation, fatigue testing, applicable welding codes and certifications, quality control, and quality assurance. It is strongly advised to carefully review these provisions before reviewing modular expansion joint shop drawings and calculations.

Any structural details, including connections, that do not clearly correspond to specific fatigue categories depicted in the LRFD shall be fatigue tested in accordance with the requirements stipulated in the modular expansion joint provisions. Documentation of these tests shall accompany the shop drawing submittal.

As stated in the modular expansion joint provisions, the Contractor shall submit documentation of a quality assurance program distinctly separate from in-house quality control. Quality assurance shall be performed by an independent agency and shall be provided by the manufacturer.

Weld procedures shall be submitted for all shop and field welds. These procedures stipulate welding process employed, end preparation of the component welded, weld metal type, preheat temperature, and welder certifications. It is critical that all welds be made in strict accordance with specifications and under very careful inspection.

Field splices of center beams require particularly careful review. WSDOT's modular expansion joint provisions recommend several mitigating measures to minimize fatigue susceptibility of center beam field splices. These measures include reducing support box spacing and optimizing field splice locations to minimize fatigue stress range. Keep in mind that the confined nature of the space in which a welder must work can make these welds very difficult to complete. The American Welding Society (AWS) Welding Code prequalifies certain end geometries because experience has shown that high quality welds can be achieved.

Non-prequalified center beam end geometries require the Contractor to submit a Procedure Qualification Record documenting that satisfactory weld quality has been achieved using samples before welding of the actual field piece. The Contractor will generally want to avoid the additional expense associated with these tests and will thus specify a prequalified end geometry.

WSDOT's modular expansion joint provisions require that adequate concrete consolidation be achieved underneath all support boxes. The reviewer should ascertain that the shop drawings detail a vertical minimum of 2 inches between the bottom of each support box and the top of the concrete blockout. Alternatively, when vertical clearance is minimal, grout pads can be cast underneath support boxes before casting the concrete within the blockout.

9.1.5.B.4 *Construction Considerations*

Temperature adjustment devices are temporarily welded to the modular expansion joints to permit the Contractor to adjust the modular expansion joint width so that it is consistent with the superstructure temperature at the time concrete is placed in the blockout. The temperature devices effectively immobilize the modular expansion joint. Once the concrete begins to set up, it is critical to remove these devices as soon as possible. If the modular expansion joint is prevented from opening and closing, it will be subject to very large, potentially damaging, forces.

Prior to placement of concrete into the blockout, temporary supports generally bridge across the expansion gap, suspending the modular expansion joint from the bridge deck surface. Following concrete placement, the modular expansion joint is supported by bearing of the support boxes on concrete that has consolidated underneath the blockout. The inspector should assure that adequate concrete consolidation is achieved underneath and around the support boxes.

Following delivery of the modular expansion joint to the jobsite and prior to its installation, the inspector should ascertain that center beam end geometries at field weld splice locations match those shown on the approved weld procedure.

9.2 Bearings

9.2.1 General Considerations

Bridge bearings facilitate the transfer of vehicular and other environmentally imposed loads from the superstructure down to the substructure, and ultimately, to the ground. In fulfilling this function, bearings must accommodate anticipated movements (thermal expansion/contraction) while also restraining undesired movements (seismic displacements). Because the movements allowed by an adjacent expansion joint must be compatible with the movement restriction imposed by a bearing, bearings and expansion joints must be designed interdependently and in conjunction with the anticipated behavior of the overall structure.

Numerous types of bearings are used for bridges. These include steel reinforced elastomeric bearings, fabric pad sliding bearings, steel pin bearings, rocker bearings, roller bearings, pot bearings, spherical bearings, disk bearings, and seismic isolation bearings. Each of these bearings possesses different characteristics in regard to vertical and horizontal load carrying capacity, vertical stiffness, horizontal stiffness, and rotational stiffness. A thorough understanding of these characteristics is essential for economical bearing selection and design. Spherical bearings, disk bearings, and pot bearings are sometimes collectively referred to as high load multi-rotational (HLMR) bearings.

Seismic isolation bearings mitigate the potential for seismic damage by utilizing two related phenomena: dynamic isolation and energy dissipation. Dynamic isolation allows a superstructure to essentially float, to some extent, while substructure elements below move with the ground during an earthquake. The ability of some bearing materials and elements to deform in certain predicable ways allows them to dissipate seismic energy that might otherwise damage critical structural elements.

Given their unique, in many instances proprietary, nature and the need to holistically incorporate their design with the overall seismic analysis and design of the structure, a separate.

9.2.2 Force Considerations

Bridge bearings must be explicitly designed to transfer all anticipated loads from the superstructure to the substructure. These forces may be directed vertically, longitudinally, or transversely with respect to the global orientation of the bridge. In accordance with LRFD provisions, most bearing design calculations are based upon service limit state stresses. Impact need not be applied to live load forces in the design of bearings.

