18.1 Introduction

Expansion joint systems are integrated, yet often overlooked, components designed to accommodate repeated cycles of movement. Properly functioning bridge expansion joint systems accommodate these movements without imposing significant secondary stresses on the superstructure. Sealed expansion joint systems also provide barriers preventing runoff water and deicing chemicals from passing through the joint onto bearing and substructure elements below the bridge deck. Water and deicing chemicals have a detrimental impact on overall structural performance by accelerating degradation of bridge deck, bearing, and substructure elements. In extreme cases, this degradation has resulted in premature, catastrophic structural failure. In fulfilling their functions, expansion joints must provide a reasonably smooth ride for motorists.

Perhaps because expansion joints are generally designed and installed last, they are often relegated to peripheral status by designers, builders, and inspectors. As a result of their geometric configuration and the presence of multiple axle vehicles, expansion joint elements are generally subjected to a significantly larger number of loadings than other structural members. Impact, a consequence of bridge discontinuity inherent at a joint, exacerbates loading. Unfortunately, specific expansion joint systems are often selected based upon their initial cost with minimal consideration for long-term performance, durability, and maintainability. Consequently, a plethora of bridge maintenance problems plagues them.

In striving to improve existing and develop new expansion joint systems, manufacturers present engineers with a multitudinous array of options. In selecting a particular system, the designer must carefully assess specific requirements. The magnitude and direction of movement, type of structure, traffic...
volumes, climatic conditions, skew angles, initial and life cycle costs, and past performance of various systems must all be considered. For classification in the ensuing discussion, expansion joint systems will be grouped into three broad categories depending upon the total movement range accommodated. Small movement range joints encompass all systems capable of accommodating total motion ranges of up to about 1.75 in. (45 mm). Medium movement range joints include systems accommodating total motion ranges between about 1.75 in. (45 mm) and about 5 in. (127 mm). Large movement range joints accommodate total motion ranges in excess of about 5 in. (127 mm). These delineated ranges are somewhat arbitrary in that some systems can accommodate movement ranges overlapping these broad categories.

### 18.2 General Design Criteria

Expansion joints must accommodate movements produced by concrete shrinkage and creep, post-tensioning shortening, thermal variations, dead and live loads, wind and seismic loads, and structure settlements. Concrete shrinkage, post-tensioning shortening, and uniform thermal variation are generally taken into account explicitly in design calculations. Thermal gradients, most commonly produced by unequal solar heating of the superstructure, cause curvature effects. These effects are much more difficult to quantify and are commonly neglected for all but complex or very deep superstructures. Because of uncertainties in predicting, and the increased costs associated with accommodating large displacements, seismic movements have generally not been explicitly included in calculations.

Expansion joints should be designed to accommodate all shrinkage occurring after their installation. For unrestrained concrete, ultimate shrinkage strain after installation, $\beta$, may be estimated as 0.0002 (WSDOT 2011). More detailed estimations can be used, which include the effect of ambient relative humidity and volume-to-surface ratios (AASHTO 2012). Shrinkage shortening of the bridge deck, $\Delta_{\text{shrink}}$, is calculated as

$$\Delta_{\text{shrink}} = \beta(\mu)\left(L_{\text{trib}}\right)$$

(18.1)

where

- $L_{\text{trib}} =$ tributary length of structure subject to shrinkage; ft.
- $\beta =$ ultimate shrinkage strain after expansion joint installation; estimated as 0.0002 in lieu of more refined calculations
- $\mu =$ factor accounting for restraining effect imposed by structural elements installed before slab is cast (WSDOT 2011)
  - 0.0 for steel girders, 0.5 for precast prestressed concrete girders, 0.8 for concrete box girders and T-beams, 1.0 for flat slabs

Uniform thermal displacements are calculated using the maximum and minimum anticipated bridge deck temperatures. These extreme values are functions of the structure’s geographic location and the bridge type. Uniform thermal movement is calculated as

$$\Delta_{\text{temp}} = \alpha(\mu)\left(L_{\text{trib}}\right)\delta T$$

(18.2)

where

- $\alpha =$ Coefficient of thermal expansion; 0.000006 in./in./°F for concrete and 0.0000065 in./in./°F for steel
- $L_{\text{trib}} =$ tributary length of structure subject to thermal variation; ft.
- $\delta T =$ temperature variation; °F

Because the AASHTO LRFD specifications stipulate that the expansion joint design be based upon strength limit state load combinations, the load factor on uniform thermal displacement, $T_U$, specified in LRFD Table 3.4.1-1, is applicable (AASHTO 2012). As noted in LRFD Article 3.4.1, the larger of the two load factors, 1.2, is used for deformations. It is presumed here that the load factor of 1.2 was correlated with the
Expansion Joints

thermal ranges specified in LRFD Article 3.12.1 for the purpose of calculating uniform thermal displacements. Some transportation agencies have established conservatively wider thermal ranges for calculating bridge movements. In such situations, a reduced value of the load factor applied to TU may be justified.

Any other predictable movements following expansion joint installation, such as concrete post-tensioning shortening and creep, should also be included in the design calculations.

18.3 Jointless Bridges

Bridge designers have employed superstructure continuously in an effort to avoid some of the maintenance problems associated with expansion joints (Burke 1989). This evolution from simple span construction was facilitated by the development of the moment distribution procedure (Cross 1932). In recent years, some transportation agencies have extended this strategy by developing jointless bridge designs. Jointless bridges are characterized by continuous spans built integrally with their abutments. In many instances, approach slabs are tied to the superstructure slab or to the abutments. The resulting designs are termed integral or semi-integral depending upon the degree of continuity developed among superstructure, substructure, and approach slab elements. Design methods and details for jointless bridges vary considerably (Burke 1989; Steiger 1991). Many transportation agencies have empirically established maximum lengths for jointless bridges (Steiger 1991).

Jointless bridges should not be considered a panacea for addressing expansion joint maintenance issues. As superstructure movements are restrained in jointless bridges, secondary stresses are induced in superstructure and substructure elements. Stresses may also be induced in approach slabs. If they are inadequately addressed during design, these stresses can damage structural elements and adjacent pavement. Damaged structural elements, slabs, and pavements are accompanied by increased probability of moisture infiltration, further exacerbating deterioration. Long-term performance and durability will determine how extensively the jointless bridge concept is applied to future construction.

18.4 Small Movement Range Joints

Many different systems exist for accommodating movement ranges under about 1.75 in. (45 mm). These include, but are not limited to, steel sliding plates, elastomeric compression seals, preformed closed cell foam, epoxy-bonded cellular neoprene seals, asphaltic plug joints, bolt-down elastomeric panels, and poured sealants. In this section, several of these systems will be discussed with an emphasis on design procedures and past performance.

