SCAN TEAM REPORT

NCHRP Project 20 68A, Scan 17-03

Experiences in the Performance of Bridge Bearings and Expansion Joints Used for Highway Bridges

Supported by the

National Cooperative Highway Research Program

The information contained in this report was prepared as part of NCHRP Project 20-68A U.S. Domestic Scan, National Cooperative Highway Research Program.


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The purpose of each scan, and of Project 20-68A as a whole, is to accelerate beneficial innovation by facilitating information sharing and technology exchange among the states and other transportation agencies, and identifying actionable items of common interest. Experience has shown that personal contact with new ideas and their application is a particularly valuable means for such sharing and exchange. A scan entails peer-to-peer discussions between practitioners who have implemented new practices and others who are able to disseminate knowledge of these new practices and their possible benefits to a broad audience of other users. Each scan addresses a single technical topic selected by AASHTO and the NCHRP 20-68A Project Panel. Further information on the NCHRP 20-68A U.S. Domestic Scan program is available at http://144.171.11.40/cmsfeed/TRBNetProjectDisplay.asp?ProjectID=1570.

This report was prepared by the scan team for Scan 17-03, *Experiences in the Performance of Bridge Bearings and Expansion Joints Used for Highway Bridges*, whose members are listed below. Scan planning and logistics are managed by Arora and Associates, P.C.; Harry Capers is the Principal Investigator. NCHRP Project 20-68A is guided by a technical project panel and managed by Andrew C. Lemer, Ph.D., NCHRP Senior Program Officer.

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Disclaimer

Scan 17-03

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REQUESTED BY THE

American Association of State Highway and Transportation Officials

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Abbreviations and Acronyms

AASHTO	American Association of State Highway and Transportation Officials
ABC	Accelerated Bridge Construction
ACWS	Asphalt Concrete Wearing Surface
ASL	Approved Supplier List
ASTM	American Society for Testing and Materials
BDM	Bridge Design Manual
BRIM	Bridge Replacement and Improvement Management
BrM	Bridge Management software (formerly Pontis)
BSIPM	Bridge and Structures Inspection Program Manual
CI	Construction Inspector
CMGC	Construction Manager/General Contractor
COBS	Committee on Bridges and Structures
CR	Chloroprene Rubber
DB	Design Build
DOT	Department of Transportation
ECC	Engineered Cementitious Composites
EOR	Engineer of Record
FDOT	Florida Department of Transportation
FHWA	Federal Highway Administration
GFRP	Glass Fiber Reinforced Polymer
HLMR	High Load Multi-rotational (bearing)
HPC	High Performance Concrete
HyFRC	Hybrid Fiber Reinforced Concrete
LaDOTD	Louisiana Department of Transportation and Development
LRFD	Load and Resistance Factor Design
MDOT	Michigan Department of Transportation
MnDOT	Minnesota Department of Transportation
NCHRP	National Cooperative Highway Research Program
NR	Natural Rubber
NYSDOT	New York State Department of Transportation
PBES	Prefabricated Bridge Elements and Systems
PCI	Precast Concrete Institute
PennDOT	Pennsylvania Department of Transportation
PTFE	Polytetrafluoroethylene
SCC	Self Consolidating Concrete
SIMS	Structure Information Management System
SPMT	Self-Propelled Modular Transporter
SREB	Steel-Reinforced Elastomeric Bearing

T 2	AASHTO COBS Technical Committee for Bearings
TxDOT	Texas Department of Transportation
UBIT	Under Bridge Inspection Truck
UDOT	Utah Department of Transportation
UHPC	Ultra High Performance Concrete
US	United States
VDOT	Virginia Department of Transportation
WSDOT	Washington State Department of Transportation

Glossary

Blockout	A recess in concrete member, typically set to accommodate another element during construction. Created by placing a box in forms to prevent concrete from filling an area.
BrM	Bridge Management software system sponsored by FHWA and owned by American Association of State Highway and Transportation Officials. Formerly known as Pontis.
Center Beam	In modular joints, the steel members parallel to joint face with formed slots on each side that hold the gland that spans between them. Beam top surface is level with roadway
Elastomeric Concrete	Concrete made using a polymer binder.
EMSEAL	Product name for expansion joints and sealants
End dam	Hardware embedded in the concrete on the two opposing faces of a joint.
Finger Joint	Expansion joint comprised of two horizontal steel plates, one on each side of the joint. The opposing faces of the plates are profiled with teeth that fit together like the teeth on two meshing gears, and so provide support for vehicle wheels when the plates move together or apart. Each joint plate can be installed as one piece for the full roadway width, or broken up into smaller segments to facilitate construction under traffic.
Header	Steel angle or other hardware, attached one to each face of the joint, to protect the corner from impact damage. In the case of angles, one leg is placed horizontally, flush with the roadway surface, and the other is placed vertically, flush with the face of the joint. The hardware is typically attached to the concrete using welded studs or bars that are embedded in the concrete.
Lateral Slide	A method of accelerated bridge construction where a new superstructure is built on temporary supports parallel to an existing bridge. The old bride is either demolished or slid out of place. The new bridge is then slid in place.
Link Slab	A section of deck slab that connects the decks on two opposing girders over an internal pier. It is usually cast separately from the two decks that it connects. When it is placed continuously with the surrounding deck concrete, the link slab may be referred to as a "continuous deck".
MiBRIDGE	A web based structure management application allowing Bridge Owners, Engineers, Inspectors, Consultants, and Managers to view and enter information for bridge and culvert assets across the State of Michigan.
MMFX	Specialty steel company providing reinforcing steel with high corrosion resistance
MOOG	Under-Bridge access platform units.
Pontis	Software application for managing highway bridges and other structures. Now known as AASHTOWare Bridge Management (BrM).

Semi-integral Abutment	An abutment, with bearings, in which the girders and back wall are made monolithic and act as a single unit. Longitudinal movement is accommodated by sliding motion of the back wall relative to the stem and by compression of a compliant material behind the back wall.
Strip Seal	Elastomeric sealing element located between, and mechanically locked into, locking edge rails anchored on each side of the joint.
Support Bar	In modular joints, the steel members perpendicular to the joint face that support center beams. Support bars slide at the support box.
Support Bar Box	In modular joints, the steel box concrete on each side of the joint that supports the Support Bar. The Support Bar Boxes are is cast into the header concrete on each side of the joint. The Support Bar is fixed on one side of the joint and free to slide on the other side.
Tooth Dam	See Finger Joint.

Executive Summary

Introduction

Bridge bearings are intended to allow free movement of the superstructure in response to thermal changes and other loadings while supporting it against gravity loads. Joints in the deck accommodate those movements by opening and closing but must be sealed to prevent water and deicing salts from leaking onto the bearings below and causing them to deteriorate. Damaged bearings can bind up and induce unintended forces and damage in other bridge elements. Rectifying that chain of damage, from joint seal to bearing to structure, costs agencies tens of millions of dollars every year, which is many times the initial cost of the items themselves. The damage also has the potential for causing significant traffic interruptions, with corresponding opportunity costs. Properly maintaining bearings and joints is therefore a cost-effective way of ensuring bridge safety and serviceability. Little work has been done on joints and bearings at the national level during the last 15 years so this is an appropriate time to conduct a domestic scan on the subject.

Scan Purpose and Scope

Domestic Scan 17-03, Experiences in the Performance of Bridge Bearings and Expansion Joints Used for Highway Bridges, was conducted March 21 through 25, 2018. It was initiated to facilitate the exchange of recent ideas and best practices for bridge bearings and expansion joints and included design, performance evaluation, and maintenance and repair/reconstruction. Discussions involved staff from design, construction, maintenance, and operations of state and other transportation agencies. Details (i.e., materials, span arrangements, and geometry) for various bridge types and sizes were examined. Lessons were drawn from both the good and bad experiences by the various participating agencies.

The team selected scan hosts that would encompass a wide range service conditions for bearings and joints. Considerations included such items as: climate challenges of the regions in which they are located, traffic volume, and project size. The states were selected on the bases of:

- States with severe climate challenges (cold and freezing conditions) - Illinois, New York and Massachusetts.
- States with considerable precipitation and cold climates - Washington State and Oregon.
- States with very high ADT’s on many bridges - California, Texas, & New York.
- Coastal states with large bridges such as Florida, Virginia, and Louisiana.
- States having success with the details and practices that they use (Minnesota) and those with lessons learned that they could share (Pennsylvania).

General topics of interest to the scan team include:

- Design and details, construction specifications and maintenance procedures for durable bearings and expansion joints that have a history of good in-service performance;
- Visual inspection and other testing of joint and bearing details;
- Specialized technology and standards used in monitoring, inspection, and repair of joint and bearing details, with the goal of ensuring safety and serviceability, minimizing downtime during bridge construction and rehabilitation; and
- Relative costs for design, construction, maintenance, and inspection of various joint and bearing details.
- Lessons learned and suggestions for improvement.

The findings of this scan are expected to be of specific interest to the AASHTO Committee on Bridges and Structures Technical Committee T-2 "Bearings and Expansion Devices", the AASHTO Committee on Materials and Pavements, and the AASHTO Committee on Maintenance. The scan report will provide current information on successful expansion joints and bearings to bridge owners. It will also provide valuable information to the AASHTO Committees for future consideration when developing their work plans and research needs. A synthesis of this information would also be of interest to State DOTs and FHWA offices, other Federal and local agencies involved in bridges, bearing and joint manufacturers, university researchers, consultants, county, city and local DOTs.

Summary of Findings and Recommendations

The scan team identified a number of commonalities among DOT's issues with bearings and joints as well as climate and volume specific issues.

Maintenance

In most states, the focus has been on maintenance and repair than on new construction. That reflects the fact that the bulk of freeway construction occurred in the 1960s and 1970s, and that the bridges built in that era are now approximately 50 years old and starting to show their age. However, limits on funding mean that bridge life may have to be extended, and hence diligent maintenance is important.

Policies for conducting maintenance were found to vary from state to state and were partly related to climate. Snowy states tend to conduct more frequent and aggressive cleaning and maintenance of both joints and bearings. Most states are also adopting electronic approaches to monitoring and record keeping and have made efforts to reduce the number of bridges in poor condition by conducting aggressive repair strategies. Those efforts depend on the maintenance resources available, such as manpower and funding, which vary widely from state to state.

Bearing Types

The types of bearing used have changed over the years. In the past, many bridges were made with steel girders and steel rocker and roller bearings, but metal bearings have been found to perform poorly, especially in earthquakes. These bearings are almost never specified today for new construction, but many still exist on older bridges. They are being, or have already been, replaced in seismic states, and in many others as well. For short to medium spans, steel reinforced elastomeric bearings are now the most widely used, because they have no moving parts, have a low first cost, and need essentially no maintenance. For higher loads and larger movements, High Load Multi-Rotational (HLMR) bearings (pots, discs and spherical bearings) are used. Of these, disc bearings are starting to capture the largest share of the market because they use less steel than pot or spherical bearings and are easier to inspect. They have also provided reliable performance over the years. However, exceptions exist. For example, California uses spherical bearings almost exclusively for high load applications and is well satisfied with their performance. The manufacturers with international sales reported that, in Europe, pot bearings are widely used, apparently without problems, and disc bearings are not used at all.

Joint Types

Joint types are described largely by their movement capacity, and are categorized as small (< 2”), medium (2” to 4”) and large (> 4”).

For large movements, finger joints were the traditional choice, but are gradually falling out of use, and modular joints are becoming more common. A third option, California’s Plate Joint Seal Assembly, has been developed recently and attracted interest. Modular joints suffered from fatigue problems early in their development, but those have largely been resolved. Nonetheless, modular joints are large, complicated mechanisms that require considerable care during fabrication and installation.

Many different types of joints are available for medium movements, and the descriptions are complicated by the fact that the same joint type is known by different names in different regions. The choice of joint depends somewhat on whether the joint is for new construction or retrofit. For new construction, gland-based systems are now the most common choice. In mechanically bonded strip seals, the gland is held in place by a slot in the steel end dam. In adhesive based strip seals (or “pre-formed silicone seals”) the gland is secured by adhesive to the header concrete, or to the armor if that exists. Other choices for medium joints include foam-based systems and elastomeric compression seals. Most states reported more problems with these types of joints than with gland-based systems.

For small movements, poured silicone seals with a foam backer rod are the most widely used. They have provided good performance and are relatively easy to replace. They have also been used as a retrofit joint over a strip seal when an overlay is applied and the joint needs to be raised. Plug seals are also quite widely used, again because they are simple to install. Asphalt plugs are most often used with asphalt overlays, while elastomeric concrete plug joints are more often used in other cases. However, several states reported poor durability for plug joints, in particular with respect to rutting.

Joint Locations

The patterns of joint use are more varied. First, most states are moving in some way towards "jointless bridges", in which the joints are either eliminated altogether or moved to a location behind the abutments, where there are no bearings to be adversely affected by joint failure. New bridges are built with no joints over the piers, and existing bridges are being retrofitted with link slabs, or even diaphragms, over the piers to eliminate a joint there. All longitudinal movement is then taken at the abutments. In fully integral abutments, the piles, stem and back wall are all made monolithic with the girders, and expansion is accommodated by bending of the piles and compressible material, such as foam, behind the back wall. In semi-integral abutments, the foundation and stem are monolithic. The diaphragm and back wall are made monolithic with the girders, but are supported on bearings and which allow movement relative to the stem. The back wall extends below the stem and prevents debris and moisture from contacting the bearings. The joint is either directly behind the back wall or is moved to the far end of the approach slab. In either location, a leak in the joint will not jeopardize the bearings.

Develop Design Specifications

For bearings, there is a need for greatly expanded design requirements for disc bearings, for which the requirements presently in the AASHTO LRFD Design Specifications are minimal. The specifications for elastomeric bearings presently contain an allowance of 0.005 radians (0.5% slope) to account for rotational errors during installation. This includes both levelling of the grout pad and estimating more accurately the girder camber. It is clear that many bearings are not installed to this accuracy, so the bearings have to sustain a permanent rotation that is larger than intended. Either verification of installation accuracy should be improved or the allowance in the design specifications should be increased. The specifications for testing elastomeric bearings, presently contained in the AASHTO materials specification M-251, also need to be updated and better correlated with the LRFD Design Specifications. The manufacturers also expressed the view that a performance-based, rather than prescriptive, specification would provide the incentive for innovation and new bearing types.

For joints, the requirements in the AASHTO LRFD Design Specifications are less prescriptive than those for bearings, and consequently more types of joints exist. The findings of NCHRP project 12-100 compare the performance of different joint types, and help guide type selection, at least for small movements. The shift towards fully- and semi-integral construction raises questions of design, at both the system and detailed levels, and research to address these would be helpful. For example, Virginia has recently developed its own abutment design, which is a variation on the semi-integral theme. An evaluation, and perhaps further development, of that design would be useful.

Knowledge Transfer

Many areas for improvement were identified. Some were associated with organizational and administrative functions, such as use of internet-based methods for tracking the condition of bridge elements (such as joints and bearings) using tablet computers for entering data in the field, ways of transferring knowledge from more to less experienced staff, the development of training tools for type selection (e.g., a selection guide), and procedures for inspection, maintenance and repair.

Future Work Identified

Some detailed questions remain with respect to particular joints. In mechanically bonded Strip Seals, the gland can be replaced, but there is as yet no agreement about whether the gland should be secured with adhesive in the slot, or about the economic benefits of using stainless steel for the end dam to avoid corrosion and facilitate gland replacement. Many of the problems associated with joints center around the headers, and studies to compare the performance of the different types of concrete (cementitious HPC, SCC, UHPC, polymeric, etc.) would be beneficial.

Planned Implementation Actions

The scan team initially presented its findings and recommendations to the AASHTO SCOBS T-2 during the June 2018 Meeting. An overview of the findings was also presented at the general session of that meeting. To disseminate information from the scan, the team is giving technical presentations at national meetings and conferences sponsored by the TRB, ASBI and other organizations and is planning to write papers for various publications.

CHAPTER 1

Introduction

Overview of the Domestic Scan Program

This study was conducted as part of the National Cooperative Highway Research Program (NCHRP) Project 20-68A, the U.S. Domestic Scan program. This program was requested by the American Association of State Highway and Transportation Officials (AASHTO) through funding provided by NCHRP. Additional support for selected scans is provided by the Federal Highway Administration (FHWA) and other agencies. The purpose of each scan, and of Project 20-68A as a whole, is to accelerate the integration of innovative ideas into practice by information sharing and technology exchange among state transportation agencies. Experience has shown that personal contact with new ideas and their application is a particularly valuable means for sharing information about practices. A scan entails peer-to-peer discussions between practitioners who have implemented practices of interest and who are able to disseminate knowledge of these practices to other peer agencies. Each scan addresses a single technical topic that is selected by AASHTO and the NCHRP 20 68A Project Panel. Further information on the NCHRP 20-68A U.S. Domestic Scan program is available at

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Background

The function of a bridge bearing is to support the applied loads and enable the free movements needed for satisfactory performance of the bridge. Generally, the bearings separate the superstructure from the substructure and the loads are vertical and due to gravity, while the movements are horizontal, caused by thermal effects, shrinkage and creep. However, in some cases, the horizontal movement must be prevented and the load that is induced must be resisted. Examples include wind, centrifugal forces and vehicle braking forces. Ideally, the forces would be resisted rigidly, and the movements would occur with no resistance at all, but in practice this ideal cannot be met, and the bearings provide high stiffness for resisting loads and low stiffness for accommodating movements.

Failure to resist the required forces may lead to misalignment of bridge components and a corresponding hazard. In extreme cases, collapse is possible. Failure to allow the required movements leads to induced forces, with the potential for damage both to the bearings themselves and to other elements.