Experience has empirically led to the following practical load capacity approximations for various bearing types:

Bearing Type	Approx. Load Capacity
Steel reinforced elastomeric (Method B)	Less than 800 kips
Fabric pad	Less than 600 kips
Steel pin	More than 600 kips
Spherical and disk	More than 800 kips
Seismic isolation	Less than 800 kips

9.2.3 Movement Considerations

Bridge bearings can be detailed to provide translational fixity, to permit free translation in any horizontal direction, or to permit guided translation. The movement restrictions thus imposed by a bearing must be compatible with the movements allowed by an adjacent expansion joint. Additionally, both bearings and expansion joints must be designed consistent with the anticipated load and deformation behavior of the overall structure. Design rotations shall be calculated as follows:

9.2.3.A Elastomeric and Fabric Pad Bearings

The maximum service limit state rotation for bearings that do not have the potential to achieve hard contact between metal components shall be taken as the sum of unfactored dead and live load rotations plus an allowance for fabrication and construction uncertainties of 0.005 radians.

9.2.3.B HLMR Bearings

Both service and strength limit state rotations are used in the design of HLMR bearings. These rotations must be shown on the plans to allow the manufacturer to properly design and detail a bearing.

The service limit state rotation shown on the plans shall include an allowance for uncertainties of +/-0.005 radians.

The strength limit state rotation is used to assure that contact between hard metal or concrete surfaces is prevented under the full range of expected loading. In accordance with the LRFD-BDS, the strength limit state rotation shown on the plans shall include allowances of:

1. For disc bearings, +/-0.005 radians for uncertainties
2. For other HLMR bearings, such as spherical, pot, and steel pin bearings, +/-0.005 radians for fabrication and installation tolerances and an additional +/-0.005 radians for uncertainties

9.2.4 Detailing Considerations

HLMR bearings shall be designed, detailed, fabricated, and installed to facilitate inspection, maintenance, and eventual replacement. Jacking points shall be identified in the contract drawings so that bearings can be reset, repaired, or replaced. In some instances, bearings may need to be reset in order to mitigate unintended displacements induced by construction sequences.

Prestressed concrete girder bridges having end Type A (semi–integral) need not be detailed to accommodate elastomeric bearing replacement at abutments. Prestressed concrete girder bridges having end Type B (L-type abutments) shall be designed and detailed to accommodate elastomeric bearing replacement at abutments. Specifically, girder stops and end diaphragms shall be detailed to accommodate the placement of hydraulic jacks. The standard end diaphragms for long-span girders may not have sufficient flexural and shear capacity to support jacking induced stresses. The designer shall check these and provide sufficient steel reinforcement to accommodate shear forces and bending moments induced by jacking. (Girder end Types A and B are depicted on Figures 5.6.2-3 and 5.6.2-4.) Incidentally, intermediate piers of prestressed concrete girder bridges having steel reinforced elastomeric bearings shall also be designed and detailed to facilitate bearing replacement.

9.2.5 Bearing Types

9.2.5.A Elastomeric Bearings

Elastomeric bearings are perhaps the simplest and most economical of all bridge bearings. They are broadly classified into three types: plain elastomeric pads, steel reinforced elastomeric pads, and cotton duck fabric reinforced elastomeric pads. Of these three types, the first two are used extensively for bridge construction. Incidentally, cotton duck fabric reinforced elastomeric pads are generally referred to as fabric pad bearings. This subsection will address steel reinforced elastomeric bearings. A subsequent section will address fabric pad bearings.

A steel reinforced elastomeric bearing consists of discrete steel shims vulcanized between adjacent discrete layers of elastomer. The vulcanization process occurs in an autoclave under conditions of high temperature and pressure. The constituent elastomer is either natural rubber or synthetic rubber (neoprene). Steel reinforced elastomeric bearings are commonly used with prestressed concrete girder bridges and may be used with other bridge types. Because of their relative simplicity and fabrication ease, steel reinforced elastomeric bearings offer significant economy relative to HLMR bearings.

Steel reinforced elastomeric bearings rely upon the inherent shear flexibility of the elastomer layers to accommodate bridge movements in any horizontal direction. This shear flexibility also enhances their rotational flexibility. The steel shims limit the tendency for the elastomer layers to bulge laterally under compressive load.

Steel reinforced elastomeric bearings can be designed by either the Method A or Method B procedure delineated in the LRFD provisions. Current WSDOT policy is to design all elastomeric bearings using the Method B provisions, which provides more relief in meeting rotational demands than Method A. The Method A design procedure is a carryover based upon more conservative interpretation of past theoretical analyses and empirical observations prior to research leading up to the publication of NCHRP Report 596 *Rotation Limits for Elastomeric Bearings*.