18.4.1 Sliding Steel Plate Joints

Sliding steel plate joints, depicted in Figure 18.1, have been used extensively in the past for expansion joints in both concrete and timber bridge decks. Two overlapping steel plates are attached to the bridge
deck, one on each side of the expansion joint opening. They are generally installed so that the top surfaces of the plates are flush with the top of the bridge deck. The plates are generally bolted to timber deck panels or embedded with steel anchorages into a concrete deck. Steel plate widths are sized to accommodate anticipated total movements. Plate thicknesses are determined by structural requirements.

Standard sliding steel plate joints do not generally provide an effective seal against intrusion of water and deicing chemicals into the joint and onto substructure elements. As a result of plate corrosion and debris collection, the sliding steel plates often bind up, impeding free movement of the superstructure. Repeated impact and weathering tend to loosen or break anchorages to the bridge deck. With the exception of sidewalk applications, sliding plate systems are rarely specified for new bridge construction today. Nevertheless, sliding plate systems still exist on many older bridges. These systems can be replaced with newer systems providing increased resistance against water and debris infiltration. In situations where the integrity of the deck anchorage has not been compromised, sliding plates can be retrofitted with poured sealants or elastomeric strip seals.

Figure 18.2 shows two variations of sliding steel plate joint applications. In the foreground is a typical sliding steel plate joint. In the background is a sliding steel plate joint that has been modified to accommodate an asphalt overlay by welding steel riser bars to the tops of the horizontal steel plates. In this photograph, the original bridge (background) was widened (foreground). Prior to the widening, the original bridge received an asphalt overlay.

18.4.2 Elastomeric Compression Seal Joints

Elastomeric compression seals, depicted in Figure 18.3, are continuous preformed elastomeric shapes, 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. Hence, design philosophy requires that they be sized and installed to always be in a state of compression. Compression seals may be installed against smooth concrete faces or against steel armoring. When the compression seal is installed directly against concrete, polymer concrete nosing material is often used to provide added impact resistance. Combination lubricant/adhesive is typically used to install the seal in its compressed state. A typical compression seal expansion joint is shown in Figure 18.4.

Because elastomeric compression seals are held in place by friction, their performance is extremely dependent upon the close correlation of constructed joint width and design joint width. If the joint opening is constructed too wide, the mobilized friction force will be insufficient to prevent the compression
seal from slipping out of the joint at wider expansion gap widths. Relaxation of the elastomer and debris accumulation atop the seal contribute to seal slippage. To minimize slippage and maximize compression seal performance, the expansion gap may be formed narrower than the design width, then sawcut immediately prior to compression seal installation. The sawcut width is calculated based upon ambient bridge deck temperature and the degree of slab shrinkage that has already occurred. As an alternative to sawcutting, block outs can be formed on each side of the joint during bridge deck casting. Prior to compression seal installation, concrete is cast into the block outs, often with steel armoring, to form an expanded gap width compatible with ambient temperature.

In design calculations, the maximum and minimum compressed widths of the seal are generally set at 85% and 40% of the uncompressed seal width (WSDOT 2011). These widths are measured perpendicular to the axis of the joint. Thus, ignoring deck shrinkage effects, it may be assumed that the width of the seal in the middle of the historical temperature range is about 62% 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% (WSDOT 2011). Additionally, the expansion gap width should be set so that the compression seal can be installed 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 bridge deck temperature.

Design relationships can be stated as follows:

\[ \Delta_{\text{temp-normal}} = \Delta_{\text{temp}} \cos \theta \quad \text{[thermal movement normal to joint]} \]  \hspace{1cm} (18.3)

\[ \Delta_{\text{temp-parallel}} = \Delta_{\text{temp}} \sin \theta \quad \text{[thermal movement parallel to joint]} \]  \hspace{1cm} (18.4)

\[ \Delta_{\text{shrink-normal}} = \Delta_{\text{shrink}} \cos \theta \quad \text{[shrinkage movement normal to joint]} \]  \hspace{1cm} (18.5)

\[ \Delta_{\text{shrink-parallel}} = \Delta_{\text{shrink}} \sin \theta \quad \text{[shrinkage movement parallel to joint]} \]  \hspace{1cm} (18.6)

\[ W_{\text{min}} = W_{\text{midrange}} \left[ \frac{T_{\text{max}} - T_{\text{install}}}{T_{\text{max}} - T_{\text{min}}} \right] \Delta_{\text{temp-normal}} > 0.4W \]  \hspace{1cm} (18.7)

\[ W_{\text{max}} = W_{\text{midrange}} + \left[ \frac{T_{\text{install}} - T_{\text{min}}}{T_{\text{max}} - T_{\text{min}}} \right] \Delta_{\text{temp-normal}} + \Delta_{\text{shrink-normal}} < 0.85W \]  \hspace{1cm} (18.8)

where \( \theta \) = skew angle of expansion joint, measured with respect to a line perpendicular to the bridge longitudinal axis; degrees, \( W \) = uncompressed width of compression seal; in., \( W_{\text{midrange}} \) = expansion gap at the midrange of temperature extremes; in., \( T_{\text{install}} \) = bridge deck temperature at time of installation; °F, \( W_{\text{min}}, W_{\text{max}} \) = minimum and maximum expansion gap widths; in., \( T_{\text{min}}, T_{\text{max}} \) = minimum and maximum bridge deck temperatures; °F.

Multiplying (18.7) by \(-1.0\), adding to (18.8), and rearranging yields:

\[ W > \frac{\Delta_{\text{temp-normal}} + \Delta_{\text{shrink-normal}}}{0.45} \]  \hspace{1cm} (18.9)

Similarly,

\[ W > \frac{\Delta_{\text{temp-parallel}} + \Delta_{\text{shrink-parallel}}}{0.22} \]  \hspace{1cm} (18.10)
Expansion Joints

Now, assuming \( W_{\text{midrange}} = 0.62W \),

\[
W_{\text{max}} = 0.62W + \left[ \frac{T_{\text{midrange}} - T_{\text{min}}}{T_{\text{max}} - T_{\text{min}}} \right] \Delta_{\text{temp-normal}} + \Delta_{\text{shrink-normal}} < 0.85W \tag{18.11}
\]

which, upon rearranging, yields:

\[
W > \frac{0.5 \Delta_{\text{temp-normal}} + \Delta_{\text{shrink-normal}}}{0.23} \tag{18.12}
\]

Equations 18.9, 18.10, and 18.12 are used to calculate the required compression seal size. Next, expansion gap widths at various construction temperatures can be evaluated.

### 18.4.3 Bonded Preformed Seal Joints

A variety of field glued preformed proprietary joint seals are marketed. These systems are designed to accommodate expansion and contraction by resisting both compression and tension. The performance of these systems is highly dependent upon the durability of the concrete headers on each side of the joint seal. Cracking or spalling of the headers adversely affect a glued seal’s performance and lead to premature failure. Advanced elastomeric concretes exhibit significantly improved performance under impact loading. They perform well as expansion joint headers, greatly improving the performance of both compression seals and glued preformed proprietary joint seals.