The primary function of a bridge joint is to accommodate movements of the superstructure, such as those caused by prestressing and by thermal and shrinkage effects. The joint must allow the movements to occur without causing a safety hazard by allowing large individual gaps into which vehicle wheels, and particularly those of motor cycles, could otherwise fall. The purpose of the joint seal is to protect the bearings by preventing rain snow, ice and dirt from falling through the joint onto them.

Failure of a joint risks damage to the bearings, especially bearings with steel components and moving parts. If the joint has metal components, such as a steel armor angle or the fingers of a steel finger joint, and these become misaligned or detached, they can create a hazard for vehicles. This problem is prevalent in snow states, where snowplow blades can catch the metal edge of the joint, thereby causing damage.

Joints and bearings are intended to move, whereas most civil engineering structures are intended not to, so the joints and bearings should be thought of as specialized mechanical devices, and designed and treated accordingly. They must be designed in the context of the bridge acting as a system, in which their interactions with the other elements are modeled appropriately. For example, if the bearings are intended to resist horizontal loads and to guide the bridge superstructure so that it displaces only longitudinally, then lateral expansion and contraction of the superstructure must be considered in the design. Using guided bearings under all the girder lines will restrain the lateral thermal movements and set up potentially large forces. Alternatives such as using a single guide system at mid-width of the superstructure, and allowing the remaining girders to move laterally, should be considered.

Joints and bearings also need to be installed using the appropriate tools and techniques, and to tolerances that take into account the state of the bridge at the time of installation, particularly with respect to shrinkage and temperature. If they are treated improperly, they exact retribution by causing repair costs far exceeding the value of the device itself.

This high leverage of costs is a major reason for studying joints bearings. However, a study of them does not follow the text-book example of undisputed theory, followed by translation into practice. The reasons are largely associated with the wide variety of devices that have been brought to market over the years, the different principles on which they depend, and the limits to which the materials can be taken. For example, how much strain can be applied to the gland of a strip seal, how many times can it be applied, at what temperature, and with what level of roadway debris in the seal, before failure will occur? These questions can best be answered by observing behavior under field conditions, and by sharing the results among participants. For this reason, a scanning tour is an ideal way to determine what works and what does not, and to disseminate this information for the benefit of all parties.

Scan Team, Host Agencies and Manufacturers.

Membership of the Scan Team, and information about the host agencies and the manufacturers who participated, are given in Appendix C. The host agencies were chosen to represent a variety of conditions, including traffic levels, meteorological conditions, approaches to maintenance, etc. The manufacturers were selected to ensure input to the team over the widest range possible of subject areas. These included the type of device sold, the number of states in which they operated, time in business, etc.

Objectives and Scope

The primary objective of the scan was to gather the best information possible on the design and performance of bridge joints and bearings and to synthesize it in a form that would allow it to be shared among agencies responsible for bridges. These include both public bodies, such as states, counties and cities, and other entities such as toll authorities, ports and harbors.

Methodology

Host Agencies and Participants

The team members, host agencies and participants are listed in Appendices A, B and C.

Initial Team Meeting

The team gathered for an initial meeting in Washington DC on 17 November 2017. The primary purpose of the meeting was to explain to all members the objectives, and to agree on a methodology for the study

Survey and Amplifying Questions

A series of amplifying questions was developed through interaction among team members, and these were sent to the eight host agency states. The purpose of the amplifying questions was to determine how the recipient states addressed the performance of joints and bearings. It was designed to be detailed enough to meet the scan team's objectives and to provide a basis on which the invited agencies could prepare their responses. To simplify response, the questions were divided into sections concerning design, fabrication and construction, and inspection and maintenance, since most DOTs have separate sections that address those issues.

The amplifying questions are provided in Appendix E.

Scan Workshop

The Scan Workshop was held 19- 23 March 2018 in San Diego, CA. The first three days (Monday – Wednesday) were devoted to presentations by, and question/answer sessions with, the host agency states. On Thursday 24 March, the manufacturers also gave presentations and responded to questions, and on Friday 25 March, the Scan Team reviewed and summarized the information provided over the previous four days.

Analysis of Findings

The information gleaned from the amplifying questions and the workshop presentations was then organized and analyzed by the team members, headed by the Subject Matter Expert and the two Consultants. It was condensed into a Summary Report on 23 April 2018, and then presented in full in this report.

The information represents a snapshot in time of the best practices developed by, and the best evidence available to, the different agencies involved in the workshop. The outcomes are subject to variations of both geography and time. From one region to another, differences exist over the best practice for any given subject, and this is true at any time, including the present. Furthermore, with the passage of time, different solutions to the present problems will be developed, and different questions will become important.

CHAPTER 2

Scan Findings and Observations

The findings that follow are divided into four groups:

- Overarching Issues.
- Bridge Management.
- Bearings.
- Joints.

Overarching issues are presented in Section 2.1, followed by bridge management issues in Section 2.2, bearings in Section 2.3, then joints in Section 2.4.

2.1 Overarching Issues

Several issues arose that affect both joints and bearings and that are not associated exclusively with technical details of either. These are presented in this section.

Funding. This was widely seen as a problem, both in the absolute and in particular for maintenance, rehabilitation and repair. The latter is a consequence of the fact that funding agencies such as state legislatures favor the high visibility of new projects. It is also affected by the trend towards project delivery by design-build, in which design responsibility, and the associated funding, is awarded by legislative mandate to the design-build team rather than to the DOT (e.g., in Washington State). As budgets become more constrained, it is important not to overlook maintenance of joints and bearings, in view of the significant effect that they have on the performance of a bridge, and therefore on its life-cycle costs.

Attraction and Retention of Staff. Many DOTs reported difficulties with staff retention, which is caused in part by the fact that private consultants are able to offer more attractive salary packages and more glamorous projects to design. These matters are closely related to the funding questions. While joints and bearings may appear simple, and they represent only a small fraction of the initial bridge cost, their behavior can be quite complex and they need the attention of experienced and technically competent staff.

Training and Transfer of Knowledge. The difficulties in retaining and hiring staff can create gaps in the professional hierarchy. When a senior engineer leaves, through retirement or a move to another employer, his or her knowledge is lost. While it is possible to encapsulate some of that professional wisdom in written documents or training videos, much is inevitably lost. The gaps caused by hiring freezes in previous economic downturns tend to remain. While this is true for all bridge elements, it is particularly true for joints and bearings, given the many types in use today and the continual evolution of new ones. Capturing the knowledge of experienced bridge engineers and maintenance personnel is vital for successful performance.

Design Responsibility and Incentives for Innovation. Innovation requires expenditure of risk capital, which constitutes a disincentive to innovation unless a concomitant reward is available. For most bearings, the AASHTO LRFD Specifications contain detailed design procedures, so manufacturers see little opportunity for innovation. The situation is somewhat different for joints, which are less closely specified by AASHTO. A plethora of different joints and materials have been developed, but no centralized system of testing and evaluation exists, so most DOTs face difficulties in determining which type will work best in a given situation. Innovation would be fostered by using a system more like the European one, where specifications are based on performance rather than prescription, and a single agency is responsible for nation-wide testing and approval of new systems.

Impediments to Clarity in Specifying and Bidding. Bearing and joint suppliers note that requirements for their products vary across states. This makes it difficult for suppliers to understand and satisfy the requirements for getting onto a state's approved supplier list (ASL).

Automated Monitoring Devices. A suggestion was made that monitoring systems should be installed with the bearings, at least in new bridges. Structural Health Monitoring has started to be used in several fields, including bridge engineering (Ko and Ni, 2005), as part of an overall asset management system. Bearings are particularly critical elements, and are therefore good candidates for monitoring. The development of high-performance, low-cost electronics in the past decades has resulted in the increasing availability of monitoring systems, and now permits a wide range of choices for automation. However, health monitoring systems are not yet widely deployed for bridges in the US.

2.2 Bridge Management

Four bridge management topics were identified as priority to DOT's. These are listed below and discussed in this section.

- Funding.
- Policies.
- Activities.
- Tools.

2.2.1 Funding

Transportation infrastructure is essential for the conduct of business, and funding is essential to keep it in good operating condition. The total amount of funding available varies from state to state, and is affected by local decision making. A gas tax is a major source of revenue in most states. Furthermore, the allocation of funds to different activities is mandated by the legislature in some states, but the extent of that control varies.

This section discusses funding sources, amounts, allocation and methods of contracting.

Sources. Gas taxes are responsible for a large portion of revenue (Utah, Pennsylvania). Pennsylvania used a combination of 58.7 cent per gallon in combination with a P3 Program to reduce bridge deficiencies by 50%, defined as NBI Condition Rating 4 or below (FHWA 1995). This was achieved by focusing on the need for rapid replacement, and the case for it was built on the previously high number of deficient bridges. County budgets contain a small amount of money for performing bridge maintenance and repair, and metrics are established to measure that a certain percentage of funding is directed to structures. However, for most areas of the state, the majority of funding for maintenance and repair comes from the capital funding program that is overseen by regional and local planning organizations

Utah's recent gas tax increase of $0.28 increased the DOT's budget by 150%, and indexing it to inflation provided a further measure of stability for the future.

State Capitol Programs also provide money for maintenance (Michigan, Pennsylvania). Michigan's State Capitol Program designates an annual percentage of the construction budget to bridge maintenance activities such as deck patching, thin polymer overlays and other various minor repairs. For that program, each region has a different strategy as determined by central office Bridge Management Unit.

In Minnesota, the state legislature determines the amounts to be spent in the broad categories of construction vs maintenance, but the Districts are given considerable control after that.

In Oregon, state funds are the primary funding source for maintenance and smaller repairs. However, long-term underfunding has caused difficulties in addressing them all in a timely fashion. Large bridge rehabilitations are generally federally funded through the State Transportation Improvement Plan (STIP). Larger maintenance projects (gland replacement, minor header repair, bearing replacements/rehab) are funded through Oregon's state funded Major Bridge Maintenance Program (MBM). Joint and bearing cleaning activities are funded at the crew level. Regardless of the funding source, there is no set amount dedicated for joint and bearing repairs; these projects must compete against other bridge needs throughout the state.

Washington relies on "Special funding" for annual cleaning of steel truss bridges and funding for strategic bridge preservation, focusing on replacing and sealing joints and decks.

Amounts. Straight budget comparisons are not easily made, as some states report budget for the department as a whole, some for maintenance only. The highest budget reported, by far, is Florida, reporting an annual budget of $10 billion for all operations. Not surprisingly, Florida indicates no trouble maintaining its bridges in good condition. Pennsylvania's 2016-17 total Highway and Bridge budget was $4.7 billion. Utah reported a budget of $48 million, while Michigan's Structure Maintenance Support Unit has an annual budget of $2 million that is exclusively used for the purchase of bridge maintenance materials, specialized equipment and specialized services. Louisiana's statewide section materials unit budget is $1.5 million, and total annual maintenance budget is $60 thousand per district. Washington State has funding for about half of its bridge preservation and maintenance needs.

Allocations. Funds are typically distributed among Replacement, Rehabilitation/Preservation, and Preventive Maintenance. Maintenance needs consume anywhere from 25% (Michigan and Texas) to 30-40% (Utah) with the remainder of funds allocated in varying percentages toward replacement and rehabilitation.

In Florida, each district is allocated funds based on its structural inventory, past needs, and predicted future rehabilitation/repair projects. Each district is required to update the Bridge Work Plan each year to prioritize maintenance and ensure the activities are captured.

Funding needs in Pennsylvania are tracked separately by each of the 11 districts within the state. The state is comprised of 25 different regional planning organizations that receive specific annual funding allocations. Every two years, the Transportation Improvement Plan (TIP) is updated and the districts, in collaboration with the local planning partners, establish spending priorities. Maintenance and repair needs are prioritized along with major rehabilitation and replacement needs. Each district/planning region is responsible for deciding how best to allocate resources from a cost-benefit perspective.

Washington has no formalized distribution of funds but the majority of bridge maintenance funding goes to corrective maintenance.

Contracting Methods. Florida contracts out most of its inspection and maintenance using different forms of contracting. The state performs safety improvements or repairs small enough not to require a detailed plan set using either in–house forces or “Push Button” contracts. A push-button contract is a contract for pre-defined pay items that are small and simple enough to require no design, and on which contractors bid ahead of time from a list advertised by FDOT. The contracts provide a way of issuing work orders quickly and simply, at a previously bid and agreed price, when the need for the work arises. The Florida DOT District 7 worked with the FHWA Florida Division to develop Push-button contracts which simplify the process for using Federal safety funds thus expediting delivery of safety improvement projects.

“Asset Maintenance” contracts are used for larger corridors or areas, and conventional contracts are used for specific works on major projects. Asset Maintenance contracts oblige the contractor to maintain the system to a particular condition for a given price over a given number of years, with deductions provided if performance criteria are not met. Florida reports that the system works well.

2.2.2 Bridge Management Policies

This section discusses the management policies employed by responding DOT’s. Included are such topics as inspection frequency, cleaning frequency, seasonal restrictions, prohibited elements, new construction details and maintenance personnel (in-house vs contracted).

Inspection Frequency. Policies for conducting inspection and maintenance vary among states. All states reported that components are inspected on a two-year cycle per FHWA requirements. Increased inspection frequencies could be initiated for bearings or joints with performance issues (e.g., leaning rocker bearing). Two states (Oregon and Pennsylvania) have a policy for replacing older joint and bearing types that have been found in principle to perform poorly, even though the particular devices have not shown problems (e.g., steel rocker bearings in seismic regions.)

Cleaning Frequency. Pennsylvania reported that expansion joints are to be cleaned annually while all other responding states state indicate cleaning frequency is “highly encouraged” (Michigan), but not defined explicitly. Bridge joint cleaning takes place every year in Oregon, but not every joint is cleaned every year.. Florida has no fixed cleaning schedule, and reports that only joints with history of significant debris build up are cleaned frequently.

Seasonal Restrictions. States have limitations on when bearing and expansion joints maintenance can occur. Oregon only allows cleaning of bearings and joints between November 1st and March 15th, when waterways are actively flowing, in order to minimize negative impacts to water quality. Oregon and Pennsylvania conduct joint repair work in snow zones between April 1 and November 1 .Other states are limited largely by times and locations of high traffic volume (Florida, Louisiana, Utah, Texas and Washington).

Prohibited Elements. Steel rocker bearings are not allowed on new bridges due to poor performance (Louisiana, Pennsylvania, Utah). Replacement of rocker bearings on existing bridges is typically delayed until an actual performance issue existed and it was no longer feasible to reset or retrofit (Pennsylvania).

Some states (Florida) identified manufacturer-specific seals that they avoid.

Pot bearings are allowed but not preferred due to maintenance and inspection challenges (Louisiana). They are explicitly disallowed in other states (e.g., Washington).

Louisiana does not permit neoprene compression seals for expansion joints on structures having an average traffic volume greater than 1,000 vehicle per day or average truck traffic volume greater than 100 per day due to historically poor performance. Current practice in Pennsylvania is to only use compression seals to replace failures as a short-term solution.

Cotton duck and fiberglass reinforced elastomeric bearing pads are explicitly excluded from use in Pennsylvania’s design manual.

Texas avoids modular joints, sliding plate joints, angled strip seal joints and finger joints due to past fatigue failures of welds causing plates to pop up and cause traffic hazards.

Jointless Bridges. States also had policies for construction types. Many states reported that leaking joints were one of their biggest maintenance problems and were consequently moving towards the use of jointless bridges, in which the superstructure is continuous over the piers and all longitudinal movement is taken at joints located beyond the abutment. This can be done by implementing link slabs (Louisiana), semi-integral abutments (Michigan, Washington State) or integral abutments (Oregon, Pennsylvania).

The Oregon Bridge Design and Drafting Manual (BDDM) specifies the follow criteria for integral abutments: 1) bent is founded on steel pipe or H piles. 2) bedrock is 12 feet minimum from the bottom of pile cap. 3) there is negligible likelihood potential of settlement. 4) radius of horizontal curve is greater than 1200 feet. 5) skew angle is less than 30 degrees.

Michigan avoids using Integral abutments at stream crossings due to scour considerations. Texas and Florida report not using integral piers or abutments as a standard practice. Louisiana built two pilot bridges with integral abutments, one for soft soil and one for stiff soil. Performance of these two bridges is monitored by a Louisiana Transportation Research Center (LTRC) research project but the design has not been implemented for statewide use.

Virginia had perhaps the most aggressive policy with respect to jointless bridges, requiring, since 2001, special permission to include a joint in a bridge. The state gives preference to the special "Virginia Abutment" and "Virginia Pier", which together avoid joints at or between the abutments. In a further effort to reduce the problems caused by leaking joints, the state also assigns joints and girder ends as specific inspection entities in the Bridge Management Systems to improve modeling of deterioration.

Skew. States report using circular bearing pads when skew is between 45 and 60 degrees (Texas, Louisiana, Florida). Other states require special approval above 45 degrees (Michigan) while others try to eliminate skew. Pennsylvania requires multi-rotational bearings for skew angles between 20-70 degrees. States report poor performance for strip seals at high skews and limit their use between 45-60 degrees (Louisiana, Oregon) while others do not provide specific limits on skew angle, but rather require specialist consultation (Pennsylvania, Texas, Washington). Oregon specifically avoids strip seal joints for replacement on a skew angle that matches the angle of their snow plow. Louisiana and Texas do not include skew in their bridge inventory.

2.2.3 Bridge Management Trends

Trends reported by DOTs as changing the face of conducting business are: retirement of senior staff or migration of experienced staff before adequate knowledge transfer occurs, outsourcing of maintenance, shift of preferential materials for bridge elements, field - office communication, construction methods and project delivery. These trends influence many bridge elements, including joints and bearings.