Both Method A and Method B design procedures require determination of the optimal geometric parameters to achieve an appropriate balance of compressive, shear, and rotational stiffnesses and capacities. Fatigue susceptibility is controlled by limiting live load compressive stress. Delamination (of steel shim-elastomer interface) susceptibility is controlled by limiting total compressive stress. Assuring adequate shim thickness precludes yield and rupture of the steel shims. Excessive shear deformation is controlled, and rotational flexibility is assured, by providing adequate total elastomer height. Generally, total elastomer thickness shall be no less than twice the maximum anticipated lateral deformation. Overall bearing stability is controlled by limiting total bearing height relative to its plan dimensions. The most important design parameter for reinforced elastomeric bearings is the shape factor. The shape factor is defined as the plan area of the bearing divided by the area of the perimeter free to bulge (perimeter multiplied by thickness of one layer of elastomer).

Axial, rotational, and shear loading generate shear strain in the constituent elastomeric layers of a typical bearing. Computationally, Method B imposes a limit on the sum of these shear strains. It distinguishes between static and cyclic components of shear strain by applying an amplification factor of 1.75 to cyclic components to reflect cumulative degradation caused by repetitive loading.

In essence, elastomeric bearing design reduces to checking several mathematical equations while varying bearing plan dimensions, number of elastomeric layers and their corresponding thicknesses, and steel shim thicknesses. Because these calculations can become rather tedious, MS Excel spreadsheets have been developed and are available for designs using both Method A and Method B procedures. See the Bearing and Expansion Joint Specialist for these design tools.

LRFD-BDS design may result in thicker steel reinforced elastomeric bearings than previous designs, particularly for shorter span bridges. This is a consequence of the increased rotational flexibility required to accommodate the 0.005 radian allowance for uncertainties and partially to inherent conservatism built into the rotational capacity equations.

Although constituent elastomer has historically been specified by durometer hardness, shear modulus is the most important physical property of the elastomer for purposes of bearing design. Research has concluded that shear modulus may vary significantly among compounds of the same hardness. Accordingly, shear modulus shall be specified on the plans as 165 psi at 73°F without reference to durometer hardness.

Elastomeric bearings shall conform to the requirements of AASHTO Specification M 251M/M *Plain and Laminated Elastomeric Bridge Bearings*. Shims shall be fabricated from ASTM A 1011 Grade 36 steel unless noted otherwise on the plans. Bearings shall be laminated in ½ inch thick elastomeric layers with a minimum total thickness of 1 inch. For overall bearing heights less than 5 inches, a minimum of ¼ inch of side clearance shall be provided over the steel shims. For overall heights greater than 5 inches, a minimum of ½ inch of side clearance shall be provided. Live load compressive deflection shall be limited to $\frac{1}{16}$ inch. AASHTO Specification M 251M/M requires elastomeric bearings to be subjected to a series of tests, including a compression test at 150 percent of the total service load. For this reason, compressive dead load and live load shall be specified on the plans.

With respect to width, elastomeric bearings shall be designed and detailed as follows:

1. For prestressed concrete wide flange girders (WF--G series and WF--DG series), the edge of the bearing pad shall be set between 1 inch minimum and 9 inches maximum inside of the edge of the girder bottom flange.
2. For prestressed concrete I-girders, bulb-tee girders, and deck bulb-tee girders, the edge of the bearing pad shall be set 1 in. inside of the edge of the girder bottom flange.
3. For all prestressed concrete tub girders, the edge of the bearing shall be set 1 in. inside of the edge of the bottom slab. Bearing pads for prestressed concrete tub girders shall be centered close to the centerline of each web.
4. For all prestressed concrete slabs, one bearing pad and corresponding grout pad is required for each end of the prestressed concrete slab. The centerline of the bearing and grout pad shall coincide with the centerline of the prestressed concrete slab. The need for steel shims shall be assessed during the bearing design.

As mentioned earlier, LRFD-BDS Article 14.4.2.1 requires that a 0.005 radian allowance for uncertainties be included in the design of steel reinforced elastomeric bearings. This allowance applies to both rotations θx and θy. The LRFD-BDS Article 14.4.2 Commentary states "An owner may reduce the fabrication and setting tolerance allowances if justified by a suitable quality control plan; therefore, these tolerance limits are stated as recommendations rather than absolute limits." Consult with the Bearings and Expansion Joint Specialist in instances in which the 0.005 radian tolerance precludes convergence to a reasonable design solution.

In order to facilitate compressive load testing, future bearing replacement, and vertical geometry coordination, the following table shall be included in the Plans:

Bearing Design Table Service I Limit State	
Dead load reaction	--------- kips
Live load reaction (w/o impact)	--------- kips
Unloaded height	--------- inches
Loaded height (*DL*)	--------- inches
Shear modulus at 73° F	--------- psi

In the construction of precast prestressed concrete girder and steel girder bridges, elastomeric bearings are generally not offset to account for temperature during erection of the girders as are most other bearing systems. Girders may be set atop elastomeric bearings at temperatures other than the mean of the temperature range. This is statistically reconciled by assuming a maximum thermal movement in either direction of:

$$\Delta_{temp} = 0.75 \cdot \alpha \cdot L \cdot (T_{MaxDesign} - T_{MinDesign})$$

where $T_{MaxDesign}$ is the maximum anticipated superstructure average temperature and $T_{MinDesign}$ is the minimum anticipated superstructure average temperature during the life of the bridge.