Closed-cell foam is one type of field glued joint seal. Evazote, an impermeable, resilient, preformed, ultraviolet resistant, flexible foam material is one proprietary example. It is a cross-linked, ethylene vinyl acetate, low density polyethylene copolymer, nitrogen blown resilient, nonextrudable foam material. Closed-cell foam is bonded in a compressed state to adjacent concrete surfaces using a two-component epoxy adhesive. Grooves are formed in the bonded faces of the material to enhance bonding. The material is typically oversized for the joint opening and cut to the length required.

Another type of glued preformed seal joint system is manufactured using flexible cellular neoprene expanded rubber produced by a relatively dense skin layer at its exterior surface to enhance durability. The sides of these seals are typically serrated to enhance bonding to substrate concrete using a two-component epoxy adhesive.

A variation of the glued preformed seal joint system consists of a voided neoprene shape similar in appearance to a compression seal. The sides of these seals are generally serrated to enhance bonding to concrete with a two-component epoxy adhesive. Complete adhesion of the epoxy to the seal and substrate surfaces is achieved by air inflation of the seal during the installation process. Once bonded, the seal can resist tension and compression.

Figure 18.5 shows three preformed proprietary seals that are used for field glued expansion joint applications. From left to right are closed-cell foam material, a preformed cellular neoprene seal, and an inflatable voided neoprene seal.

### 18.4.4 Asphaltic Plug Joints

Asphaltic plug joints comprise liquid polymer binder and graded aggregates compacted in preformed block outs as depicted in Figure 18.6. The compacted composite material is referred to as polymer modified asphalt (PMA). These joints have been used to accommodate movement ranges up to 2 in. (51 mm). This expansion joint system was originally developed in Europe and can be adapted for use with concrete or asphalt bridge deck surfaces. The PMA is installed in multiple lifts within a block out to center over the expansion joint opening with the top of the PMA flush with the roadway surface. A steel plate retains the PMA at the bottom of the block out during installation. The polymer binder material is
generally installed in heated form. Aggregate gradation, binder properties, and construction quality are critical to asphaltic plug joint performance.

The asphaltic plug joint is designed to provide a smooth, seamless roadway surface. It is relatively quick and easy to install, fairly inexpensive, and relatively easy to repair. It is not as susceptible to snow plow damage as other expansion joint systems, and can be cold milled and/or built up for roadway resurfacing. Given these factors, it is a particularly attractive alternative for rural applications with low traffic demands.

As with other expansion joint systems, asphaltic plug joints have their own set of disadvantages, which must be considered in the selection of appropriate expansion joint systems. The performance of asphaltic plug joints in the United States has been somewhat erratic (Bramel et al. 1996). The material properties of PMA vary with temperature. Asphaltic plug joints have demonstrated a proclivity to soften and creep at warmer temperatures, exhibiting wheel rutting and migration of PMA out of the block outs under high traffic volumes. At warmer temperatures and lower traffic volumes, they tend to heave as joints are compressed. In very cold temperatures, the PMA can become brittle and crack at the plug joint-to-pavement interface, making the joint susceptible to water infiltration. Figure 18.7 shows a failed asphaltic plug joint application.
Some of the performance problems exhibited by asphaltic plug joints can be attributed to inadequate blockout preparation or the use of incompatible binder materials. In other instances, unsatisfactory performance is a result of applying the system to inappropriate applications. Research has been performed to investigate these issues and develop objective design guidelines, material specifications, and installation procedures to improve performance (Bramel et al. 1999).

Anecdotal observations indicate that asphaltic plug joint installations have higher success rates when they are installed near the center of the temperature range to which they will be subjected (Kazakavich, V., Personal Communication, March 2, 2012, Schenectady, NY). Experience has also shown that installations exhibit better cold weather performance after an adequately long warm weather curing period, indicating the preferability of installing asphaltic plug joints during the late spring or early summer seasons (Dolan, C.W., Personal Communication, March 2, 2012, University of Wyoming—College of Engineering and Applied Science—Civil and Architectural Engineering, Laramie, WY).

As with all expansion joint systems, designers must understand the limitations of asphaltic plug joints. These joints were not designed for, and should not be used in, accommodating differential vertical displacements, as may occur at longitudinal joints. Because of the PMA creep susceptibility, asphaltic plug joints should not be used where the roadway is subject to significant traffic acceleration and braking. Examples include freeway off ramps and roadway sections in the vicinity of traffic signals. Asphaltic plug joints have also performed poorly in highly skewed applications and in applications subjected to large rotations. Maintaining the minimum block at depth specified by the manufacturer is particularly critical to successful performance. In spite of these limitations, asphaltic plug joints do offer advantages not inherent in other expansion joint systems. However, they should not be considered as maintenance-free, long-term solutions to accommodate movement.

**FIGURE 18.7** Failed asphaltic plug joint application.
Ongoing research in Europe has developed an advanced polyurethane variation of the asphaltic plug joint (Gallai 2011). The aim of this research has been to combine the advantages of existing asphaltic plug joints with new less temperature sensitive materials to achieve an increase in movement capability and working temperature range. The developers of this new system assert that the two-component advanced polyurethane incorporated into the new plug joint does not exhibit the same adverse temperature dependent characteristics as bituminous polymer used in standard asphaltic plug joints and can accommodate movements of up to 4 in. (102 mm). The researchers report that trial installations in Austria, the United Kingdom, and Italy have performed well for over two years (Gallai 2011).

18.4.5 Poured Sealant Joints

Durable low-modulus sealants, poured cold to provide watertight expansion joint seals as depicted in Figure 18.8, have been used in new construction and in rehabilitation projects. Properties and application procedures vary between products. Most silicone sealants possess good elastic performance over a wide range of temperatures while demonstrating high levels of resistance to ultraviolet and ozone degradation. Rapid-curing sealants are ideal candidates for rehabilitation in situations where significant traffic disruption from extended traffic lane closure is unacceptable. Other desirable sealant properties include self-leveling and self-bonding capabilities. Installation procedures vary among different products, with some products requiring specialized equipment for mixing individual components. Designers must assess the design and construction requirements, weighing desirable properties against material costs for alternative sealants. Figure 18.9 shows a typical poured sealant expansion joint application.

Most sealants can be installed against either concrete or steel. Particularly in rehabilitation projects, it is extremely critical that the concrete or steel substrates be thoroughly cleaned before the sealant is placed. Some manufacturers require application of specific primers onto substrate surfaces prior to sealant placement to enhance bonding. Debonding of sealant from substrate concrete or steel, compromising the integrity of the watertight seal, have previously plagued poured sealant joints. More recently developed sealants have demonstrated very favorable performance and versatility of use in bridge rehabilitation. Continuing improvements in durability can be expected to extend their range of future application.

Poured sealant joints should be designed based upon the manufacturers’ recommendations. Maximum and minimum working widths of the poured sealant joint are generally recommended as a percentage of the sealant joint width at installation. A minimum recess is typically required between the top of the roadway surface and the top of the sealant. This recess is critical in preventing tires from contacting and debonding the sealant from its substrate material.