Workforce Turnover and Maintenance Outsourcing. Many DOTs reported that keeping experienced in-house staff members was becoming more difficult, and that they were concerned that the gradual loss of senior staff was hindering the transfer of knowledge to younger, less experienced members. However, the measures taken to counter those trends varied widely.

Minnesota was one of the most pro-active states, using outreach to attract new staff and offering extensive training to both professional staff and field crews to hone skills and disseminate their knowledge base. The state has also invested in rigorous record-keeping efforts, the better to keep track of maintenance needs and to execute repairs before they become too serious. The Bridge Office reviews all shop drawings in-house and is now considering the use of electronic submission and approval for them. They also meet bi-monthly to discuss potential changes to standards.

Many states use in-house maintenance resources exclusively with the possible exception of major emergency work (Louisiana, Minnesota, Michigan, Oregon, Utah, Washington). Other states use maintenance contracts (Utah for bearing cleaning, Texas) while others use various combinations of Asset Maintenance contractors (A.M. contractors), in-house forces, push button contracts, and standard design/bid/build contracts (Florida and Pennsylvania). Some deck sealing or thin bonded deck overlays are bundled and contracted in specific projects (Oregon).

Florida and Texas both do not have in-house bridge maintenance personnel. The outsourcing appears to be working well at present, but questions remain over its sustainability, because the knowledge drain will make effective oversight difficult in the future.

Element Materials. The physical nature of bridges has changed over the years, as has the way they are built, and the changes have caused the need for continuous updating of knowledge. For short-to-medium span bridges, the girder of choice in most states today is a precast, prestressed concrete girder. Benefits of precast girders include economy of initial cost and low maintenance requirements in service. Elastomeric bearings also work well with prestressed girders and they, too, are economical and virtually maintenance free. Some midwest states still make wide use of steel girders (Louisiana, Michigan, Pennsylvania) and California is still the home of the cast-in-place post-tensioned box, but prestressed girders have captured a large portion of the market.

Field - Office Communication. Louisiana has plan review meetings with districts in various project development stages. They also use inspection reports, maintain general communication with project engineer (who manages the construction), holds project construction progress meetings, conducts field visits and lesson learned sessions at end of project (for critical and large projects only). Michigan has a position designated as the Statewide Bridge Construction Engineer. This position is responsible for coordinating all construction problems and bringing discrepancies to the attention of the designers. Additionally, Michigan has a Statewide Bridge Committee that meets monthly and has representatives from Bridge Design, Bridge Standards and Bridge Construction.

Oregon has an annual bridge design conference (open to the bridge community) and annual bridge designer exchange (in-house meeting). When significant problems arise, a presentation is made and the issues are discussed in Q&A session at both events. Designers have access to Bridge Inspection Reports that list any deficiencies. Designers accompany bridge maintenance leads or bridge inspectors on field visits during the design phase or when significant concerns are raised. In-house inspection staff performs field quality reviews on a percentage of bridge inspections. The inspection staff manager manages design standards staff which facilitates getting field issues resolved by the design standards staff.

Pennsylvania documents In-service performance via standard forms completed as part of routine NBIS inspections. These forms containing performance observations are made available to designers. Additionally, many district offices in the state conduct after-action reviews upon completion of each construction project. Construction personnel complete a standard form documenting things that worked well and things that could use improvement. If there were any installation or plans presentation issues regarding joints or bearings, such information would get documented.

Texas and Florida report no formal feedback mechanism although Florida engineers discuss issues at construction progress meetings; if a recurring issue arises, it is elevated for discussion. Some Florida districts use “Pass the Torch”, “Extinguish the Torch” meetings to pass along this information.

Washington has a Bridge Technical Advisor (BTA) program. The bridge designer is generally assigned as BTA during construction. The protocol is to direct issues / questions of a structural nature from the Project Engineer’s Office to the Headquarters Construction Office and then to the assigned BTA.

Construction Methods. Accelerated Bridge Construction popularity continues to grow as a solution to the problem of building bridges in heavy traffic areas. Utah has led the way in this regard, and ABC is now the new normal in the state. There, engineers have used SPMTs (Self-Propelled Modular Transporters) or lateral slides when conditions favor them, and precast elements in others. The state’s outreach efforts have been successful and have persuaded the public that the slight increase in construction costs is easily outweighed by the savings in business opportunity costs achieved by the shorter construction time. The ABC premium has also dropped as contractors gain experience with the technique. This process has been aided by the requirement that consultants follow up on ABC projects to record and disseminate the lessons learned.

2.2.4 Tools

The scan team sought to document tools that DOTs used in design, management and inspection of their assets. This section summarizes the design standards, standard details, inventory categories, software and inspection technology used by reporting agencies.

Design Standards and Standard Details. Responding DOTs cited AASHTO LRFD Bridge Design Specifications, AASHTO LRFD Bridge Construction Specifications as well as their state-based design manuals. Some specifically mentioned Method A (Texas) or Method B (Florida) for design of elastomeric bearings. Each state also reported developing their own standard details for bearings and expansion joints.

Most DOTs have manuals, standard plans and details, etc., on their websites. URLs for the agencies presenting at this workshop are listed below. This offers a convenient way of keeping such information up to date and provides one way of archiving knowledge that might otherwise be lost. Minnesota offers a particularly wide variety of information, including their Structure Information Management System (SIMS) and several manuals (e.g., inspection manuals), that are supplemented by others developed by FHWA (e.g., inspection and maintenance manuals). FHWA also offers training courses through the National Highway Institute (NHI) on a variety of topics, including maintenance.

FL www.fdot.gov

LA www.dotd.la.gov

MI https://www.michigan.gov/mdot/

MN http://www.dot.state.mn.us/bridge/index.html

NY https://www.dot.ny.gov

OR https://www.oregon.gov/ODOT

PA https://www.penndot.gov/Pages/default.aspx

TX https://www.txdot.gov/

VA www.virginiadot.org/

WA https://www.wsdot.wa.gov/

Inventory Categories. Louisiana catalogs seven types of joints and seven types of bearings, according to the Manual for Bridge Element Inspection. Michigan has eight joint types, seven bearing types and categorizes according to their own Michigan Element Inspection Manual. Florida, Oregon and Utah recognize 6 joint and 6 bearing types but Oregon notes they do not record manufacturer information. Pennsylvania recognizes 18 joint and 21 bearing types.

Software. Most participating states indicated they have their own proprietary software for condition and inspection tracking (Michigan, Pennsylvania, Washington). Florida, Oregon, Utah and Louisiana all cited AASHTOWare Bridge Management System (BrM, formerly PONTIS). Louisiana also cited INSPECTTECH.

Inspection Technology. Louisiana and Michigan inspectors use iPads (tablets) and enter inspection reports in real time. Pennsylvania has an internally developed electronic form software called "iForm" that is transferred to their permanent database after collection. Utah, Louisiana and Florida each note being in the exploratory phase of using drones for hard to access locations. Under-bridge inspection trucks (UBIT), scissor lifts, man lifts, suspended staging, traffic control are still standard inspection methods.

2.3 Bearings

Design, Construction and Maintenance are normally carried out by separate divisions within a Department of Transportation. Consequently, the discussion of joints and bearings are subdivided here according to those disciplines. In each case, performance is addressed in the section on design.

2.3.1 Design of Bearings

2.3.1.1 General

Most of the states in attendance have an in-house engineer who acts as the bearing specialist, although in most cases the individual has other duties as well. This arrangement offers benefits through familiarity of the subject and continuity of specialized understanding. The individual can also act as a repository of knowledge about what works and what does not, in the environment of the particular state. The assignment of one individual to specialize in joints and bearings also improves communications with manufacturers and contractors, and reduces the chances of misunderstanding and mistakes.

Some states (e.g., Minnesota, Louisiana) have a table of standard elastomeric bearing sizes and guidance for selecting them. Standardization reduces the chances of error in bearing selection, and is also helpful because different designers might otherwise develop different bearing sizes for the same girder application. Girders are fabricated in standard sizes, each of which is suited to a limited range of spans, so each girder size has a load and longitudinal movement that vary little from one application to the next. Standard bearing sizes are thus possible. This also helps manufacturers of elastomeric bearings, who can take advantage of standard sized molds. This is most important in the case of circular bearings, because a rectangular mold can be adjusted relatively easily to fabricate a smaller bearing through the use of packing plates. However, circular bearings are not commonly used, being restricted largely to bridges that are curved or heavily skewed.

Bearings may be bid and paid for as separate items, or as a package with the girders. The latter was common in the last century, when most bridges and bearings were made of steel, and the same fabricator would make both. Today, the wide use of concrete girders and a variety of bearing types means that the girder and bearing will likely be fabricated by different companies. The advantages of a combined bid then diminish. For most of the states, bearings other than elastomeric pads are separate bid items, which allows their true cost to be identified. Elastomeric pads, which are typically lower cost items, are bid in ways that vary from state to state.

The lead time needed for purchase should be accounted for at design if the construction schedule is likely to be tight. Typical times are in the range 4-5 weeks. While lead time is strictly the contractor's problem and not the bridge owner's, planning for it can still help to avoid problems.

Several manufacturers expressed frustration at the large number, and variation, of state requirements for joints and bearings. The situation in the US was contrasted with that in Europe, where approval is centralized, and test requirements are established by a central authority (EOTA) that has the capability for developing and evaluating test procedures. Such a system would help not only manufacturers, but would also reduce the probability of misunderstandings, and so help states as well. However, it represents a significant departure from the present system of state control.

Record-keeping varied widely from state to state. Minnesota, for example, (SIMS) keeps detailed records of joints and bearings, including modifications and repairs. Such records provide a body of knowledge about the strengths and weaknesses of the systems installed, which can inform future choices. They also facilitate emergency repairs.

2.3.1.2 Bridge System Design

Bearings represent a small part of the first cost of the bridge, but failure of a bearing has the potential to cause repair costs far out of proportion to its capital value. (Even a high load, multi-rotational bearing typically costs in the range $5,000 to $10,000, which is much less than, say, a girder). The difference is largely associated with the opportunity costs of bridge closure or traffic control, and the costs of lifting the bridge. Consequently, the bridge and bearings should be thought of a system, all parts of which should be designed to work together.

Provision should be made for replacing bearings. Many, but not all, states already do this. At a minimum, jacking points should be provided, but other details, such as designing anchor bolts so that they and the bearing can be removed easily and replaced without the need for excessive lift height, is important. Some states (e.g., Florida) do not provide jacking points for bridges supported by elastomeric bearings, on the basis that the bearings have a very low failure rate, and that provision of specific jacking points is not useful because, by the time jacking is needed, the jacking plans have usually changed from those envisioned during the original construction. Pennsylvania has found that provision of jacking points under pier diaphragms in concrete bridges requires space that may not be readily available, and the state has developed a procedure whereby through-holes are installed in the cap beam by which temporary lifting brackets can be attached when needed (see Figure 2.3-36). Bearing replacement is addressed in detail in Section 2.3.3. In special cases, such as at in-span expansion hinges, access to the bearing location is inherently so poor that providing jacking points is difficult.

In unusual cases the bearing might even need to be re-located within the bridge. Minnesota reported a case in which the bridge was constructed on fat clays that were subject to large movements. The movement was too large to be accommodated within the bearing displacement capacity, so additional bearing stiffeners were provided in the steel girders that allowed the bearing to be moved along the girder to the new location, without risking local damage at the concentrated load, as the soil and columns moved.

The loads on bearings are influenced by the relative stiffnesses of the bridge system components. For example, California does not allow guided sliding bearings, but rather requires a separate shear key system to guide the direction of sliding. The lateral loads are then taken by the shear key and not the bearing, and the risk of local distortion, or even overturning, of the bearing is reduced. All other states reported using guided bearings.

The load distribution among bearings at a support line depends, at least in part, on the superstructure properties. If it is stiff in torsion, and particularly if the bridge is curved or skewed, the effects of temperature or post-tensioning may cause the superstructure to twist and the bearing loads to be distributed unevenly. Oregon experienced this problem, as described in Section 2.4.2.10. A similar problem arises with slab beams. In them, Texas supports one end on two bearings and the other end on one bearing. This three-point support is statically determinate and leads to equal load on each of the two bearings in the pair. Pennsylvania also reported a related problem with box beams, resting on two bearings each end, aligned with the support. One bearing of the pair lifted off and the other was overloaded. To equalize the loads, it was necessary to cut down the concrete beam seat to lower one bearing. The lesson was that, for such statically indeterminate systems, it is better to avoid skew if possible (i.e., for new bridges), and to analyze the rotations carefully for others.

Last, the bearings in skewed or curved bridges should be oriented to accommodate the imposed rotations These may be due to traffic and other effects, such as thermal or post-tensioning. However, the separate sources of rotation may not cause rotations about the same axis, so, in such cases the orientation of the largest rotation may not be clear and it is advisable to use a bearing type with no preferential rotation axis such as a HLMR (High Load Multi-Rotational) or circular elastomeric. The team discussed at some length the ideal orientation for rectangular bearings, but no consensus was reached between setting the bearing axis perpendicular to the girder axis (the most common choice) or parallel to the support line. Pennsylvania reported using the former for steel multi-girder bridges and the latter for prestressed concrete girders. Michigan also orients the bearing parallel to the support for box beams, and perpendicular to the girder axis for others. In practice the ideal orientation is likely to depend on the ratio between the torsional stiffness of the girder and the bending stiffness of the deck and end diaphragm.

Bearings and joints are usually ordered by the General Contractor from the supplier of his choice. Large bearings may require significant lead time, and this can lead to problems. Minnesota reported pressure from the contractor to approve the bearings faster than usual and are considering switching to electronic transfer of drawings to speed the process. A further caveat exists for HLMR bearings that are designed by the supplier; the Engineer of Record typically shows approximate dimensions on the plans to ensure that sufficient space exists, but it is necessary to verify that it does, once the real sizes become available.

2.3.1.3 Movements and Forces

The movements and forces for which a bearing must be designed depend on the meteorological environment, and that varies significantly among states. Florida is largely hot and humid, and experiences hurricanes but no earthquakes, whereas Washington has two different climate regions (coastal and inland) and suffers from earthquakes but not hurricanes.

Design movements of the superstructure are largely dictated by the AASHTO LRFD Specifications (AASHTO, 2017), which provide temperature ranges for all locations in the US for overall expansion and contraction, plus thermal gradients that control camber. For many regions of the US, the total thermal movement (summer to winter) is about 1 inch per 100ft of span. AASHTO recommends using 65% of this in each direction, to allow for a setting temperature that is not exactly mid-range. However, some states require more stringent design movements than AASHTO; California designs for one-way movements corresponding to 75% of the total specified range of temperature, and Utah uses 80%. The design setting temperature also varies from state to state (e.g., 45 degrees in Minnesota and 70 degrees in Florida). Creep, shrinkage and post-tensioning (if any) all cause shortening of the superstructure, but their values depend strongly on the type of construction.

The foregoing movements are those of the centroid of the girders. In addition, the bottom flange moves longitudinally when the girder end rotates.

Uplift effects on the bearing have the potential for introducing major problems. They usually occur when adjacent spans have significantly different lengths. If the bearings are held down, tension forces are induced between the bearing and supporting structure, and they can cause any sliding elements to bind, or bolts to break, especially if prying action occurs. If the superstructure is free to lift off the bearings (possible with elastomeric bearings) it may move horizontally relative to the bearing and not return to the intended location on the bearing. In the Kesen Bridge in Japan, the bearings failed in tension (Figure 2.3-1) when a tsunami lifted the bridge.

Figure 2.3 1. Kesen Bridge, Japan, 2011

California does not allow bearings to be designed to resist uplift, and Utah does not use HLMR bearings when vertical seismic accelerations are significant. Two of the manufacturers advised strongly against designing to resist uplift forces. The team members agreed that the best solution, if possible, is to re-configure the spans so that uplift does not occur. If that is not possible, tie-down elements, separate from the bearing, are preferable to carrying the tension in the bearing itself, although they are likely to complicate bearing replacement, should it be necessary.

The ends of the girders will rotate under live load, especially in the case of simple spans. The AASHTO mid-span deflection limit of L/800 corresponds (PCI, 2014) to an end rotation of approximately 0.004 radians (0.4%). Most bearing types can easily accommodate a rotation this small. However, other causes of end slope, such as camber or grade, add significantly to the total. In prestressed concrete girders, the camber due to elastic effects plus shrinkage, creep and prestress loss are likely to be both more difficult to estimate, and larger than, the end rotation due to live load. If the rotation is permanent, it may be corrected by means of a beveled sole plate (e.g., Utah) or an inclined recess in the bottom flange of the (concrete) girder. Of course, the inclination must be oriented correctly, or it will make the problem worse rather than better (Figure 2.3-2).

Most states have a threshold for installing such corrective measures. Utah uses tapered plates only when the final end slope is greater than 1%. Texas uses tapered elastomeric bearings for grades up to 5%, although the AASHTO Specifications do not allow them. The state justifies the choice on the basis that fabricating tapered bearings is cheaper than machining a steel bevel plate, and reports no problems. Florida finishes the seat to be parallel to the final girder slope for slopes between 0.5% and 2%, and uses level seats and beveled plates at steeper slopes. Note that the sloped seat implies a permanent shear force in, and consequent shear deformation of, the bearing.

Figure 2.3-2. Girder with reversed bearing recess

Anchor bolts are usually designed prescriptively, based on historical practice, because the loading on them cannot be defined easily; in most cases friction alone may be sufficient to prevent movement, and accordingly many elastomeric bearings are designed and installed without any positive anchorage. Some states (e.g., Florida) use a nominal value for the horizontal design load on the anchor bolt group, such as 10% of the downwards vertical load, and provide an equation for the bolt size in their Structures Design Guidelines (Florida DOT, 2018).