For precast prestressed concrete girder bridges, the maximum thermal movement, Δ_{temp}, shall be added to shrinkage and long-term creep movements to determine total bearing height required. The shrinkage movement for this bridge type shall be half that calculated for a cast-in-place concrete bridge, as noted in Section 9.1.2-A.

For cast-in-place concrete bridges, it is assumed that the temperature of concrete at placement is equal to the normal temperature, as defined by the *Standard Specifications*. Total shrinkage movement, calculated as noted in Section 9.1.2-A, is added to the maximum thermal movement, Δ_{temp}, to determine required total height of the elastomeric bearing.

9.2.5.B Fabric Pad Sliding Bearings

Fabric pad sliding bearings incorporate fabric pads with a polytetrafluoroethylene (PTFE)–stainless steel sliding interface to permit large translational movements. Unlike a steel reinforced elastomeric bearing having substantial shear flexibility, the fabric pad alone cannot accommodate translational movements. Fabric pads can accommodate very small amounts of rotational movement; less than can be accommodated by more flexible steel reinforced elastomeric bearings. Practical size considerations limit the use of fabric pad bearings to total service load reactions under about 600 kips. Fabric pad sliding bearings shall conform to the requirements of AASHTO Specification M 351 *Cotton Duck Fabric Bridge Bearings*.

PTFE, also referred to as Teflon, is available in several forms: unfilled sheet, dimpled lubricated, filled, and woven. Filled PTFE contains glass, carbon, or other chemically inert fibers that enhance its resistance to creep (cold flow) and wear. Interweaving high strength fibers through PTFE material creates woven PTFE. Dimpled PTFE contains dimples, which act as reservoirs for silicone grease lubricant.

Friction coefficients for PTFE – stainless steel surfaces vary significantly as a function of PTFE type, contact pressure, and ambient temperature. The LRFD-BDS provides friction coefficients as a function of these variables. Dimpled lubricated PTFE at high temperatures and high contact pressures typically yield the lowest friction coefficients. Filled PTFE at low temperatures and low contact pressures yield the highest friction coefficients.

In order to minimize frictional resistance, a Number 8 (Mirror) finish should be specified for all flat stainless steel surfaces in contact with PTFE. The low-friction characteristics of a PTFE – stainless steel interface are actually facilitated by fragmentary PTFE sliding against PTFE after the fragmentary PTFE particles are absorbed into the asperities of the stainless steel surface.

In fabric pad sliding bearings, the PTFE is generally recessed half its depth into a steel backing plate, which is generally bonded to the top of a fabric pad. The recess provides confinement that minimizes creep (cold flow). The stainless steel sheet is typically seal welded to a steel sole plate attached to the superstructure.

Silicone grease is not recommended for non-dimpled PTFE. Any grease will squeeze out under high pressure and attract potentially detrimental dust and other debris.

9.2.5.B.1 *Fabric Pad Design*

WSDOT's design criteria for fabric pad bearings are based upon manufacturers' recommendations, supported by years of satisfactory performance. These criteria differ from LRFD-BDS provisions in that they recognize significantly more rotational flexibility in the fabric pad. Our maximum allowable service load average bearing pressure for fabric pad bearing design is 1,200 psi. WSDOT's maximum allowable service load edge bearing pressure for fabric pad bearing design is 2,000 psi. A 1,200 psi compressive stress corresponds to 10 percent strain in the fabric pad while a 2,000 psi compressive stress corresponds to 14 percent compressive strain. Based upon this information, the following design relationship can be established:

$$\theta = \frac{2 \times (.14 - .10) \times T}{L}$$

$$\theta = \frac{.08 \times T}{L}$$

$$T = 12.5 \times \theta \times L$$

Where

θ = rotation due to loading plus construction tolerances
L = pad length (parallel to longitudinal axis of beam)
T = fabric pad thickness required

9.2.5.B.2 *PTFE*

Stainless Steel Sliding Surface Design – PTFE having a maximum dimension less than or equal to 24 inches shall be 3⁄16-inch thick and shall be recessed 3⁄32 inch into a ½-inch thick steel plate that is bonded to the top of the fabric pad. PTFE having a maximum dimension greater than 24 inches. shall be ¼-inch thick and shall be recessed ⅛ inch into a ½-inch thick steel plate that is bonded to the top of the fabric pad. With the PTFE confined in this recess, the LRFD-BDS code permits an average contact stress of 4,500 psi for all loads calculated at the service limit state and an average contact stress of 3,000 psi for permanent loads calculated at the service limit state. The LRFD-BDS code permits slightly higher edge contact stresses.