FIGURE 18.8  Poured sealant joint (cross section).
18.4.6 Design Example 1: Elastomeric Compression Seal

Given:

A reinforced concrete box girder bridge has an overall length of 300 ft. (61 m). A compression seal expansion joint at each abutment will accommodate half of the total bridge movement. These expansion joints are skewed 20°. Superstructure temperature range shall be determined using Procedure A as defined in Article 3.12 of the AASHTO LRFD Bridge Design Specifications, 6th Edition (AASHTO 2012). A moderate climate is assumed.

Requirements:

Elastomeric compression seal sizes and construction gap widths at 40°F (4.4°C), 65°F (18.3°C), and 80°F (26.7°C).

Solution:

Step 1: Determine the appropriate temperature range for the bridge.

AASHTO LRFD Article 3.12.2 allows the use of either Procedure A or Procedure B for determining the design thermal movement associated with a uniform temperature change for a concrete deck bridge supported on concrete girders. LRFD Table 3.12.2.1-1 identifies the temperature range for a concrete girder bridge located in a moderate climate as being 10°F (−12.2°C) to 80°F (26.7°C). A moderate climate is defined as one in which there are less than 14 days per year in which the average temperature is less than 32°F.
Step 2: Calculate temperature and shrinkage movements.
The expansion joint design is based upon the strength limit state, which applies a load factor of 1.2 on uniform thermal displacements. For the purpose of calculating design movements, the 1.2 load factor will be applied to the temperature range determined from LRFD Table 3.12.2.1-1, with minimum and maximum temperatures adjusted accordingly.

\[ T_{\text{range}} = 1.2(80 - 10) = 84^\circ F \]

\[ T_{\text{min}} = 10 - \frac{(84 - (80 - 10))}{2} = 3^\circ F \]

\[ T_{\text{max}} = 80 + \frac{(84 - (80 - 10))}{2} = 87^\circ F \]

\[ T_{\text{midrange}} = \frac{3 + 87}{2} = 45^\circ F \]

\[ \Delta_{\text{temp}} = \left(\frac{1}{2}\right)(0.000006)(87 - 3)(300)(12) = 0.91 \text{ in.} \]

\[ \Delta_{\text{shrink}} = \left(\frac{1}{2}\right)(0.0002)(0.8)(300)(12) = 0.29 \text{ in.} \]

\[ \Delta_{\text{temp}} + \Delta_{\text{shrink}} = 0.91 + 0.29 = 1.20 \text{ in.} \]

\[ \Delta_{\text{temp-normal}} + \Delta_{\text{shrink-normal}} = (1.20)(\cos 20^\circ) = 1.13 \text{ in.} \]

\[ \Delta_{\text{temp-parallel}} + \Delta_{\text{shrink-parallel}} = (1.20)(\sin 20^\circ) = 0.41 \text{ in.} \]

Step 3: Determine required compression seal width from Equations 18.9, 18.10, and 18.12.

\[ W > \frac{1.13}{0.45} = 2.51 \text{ in.} \]

\[ W > \frac{0.41}{0.22} = 1.86 \text{ in.} \]

\[ W > \frac{[0.5(0.91) + 0.29]\cos(20^\circ)}{0.23} = 3.04 \text{ in.} \]

→ Use 3 in. (76 mm) compression seal

Step 4: Evaluate construction gap widths for various temperatures for a 3 in. compression seal.

Construction width at 45°F = (0.62)(3) = 1.86 in.

Construction width at 40°F = 1.86 + \(\frac{(45 - 40)}{(87 - 3)}(0.91)(\cos 20^\circ)\) = 1.91 in.
Construction width at 65°F = 1.86 - \( \frac{65 - 45}{87 - 3} \)(0.91)(\cos 20°) = 1.66 \text{ in.}

Construction width at 80°F = 1.86 - \( \frac{80 - 45}{87 - 3} \)(0.91)(\cos 20°) = 1.50 \text{ in.}

**Conclusion:**

Use 3 in. (76 mm) elastomeric compression seals. Construction gap widths for installation temperatures of 40°F, 65°F, and 80°F are 1.91 in. (49 mm), 1.66 in. (42 mm), and 1.50 in. (38 mm), respectively.

### 18.5 Medium Movement Range Joints

Medium movement range expansion joints accommodate movement ranges from about 1.75 in. (45 mm) to about 5 in. (127 mm) and include sliding steel plate systems, bolt-down panel joints (elastomeric expansion dams), strip seal joints, and steel finger joints. Sliding steel plate systems were previously discussed under small motion range joints.

#### 18.5.1 Bolt-Down Panel Joints

Bolt-down panel joints, also referred to as elastomeric expansion dams, consist of monolithically molded elastomeric panels reinforced with steel plates as depicted in Figure 18.10. They are bolted into block outs formed in the concrete bridge deck on each side of an expansion joint gap. Manufacturers fabricate bolt-down panels in varying widths roughly proportional to the total allowable movement range. Expansion is accompanied by uniform stress and strain across the width of the panel joint between anchor bolt rows. Unfortunately, the bolts and nuts connecting bolt-down panels to bridge decks have historically been prone to loosening and breaking under high speed traffic. The resulting loose panels and hardware in the roadway present hazards to vehicular traffic, particularly motorcycles. Consequently, to mitigate liability, some transportation agencies have phased out their use of bolt-down panel joints. With the increased use of cast-in-place and adhesive anchors in lieu of expansion anchors, bolt-down panel joints have exhibited improved performance and experienced some resurgence in recent years (Kazakavich, V., Personal Communication, March 2, 2012, Schenectady, NY). A typical bolt-down panel expansion joint application is shown in Figure 18.11.

![Bolt-down panel joint (cross section).](image-url)
18.5.2 Elastomeric Strip Seal Joints

An elastomeric strip seal expansion joint system, depicted in Figure 18.12, consists of a preformed elastomeric gland mechanically locked onto metallic edge rails embedded into concrete on each side of an expansion joint gap. Movement is accommodated by unfolding of the elastomeric gland. Steel studs or reinforcing bars are generally welded to the edge rails to facilitate bonding with the concrete in forming block outs. In some instances, the edge rails are welded or bolted in place. Edge rails provide armoring for the adjacent bridge deck concrete. Properly installed strip seals have demonstrated exceptionally good performance. Damaged or worn glands can be replaced with minimal traffic disruptions. The elastomeric glands exhibit a proclivity for accumulating debris. In some instances, this debris can resist joint movement and result in premature gland failure. A typical elastomeric strip seal expansion joint application is shown in Figure 18.13.

The preformed silicone joint sealing system is a variation of the conventionally armored elastomeric strip seal expansion joint system (Watson 2011). This system uses a preformed silicone joint sealing gland that is bonded to vertical concrete or steel joint faces using a single component silicone-based adhesive. The constituent silicone elements of this system have exhibited good resistance against weathering and other types of environmental exposure.