2.3.1.4 Bearing Type Selection

The AASHTO LRFD Design Specifications contain design procedures for ten bearing types:

- Steel rocker bearings,
- Steel roller bearings,
- Curved bronze sliding bearings,
- Pot bearings,
- Disc bearings,
- PTFE sliding surfaces,
- Plain elastomeric pads,
- Cotton duck reinforced elastomeric pads,
- Glass fiber reinforced elastomeric pads and
- Steel reinforced elastomeric bearings

Of these, steel rocker and roller bearings are almost never specified for new bridges, because they have a history of poor performance; they tend to corrode and lock up, or topple over. None of the attending states specified them for new bridges, and some outright banned them, However, many steel bearings exist in older bridges, and they need to be maintained or replaced. Occasionally a steel rocker bearing in a bridge will be replaced by another similar bearing, in a like-for-like exchange, if it is the only one being replaced. If many are being replaced, they will normally be replaced by a different type, such as steel reinforced elastomeric. The aggressiveness with which they are replaced depends on the dangers that they present and is greatest in states that experience more frequent earthquakes, such as those on the West Coast. Some states (e.g., New York) have a specific program to replace all high rocker bearings by a particular date.

Similar findings hold true for curved bronze bearings. Bronze is weaker than steel and has a tendency to crack if it is loaded along a line, rather than uniformly over a surface. Utah reported replacing the bronze sliding element in one bridge where the original bronze was damaged, and Pennsylvania reported that they were open to replacing worn bronze bearings with new ones.

Of the elastomeric types, Fiberglass reinforced pads are no longer available commercially, so they too are not specified for new bridges. A few are believed to remain in existing bridges. Kirkhill Rubber developed them and was the only company to manufacture them, but went out of business in 1997. They could be made in large sheets, stocked, and cut to the desired size when a customer ordered them, which potentially speeded up delivery. However, they could carry less compressive stress than could a comparable steel-reinforced elastomeric bearing and never commanded a large market share. Pennsylvania explicitly disallows their use, as it also does for random fiber pads and cotton duck pads.

The great majority of bridge bearings used today are either Steel-Reinforced Elastomeric Bearings (SREBs) or High-Load Multi-Rotational (HLMR) Bearings, such as pot, disc or spherical sliding bearings. Elastomeric bearings are the most widely used; they are favored because they have sufficient load capacity for most common situations, have a low first cost, need essentially no maintenance, have good resistance to corrosion because they are encapsulated by rubber, and are relatively forgiving of overstress. The AASHTO specifications allow them to carry compressive stresses up to a maximum of nearly 2 ksi, depending on the accompanying shear and rotation demands. For higher loads, pot, disc and spherical bearings are used to accommodate the rotation, with a flat sliding surface as well, if needed, to accommodate horizontal movement. Flat sliding surfaces are also referred to as “sliders”.

Bearings are chosen on the basis of load and displacement capacity, but the ranges for the different bearing types overlap. For example, the elastomeric bearing shown in Figure 2.3-3 is one of the largest ever made and has a load capacity of approximately 4500 kips. For that load, an HLMR bearing would be the usual choice. Most states limit the height of a steel-reinforced elastomeric bearing, which implicitly limits its displacement capacity, and hence, the bridge span length. For example, Michigan routinely uses SREBs for spans up to 120 ft.

HLMR bearings are sometimes made with sliders on top, and their displacement capacity is then limited only by the space available for the stainless steel sheet. Thus, they are commonly used when large displacements are needed. However, an SREB can also be equipped with a sliding surface and would then have the same large displacement capacity as the HLMR bearing. New York is considering that combination. It is attractive, because the slider moves only after a displacement large enough to generate a shear stress in the elastomer that will overcome the friction. Consequently, small, frequent displacements, such as those due to traffic, are absorbed by shear deformation of the elastomer, while the larger displacements, due to seasonal thermal effects and concrete creep and shrinkage, are taken up by sliding. This arrangement minimizes the slide path of the PTFE, and so reduces the wear on it. This is useful because there is some evidence (Campbell and Kong, 1987) that, for a given total slide path, many small movements at a high sliding speed cause more wear than a few larger and slower ones. However, the thinner bearing that the slider permits is less able to accommodate rotation, and this should be borne in mind when designing the composite bearing.

Figure 2.3-3. Large elastomeric bearing

In steel bridges where the load is high and the flange is relatively narrow, the bearing dimensions may be so limited as to cause a compressive stress that is too high for an SREB. Then an HLMR bearing is the only real choice. In the past, HLMR bearings tended to be used in steel bridges, and SREBs in concrete bridges. That was a consequence of history, perhaps a descendent of the practice of a steel fabricator making both the girder and the rocker bearing, and was not driven by engineering needs. The widespread practice of using anchor bolts for steel-based bearings (e.g., HLMR) but not for SREBs, is similarly related to historical practice, not engineering reason.

The three HLMR types (pot, disc, spherical) share many properties but also have some differences. Pot and disc bearings both use the deformation of an elastomeric element to accommodate rotation, so they have a non-zero stiffness and their rotation is limited. That is important because the resisting moment may cause problems at flat PTFE-stainless steel interface on top of the bearing; too high a moment will cause lift-off on one side of the PTFE and excessive edge loading at the other. Spherical bearings depend on sliding, so their resistance does not increase with the applied rotation, once sliding has initiated. California uses them exclusively for HLMR applications for that reason, and machines the convex plate from solid stainless steel plate to avoid the potential for corrosion at a carbon-stainless steel interface.

Pot bearings suffered failures in several states in the early 1980s, soon after they were introduced into the USA from Europe, and this caused some states (e.g., Washington) to ban them. The pot also has to be machined from a single piece of plate. This requires thicker plate, and more machining, than required for a comparable disc bearing, thereby driving up costs. Consequently, disc bearings are gaining in popularity. However, pot bearings are widely used in Europe today, and problems are not widespread, either there or in the domestic states that still use them. The early failures were associated with poor seal performance; the seals squeezed out of the gap between piston and pot and allowed the elastomer to follow. However, many of those seals were made from PTFE, whereas brass is used in the USA today, and there is also evidence that some of the early pots and pistons were not well machined to match. Therefore, the causes of the early failures may no longer be applicable today.

Disc bearings consist of a circular polyurethane disc placed between two steel plates. The polyurethane disc behaves in much the same way as a plain elastomeric pad, in that it is prevented from squeezing out laterally by friction against the steel plates above and below. The vertical compressive stiffness and the rotation stiffness thus depend on the ratio of thickness to diameter, which is a compromise between the need for compressive stiffness and rotational flexibility. A central pin transmits horizontal shear force between the top and bottom plates.

Disc bearings are easier to inspect than pots, and they use less steel, so are typically more economical for the same load capacity. Thus, with time, they are becoming more common. This is in contrast to the European marketplace, where disc bearings are not sold and pots are viewed as being reliable and cost-effective. In the US, Minnesota is starting to avoid new pot bearings due to initial and maintenance costs, and they have banned steel rocker bearings, sliding plates and PTFE bearings. In most of the other states in attendance, pots are either not permitted at all or are given a lower priority than discs or spherical bearings.

2.3.1.5 Steel Rocker and Roller Bearings – Details

Steel bearings can accommodate horizontal movement and rotation where these are allowed (see Figure 2.3-4). Many configurations are possible, and the naming system is not unique. Here they are classified according to the behavior at the two contact surfaces. In a roller bearing, both contact interfaces (top and bottom) have the same diameter and form part of a common cylindrical surface. Movement is by rolling at both surfaces. Other bearings are referred to here as rocker bearings.

Figure 2.3-5 shows a single roller bearing. (In this case, the rolling element is a cylinder with the sides cut off.) Figure 2.3-6 shows a roller nest, in which several cylindrical roller elements are connected by a mechanism to maintain their alignment and spacing. In Figure 2.3-7 the rollers in the nest are functionally the same but are shaped differently. The guiding mechanism consists only of pintles. Figure 2.3-8 shows a roller nest that can no longer move because of the severe corrosion.

Figure 2.3-9 shows some movable high rocker bearings. The bottom surface rolls on the flat steel plate, while the pin at the top rotates by sliding. Figure 2.3-10 shows a high rocker bearing that has toppled over – a common mode of failure. Figure 2.3-11 shows a fixed high rocker bearing, sometimes referred to as a pin bearing, that allows rotation but not horizontal movement. Figure 2.3-12 shows a low rocker bearing. Because it is so low and has a flat top surface, it allows essentially no horizontal movement, but only rotation. A sliding plate under the rocking element (optional) allows horizontal movement.

The primary modes of failure of steel bearings are:

- Corrosion. This is particularly damaging in roller bearings. When they seize up, bridge expansion induces large forces, and probably damage, in the abutments.
- Toppling over. This primarily affects high rocker bearings, and typically leads to the bridge superstructure dropping, to leave a step in the roadway. Seismic motions are a common cause.

Many states, both those present and others, but particularly those traditionally associated with the steel industry, still have bridges with steel rocker and roller bearings. States subject to earthquakes (California, Oregon, Washington, New York) are generally replacing steel bearings as rapidly as resources allow. In others, and if there is no reason to anticipate toppling, replacement depends on the bearings' condition. If the bearings show little corrosion, periodic cleaning and lubrication may be more satisfactory and is cost-effective. The cleaning interval depends on the environmental conditions. For example, Minnesota attempts to clean and lubricate every four years. However, Oregon reports that about 80% of their bearings that are in Condition 2 or lower are steel bearings. Other states, including those without serious seismic hazard, are nonetheless replacing steel bearings to reduce their exposure to risk. In cases of high loads or restricted space, they are replaced with a HLMR bearing. Otherwise an SREB is the bearing of choice.

States have also instituted temporary measures while the steel bearings are waiting their turn for replacement. Based on a detail from FHWA's Seismic Retrofitting Manual (FHWA 2006), Pennsylvania has added small wedges to inhibit over-rotation of rocker bearings and stops to inhibit lateral displacement. The detail is reproduced in Figure 2.3-4. No feedback was available to determine its effectiveness because the bridge has not yet been subjected to seismic loading.

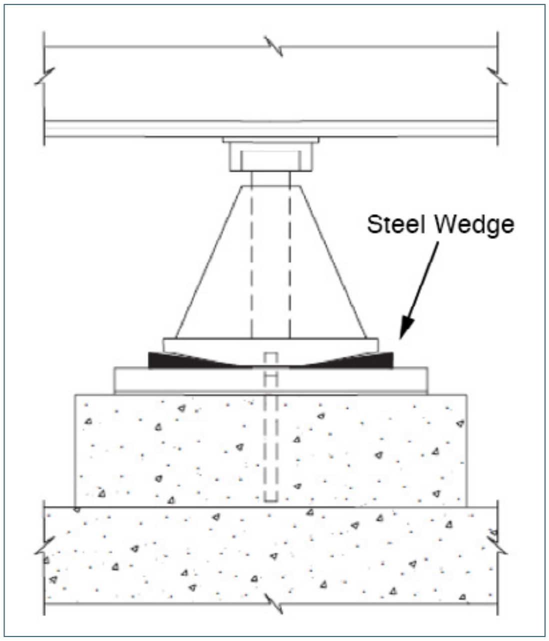

Figure 2.3-4 Steel wedge detail for rocker bearing (from FHWA).

Figure 2.3-5. Single roller bearing (OR)

Figure 2.3-6. Roller nest

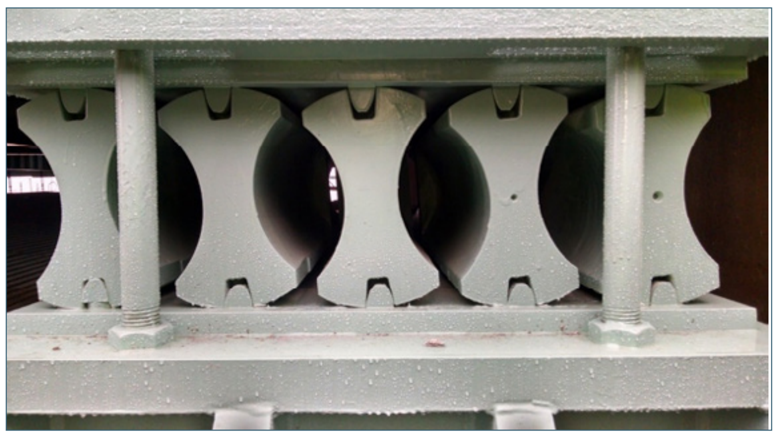

Figure 2.3-7. Roller nest – Alternative roller shape (PA)

Figure 2.3-8. Roller nest showing corrosion (CA)

Figure 2.3-9. High movable rocker bearings. (LA)

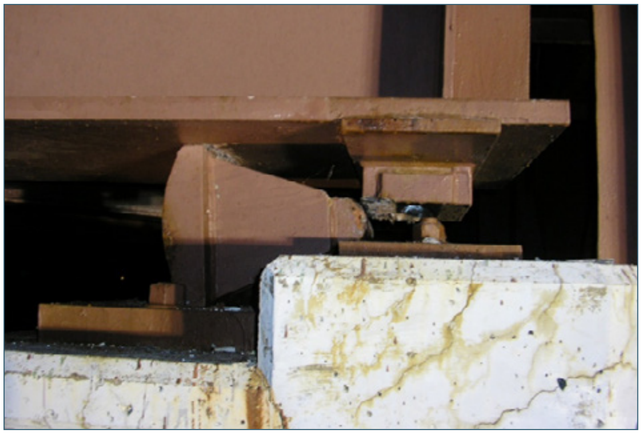

Figure 2.3-10. Movable high rocker bearing –Toppled. (NY)

Figure 2.3-11. Fixed high rocker (CA)

Figure 2.3-12. Low rocker (with sliding plate under)

2.3.1.6 Steel-Reinforced Elastomeric Bearings – Details

Elastomeric bearings are available in several forms, differentiated by the details of the internal reinforcement, which inhibits the outward movement of the elastomer caused by the application of vertical load.

Steel-reinforced elastomeric bearings (SREBs) (Figure 2.3-13) contain internal steel plates, or "shims". SREBs have the highest vertical load capacity and stiffness of all the elastomeric types, and are the bearing of choice for most prestressed concrete and many steel girder bridges. For example, California allows only SREBs, and not other elastomeric types, such as fiber-reinforced or plain pads.

Figure 2.3-13. Steel reinforced elastomeric bearing pad (shown cut open). (CA)

Several variants on the basic design have been used. For example, New York uses a fixed pin in the bearing (Figure 2.3-14) to prevent horizontal movement, but it appears to be only partially successful because the pins tend to bend. (Note also that the bearing in the figure has been poorly made, because the layer thicknesses are uneven).

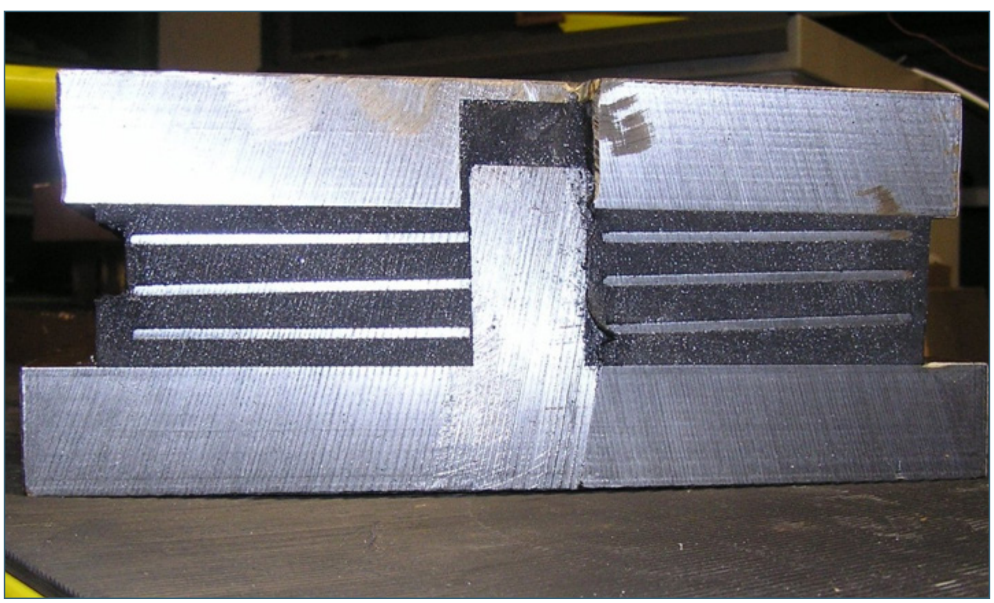

Figure 2.3-14. SREB with central pin. (NY)

Minnesota uses a special composite bearing (Figure 2.3 15), with a low steel rocker on top of an SREB for expansion bearings, and a cotton duck pad for fixed bearings. The low rocker provides the rotational function and the SREB provides for the horizontal displacement. Until recently, they used plain elastomeric pads at the fixed bearings, but those tended to squeeze out. The switch to cotton duck pads appears to have cured that problem.

Figure 2.3-15. SREB with rocker plate. (MN)

Some states, such as Florida and Louisiana, have adopted standard sizes for SREBs, with limiting combinations of span and load for each size. Australia has done the same for many years (Australian Standards, 2006). Because prestressed concrete girders are also produced in standard sizes (although the sizes may differ from one region to another) they provide a further incentive for standardization at least for the bearing width. Using the widest bearing consistent with the bottom flange width inhibits lateral rolling of the girder during erection.