For example, suppose:

DL = 150 kips

LL = 90 kips

A_{PTFE} > (150 kips + 90 kips)/4.5 ksi = 53.3 in^2

A_{PTFE} > 150 kips/3 ksi = 50.0 in^2

Selected area of PTFE must exceed 53.3 in^2

Stainless steel sheet shall be finished to a No. 8 (Mirror) finish and seal welded to the sole plate.

9.2.5.C Pin Bearings

Steel pin bearings are generally used to support heavy reactions with moderate to high levels of rotation about a single predetermined axis. This situation generally occurs with long straight steel plate girder superstructures.

9.2.5.D Rocker and Roller Type Bearings

Steel rocker bearings have been used extensively in the past to allow both rotation and longitudinal movement while supporting large loads. Because of their seismic vulnerability and the more extensive use of steel reinforced elastomeric bearings, rocker bearings are no longer specified for new bridges.

Steel roller bearings have also been used extensively in the past. Roller bearings permit both rotation and longitudinal movement. Pintles are generally used to connect the roller bearing to the superstructure above and to the bearing plate below. Nested roller bearings have also been used in the past. Having been supplanted by more economical steel reinforced elastomeric bearings, roller bearings are infrequently used for new bridges today.

9.2.5.E Spherical Bearings

A spherical bearing relies upon the low-friction characteristics of a curved PTFE–stainless steel interface to provide a high level of rotational flexibility in multiple directions. An additional flat PTFE–stainless steel surface can be incorporated into the bearing to additionally provide either guided or non-guided translational movement capability.

Woven PTFE is generally used on the curved surfaces of spherical bearings. Woven PTFE exhibits enhanced creep (cold flow) resistance and durability characteristics relative to unwoven PTFE. When spherical bearings are detailed to accommodate translational

movement, woven PTFE is generally specified on the flat sliding surface also. The LRFD code permits an average contact stress of 4,500 psi for all loads calculated at the service limit state and an average contact stress of 3,000 psi for permanent loads calculated at the service limit state. The LRFD-BDS code permits slightly higher edge contact stresses.

Both stainless steel sheet and solid stainless steel have been used for the convex sliding surface of spherical bearings. According to one manufacturer, curved sheet is generally acceptable for contact surface radii greater than 14 in. to 18 in. For smaller radii, a solid stainless steel convex plate or a stainless steel inlay is used. The inlay is welded to the solid conventional steel. If the total height of the convex plate exceeds about 5 in., a stainless steel inlay will likely be more economical.

Most spherical bearings are fabricated with the concave surface oriented downward to minimize dirt infiltration between PTFE and the stainless steel surface. Structural analysis of the overall structure must recognize the center of rotation of the bearing not being coincident with the neutral axis of the girder above.

The contract drawings must show the diameter and height of the spherical bearing in addition to all dead, live, and seismic loadings. Total height depends upon the radius of the curved surface, diameter of the bearing, and total rotational capacity required. Consult the Bearing and Expansion Joint Specialist for design calculation examples. Additionally, sole plate connections, base plate, anchor bolts, and any appurtenances for horizontal force transfer must be detailed on the plans. The spherical bearing manufacturer is required to submit shop drawings and detailed structural design calculations of spherical bearing components for review by the Engineer.

9.2.5.F Disk Bearings

A disk bearing is composed of an annular shaped polyether urethane disk designed to provide moderate levels of rotational flexibility. A steel shear-resisting pin in the center provides resistance against lateral force. A flat PTFE–stainless steel surface can be incorporated into the bearing to also provide translational movement capability, either guided or non-guided.

9.2.6 Miscellaneous Details

9.2.6.A Temporary Support before Grouting Masonry Plate

The masonry plate of a HLMR bearing is generally supported on a grout pad that is installed after the bearing and superstructure girders above have been erected. This procedure allows the Contractor to level and slightly adjust the horizontal location of the bearing before immobilizing it by placing the grout pad. Several methods have been developed to temporarily support the masonry plate until the grout is placed. The two most commonly used methods will be discussed here.

9.2.6.A.1 *Shim Packs*

Multiple stacks of steel shim plates can be placed atop the concrete surface to temporarily support the weight of the girders on their bearings before grouting. Engineering judgment must be used in selecting the number and plan size of the shims taking grout flowability and shim height adjustability into consideration.

9.2.6.A.2 *Two-step Grouting with Cast Sleeves*

A two-step grouting procedure with cast-in-place voided cores can be used for smaller HLMRs not generally subjected to uplift. Steel studs are welded to the underside of the masonry plate to coincide with the voided cores. With temporary shims installed between the top of the concrete surface and the underside of the masonry plate, the voided cores are fully grouted. Once the first stage grout has attained strength, the shims are removed, the masonry plate is dammed, and grout is placed between the top of the concrete surface and the underside of the masonry plate.