18.5.3 Steel Finger Joints

Steel finger joints, depicted in Figure 18.14, have been used to accommodate medium and large movement ranges. These joints are generally fabricated from steel plate and are installed in cantilever or prop cantilever configurations. The steel fingers must be designed to support traffic loads with sufficient stiffness to preclude excessive vibration. In addition to longitudinal movement, they must also accommodate any
Expansion Joints

Heavy duty anchorage shown

Standard anchorage shown

Steel plate—welded to extruded steel shape at specified spacing

Bent steel reinforcement—welded to steel plate

Block out limits

Elastomeric strip seal

Extruded steel shape

Welded steel studs

Spaced alternately

FIGURE 18.12 Elastomeric strip seal joint (cross section).

FIGURE 18.13 Elastomeric strip seal application.
rotation or differential vertical deflection across the joint. To minimize the potential for damage from snowplow blade impact, steel fingers may be fabricated with a slight downward taper toward the joint centerline. Generally, steel finger joints do not provide a seal against water intrusion to substructure elements. Elastomeric or metallic troughs can be installed beneath the steel finger joint assembly to catch and redirect water and debris runoff. However, unless regularly maintained, these troughs clog with debris and become ineffective (Burke 1989). Two steel finger joint applications are shown in Figure 18.15.

Steel finger joints may be fabricated and installed in full roadway width segments, partial roadway width segments, or shorter modular segments. Robust anchorage of the steel finger joint segments to the bridge superstructure is critical in achieving satisfactory longevity of the overall finger joint system. The most common failures of older steel finger joints are related to anchorage. Impact loading, moisture penetration, and corrosion make these anchorages particularly susceptible to fatigue. Some companies are now marketing standardized steel finger joint panels of shorter modular lengths. Some of these proprietary systems incorporate post-tensioned anchorages for improved durability. In some situations, failed steel finger joint systems cannot be replaced with more modern watertight systems because of spatial limitations or other considerations. In these instances, standardized finger joint panel systems can be installed as replacements. The shorter modular panel segments allow construction to be staged more easily to accommodate traffic demands during construction.

Steel fingers joints can present particular hazards to bicyclist and pedestrians. In these situations, the finger joint assemblies can be modified with cover plates to minimize these hazards. A non-skid surfaces further minimizes any hazard.

18.5.4 Design Example 2: Elastomeric Strip Seal

Given:

A steel plate girder bridge located in east central Indiana has a total length of 600 ft. (183 m). It is symmetrical and has a strip seal expansion joint at each end. These expansion joints are skewed 15°. Superstructure temperature range shall be determined using Procedure B as defined in Article 3.12 of the AASHTO LRFD Bridge Design Specifications, 6th Edition (AASHTO 2012). Assume an approximate installation temperature of 65°F (18.3°C).

Requirements:

Type A and Type B elastomeric strip seal sizes and construction gap widths at 40°F (4.4°C), 65°F (18.3°C), and 80°F (26.7°C). Type A strip seals have a ½ in. (13 mm) gap at full closure. Type B strip seals are able to fully close, leaving no gap.
Expansion Joints

Solution:

Step 1: Determine the appropriate temperature range for the bridge.

AASHTO LRFD Article 3.12.2 allows the use of either Procedure A or Procedure B for determining the design thermal movement associated with a uniform temperature change for a concrete deck bridge having steel girders. AASHTO LRFD Figures 3.12.2.2-3 and 3.12.2.2-4 identify the maximum and minimum design temperature for a steel girder bridge located in an east central Indiana as being 115°F (46.1°C) and −10°F (−23.3°C), respectively.

Step 2: Calculate temperature and shrinkage movement.

The expansion joint design is based upon the strength limit state, which applies a load factor of 1.2 on uniform thermal displacements. For the purpose of calculating design movements, the 1.2 load factor will be applied to the temperature range determined from LRFD Figures 3.12.2.2-3 and 3.12.2.2-4, with minimum and maximum temperatures adjusted accordingly.

\[
T_{\text{range}} = 1.2(115 + 10) = 150^\circ\text{F}
\]

\[
T_{\text{min}} = -10 - \frac{(150 - (115 + 10))}{2} = -22.5^\circ\text{F}
\]

\[
T_{\text{max}} = 115 + \frac{(150 - (115 + 10))}{2} = 127.5^\circ\text{F}
\]

\[
T_{\text{midrange}} = \frac{-22.5 + 127.5}{2} = 52.5^\circ\text{F}
\]

\[
\Delta_{\text{temp}} = \left(\frac{1}{2}\right)(0.0000065)(150)(600)(12) = 3.51 \text{ in.}
\]

FIGURE 18.15 Steel finger joint applications.
\[ \Delta_{\text{shrink}} = 0.0 \text{ (no shrinkage, } \mu = 0.0 \text{ for steel bridge)} \]

\[ \Delta_{\text{temp}} + \Delta_{\text{shrink}} = 3.51 + 0 = 3.51 \text{ in.} \]

\[ \Delta_{\text{temp-normal-closing}} = \frac{127.5 - 65}{127.5 + 22.5} \times 3.51 \cos(15^\circ) = 1.41 \text{ in.} \]

\[ \Delta_{\text{temp-normal-opening}} = \frac{65 + 22.5}{127.5 + 22.5} \times 3.51 \cos(15^\circ) = 1.98 \text{ in.} \]

**Step 3: Determine required strip seal size. Assume a minimum construction gap width of 1.50 in. at 65°F in order to assure that the seal can be replaced at that temperature in the future.**

*Type A*: Construction gap width of 1.50 in. at 65°F will not accommodate 1.41 in. closing and still allow a 0.50 in. gap at full closure. Therefore, construction gap width at 65°F must be at least 1.41 in. + 0.50 in. = 1.91 in. → Use 2 in.

Size required = 2.00 + 1.98 – 0.50 = 3.48 in. < 4.00 → **Use 4 in. (100 mm) strip seal**

*Type B*: Construction width of 1.50 in. at 65°F is adequate.

Size required = 1.50 + 1.98 = 3.48 in. < 4.00 → **Use 4 in. (100 mm) strip seal**

**Step 4: Evaluate construction gap widths for various temperatures for a 4 in. strip seal.**

*Type A*: Required construction gap width at 65°F = 0.50 + 1.41 = 1.91 in. → Use 2 in.

Construction gap width at 40°F = 2.00 + \( \frac{65 - 40}{65 + 22.5} \) \times 1.98 = 2.57 in.

Construction gap width at 80°F = 2.00 – \( \frac{80 - 65}{127.5 - 65} \) \times 1.41 = 1.66 in.

*Type B*: Construction width of 1.50 in. at 65°F is adequate.

Construction gap width at 40°F = 1.50 + \( \frac{65 - 40}{65 + 22.5} \) \times 1.98 = 2.07 in.

Construction gap width at 80°F = 1.50 – \( \frac{80 - 65}{127.5 - 65} \) \times 1.41 = 1.16 in.