The capacity of SREBs rose in 2008 when the new AASHTO Design Method B was introduced, but some participating states, and some manufacturers, nonetheless felt that SREBs' true capacity was still being under-utilized and that the allowable stress could be raised further. Others saw little advantage in so doing, because the minimum bearing size is often controlled by considerations other than stress, such as the need for a bearing as wide as the girder bottom flange. The use of more rigorous testing, associated with Method B design, to justify higher stresses was also questioned. The manufacturers also pointed out that it is not practical for them to run two production lines in parallel, one with higher QC for Method B bearings, and another, with lower QC, for bearings designed by Method A, so more rigorous testing is unlikely to lead to higher quality in bearings designed to Method B.

Many states, both those in attendance and others, provide guidance for bearing type selection and recommend the use of SREBs up to a certain total load level, span length, or both. In most cases this is in the range 400 to 500 kips, but Utah has a limit of 650 kips. For span length, Michigan provides an example, with SREBS recommended for spans up to 120 ft. These limits are really imposed by practicality; for a larger load, the bearing would become larger in plan, and for a longer span, the change in length (and so, the bearing thickness) increases. Either leads to problems both with fabrication and with finding the space to place the bearing on the pier cap.

Gravity loading and rotation cause compressive stresses that add on one side of the bearing, so in AASHTO's Design Method B the amplitudes of the compressive stress and rotation are linked. The design is analogous to, but slightly more complicated than, that for a steel column subjected to axial load and bending. The complications occur for two reasons. First, the rotational "loading" is represented by a rotation that must be accommodated rather than a moment that must be resisted. Thus the load combination consists of one force-based load (the gravity load) and one displacement-based load (the rotation). Second, the stresses in the elastomer are a function of the shape factor, S, of a typical interior layer, given by

$$S = \frac{\textit{plan area}}{\textit{area of perimeter free to bulge}}$$

Layers that are large in plan but thin are excellent for resisting compression, but bad at accommodating rotation, and vice versa. The selection of a suitable shape factor is thus a compromise that reflects the relative magnitudes of the compressive and rotation loading. While Utah recommends SREBs for loads up to 650 kips and rotations up to 4%, these two could not be accommodated simultaneously if the bearing were to be of practical proportions.

The fact that rotations induce significant stresses emphasizes the need to establish realistic values for them. In a number of the cases in which elastomeric bearings have split or bulged, the problem has been traceable to poor leveling during installation. The component caused by live load is relatively easy to predict, and is seldom larger than 0.004 radians (Stanton et al, 2007). The dead load rotation may involve a creep component, but the components due to camber and installation tolerances are the most difficult to predict and control. The AASHTO LRFD Design Specifications contain an allowance of 0.005 radians for installation. This is very small (about one tenth of a bubble on a standard builder's level), but no participating state reported having a procedure for checking that even the grout pad is leveled to this accuracy, much less the combined slope error due to camber and grout pad. Figure 2.3-16 suggests the specified tolerance is not being met. Either the allowance for inaccurate installation should be increased, or quality control over the rotations achieved on site should be tightened. Increasing the allowance would make SREBs more bulky and might rule out their use for some applications where they are commonly used today.

Figure 2.3-16. Over-rotation, possibly due to initial camber. Note uplift at front edge and bulging at back. (NY)

Many states have some upper limit on permissible elastomeric bearing thickness, with most reporting a 6” limit and none reporting bearings thicker than 8”. The limit is based on several considerations, including cost, bearing stability and the difficulty of manufacturing thick bearings, for which uniform curing of the rubber becomes more problematic. The total thickness of an SREB is largely controlled by the horizontal displacement capacity required and, to minimize thickness, some states add a PTFE slider. Post-tensioning of concrete bridges induces the greatest one-time displacement. California uses post-tensioned bridges extensively, and the state accommodates the movement by greasing the PTFE in the flat sliding surface. Other states, such as New York, are considering adding a slider. Some states also have special policies for elastomeric bearings in skewed bridges. For example, the bearing is placed with the long dimension parallel to the support line and the corner is clipped off to avoid the bearing’s projecting beyond the girder flange.

The primary problem reported for SREBs is walking out of place, although in some cases the bearings that walked appear to have been plain (unreinforced) pads. Texas funded a study (Muscarella and Yura, 1995) to investigate the problem and concluded that it was caused by the migration to the rubber surface of the anti-ozonant waxes used in Natural Rubber (NR) bearings. They solved the problem by banning NR bearings and allowing only chloroprene rubber (CR, or neoprene) bearings.

Contemporary SREBs use chemistry other than wax to achieve ozone resistance, so there is reason to believe that walking of NR bearings should be less of a problem than in the past. Some states, such as California, view the resistance to ozone or ultra-violet light as a cosmetic, surface problem that has essentially no influence on the performance of the bearing as a whole. In that case, reducing the anti-ozonant wax has no drawback.

Some states specify that SREBS be made only from neoprene (i.e., chloroprene rubber). This requirement appears to be a long-term consequence of a successful marketing campaign by the producer when elastomeric bearings were first introduced in the late 1950s. Apart from the anti-ozonant wax issue described above, there are several reasons for considering bearings made from Natural Rubber (NR). They include cost, material availability, and performance, especially at low temperatures. At the time of writing, NR is less than half the price of CR on a volume basis. The mechanical properties of NR (such as elongation at break) are superior to those of CR, as evidenced by the fact that elastomeric seismic isolation bearings, which are made to stricter standards, and need to sustain larger deformations, than conventional bridge bearings, are almost exclusively made from NR. NR is also used by most states that experience low temperatures, because bearing fabricators cannot meet AASHTO’s low temperature stiffness requirements with Neoprene.

A certain way of preventing the bearing from walking out of place is to vulcanize it to upper and lower steel plates, and to secure the plates to the supports with anchor bolts. See, for example, Figure 2.3-16. This adds some cost to the bearing, but ensures that the bearing remains in place. The steel plates need to be protected against corrosion before vulcanization. Protection by galvanization causes difficulties with the subsequent vulcanization, but metallization has been found to be a satisfactory solution. If the plates are to be fixed to the girder, for prestressed girders, this means embedding a steel plate in the bottom flange. Louisiana considered the use of a 1/8” recess in the bottom of the girder instead of direct attachment by vulcanized plates. However, they discussed the approach with staff from Florida and Minnesota, where the detail had been tried previously, determined that it was not effective because the bearing “climbs out” of the recess, and decided not to use it.

Other states have addressed the problem by requiring, during design, that the compressive stress never be less than 200 psi at any time, and that the friction resistance at all times be larger than the demand. Caltrans reported success using this approach.

The manufacturers pointed out that the ¼” cover commonly used, and the 1/8” occasionally used, for side cover is insufficient for use with thick elastomeric layers, which usually occur in large bearings. The thin cover leads to large local strains in the critical location, near the outer edge of the steel shims. They suggested that the cover should be related to the layer thickness.

2.3.1.7 Fiberglass-reinforced Elastomeric Pads

Fiberglass-reinforced elastomeric bearings are no longer made commercially for the bridge market, although research is being conducted to develop a fiberglass-reinforced elastomeric seismic isolation bearing (Toopchi-Nezhad et al, 2008). Such isolators have not yet been deployed in bridges.

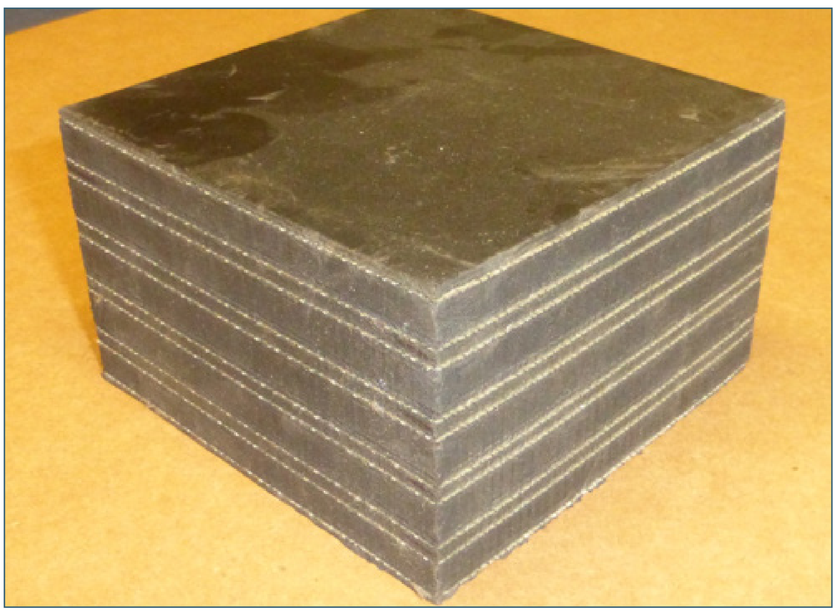

Figure 2.3-17. Fiberglass-reinforced bearing pad (CA)

2.3.1.8 Cotton Duck Pads – Details.

Cotton duck pads are made from rubber reinforced with closely spaced layers of cotton fabric. The layers are so thin (approximately 60 layers per inch) that the bearing has high strength, but it has almost no ability to deform in shear or rotation. Thus, it is useful either as a fixed bearing, or as an expansion bearing when equipped with a PTFE slider. They are used sparingly today, and the team heard no recommendations for change.

2.3.1.9 Plain Elastomeric Pads - Details

Plain pads consist solely of elastomer. They are seldom used as a primary bearing, because their vertical stiffness and strength depend on the friction at their upper and lower surfaces that inhibits lateral spreading. That friction has been found to be unpredictable, and the pads have been found, both in tests (Tetzlaff, 2013) and in the field, to squeeze out laterally under loads greater than 400 - 500 psi, regardless of the surface roughness. Nonetheless Utah still uses them for light loads. Some states also use plain elastomeric strips, rather than individual bearings, under the ends of voided slab bridges. There, they cannot be seen, so their performance is unknown.

Thin pads (e.g., 1/8") are sometimes used just to correct an uneven surface beneath another bearing type. For example, until recently, Minnesota used ½" plain pads below a steel plate that in turn supported a curved plate that accommodated rocking. The plain pads squeezed outwards, as shown in Figure 2.3-18. (In the figure, the pad is intermittently restrained against bulging by pins.) Figure 2.3-19 shows similar behavior in a test. The state now uses the same bearing configuration, but with thin cotton duck pads, instead of plain pads, under the plate.

The AASHTO LRFD Design Specifications allow compressive stresses on plain pads up to 800 psi. The consensus was that that stress is too high.

Figure 2.3-18. Plain elastomeric pad squeezing out from under masonry plate (MN)

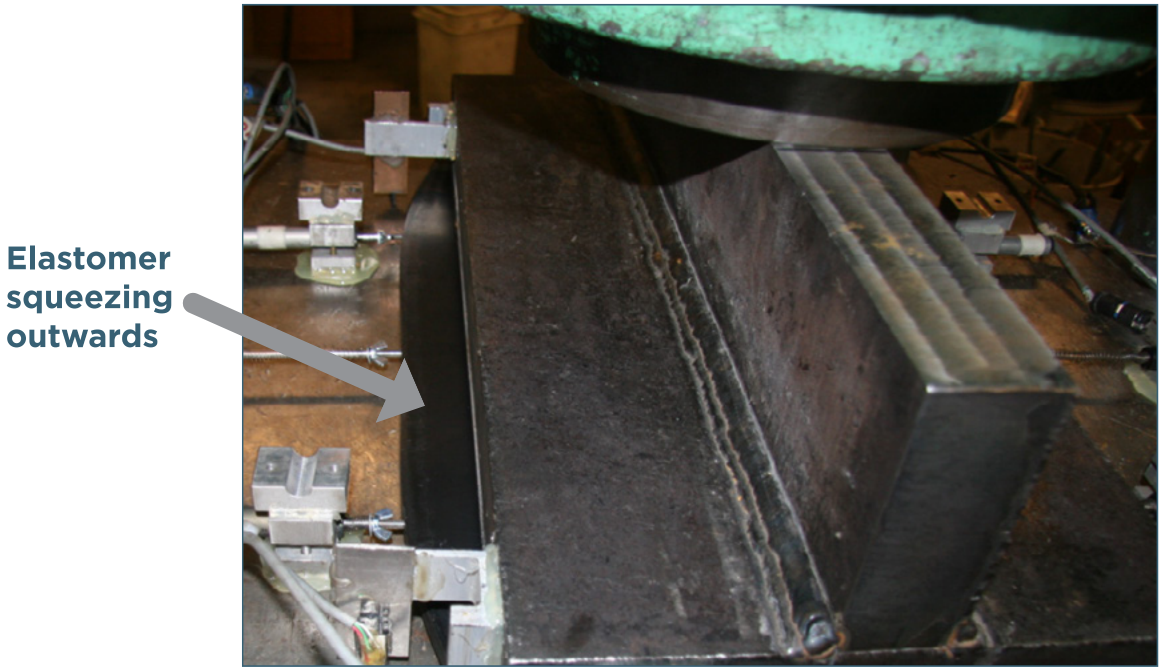

Figure 2.3-19. Plain elastomeric pad squeezing out during test

2.3.1.10 HLMR Bearings – Details

High-Load Multi-Rotational bearings include pot, disc and spherical bearings. They are functionally quite similar to each other, and are capable of resisting higher loads and accommodating larger rotations than SREBs, but differences exist between the types in cost and performance. Because all HLMR bearings typically have rotation elements that are circular in plan, they can rotate about any horizontal axis with equal ease. This is beneficial when the axis of the applied rotation is uncertain, such as in skewed or curved bridges.

The rotational stiffness of the bearing is important primarily in bearings equipped with a flat PTFE sliding surface to accommodate horizontal movements; high rotational stiffness may lead to an induced moment large enough to cause the stainless steel to lift off from the PTFE on one side and to apply concentrated edge loading on the other, potentially leading to damage of the PTFE.

Caltrans heavily favors spherical bearings, largely on the basis of their predictable and low resistance to rotation. Disc bearings are becoming more popular than pots due to their lower cost (they use less steel) and because they are easier to inspect. However, they have the highest rotational stiffness of the three types. Pot bearings attracted a bad reputation in their early history in the US due to a tendency to leak elastomer (Figure 2.3-21), but they are widely used, apparently without problem, in Europe. Most states reported that pots are less often chosen than discs or sphericals, and some states (e.g., Washington), explicitly ban them, based on past problems with leaking elastomer.

2.3.1.11 Pot Bearings - Details

A pot bearing consists of a shallow circular steel cylinder, or pot, in which an elastomeric pad is placed

(Figure 2.3-20). A steel piston rests on top of it, and brass seals prevent the elastomer from escaping through the gap between piston and pot. When vertical load is applied, the elastomer is confined in the pot and experiences hydrostatic compression. The bearing is thus very stiff in compression. When the piston tilts to accommodate rotation, the elastomeric pad is often said to behave like a fluid. However, it deforms, becoming thinner on one side than the other, which involves lateral movement of material within the pot. To facilitate that movement, the bottom surface of the pot has sometimes been lubricated, using either grease or thin sheets of PTFE. However, the PTFE sheets tend to buckle, and the grease eventually squeezes out, rendering both procedures ineffective in the long term. However, lubrication may still be useful for one-time rotations early in the life of the bearing.

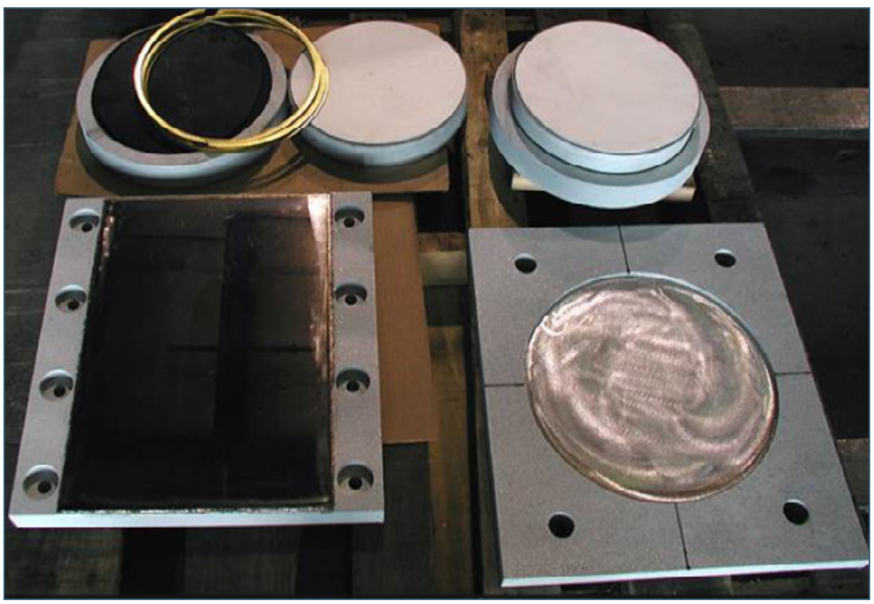

Figure 2.3-20. Pot bearing components (DS Brown)

Pot bearings were developed in Germany after the Second World War and the concept was introduced into the USA in the 1970s. Early US pot bearings suffered from problems of elastomer leakage (see Figure 2.3-21), which has been attributed to poor design of the seals, and possibly to poor quality machining. (A coarse machined finish on the inside of the pot walls causes wear on the seal as the piston rotates (Stanton et al., 1999). Although brass seals are used almost exclusively in the US, largely because they are specified in the AASHTO LRFD Design Specifications, other proprietary sealing systems are used in Europe, and have performed well in tests (Stanton et al., 1999). In some cases they are embedded in the rubber during vulcanization, which offers performance advantages.