9.2.6.B Anchor Bolts

Anchor bolts shall be designed to resist all horizontal shear forces and direct tension force due to uplift.

Anchor bolts shall be ASTM A 449 where strengths equal to ASTM A 325 are required and ASTM A 354, Grade BD, where strengths equal to ASTM A 490 are required. ASTM F 1554 bolts with supplemental Charpy test requirements shall be specified in applications in which the bolts are subject to seismic loading.

9.2.7 Contract Drawing Representation

High load multi-rotational bearings are generally depicted schematically in the contract drawings. Each bearing manufacturer has unique fabricating methods and procedures that allow it to fabricate a bearing most economically. Depicting the bearings schematically with loads and geometric requirements provides each manufacturer the flexibility to innovatively achieve optimal economy.

9.2.8 Shop Drawing Review

The manufacturer designs and develops shop drawings for high load multi-rotational bearings. The Engineer is responsible for checking and approving the calculations and shop drawings. The calculations shall verify the structural adequacy of all components of the bearing. Each bearing shall be detailed to permit the inspection and replacement of components.

9.2.9 Bearing Replacement Considerations

In some situations, existing bearings, or elements thereof, must be replaced consequent to excessive wear or seismic rehabilitation. Bearing replacement operations generally require lifting of superstructure elements using hydraulic jacks. The designer is responsible for calculating anticipated lifting loads and stipulating these loads on the contract drawings. Limitations on lift height shall also be specified. Consideration shall be given to lift height as it relates to adjacent expansion joints elements and adjoining sections of railing. Stresses induced by differential lift height between multiple hydraulic jacks are generally addressed by stipulating restrictions in the plans or special provisions.

Experience has shown that actual lifting loads nearly always exceed calculated lifting loads. Many factors may contribute to this phenomenon, including friction in the hydraulic jack system and underestimation of superstructure dead loads. Unless the Bearing and Expansion Joint Specialist or the State Bridge Design Engineer approves a variance, contract documents shall require that all hydraulic jacks be sized for 200 percent of the calculated lifting load. In all cases, the designer shall verify from manufacturer's literature that appropriate hydraulic jacks are available to operate within the space limitations imposed by a particular design situation.

9.3 Seismic Isolation Bearings

9.3.1 General Considerations

Numerous seismic isolation bearings exist, each relying upon varying combinations of dynamic isolation and energy dissipation. These devices include lead core elastomeric bearings, high damping rubber, friction pendulum, hydraulic dampers, and various hybrid variations.

Effective seismic isolation bearing design requires a thorough understanding of the dynamic characteristics of the overall structure as well as the candidate isolation devices. Isolation devices are differentiated by maximum compressive load capacity, lateral stiffness, lateral displacement range, maximum lateral load capacity, energy dissipation per cycle, functionality in extreme environments, resistance to aging, fatigue and wear properties, and effects of size.

The Highway Innovative Technology Evaluation Center (HITEC) has developed guidelines for testing seismic isolation and energy dissipating devices. With the goal of disseminating objective information to design professionals, HITEC has tested and published technical reports on numerous proprietary devices. These tests include performance benchmarks, compressive load dependent characterization, frequency dependent characterization, fatigue and wear, environmental aging, dynamic performance at extreme temperatures, durability, and ultimate performance.

9.3.2 Suitability and Selection Considerations

The decision to use seismic isolation bearings should be made during the early stages of project development based upon complexity of the geotechnical issues and bridge structural design. A cost-benefit analysis comparing Type 1 (ductile substructure) design vs. Type 3 (seismic isolation) design shall be performed and submitted for approval to the State Bridge Design Engineer. The designer shall perform two separate designs, one with and one without seismic isolation bearings. The cost-benefit analysis shall, as minimum, address the following:

- Longer initial design time and increased analysis complexity
- Impact of preliminary and final design time on the project delivery schedule
- Time required for feasibility assessment of seismic isolation and consultation with prospective isolation bearing suppliers
- Life cycle cost associated with additional specialized bearing inspections
- Life cycle cost associated with potential bearing and expansion joint replacements
- Long-term performance and maintenance issues
- Necessity for larger movement range expansion joints to accommodate isolation effects

Seismic isolation bearings shall not be used between top of column and bottom of crossbeam in either single or multiple column bents.

Following approval, by the State Bridge Design Engineer, to use seismic isolation bearings, the designer shall send a set of preliminary plans and mitigation requirements to at least three seismic isolation bearing suppliers for evaluation to determine if they can meet the design and specification requirements. Inspection and maintenance requirements shall be solicited from the suppliers to ascertain that their bearings will function satisfactorily throughout the design life of the bridge, including after any seismic events. Comments

from suppliers shall be considered and appropriately assimilated before commencing final structural design. Sole source procurement may be considered and pursued upon approval by the State Bridge Design Office and the Project Engineer.