**Conclusion:**

Use 4 in. (100 mm) elastomeric strip seals. Construction gap widths for Type A strip seals at installation temperatures of 40°F (4.4°C), 65°F (18.3°C), and 80°F (26.7°C) are 2.57 in. (65 mm), 2.00 in. (51 mm), and 1.66 in. (42 mm), respectively. Construction gap widths for Type B strip seals at installation temperatures of 40°F, 65°F, and 80°F are 2.07 in. (53 mm), 1.50 in. (38 mm), and 1.16 in. (30 mm), respectively.

**18.6 Large Movement Range Joints**

Large movement range joints accommodate more than 5 in. (127 mm) of total movement and include bolt-down panel joints (elastomeric expansion dams), steel finger joints, and modular expansion joints. Bolt-down panel and steel finger joints were previously discussed as medium movement range joints.
18.6.1 Modular Expansion Joints

Modular expansion joints (MEJ), depicted in Figure 18.16, are more complex structural systems designed to provide watertight wheel load transfer across wide expansion joint openings. These systems were developed in Europe and introduced in the United States in the 1960s (Kaczinski et al. 1996). They have been used to accommodate the movements of over 7 ft. (2.1 m). MEJs are generally shipped to the construction site for installation in a fully assembled configuration. A typical MEJ application is shown in Figure 18.17.

Early generation MEJs were designed to accommodate movement in one primary direction. In response to the need to accommodate more complex movements, manufacturers have developed proprietary enhancements to standard MEJs, allowing them to articulate in multiple directions and to accommodate multi-axes rotations. These enhanced MEJs have been used to accommodate complex movements ranging from seismic response to floating bridge transition span movements (Dornsife and Kaczinski 2011). The increased acceptance and use of seismic isolation to mitigate seismic hazards for new and existing bridges has resulted in an increased need to accommodate complex movements between superstructure and substructure elements. In response to this need, manufacturers have further improved MEJ systems and dynamically tested them under laboratory simulated seismic displacement and velocity demands.

MEJs comprise a series of center beams supported atop support bars. The center beams are oriented parallel to the joint axis while the support bars span across the joint opening. MEJs can be classified as either single support bar systems or multiple support bar systems. In multiple support bar systems, each center beam is supported by a separate support bar at each support box location. Figure 18.16 depicts a multiple support bar system. 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 joint opens and closes. This is accomplished by attaching steel yokes to the underside of the center beams. The support bar passes through the openings

**FIGURE 18.16** Modular expansion joint (multiple support bar system; cross section).
in the yokes. Elastomeric springs and sliding bearing surfaces between the underside of each center beam and the top of the support bar and between the bottom of the support bar and the bottom of the yoke support each center beam and permit it to translate along the longitudinal axis of the support bar.

The support bars are, in turn, supported on sliding bearings mounted within support boxes. PTFE (PolyTetraFluoroEthylene) or other proprietary low friction material-to-stainless steel interfaces between elastomeric support bearings and support bars facilitate movement of the support bars as the expansion gap opens and closes. Control springs between adjacent support bars and between support bars and support boxes of multiple support bar MEJs are designed to maintain equal distances between center beams as the expansion gap varies. The support boxes are embedded in bridge deck concrete on each side of the expansion joint. Elastomeric strip seals or elastomeric box type seals attach to adjacent center beams, providing resistance to water and debris intrusion.

The highly repetitive nature of axle loads predisposes MEJ components and connections to high fatigue susceptibility, particularly at center beam-to-support bar connections. Bolted connections have generally performed poorly. Welded connections are preferred, but must be carefully designed, fatigue tested, fabricated, and inspected to assure satisfactory performance and durability. Lack of understanding of the dynamic response of these systems, connection detail complexity, and the competitive nature of the marketplace have exacerbated fatigue susceptibility. Fortunately, research has developed fatigue resistant structural design specifications in addition to minimum performance standards, performance and acceptance test methods, and installation guidelines for MEJs (Kaczinski et al. 1996; Dexter et al. 1997).

As mentioned earlier, modular expansion joints may need to be shipped and installed in two or more segments in order to accommodate projected staging requirements or shipping length restrictions. The center beams are the elements that must be field spliced. These field connections may be either welded, bolted, or a combination of both. Center beam field splices have historically been weak links in the durable performance of MEJs because of their high fatigue susceptibility and tendency to initiate progressive system failures. The reduced level of quality control achievable in the field vis-à-vis a shop
operation contributes to this susceptibility. Mitigating measures include reducing support box spacing, using bolted shear-type connections, and careful detailing, control, and inspection over any field welding operations.

Total movement demands establish MEJ size. Because today’s fatigue resistant MEJ systems, unlike other simpler and less expensive expansion joints, are expected to provide durability for the full life of the structure without the need for replacement, some transportation agencies apply a nominal safety factor on the calculated movement range. The safety factor also permits some latitude in anchoring a very large MEJ at its appropriate gap setting.

Presently available systems permit 3 in. (76 mm) of movement per strip seal element; hence the total movement rating provided will be a multiple of 3 in. (76 mm). To minimize impact and wear on bearing elements, the maximum gap between adjacent center beams is limited, typically, to about 3.5 in. (89 mm) (Van Lund 1991). To facilitate installation within concrete block outs, contract drawings should specify the distance face-to-face from edge beams as a function of superstructure temperature at the time of installation.

Design relationships can be expressed as

\[
\begin{align*}
  n &= \frac{MR}{mr} \quad (18.13) \\
  G_{\text{min}} &= (n-1)(w) + ng \quad (18.14) \\
  G_{\text{max}} &= G_{\text{min}} + MR \quad (18.15)
\end{align*}
\]

where

- \( MR \) = total movement rating of the MEJ system; in.
- \( mr \) = movement rating per strip seal element; in.
- \( n \) = number of seals
- \( n - 1 \) = number of center beams
- \( w \) = width of each center beam; in.
- \( g \) = minimum gap per strip seal element at full closure; in.
- \( G_{\text{min}} \) = minimum distance face-to-face of edge beams; in.
- \( G_{\text{max}} \) = maximum distance face-to-face of edge beams; in.

Structural design of MEJs is generally performed by the manufacturer. Project specifications should require that the manufacturer submit structural calculations, detailed fabrication drawings, and applicable fatigue test results for approval. All elements and connections must be designed and detailed to resist fatigue stresses imposed by repetitive vertical and horizontal wheel loadings. Additionally, MEJs should be detailed to provide access for inspection and periodic maintenance, including replacement of seals, control springs, and bearing components.

### 18.6.2 Design Example 3: Modular Expansion Joint System

**Given:**

Two cast-in-place post-tensioned concrete box girder bridge frames meet at an intermediate pier where they are free to translate longitudinally. Skew angle is 0°. The transportation agency owning this bridge has established its own historical temperature range for evaluating superstructure uniform thermal displacements. That temperature range is conservative relative to the procedures delineated in AASHTO LRFD Article 3.12.2, thus a strength limit load factor of 1.0 on uniform thermal movement is justified. Superstructure ambient temperatures are deemed to range from 5°F (−15°C) to 120°F (48.9°C). A MEJ will be installed 60 days after post-tensioning operations have been completed. Specified creep
is 150% of elastic shortening. Assume that 50% of shrinkage has already occurred at installation time. The transportation agency has an internal policy of sizing MEJ for 115% of the calculated movement demand. The following longitudinal movements were calculated for each of the two frames:

### Requirements:

MEJ size required to accommodate the total calculated movements and the installation gaps measured face-to-face of edge beams, $G_{\text{install}}$, at 40°F (4.4°C), 65°F (18.3°C), and 80°F (26.7°C).