Pot bearings are sensitive to rotations beyond the design limit, because then the piston might lift clear of the pot, providing a path by which the elastomeric pad could extrude. Live load rotations due to traffic are unlikely to exceed 0.004 radians (see Section 2.3.1.3) so over-rotation will occur primarily because of poor levelling during installation or because of excessive one-time rotations. The latter might occur due to end rotation of the girder when the deck is poured.

Figure 2.3-21. Elastomer leaking from pot bearing (OR)

Horizontal load, if any, is transmitted either by a guide bar system in guided bearings or by direct bearing between the edge of the piston and the interior of the pot wall in fixed bearings. If the horizontal load is sustained, the friction that it causes may prevent rotation of the piston and inhibit the proper functioning of the bearing under live load.

Experience with pot bearings in the US has been mixed, and perhaps colored by history. Some states (e.g., Washington) do not allow them, based on poor performance soon after they were introduced. By contrast Pennsylvania uses them, and reports that they are performing well. Most other states prefer not to use them, based both on the reluctance of others and the fact that they are difficult to inspect. If the plans call for an HLMR bearing without specifying the type, the choice then lies with the contractor, who will likely choose the lowest cost bearing. Today, that is unlikely to be a pot.

2.3.1.12 Disc Bearings - Details

Disc bearings (see Figure 2.3-22) were first developed and deployed in Canada in 1970 and have generally performed well over the years. Their use in the USA was limited while the concept remained under patent and the bearing remained a sole-source item, but has expanded in the last 20 years, since the patent expired.

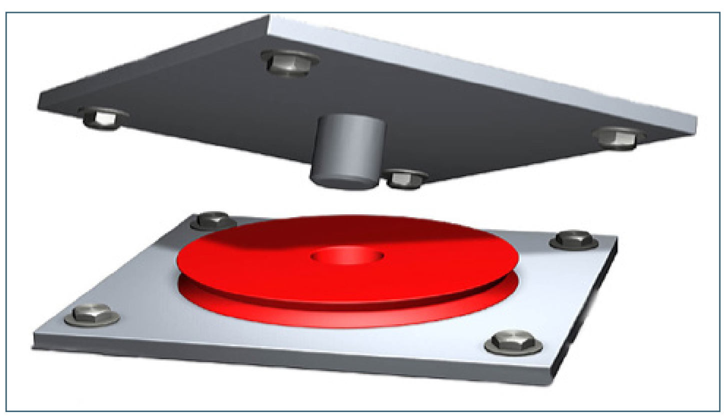

Figure 2.3-22. Fixed Disc Baring (RJ Watson)

Because their compressive capacity depends on friction between the disc and the plates above and below (Stanton et al., 1999) they are sensitive to any material, such as paint, on the steel surfaces that might reduce the friction. Some early designs used a partial height retainer ring to prevent outwards movement of the urethane disc, which was sometimes made from a more flexible material than that in common use today. One manufacturer now offers a design for larger rotations with a full-height restrainer ring, making the device a hybrid between a pot and a disc bearing.

The polyurethane material is much stiffer than the elastomer typically used in pot bearings (typically NR or CR), so disc bearings have the highest rotational stiffness of the three HLMR types.

Disc bearings are not used at all in Europe.

Design requirements for disc bearings in the AASHTO LRFD Design Specifications are much less detailed than those for pots or spherical bearings. This reflects the fact they were a proprietary, sole source, item when the bulk of the specification was written. Now that the original patent for disc bearings has expired and they may be built by any manufacturer, there is a need for comprehensive design procedures. A proposed set has been developed (Bradford et al, 2017) but has not yet been reviewed by Committee T2 (bearings) of AASHTO's COBS. A difficulty that is likely to arise is the definition of failure. This problem has occurred frequently for other bearing types, because in most cases failure occurs by gradual deterioration rather than sudden fracture. A consensus is then needed on "how much is enough?" damage to constitute failure.

2.3.1.13 Spherical Bearings - Details

Spherical bearings have been in use since the 1960s, although they have only become common since the late 1980s, when pot bearings started to go out of favor. They consist of two steel spherical surfaces, one convex and one concave, separated by a layer of low-friction material, as shown in Figure 2.3-23. The convex surface rotates by sliding within the concave one. The geometry of a spherical bearing is such that two sliding surfaces, one curved and one flat, are needed to permit rotation if no horizontal movement is possible. In the US today, the low-friction material is usually woven PTFE. Early bearings used sheet PTFE, but there are difficulties in making the flat sheet conform to a spherical surface.

Spherical bearings typically have the lowest resisting moment of all HLMR bearings, especially at higher rotations (say greater than 2%). They are also the most expensive, largely on account of the machining and the volume of steel needed. The concave plate is usually made from carbon ("black") steel, and the convex plate has in the past been made from either solid stainless steel (common) or carbon steel with a stainless steel welded overlay. However, the overlays have been found to crack over time and so that system is no longer recommended. California uses spherical bearings extensively and is well satisfied with their performance. The state uses a minimum contact pressure of 2000 psi at the sliding interface, and reports no significant problems in the 25 years that they have been using spherical bearings. Most of the other attending states allow spherical bearings.

The AASHTO Specifications allow horizontal load to be carried by a spherical surface, but this is seldom done. The spherical elements are generally quite low-profile, and the slope near the edge is consequently low. This is done to avoid the need for machining from thick, stainless steel plate, which is difficult to obtain commercially. The edge slope largely determines the ratio of horizontal to vertical load that can be carried. Thus, when horizontal loads need to be resisted, use of an external guide system is preferable.

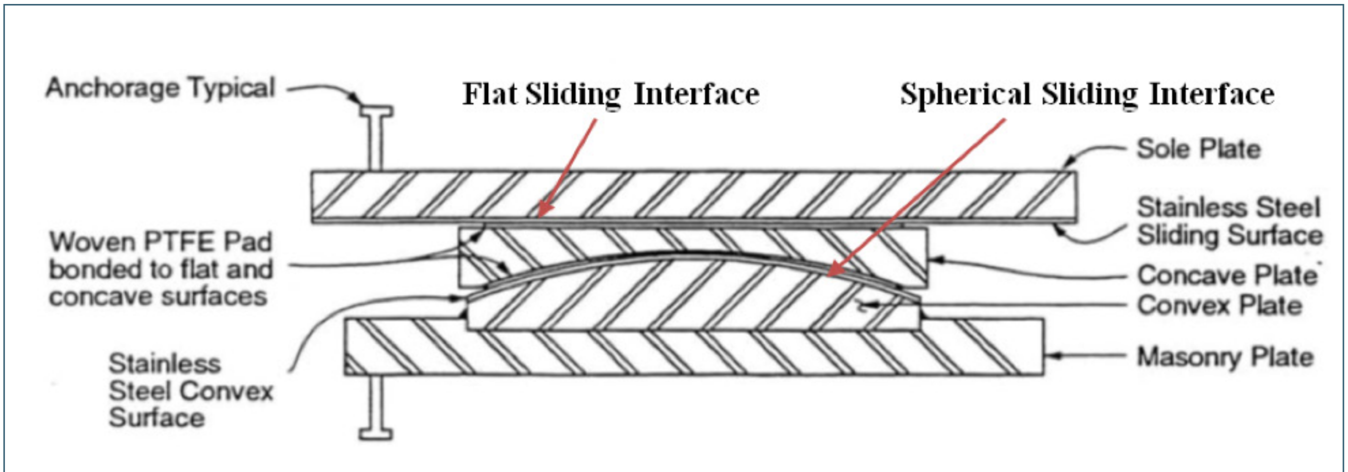

Figure 2.3-23. Spherical Bearing (CA)

2.3.1.14 PTFE sliders

Sliding surfaces are typically made from a low friction polymer (e.g., PTFE) sliding on polished stainless steel. In most cases they are planar and provide for horizontal movement. They may also be curved in one direction (cylindrical) or two (spherical), in which case they are called cylindrical or spherical bearings (see Section 2.3.1.10). In planar (flat) sliders, the stainless steel is larger than the PTFE, so that the PTFE never risks damage by sliding over the sharp edge of the stainless steel, which is usually welded to a carbon steel backing plate. The stainless steel is also uppermost, to prevent dust and dirt from settling on it. Both sheet and woven PTFE are used for planar sliding surfaces.

Almost all states accept the use of sliding surfaces, often coupled with a rotational element such as an HLMR bearing, to permit longitudinal movements larger than can be handled by an SREB of practical thickness.

The surfaces may or may not be lubricated. In Europe and Canada, lubrication is the norm, and dimples are machined into the (sheet) PTFE to acts as grease reservoirs. The goal is to prolong the lifetime of the lubricated system, because grease quickly squeezes out from a flat surface. In the US, most sliding surfaces are not lubricated and not dimpled, on the basis that even “dry” PTFE offers a low enough friction coefficient for the needs of the structure. A flat sliding surface may be lubricated to provide short-term low friction, for example to reduce resistance during post-tensioning. It is important not to allow the surfaces to separate (see Section 2.3.1.2), in order to prevent ingress of contaminants that could damage the surfaces. This is particularly true for greased surfaces, in which the grease would more readily trap wind-blown dust.

PTFE is the only low-friction polymer presented in the AASHTO LRFD Design Specifications, so it is almost universally used in the US. However, it is relatively soft and wears progressively with slide path and other factors (Campbell and Kong, 1987). Since the Specifications were first written, other, proprietary low-friction polymers with better wear characteristics have been developed. Acceptance of them is hindered by the prescriptive nature of the Specifications (see Section 2.3.1.17).

The PTFE must be firmly attached to its backing material (usually steel). This is typically done by bonding the PTFE to a material such as fiberglass or Kevlar, and mechanically interlocking that to the steel substrate. The bonding requires special epoxies, because PTFE's desirable characteristics in service are that it is chemically inert and slippery. California has found that high-temperature epoxy or phenolic adhesive works. The bonding agent must also be spread evenly to avoid high spots in the surface, which would promote wear.

The stainless steel is usually specified as having a #8 mirror finish, again, because this is the only surface defined in the AASHTO LRFD Specifications. The requirement can cause difficulties both with availability (because special polishing is needed to produce it) and because the surfaces are vulnerable to scratching during handling, in which case they risk rejection. The need for such a highly finished surface appears to have originated in Germany (Hakenjos and Richter, 1975) and has seldom been questioned. However, a recent report (Taylor, 2009) found that the benefits of the highly polished surface were short-lived, and that a lower grade finish is readily available commercially and actually gave better results over the long term. That result was obtained specifically for PTFE, and may differ for other polymers.

California developed a special version of an SREB with a PTFE slider. The original version is shown on the left of Figure 2.3-24. The goal was to maintain a high contact stress on the PTFE to keep the friction coefficient low, but to have a lower stress on the elastomeric component consistent with its capacity. This required the PTFE area to be less than the elastomer one. However, the system did not perform as intended, because the elastomeric bearing was stiff in rotation and the stainless steel tilted on the PTFE and caused loading on edge of the circular pad of PTFE. The state no longer uses PTFE sliders on SREBs.

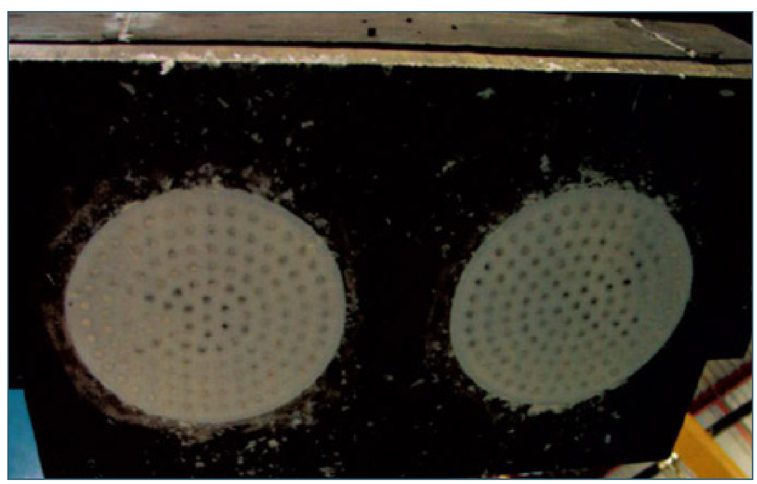

Figure 2.3-24. PTFE slider on SREB. (CA)

2.3.1.15 Seismic Isolation Bearings

Seismic isolation bearings are used to reduce the seismic forces on bridge substructures. The primary need for them is in regions vulnerable to earthquakes, and they are therefore of interest only to a small number of states, primarily those on the West Coast (California, Oregon and Washington.) Seismic isolation has been embraced less readily in the US than in other countries, such as Japan, Italy and Turkey. For new bridges in the US, engineers have generally preferred to rely on the traditional approach based on strength and ductility. However, seismic isolation has been used in several retrofit projects, where its ability to reduce the seismic forces has allowed the existing structure to remain in place and thus has resulted in the most economical solution. In the US, no isolated bridge has yet been through a severe earthquake, but evidence of the approach's effectiveness is available from other countries (Feng and Chu, 1996),

The design of seismic isolation bearings is governed by the AASHTO Guide Specification on Seismic Isolation (AASHTO, 2014), rather than the LRFD Design Specifications. The Guide Specification uses an approach based at least partly on performance, and this provides more room for innovation and the introduction of new systems. Unfortunately, the Guide Specification's bearing provisions are not well coordinated with those of the LRFD Design Specifications. For example, for steel-reinforced elastomeric bearings, not only are the strain limits different, which might be expected, but the method of calculation also differs.

California designs isolation systems for at least a 1000 year return period and, for special bridges, for a 2500 year return period. This hazard level defines the necessary displacement capacity of the bearing. In the event of a larger earthquake, inelastic action and some damage are expected, so the state requires the bridge to be designed and detailed for a plastic hinge mechanism as well. The design objective for the isolation is thus to minimize damage and downtime under the design earthquake.

Two types of bearing dominate the seismic isolation market: lead-rubber bearings and friction pendulum bearings. The former are based on the same principles as SREBs, but have significantly higher shape factors to allow higher vertical stresses and a lead core for energy dissipation (Figure 2.3-25). The friction pendulum bearings use a slider that floats in a spherical dish, and returns to the central low point through the action of gravity (Figure 2.3-26). Other types, based on planar sliding surfaces and independent return springs, are also available.

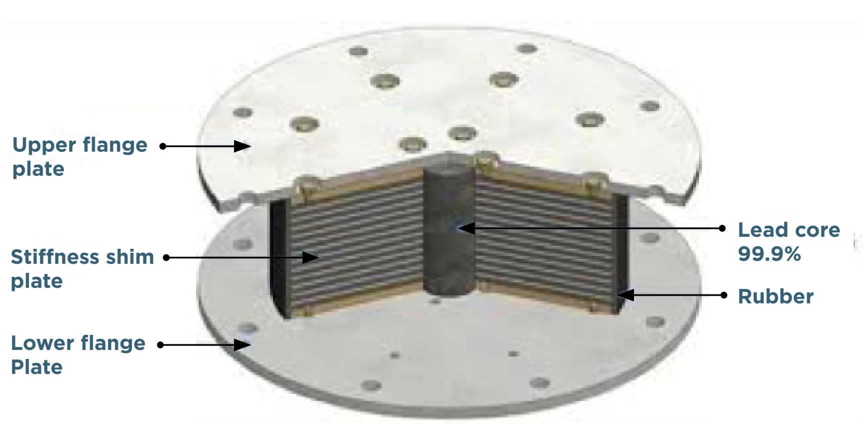

Figure 2.3-25. Lead Rubber Isolation Bearing (DIS)

The original patents on both systems have now run out, so they may be made by any manufacturer, although more complex versions of the basic friction pendulum have now been developed and are still under patent. California allows only prequalified manufacturers to bid on isolation work. The state tends to select friction pendulum bearings when large displacements and a low profile are critical, and lead-rubber bearings when corrosion resistance is important. Accommodating large displacements with any bearing system means a large bearing, and a large space in which to put it. The friction pendulum bearings in the Benicia-Martinez Bridge are 12 ft. in diameter (Figure 2.3-27). Washington State does not specify a type, but provides an envelope for the load-displacement curve, and accepts any bearing (from a prequalified manufacturer) that satisfies the envelope.

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The corrosion question affects sliding systems more than elastomer-based ones, because any corrosion of the steel component of the sliding surface affects the friction coefficient, which in turn affects the structural response. Washington, for example, has had problems with water ingress and corrosion in friction pendulum bearings (Figure 2.3 28). California has also experienced some corrosion with

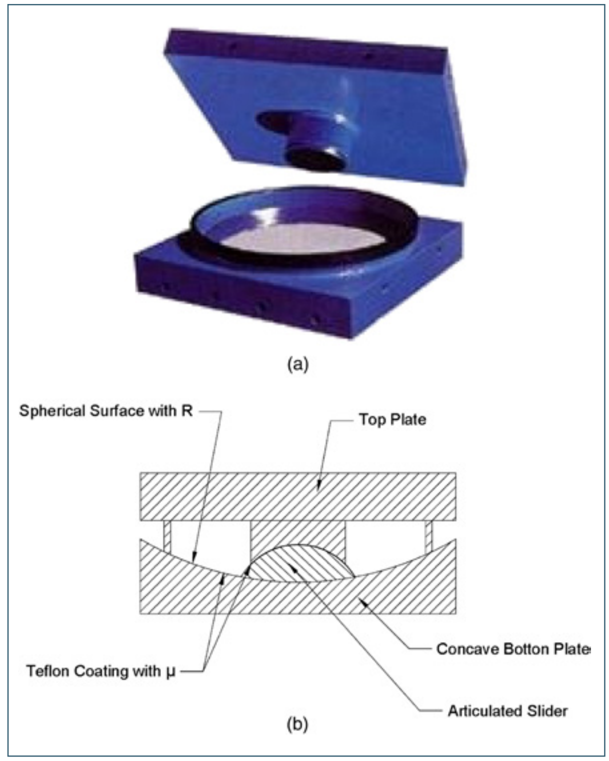

Figure 2.3-26. Friction Pendulum Isolation Bearing

Figure 2.3-27. Friction Pendulum Bearing for the Benicia-Martinez Bridge (EPS)

Figure 2.3-28. Corroded isolation bearing (WA)

them. Conventional SREBs, designed to accommodate thermal expansion, lengthen the natural period of the superstructure by introducing elastic flexibility, provided that no point on the bridge superstructure is completely fixed. In some states such as Illinois, where there is a small risk of a large earthquake, there is interest in the extent to which conventional expansion bearings can provide some measure of isolation (Ash et al., 2002, Kelly and Konstantinidis, 2011). The AASHTO LRFD Design Specifications offer no guidance on the matter.