Seismic isolation bearings may not provide significant benefit for concrete bridges less than 700 feet long, steel bridges less than 800 feet long, bridges having skew in excess of 30 degrees, or bridges with geometrical complexities, variable superstructure width, or drop-in spans. As such, seismic isolation bearings are not recommended for bridges having these characteristics.

The suitability of seismic isolation bearings for a specific project should be carefully evaluated prior to approval. Seismic isolation bearings may not be an effective solution for some combinations of bridge types and site conditions. For example, increasing the fundamental period of a structure founded on soft soils may not reduce the force demand. Design shall include near fault effects and soil-structure interaction associated with soft soil sites.

Expansion joints must accommodate seismic movements for seismic isolation bearings to function properly. The effect of this increased movement upon expansion joint demands shall be carefully considered. Modular expansion joints are generally designed to accommodate longitudinal service movement only. Design of modular expansion joints to accommodate longitudinal service movement is generally based upon limiting the movement capacity per elastomeric seal to 3 inches maximum in order to limit the fraction of wheel load imposed upon any one center beam and to assure that elastomeric seals will not detach under service load conditions. Because the fatigue limit state almost always controls center beam and support bar design, a larger movement capacity per cell is acceptable to accommodate seismic movement provided that 1) support bars and boxes are detailed to accommodate the increased movement and 2) elastomeric seal detachment is acceptable. Standard modular expansion joints are not designed to accommodate transverse bridge movements. Seismic modular expansion joints must be used if transverse movement must also be accommodated.

9.3.3 General Design Criteria

Seismic isolation bearings shall be designed in accordance with this manual and the requirements of the LRFD-BDS, LRFD-SGS, and AASHTO Guide Specifications for Seismic Isolation Design (ISOLATION). The response modification factors (R-factors) contained in Article 6 of the LRFD-SGS shall not be used if the provisions of the LRFD-SGS are being followed for the design of the bridge. Expansion joints shall accommodate seismic movements for seismic isolation bearings to function properly. Adequate clearance for the seismic displacement shall be provided between the girders and abutment to limit the damage at top of backwall. After a major earthquake, damage is easily inspected and repaired.

Performance requirements:

An isolated bridge shall meet the same seismic hazard, seismic design requirements, and performance criteria used to design the conventional bridge.

Combinations of isolation bearings and conventional bridge column fixity to the superstructure are not allowed.

The following target values to optimize the performance of isolated structures shall be considered:

Table 9.3-1

Parameter	Transverse	Longitudinal
Seismic Total Design Displacement Demand (TDD)	6 to 24 inches	6 to 24 inches
Isolated Structure Effective Period (T)	2 to 3 seconds	2 to 3 seconds

An isolated bridge system shall meet the following requirements:

1. Superstructure unseating shall be prevented if the displacement demand exceeds 125% of the bearing total design displacement demand (TDD). For existing bridges, prevention strategies include large platform seats and/or catcher blocks. Any drop of the superstructure onto the catcher or seat shall be made as small as possible to prevent damage from impact.
2. The column shear capacity shall meet or exceed the lateral isolator force at 125% of the bearing total design displacement, F1.25TDD.
3. Isolated bridge systems shall be designed for a minimum local displacement ductility capacity for columns, shafts, and piles of 3.0. Local displacement ductility capacity is based on an equivalent member that approximates a fixed base cantilever element. Fixed-free and fixed-fixed columns are idealized as one and two cantilever elements.
4. The displacement capacity of the isolation devices shall be determined from the manufacturer's recommendation, but not less than 125% of the bearing total design displacement.

9.3.4 Seismic Isolation Bearing Submittal Requirements

The selected manufacturer shall develop and submit seismic isolation bearing design calculations and shop drawings to WSDOT for review and approval. Design calculations and shop drawings shall be based upon the loads, movement demands, schematic details, and engineering requirements contained in the contract drawings and Special Provisions. All suppliers are required to provide calculations and shop drawings regardless of the contracting method or whether sole source procurement has been approved. The use of seismic isolation bearings for WSDOT projects shall fully comply with the requirements herein. A manufacturer's design report will not be accepted in lieu of complying with the following five requirements:

9.3.4.A Shop Drawings

Fully engineered shop drawings shall be submitted to document compliance with contractual material and design requirements and to provide a baseline reference for future engineering evaluation during the design life of the bridge. Shop drawings shall clearly show all individual components, constituent materials, connections, dimensions, surface finishes, coatings, and tolerances necessary to fabricate components and fully assemble the bearing. Shop drawings do not need to identify proprietary aspects of individual components or fabrication procedures (for example, low-friction sliding material and its attachment to steel components).