### Solution:

#### Step 1: Determine MEJ size.

Total opening movement (Frame A) = $(0.5)(1.18) + 2.13 + 2.99 = 5.71$ in.
Total opening movement (Frame B) = $(0.5)(0.59) + 1.18 + 1.50 = 2.99$ in.
Total opening movement (Both Frames) = $5.71 + 2.99 = 8.70$ in.
Total closing movement (Both Frames) = $2.60 + 1.30 = 3.90$ in.

Determine size of MEJ, including a 15% allowance:

$1.15(8.70 + 3.90) = 14.49$ in.

→ Use 15 in. (381 mm) movement rating MEJ

#### Step 2: Evaluate installation gaps measured face-to-face of edge beams at 40°F, 65°F, and 80°F.

MR = 15 in. (MEJ movement range)
$mr = 3$ in. (maximum movement rating per strip seal element)
$n = 15/3 = 5$ strip seal elements
$n - 1 = 4$ center beams
$w = 2.5$ in. (center beam top flange width)
$g = 0$ in.

$$G_{\text{min}} = (n-1)(w) + ng = (4)(2.5) + (5)(0) = 10 \text{ in.}$$

$$G_{\text{max}} = G_{\text{min}} + MR = 10 + 15 = 25 \text{ in.}$$

Recognizing that shrinkage and creep effects will cause the joint to permanently open with time, the installation strategy will be to set the joint as closely as possible to being fully closed if it experiences the maximum temperature extreme immediately following installation.

$$G_{65F} = G_{\text{min}} + 1.15(\text{Total closing movement}) = 10 + 1.15(3.90) = 14.48 \text{ in.}$$

→ Use 14.5 in.

$$G_{65F} = 14.5 + \left(\frac{65 - 40}{65 - 5}\right)(2.99 + 1.50) = 16.37 \text{ in.}$$
Expansion Joints

\[ G_{80F} = 14.5 - \left( \frac{80 - 65}{120 - 65} \right) (2.60 + 1.30) = 13.44 \text{ in.} \]

Check spacing between center beams at minimum temperature:

\[ G_{SF} = 15 + 8.70 = 23.70 \text{ in.} \]

Maximum spacing = \[ \frac{23.70 - (4)(2.5)}{5} = 2.74 \text{ in.} < 3.5 \text{ in.} \]

O.K.

Check spacing between center beams at 65°F for seal replacement:

\[ \text{Spacing} = \frac{15 - (4)(2.5)}{5} = 1.00 \text{ in.} < 1.5 \text{ in.} \]

Therefore, center beams must be mechanically jacked in order to replace strip seal elements.

**Conclusion:**

Use a MEJ with a 15 in. (381 mm) movement rating. Installation gaps measured face-to-face of edge beams at installation temperatures of 40°F (4.4°C), 65°F (18.3°C), and 80°F (26.7°C) are 16.37 in. (416 mm), 14.5 in. (368 mm), and 13.44 in. (341 mm), respectively.

18.7 Installation Considerations

Proper installation of an expansion joint system is a critical element of assuring its long-term performance and durability. Proper installation requires an understanding of how a structure and its expansion joints respond to thermal effects during the installation process. Particularly for a more complex proprietary expansion joint system, it also requires an intimate understanding of the critical aspects of the installation process as they relate to the performance of the system. For example, it is extremely critical that good concrete consolidation be achieved underneath modular expansion joint support boxes.

It is highly advisable that the contract specifications require a qualified installation technician be present at the job site in order to assure proper installation of more complex expansion joint systems. The specifications should preferably stipulate that the technician is a full-time employee of the manufacturer of the expansion joint system in order to assure that the technician is adequately trained and knowledgeable.

For less complex expansion joint systems, provision shall be included to assure that a contractor’s crew is adequately trained in the nuances of installation prior to performing its first installation of that system. For example, installation of a rapid-cure silicone sealant expansion joint system requires adequate knowledge and training in joint surface preparation, primer application, and sealant installation and curing requirements.

18.7.1 Thermal Effects

Bridge expansion joints are constantly moving in response to a number of different phenomena. The most significant of these phenomena is uniform variation in structure temperature with time. Fixed dimensions identified on contract drawings are generally associated with a *mean* or *construction* temperature. A contractor is required to adjust his concrete forms accordingly when the structure is built at other temperatures. Similarly, a steel fabricator is required to adjust his dimensions commensurately when fabricating at other temperatures.
Likewise, expansion joint gaps vary with changes in bridge superstructure temperature. For expansion joint hardware installed in preformed blockouts that are subsequently filled with concrete, this requires some accommodation during the hardware setting process. Specifically, a strip seal, steel finger, or the modular expansion joint system must be anchored in blockouts at a gap setting corresponding to the temperature of the superstructure at the time of anchorage. A compression seal installed in a sawcut gap requires similar accommodation. The gap must be sawcut to an opening width that corresponds to the temperature of the superstructure at the time of sawcutting. The temperature—gap setting relationship is best communicated to the contractor by including a temperature versus gap setting table in the contract drawings.

Structures are subject to several modes of heat transfer: radiation, convection, and conduction. Each mode affects the thermal response of a structure differently. Variations in ambient air temperature produce uniform thermal variation in the superstructure with concomitant lengthening or shortening if unrestrained. Solar radiation imposed upon a bridge deck produces thermal gradient effects in the superstructure. The overall thermal response of a superstructure lags changes in ambient air temperature in accordance with thermodynamic principles. More massive concrete bridges respond more slowly than steel bridges composed of thinner, more heat conductive, elements. The slower response attenuates both diurnal and seasonal temperature extremes of the superstructure relative to ambient air temperature. Objective evaluation of the superstructure’s temperature at any time must take these factors into consideration.

Installation of a larger expansion joint system requires a well-thought out logical procedure for temporarily supporting the system and anchoring it at the appropriate gap setting. The procedure must include reasonably accurate evaluation of the superstructure temperature and recognition that the superstructure is constantly moving under varying temperature. The latter point is important because the anchorage and concrete placement sequence may take several hours to complete. The installation process must assure that any temporary restraints against opening or closing of the expansion joint system are promptly removed once concrete begins to set up. Otherwise, the restraint will damage or tear out the anchorages from the concrete.