Isolation bearings are designed to move during an earthquake, but may need to resist wind or other service loads at other times. Then they are equipped with “wind locks”. These are generally bolted devices that are calibrated to break at a given load, higher than the expected service load but smaller than the seismic load. Both tension and shear devices have been used, and they may be either an integral part of the bearing or a separate device that works in parallel with the bearing. In some shear-based wind locks, the bolt-holes are slotted to allow free movement in one horizontal direction and to resist wind forces in the other.

2.3.1.16 Special Bearing types

Several states use bearing types that were developed in-house, and which have been used for many years. Minnesota uses a metal rocker system, in which a cylindrical steel element acts as a rocking surface (see Figure 2.3-15 and Figure 2.3-29). The cylinder is welded to a larger steel plate to spread the load, and that plate rests on an elastomeric pad whose function is to correct any unevenness in the concrete supporting surface. Plain pads were used until recently. They were found to bulge excessively, and cotton duck pads are now used.

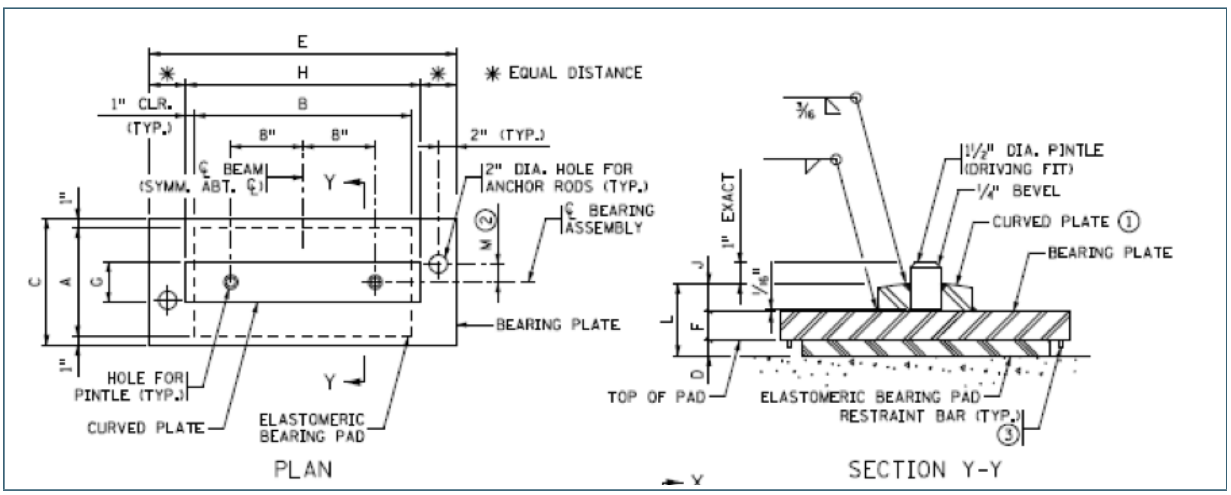

Figure 2.3-29. Minnesota bearing

Texas uses tapered SREBs, in which the rubber layers are tapered and the internal steel shims are splayed, to accommodate permanent girder end slopes. The distribution of taper among the layers is usually uniform for large slopes, but for smaller slopes it may all be concentrated in the top layer.

Some cylindrical bronze sliding bearings are still in use (Figure 2.3-30). Utah recently had to replace a broken one. Because only one bearing of many had failed, the bronze element was replaced rather than replacing all the bearings with a different type.

Michigan used an “H-bearing” which consisted of a piece of steel H-pile welded to the girder, with a slider under it. Some existing bearings remain in service, but no new ones are being implemented.

Figure 2.3-30. Cylindrical bronze bearing (UT)

California used to use a special elastomeric bearing with a PTFE slider set into the top, but has now abandoned the design. In order to maximize the stress on the PTFE (which minimizes the friction, because the friction coefficient varies with stress), but keep the stress on the elastomer within limits, a rectangular steel plate was bonded to the top of the elastomer (Figure 2.3-24). It had two circular recesses, in each of which was bonded a circular piece of PTFE that slid on the stainless steel.

2.3.1.17 Specifications

In the US, design of bearings is guided by the AASHTO LRFD Design Specifications, and testing of elastomeric bearings, which used to be controlled by the AASHTO Construction Specifications, now resides in the AASHTO M251 Materials Specification. The team heard input on three fronts: general comments on the Design Specifications, comments on the disc bearing provisions, and comments on the elastomeric bearing provisions.

The general comments focused on two points. First, each state has its own specification and requirements, for both design and testing. Even though the individual state requirements are built around same national specifications, the multiplicity of different state requirements makes the manufacturers' job difficult, and contributes to misunderstandings. The present system is therefore inefficient for fabricators supplying clients countrywide. A single, national set of specifications was seen by the manufacturers as preferable.

Second, the AASHTO LRFD Design Specifications are prescriptive, and therefore inhibit innovation and the introduction of new materials or bearing configurations. This situation stands in contrast to the European Specifications, Eurocode EN1337, which is written in terms of performance. Improved low-friction polymers provide an example of materials that are not addressed by the AASHTO specifications, and which are therefore difficult to introduce in the US market place.

The provisions for disc bearings in the AASHTO LRFD Design Specifications are minimal. Expanded provisions, and clear guidance, are needed for design, especially now that disc bearings may be fabricated by any manufacturer, and they are becoming more widely used as some states move away from pot bearings. It is in the interests of owners (states) to have a universal set of design rules for all manufacturers. R.J. Watson has conducted a study and developed design rules from it (Bradford et al. 2017). The team had not yet seen details of this proposal, but agreed that it might provide a starting point. Until national guidance is developed, some states (e.g., Minnesota) are drafting their own design specifications for disc bearings. Some features of the proposed design rules raised concerns. For example, they contain provision for higher stresses on disc bearings, and questions were raised by attending state representatives as to whether the risks involved were justified by the benefits of smaller, and presumably cheaper, bearings.

The design rules for elastomeric bearings were changed significantly in 2008, when they were first formulated in terms of a limit on the total shear strain from all sources. This change was seen as an improvement in the logic and clarity of the design process, and as being in line with the European specification EN 1337. AASHTO Design Method B, which takes advantage of this approach, appears to be widely used, despite its greater complexity. Many states have offset the problem of complexity by developing in-house spreadsheets for use in design.

The testing requirements for elastomeric bearings were seen by participants as being in need of revision, both in the absolute and because they are not well linked to the Design Specifications. Inconsistencies, and sometimes conflicts, exist over the number of specimens to be tested and the tests that are needed. Test requirements are discussed further in Section 2.3.1.18.

2.3.1.18 Testing

All of the discussion about test requirements concerned the tests for elastomeric bearings, which are contained in AASHTO Materials Specification M251. The elastomer itself must satisfy certain ASTM material tests. These are typically laboratory scale tests; they do not use the manufacturer's production machinery, they form part of the in-house QC procedures, and were not questioned.

The finished bearing must be tested to 150% of the design (compressive) load in a short-term test. This test is relatively quick and easy, because it can be conducted in the press in which the bearing was made, and was seen by the manufacturers as a useful way to detect obvious flaws, such as misplaced internal steel plates. Bearings subjected to this test can be used in the project for which they were fabricated.

Long-term load tests pose a greater logistical problem, because they prevent the press from being used to fabricate another bearing, and their usefulness was questioned. First, there is no known scientific study to demonstrate the effectiveness of the long-term test procedures in preventing failures in the field, and it was felt that a risk-based analysis of field performance would help to better define the amount of testing needed.

A subsidiary question relates to the number of bearings to be tested, because destructive testing wastes bearings. This is particularly true when the number of bearings in the lot is small. If the lot consists of only two bearings, is it cost-effective to make a third for destructive testing? Could long-term deterioration tests be carried out on prototype bearings, rather than on a per project basis? Last, the test requirements contained in AASHTO M251 were seen as poorly correlated with the LRFD Design Specifications, and lacking in clarity. Florida, for one, has modified the requirements of M 251 with the goal of simplifying them.

2.3.2 Fabrication and Installation of Bearings

2.3.2.1 Fabrication of Bearings

No serious problems were seen with fabrication of SREBs. In some cases dimensional tolerances are violated, but small variations have little effect on the performance of the bearing, although when the bearing has to fit in a space of given dimensions, maintaining dimensional tolerances is clearly important. The most common other fabrication problem in practice is inadequate bond between the elastomer and the internal steel plates. This can usually be detected by a discrepancy in the bulge pattern under the short-term load test, but occasionally it appears for the first time in the field.

Pot bearings are typically machined from thick plate, which creates a significant amount of wastage and is consequently expensive, especially if the market for steel is tight. The possibility of local distortions in the domestic market, causing higher prices, material shortages and delays, especially for thicker plates, should be considered when choosing to use a pot bearing.

Corrosion protection of the steel is required, and several attending states have received requests from fabricators to permit metallization instead of hot dip galvanization, because galvanization makes achieving the specified flatness tolerance on the plates almost impossible. Metallization has been found to be almost effective as galvanization, so it appears to be a good solution. In particularly corrosive environments, painting on top of the metallization may be used for additional protection.

For pot bearings, the seal is typically the critical element while, for discs, care is needed not to paint or otherwise contaminate the surfaces in contact with the polyurethane disc. For spherical bearings, the two spherical surfaces must be machined to provide matching geometry, with an appropriate allowance for the PTFE thickness, in order to avoid local high pressure and premature wear. These issues are well-known to manufacturers.

Sliding materials such as PTFE are normally secured to a backer plate using adhesive. Conventional epoxies do not perform well. Specially formulated epoxies are available and are suitable for non-seismic applications. The high-speed movements that occur in seismic bearings risk over-heating the surfaces and breaking down the adhesive. In such cases, a special, high-temperature adhesive is necessary. California has found that phenolic or epoxy adhesives formulated for high temperature work well.

2.3.2.2 Installation of Bearings

Site operations are conducted by the General Contractor, and bearings are sensitive to mishandling by unskilled labor. One example was quoted in which an elastomeric bearing was found to be damaged; closer inspection showed that it had been installed with the aid of a sledge-hammer. Bearings that are composed of multiple components, such as any bearing with a slider or a spherical bearing, should not be disassembled on site, lest dust or dirt affect moving parts, or UV chemically affect materials such as PTFE. Some states reported requiring the presence on site of a manufacturer's representative during installation, while others (e.g., Washington) do not.

The General Contractor also is responsible for setting the bearing to account for thermal displacements. The requirements vary by state. For example, Minnesota adds disc bearing setting tables to plans, while New York requires the bridge to be lifted and the bearings to be re-set after installation is complete. Both procedures represent an improvement over doing nothing, but their success depends on how well the bridge temperature can be correlated with the air temperature, which is used as the setting criterion. Re-setting can only be done approximately, so, for example, Louisiana requires it only if the bearing is placed at a temperature outside the range 50 – 85 deg. F.

The bearing seat must be set at the correct elevation, and this is normally achieved with a grout pad. Louisiana uses a grout pad, or "riser" with a minimum height of 4 inches, to allow for reducing the height if necessary during construction. That may be needed, for example, if the girder camber is excessive. While no problems were reported, it should be noted that grout pads under bearings experience some of the highest service stresses of any cementitious material in the bridge, yet they are typically unreinforced. Therefore, using the thinnest pad consistent with other requirements would minimize the chance of structural damage. There is also a question of durability of the grout, which is typically mixed on site, by site labor. Utah avoids the use of grout by requiring the contractor to cast the concrete too high and then grind it down to level. Hard data were not available to show which is the better procedure.

Levelling the grout pad to be truly horizontal is particularly important with SREBs, as discussed in Section 2.3.1.6, because their rotation capacity is linked to the applied compressive stress. Yet no participating state reported that they monitored the accuracy of the finished level. Elastomeric bearings are often regarded as being tough and forgiving of overload. Unfortunately, they are not well suited to accommodating rotations, so careful leveling during installation is important.

Many SREBs are held in place by friction alone. This has been traditional practice for bearings supporting prestressed concrete girders and for the most part has proved adequate, with two notable exceptions. First, Texas found that pads were walking out of place, which they attributed to the anti-ozonant waxes used at the time in Natural Rubber (see Section 2.3.1-6). Second, Pennsylvania found that setting the pads on epoxy-coated concrete beams seats caused the pads to squeeze out (see Figure 2.3 31). This behavior is attributed to the fact that the surface of the epoxy was smooth, which led to progressive slip across the interface.

Figure 2.3-31. Elastomeric bearing set on epoxy base and squeezing out

For fixed SREBs, some states provide anchor bolts, as shown in Figure 2.3 32 and Figure 2.3 33. The bolts are subjected to both shear and bending, and must be appropriately sized to avoid failure. For example, Florida's Structure Design Guidelines (Florida DOT 2018) provide an equation that relates the required bolt size to the applied load. However, the bolt forces also cause stress in the (unreinforced) grout pad (see Figure 2.3 33), and may damage it. For restraining movements of the superstructure, an external bracket such as that shown in Figure 2.3-34, provides a better solution. If the purpose of the anchorage is simply to prevent the bearing from walking out of place, external plates can be vulcanized to the rubber and bolted to the concrete seat.

Figure 2.3-32. Steel reinforced elastomeric bearing – broken anchor bolt

Figure 2.3-33. Steel reinforced elastomeric bearing – cracked grout pad

Figure 2.3-34. External bracket for steel reinforced elastomeric bearing (PA)

2.3.3 Inspection, Maintenance and Repair and Replacement of Bearings

2.3.3.1 All Bearing Types

Bearings are special items and inspectors need training, for example, in measuring and recording the displacements, rotations and ambient temperature at the time of inspection. These measurements are needed to know whether the bearings are functioning as intended. Minnesota, for example, runs a Bridge Maintenance Academy to supply the training.

Traffic control is one of the biggest problems in arranging inspection and maintenance. For example, Michigan limits inspection and maintenance in some cases because traffic precludes access. Inspecting and maintaining bearings depends on being able to see them. In some cases, such as in in-span hinges, this is difficult (Figure 2.3-35). If a bearing cannot be seen, replacing it will be very difficult. This emphasizes the importance of planning for replacement at design time.

Figure 2.3-35. Bearing in in-span hinge (WA)

Methods for inspection vary among states. Texas described it as “always an ongoing battle”, and many other states reported that both access, restricted on roads with heavy traffic, and limited funding, present real challenges. At one end of the scale, Florida relies, for inspecting high-level bridges, almost exclusively on UBITs, which are likely to cause congestion in heavy traffic conditions. At the other end, Minnesota uses cameras on poles, drones, two different types of UBITs (Conventional snooper trucks and MOOGs). Utah, Florida and Louisiana are also considering the use of drones. Minnesota is presently developing policies for optimal use of the drones. While they appear promising, detailed data on their effectiveness is not yet available.

Frequency of inspection (and cleaning) depend strongly on the environment in the state. Minnesota experiences hard winters and so cleans joints every spring (and more often for some joints). During that cleaning, joint and bearing deficiencies are both tracked and are logged into the Maintenance module of the Structures Information Management System (SIMS, 1999). Steel bearings show the most widespread and serious deficiencies, and these need additional inspection that may not be possible during routine joint cleaning, because additional equipment may be needed for access. Florida uses FHWA’s BrM software (the former Pontis) and the associated Condition Indices (Chase et al, 2016).

Other considerations were are also of concern. Pennsylvania reported that cleaning and maintenance during winter months was less effective, and was largely restricted to daylight hours, while Oregon reported that cleaning (for joints and bearings) was allowed only during the winter months (November – March) to restrict contamination of waterways. Florida reported restrictions to limit disturbances to bats, and Washington reported impediments to access caused by encampments of homeless people. By contrast, Florida found no problems with access.

These problems with access and funding have led most states to replace bearings only when strictly necessary, or when the bridge is slated for some other major rehabilitation, such as a seismic upgrade. A major consideration is whether the repair or replacement can be conducted under traffic. Pennsylvania conducts such work under traffic, closing the roadway only for the short time (approximately 20 minutes) needed for the actual lifting operation. Other states use different procedures, depending on the circumstances. Florida reported closing the roadway and lifting the bridge for replacing SREBs, while Utah and Oregon would typically replace bearings under traffic, and Texas would do so for certain types of bearing.

Bearings at in-span hinges (see Figure 2.3-35) pose particular difficulties. Inspection is at best difficult, while replacement typically requires complex lifting arrangements (e.g., Michigan, 2014) and almost inevitable lane closures.