9.3.4.B Bearing Materials and Fabrication

Stainless steel shall be incorporated as specified on all sliding or articulating surfaces. Bearings with seals shall be watertight systems. Mill certification documentation shall be submitted for all constituent components. Quality Assurance (QA) inspection affords WSDOT the opportunity to verify and document materials, witness modular fabrication and coating processes, and verify "Buy America" material requirements. Modular methods of bearing manufacturing shall accommodate the full WSDOT QA process.

9.3.4.C Paint

All non-stainless steel surfaces shall have a full four-coat (primer, intermediate, intermediate stripe, and top coat) paint system applied in accordance with *Standard Specifications* and Special Provisions. The full four-coat paint system shall be applied to all non-stainless steel and non-plastic internal and external surfaces.

9.3.4.D Bearing Specifications

WSDOT bearing specifications constitute part of the contract. Disregard of or noncompliance with the bearing specification requirements in the Special Provisions constitutes a violation of the contract.

9.3.4.E Fabrication

Third-party inspection shall be provided by the manufacturer. The manufacturer shall provide access for Contracting Agency QA inspection as stipulated in *Standard Specifications* Section 1.5.6 and the bearing Special Provisions. QA inspection shall include all aspects of the bearing fabrication and assembly. The manufacturer shall adhere to all hold points, as specified in the *Standard Specifications* and Special Provisions.

9.3.5 Seismic Isolation Bearing Review Process

The manufacturer shall submit design calculations and shop drawings to the Engineer for review and approval prior to commencement of fabrication operations. The Engineer shall review the contract drawings and Special Provisions to assure familiarity with the design, fabrication, and inspection requirements.

9.3.5.A Design Calculations

The Engineer shall review the design calculations to assure that

- All design calculations are stamped and signed.
- The design incorporates all load cases specified in the contract documents.
- The design incorporates all displacements and rotations specified in the contract documents.
- All allowable stresses used in the design are consistent with LRFD provisions and that these allowable stresses are not exceeded.

9.3.5.B Shop Drawings

The Engineer shall review the shop drawings to assure that

- All shop drawings are stamped and signed.
- Shop drawings include plan and elevation view of the assembled bearings and details of each constituent component. Such details shall include all dimensions and tolerances necessary to complete manufacturing.

- All component materials shall be specified on the shop drawings and shall conform with the requirements of the Special Provisions.
- All component flatness tolerances and surface roughness requirements are depicted on the shop drawings and are consistent with the Special Provisions.
- All corrosion protection system details (galvanization, paint) for steel components, bolts, and washers are designated on the shop drawings and are in conformance with the Special Provisions and the *Standard Specifications*.
- Bearings have been designed and detailed to accommodate full inspection and removal and replacement of all components subject to wear or other anticipated damage.
- Adequate clearances, including applicable tolerances, have been provided between components to accommodate assembly and service movements.
- Positive connections are provided between all components to assure individual components will not separate under unanticipated seismic movements. All connections have been designed to accommodate loads shown on the contract drawings.
- Shop drawings stipulate handling and storage requirements for both shipment and jobsite storage.
- Masonry and sole plate connections are integrated into the bearing design.
- Specific directives are provided for setting the bearings as a function of the bridge temperature. These directives shall include a rational method for assessing the temperature of the superstructure.
- Grouting procedure and temporary shim requirements underneath the masonry plate are clearly depicted on the shop drawings.
- Shop drawings stipulate that all bearings shall be marked for location and orientation as required by the Special Provisions

9.3.6 Seismic Isolation Bearing Inspection

9.3.6.A Fabrication Inspection

The manufacturer shall provide access for third-party QA inspectors to observe the fabrication and testing of the seismic isolation bearings in accordance with WSDOT *Standard Specifications* Section 1-05.6.

9.3.6.B Field Inspection

Following arrival at the jobsite, prior, during, and after installation, WSDOT inspectors shall perform the following inspections

- Inspect all external surfaces for paint distress or presence of corrosion.
- Inspect perimeter seals for damage that could compromise watertightness.
- Inspect grout, concrete, and other structural elements connected to the bearings for damage.
- Verify that adequate lateral and vertical clearances exist around the bearing to assure that lateral and vertical displacement capacities can be achieved. Confirm that no structural components obstruct the bearing from achieving these movement capacities. Evaluate any non-structural obstructions that could impede attainment of movement capacities.
- Inspect the perimeter of the isolated structure to ascertain that it is free to move as needed to attain the horizontal and vertical displacement capacities of the bearings.

9.4 Bridge Standard Drawings

9.4.1 Expansion Joints

9.1-A1-1 Compression Seal (PDF 131KB)(DWG 132KB)

9.1-A2-1 Expansion Joint Details Strip Seal (PDF 169KB) (DWG 199KB)

9.1-A3-1 Silicone Seal (PDF 107KB)(DWG 104KB)

9.99 References

1. See Section 1.99.