18.7.2 Design Example 4: Finger Joint Installation Procedure

Given:
An existing steel finger expansion joint system has reached the end of its serviceable life and needs to be replaced. Spatial constraints associated with the existing steel truss structure and concrete approach span preclude replacement of the existing system with a watertight modular expansion joint system. A proprietary steel finger joint system composed of modular segments is selected to accommodate traffic staging requirements. This system incorporates post-tensioned anchorages of the finger joint panels for enhanced durability. A single row of vertical bolts anchor the panels on each side of the joint. The contract drawings include a temperature versus gap setting table showing the horizontal distance between the two rows of bolts to be 1’–9” @ 40°F (533 mm @ 4.4°C), 1’–7¼” @ 64°F (489 mm @ 17.8°C), and 1’–6” @ 80°F (457 mm @ 26.7°C).

Requirements:
Develop a logical procedure for installing the finger joint panels at the appropriate expansion gap setting. The procedure shall include a rational approach to measure the temperature of the superstructure for the purpose of locating the finger joint panels, for temporarily supporting the finger joint panels prior to concrete placement, and for the timely removal of any temporary restraint devices.
Expansion Joints

Solution:

Exposure to direct sunlight and varying ambient air temperature makes it difficult, if not impossible, to approximate average superstructure temperature based upon ambient air temperature. The bridge deck surface temperature on a sunny afternoon, as opposed to a very cloudy overcast day, is not representative of the average superstructure temperature. After several hours of relatively stable nighttime temperature conditions, ambient air temperature best approximates bridge superstructure temperature. With this in mind, take an air temperature reading within one-half hour (before or after) sunrise. The stepwise procedure for installing the steel finger joint panels is as follows:

Step 1: Measure the air temperature reading at sunrise as 52°F (11.1°C). Interpolate from the temperature versus gap setting table to determine that the appropriate distance between the two bolt rows at 52°F (11.1°C) is 1′–8⅛″ (511 mm).

Step 2: Immediately paint, or otherwise mark, two fine parallel lines along the axis of the expansion joint. One line is approximately centered at the bottom of the concrete blockout on the approach span side of the expansion joint. The second line is on the inside bottom of the stay-in-place steel form on the truss span side of the expansion joint, separated horizontally from the first line by 1′–8⅛″. The stay-in-place steel form serves as the bottom and sides of a reinforced concrete beam supporting the finger joint panels. These two painted lines establish the location of the two rows of anchor bolts. Henceforth, the distance between these two lines will vary as the bridge expands and contracts throughout the day.

Step 3: Place temporary support beams atop the bridge deck with both sides of the finger joint segments bolted underneath. The temporary support beams span fully across the expansion joint blockouts perpendicular to the axis of the expansion joint. Anchor the support beams to the top of the concrete deck on the approach span side of the expansion joint using temporary expansion anchors. (The temporary support beams remain free to slide relative to the top of the bridge deck on the steel truss span side of the expansion joint.)

Step 4: Adjust the finger joint segments on the approach span side of the expansion joint so that their anchor bolts line up vertically with the painted line at the bottom of the concrete blockout.

Step 5: Place concrete in the formed blockout underneath the finger joint segments on the approach span side of the expansion joint.

Step 6: After concrete has set, loosen and remove bolted attachment of the steel finger joint segments to the temporary support beams on the approach span side of the expansion joint.

Step 7: Remove the temporary expansion anchors anchoring the temporary support beams to the approach span side of the expansion joint.

Step 8: Anchor the support beams to the top of the concrete deck on the steel truss span side of the expansion joint using temporary expansion anchors. (The temporary support beams are now free to slide relative to the top of the bridge deck on the approach span side of the expansion joint.)

Step 9: Adjust the finger joint segments on the steel truss span side of the expansion joint so that their anchor bolts line up vertically with the painted line on the inside bottom of the stay-in-place steel form.

Step 10: Place concrete in the blockout underneath the finger joint segments on the steel truss span side of the expansion joint. Note: It is important that adequate time be provided between the two concrete pours to allow the first concrete pour to set, to permit the temporary support beam expansion anchors to be removed and reset, and to reposition all finger joint segments on the steel truss span side of the bridge.

Step 11: After concrete in the second blockout has set, loosen and remove bolted attachment of the steel finger joint segments to the temporary support beams on the steel truss span side of the expansion joint.
Step 12: Remove the temporary expansion anchors anchoring the temporary support beams to the steel truss span side of the expansion joint.

Step 13: Remove all temporary support beams and hardware. Fill all expansion anchor holes in the bridge deck with approved grout.

18.8 Summary

A wide range of different expansion joint systems is available for accommodating bridge superstructure movements. Appropriate selection and design procedures, quality fabrication, competent installation practices, careful inspection, and routine maintenance all contribute to enhancing the long-term performance and durability of expansion joint installations. Expansion joint components and connections experience severe loading under harsh environmental conditions. An adequately designed system must be properly manufactured, installed, and maintained to assure adequate performance under these conditions. The importance of quality control cannot be overemphasized. Contract drawings and specifications must explicitly state all design, material, fabrication, installation, and quality control requirements. Structural calculations and detailed fabrication drawings should be submitted to the bridge designer for careful review and approval prior to fabrication. A qualified installation technician employed by the expansion joint system manufacturer should be present during installation to assure that all the manufacturer’s installation recommendations are being followed.

Research and experience continues to improve expansion joint system technology (Stoyle 1991; Atkinson 1996). It is vitally important that design engineers keep abreast of new technological developments. Interdisciplinary and inter-agency communication plays a vital role in the exchange of important information. Maintenance personnel can furnish valuable feedback to designers for implementation in future designs. Designers can provide valuable guidance to maintenance personnel with the goal of increasing service life. Manufacturers furnish designers and maintenance crews with guidelines and limitations for successfully designing and maintaining their products. In turn, designers and maintenance personnel provide feedback to manufacturers on the performance of their products and how they might be improved. Communication among disciplines is paramount to improving the long term performance and durability of expansion joint systems.

References


Atkinson, B. 1996. *Fourth World Congress on Joint Sealants and Bearing Systems for Concrete Structures*, American Concrete Institute, Farmingham Hills, MI.


Dornsife, R.J., and M.R. Kaczinski. 2011. “Homer M. Hadley bridge large modular expansion joint replacement,” *Seventh World Congress on Joints, Bearings, and Seismic Systems for Concrete Structures*, American Concrete Institute, Farmingham Hills, MI.
Gallai, G. 2011. “A new flexible plug joint—polyflex advanced PU,” Seventh World Congress on Joints, Bearings, and Seismic Systems for Concrete Structures, American Concrete Institute, Farmingham Hills, MI.


Steiger, D.J. 1991. “Field evaluation and study of jointless bridges,” Third World Congress on Joint Sealing and Bearing Systems for Concrete Structures, ed. Stoyle, J.E., American Concrete Institute, Farmington Hills, MI, 227.

Stoyle, J.E. 1991. Third World Congress on Joint Sealing and Bearing Systems for Concrete Structures, American Concrete Institute, Farmingham Hills, MI.


Watson, E.S. 2011. “High performance joint sealing system for the 21st century,” Seventh World Congress on Joints, Bearings, and Seismic Systems for Concrete Structures, American Concrete Institute, Farmingham Hills, MI.