The condition of the different elements, and their expected lives, also depends on the local environment. Minnesota reported that the Condition Index depended less on age, but more on element type and exposure, with bearings under fascia girders suffering the most. Damage requiring attention was concentrated in two areas: elastomeric bearings moving out of place (e.g., Louisiana, Pennsylvania, Texas, Utah) and steel bearings suffering corrosion, largely as a consequence of joints failing and leaking. The latter problem was reported by almost all states, but is particularly prevalent in states (e.g., Pennsylvania) that use deicing salts and anti-skid materials, because those create can damage joints. Most states reported that, in general, their modern elastomeric bearings were performing well. Oregon believed that SREBs merited little inspection, and Texas has some elastomeric bearings that are 70 years old and still functioning properly. (It should be noted that Texas used to use 70 durometer unreinforced pads in preference to SREBs, and it is not clear whether the older bearings are of this type.) Other, less common types displayed their own particular shortcomings. Texas reported locked-up lead slider bearings, Washington reported frozen brass bearings, and Utah has replaced slipped or cracked bronze bearing elements.

Causes other than corrosion were reported by several agencies. Louisiana cited over-weight vehicles, extremes of temperature and other sources of environmental deformations, and incorrect installation. Oregon pointed to a related problem, namely that of ensuring even loading among multiple bearings. This is particularly difficult in box girder bridges, because of their high torsional stiffness.

Replacing bearings requires jacking the bridge, and this is made easier by providing jacking points at design time. Many states require space for the jacks under the girders or diaphragms, but Oregon reported that, while this may be feasible in steel bridges, where space can be made available under the diaphragm, the geometry of concrete diaphragms resulted in little available space. Then they had to resort to corbels drilled in and bolted on at replacement time. Pennsylvania has in some cases embedded steel tubes through the cap beam so that a temporary steel bracket can be bolted on if the need arises. Figure 2.3-36 shows one of the experiences that led to that decision; to attach lifting brackets, it was necessary to use threaded rods that were drilled in and anchored with epoxy. In the figure, the rods have been cut off after the completion of jacking. Florida used to require jacking points, but have now abandoned that policy, at east for elastomeric bearings, because they seldom require maintenance, much less replacement.

Figure 2.3-36. Provision for bolted lifting bracket (PA)

Lifting the superstructure may require lane closures, and those are clearly undesirable. The need for lane closures may depend on the lift height required. Florida reports having replaced elastomeric bearings, which require a minimum lift height (about ½"), but they still closed lanes during the jacking operation. Michigan has a 3/8" minimum lift height, which should be adequate for removing even a maximum height (6") SREB. Replacing the bearings at semi-integral abutments may pose special problems, because the backfill in contact with the backwall may inhibit lifting.

The necessary lifting height is dictated by the attachment details. If the bearing is secured by welds alone, these can be cut and the bearing can be slid out with minimal lifting. If anchor bolts project up from the seat, through the masonry plate, the bridge has to be lifted at least by the projecting length of the bolts. However, if the sole (and masonry) plates are bolted to embedded plates (see Figure 2.3 37) the bolts can be removed and the bearing can be removed with minimal lifting. Note that the bolts must be short enough to be withdrawn in the vertical space available. If the bolts are secured in threaded holes in the embedded plate, they will not develop their full strength, because the plate steel will be weaker than the high strength bolt steel. High strength nuts welded to the back of the embedded plate solve the problem.

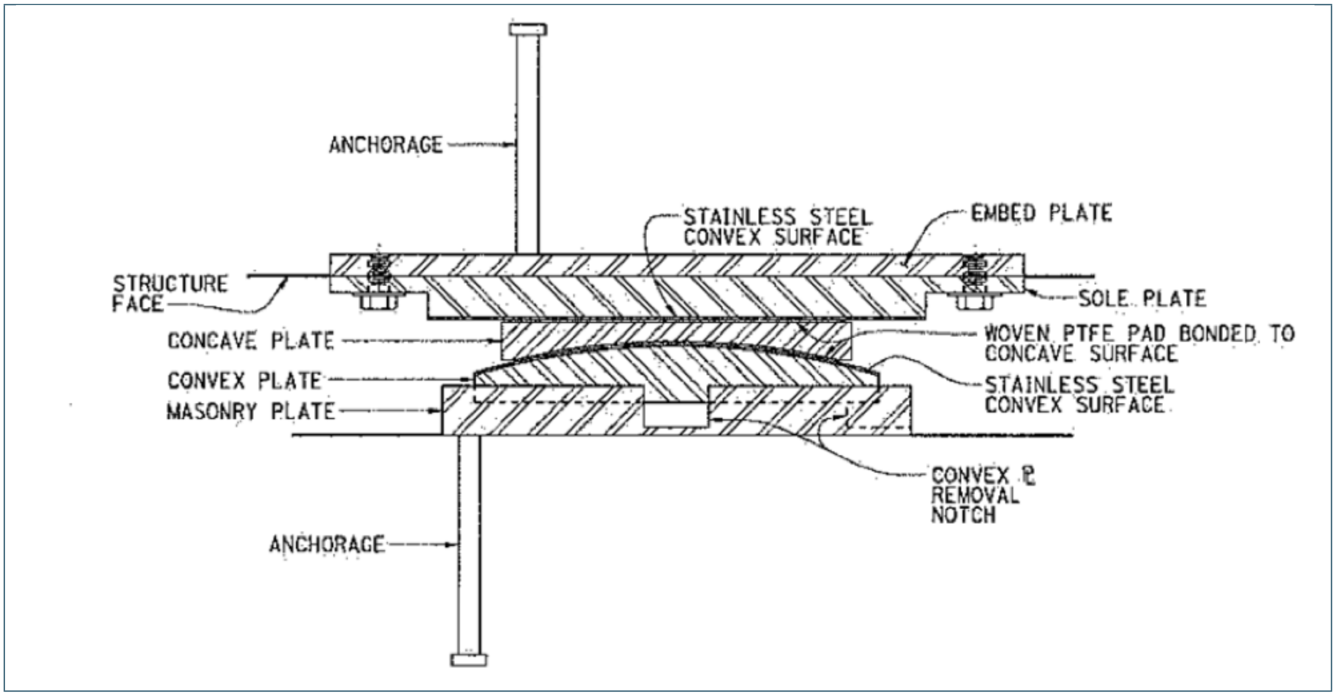

Figure 2.3-37. Bolted attachment detail for minimal lifting

The available lifting height is related to the stroke of the jack, which is in turn related to the collapsed length of the jack and therefore to the vertical space available. If this is insufficient, it is possible to “ratchet” the bridge up, using a jack and a stack of plates. Such a procedure takes additional time and space.

2.3.3.2 Performance Monitoring

Two suggestions were raised for establishing records of bridge performance, and both used the bearings as sensors. First, the position of the bearing could be monitored and recorded during routine maintenance, along with the concurrent temperature. Examples of position include the tilt angle of a rocker bearing, the shear displacement of an elastomeric or the slip distance of a PTFE sliding bearing. These measurements, albeit intermittent, would provide a record of the movements of the superstructure, so that future anomalous movements that did not the pattern of previous readings could alert the inspectors to possible problems. Second, the same concept could be adapted to wireless, digital recording methods to provide a real-time record of movements, and possibly forces, with time. Suitable equipment suitable for such structural health monitoring of bridges is available today, and is presently in use in the I-35W bridge in Minneapolis, for one example. Collins et al (2014) prove an overview of the field.

2.3.3.3 Steel Bearings

Steel bearings (rocker, roller, sliding plate) were reported by all states from Florida to Washington to require the most maintenance of all bearing types. They are almost always associated with steel bridges. They are, however, easier to inspect than other types, because their greater clearance offers better access. Some states clean them during inspection, while others send separate crews to clean them. Minnesota cleans and lubricates them every four years, but Oregon questioned whether cleaning and painting steel bearings was cost-effective. Other states typically paint the bearings with the bridge. Minnesota has removed some rocker bearings to re-furbish them in the shop before re-installing them in the field. In cases of serious decay (e.g., Figure 2.3-38), roller nest bearings have been replaced, usually by SREBs. Then, the difference in height of the old and new bearings requires consideration.

Figure 2.3-38. Roller nest – serious corrosion

The maintenance activities for steel bearings include:

- Adding steel edges to prevent excessive rotation (PA) (see Figure 2.3-39),
- Shimming (MN),
- Re-greasing lubricated bronze bearings,
- Re-setting rocker bearings that had over-rotated,
- Cleaning and lubricating pins. They may be difficult to extract, and need heat and/or small hydraulic jacks to release them.

Florida reported that cleaning and spot painting can be done during inspection and that some of the rehabilitation, including metallizing, can be done under traffic. Bearing replacement requires traffic diversions.

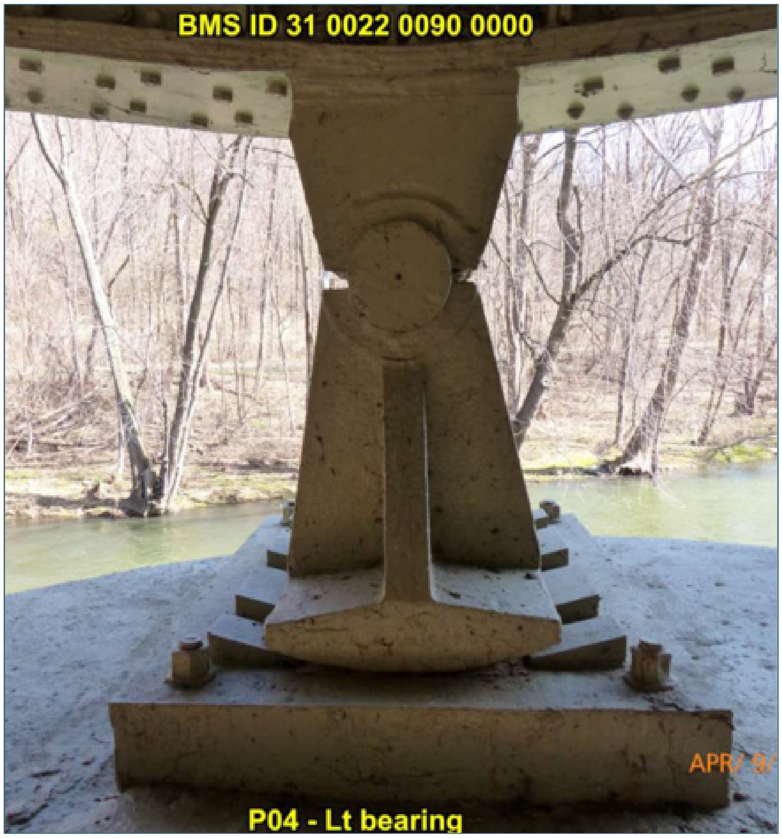

Figure 2.3-39. Keeper plate retrofit for rocker bearing (PA)

2.3.3.4 SREBs

For elastomeric bearings, inspection and maintenance is largely a question of verifying that the bearings are still in place, and have not moved or walked out. Occasionally, damage such as bulging or splitting occurs (see Figure 2.3-40) and is usually a consequence of edge loading, overloading, or poor manufacturing procedures. (As discussed in Section 2.3.1.4, overloading can be caused by an uneven distribution of load among bearings, just as much as by excessive total load).

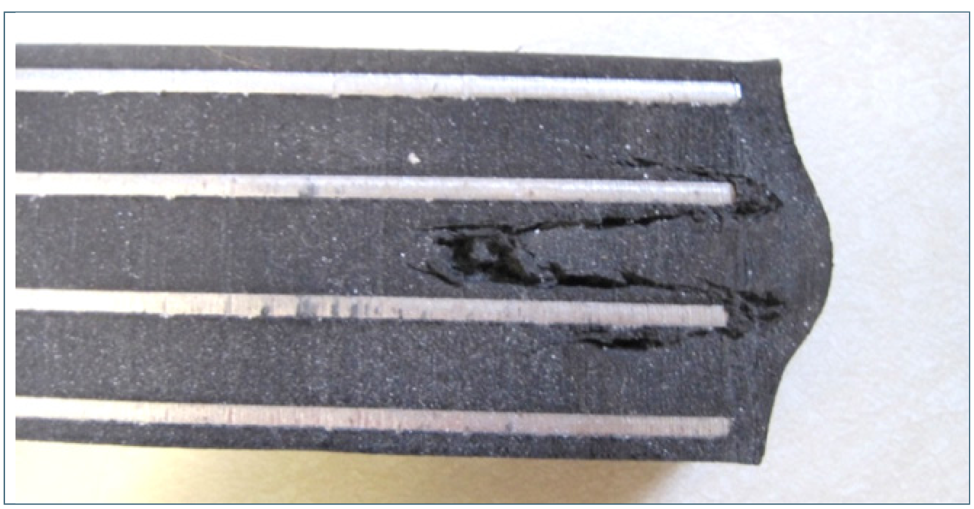

Figure 2.3-40. Damage (in Test) to SREB due to severe overloading

Oregon reported that their primary problems with elastomeric bearings were:

- Pads becoming mis-aligned, or walking out of place (mainly plain pads, rather than SREBs)
- Uneven loading
- Poor grout pad construction

To prevent the pads walking they have developed a steel stopper plate detail, shown in Figure 2.3-41. It has an advantage over vulcanized plates that are bolted to the seat in that the stopper plate bolts can be removed easily, after which the bearing can be removed with only a minimal girder lift. If an SREB with vulcanized plates is set over bolts set into the concrete seat, the girders must be lifted high enough to clear the projecting bolts.

Figure 2.3-41. Bolted stopper plates for elastomeric bearings

2.3.3.5 HLMR Bearings

For all HLMR bearings, over-rotation and steel-to-steel contact is a potential problem, although seldom seen in practice. For pot bearings, the primary question is whether the seal has failed and elastomer is leaking. The critical elements in a pot bearing (the rubber pad and the seals) cannot easily be seen, which makes inspection difficult. By contrast, the pad of a disc bearing is readily visible, and that ease of inspection contributes to their increasing popularity.

In a spherical bearing, the PTFE sliding material is the most likely to wear but cannot be seen without dis-assembling the bearing. However, no state reported problems with wear, and California (perhaps the most extensive user) reported being happy with the results, to the extent of using spherical bearings for all HLMR uses. Because the PTFE cannot be seen, the extent of the wear is of interest. PTFE is known to wear with slide path distance, contact pressure and sliding speed (Campbell and Kong 1987). Rotations that would cause movement at the curved interface might occur in several categories:

- One-time movements during construction,
- Seasonal thermal movements
- Daily thermal movements
- Traffic movements

The first three are unlikely to induce significant contributions to slide path. For the traffic movements, the induced moments may be too small to overcome the friction at the sliding interface, instead causing bending deformations in some other component such as the anchor bolts or column. If the PTFE does not move, it does not wear. If this is the case, it may explain why the PTFE appears to be quite durable in the field, even though comparisons with other, more modern, materials suggest that it wears more quickly than they do.

2.3.3.6 Isolation Bearings

Seismic isolation bearings are more complex devices than conventional expansion bearings, and they are typically designed by the supplier to meet the requirements of the owner. The principles of the two main designs (lead-rubber and friction pendulum) are well established, so the outcome is largely determined by the quality of the materials and workmanship. Those depend on parameters that include

- Complete Shop Drawings
- Stainless Steel for all machined/polished sliding surfaces
- Robust painting/coating practices
- Water-Tight Bearing Enclosures (for sliding isolators)
- Third Party Inspection

No isolated bridge in the US has yet been through a severe earthquake, so true field data is not available, but at least a sample of each bearing lot is tested prior to installation. Thus, although the tests are expected to have proven that the desired behavior occurs, field experience in an earthquake might yet reveal a hitherto unknown shortcoming. It took the 1971 San Fernando earthquake to reveal the need for heavy ties in concrete columns and the 1994 Northridge earthquake to reveal the shortcomings in steel welded moment frames.

Theoretically, both systems (lead-rubber and friction pendulum) will not re-center exactly and return to their pre-quake configuration, prevented in one case by the yield strength of the lead plug and in the other by the friction. In practice, the bridge could be jacked back into position, but ambient vibrations and creep might also bring it back without assistance.

Both bearing types must be installed horizontal to a high degree of accuracy. In the case of lead-rubber bearings, they are typically built with a high shape factor (around 20), in which case even a small rotation induces a significant stress. For the friction pendulum bearings, any slope discrepancy will cause the bearing not to return to its original position. The presence of a manufacturer's representative on site during installation is essential.

2.4 Joints

All joints deteriorate over time. If, eventually, they start to leak, they risk causing corrosion and damage to the bearings. However, joints are necessary to accommodate movements. One solution to this apparent dilemma is to move the joints off the bridge, for example to the approach slabs, where leaks may still occur but they do so without jeopardizing the bearings. Such bridges are referred to as "Jointless". The terminology is somewhat misleading, because joints do exist; they are simply not within the domain of the bridge itself, namely between the abutments. There are no joints over the piers, so all the expansion has to be taken at the ends of the bridge. There, the movement is accommodated by either a fully integral or a semi-integral abutment. Jointless bridges offer a real advantage, and many DOTs now view a hierarchy of choice, ranging from fully integral to semi-integral to jointed, in order of decreasing preference, with some states disallowing joints altogether between abutments unless special permission is granted to include them. Jointless bridges are discussed in Section 2.4.1.

Joints are distinguished largely by their movement capability, which can be roughly divided into small (< 2") medium (2" to 4") and large (> 4"). These values are specified by AASHTO, but are modified slightly by some states. The terminology for joints varies widely among states, and carries the potential for confusion in any country-wide comparison. The type of joint used is primarily dictated by its movement, as listed below. Details of individual joint types are provided in Sections 2.4.2.3 to 2.4.2.11.

- **Large movements** can be accommodated by finger joints, modular joints or large sliding plates, such as Caltrans' seismic sliding plate joint.
- **Medium movements** can be accommodated by gland-based systems, such as strip seals, by bolt-down elastomeric panels, and by sliding plates.
- **Small movements** can be accommodated by plug-type systems and a variety of systems that consist of a joint filler material and a waterproofing system.

The choice of joint type is also influenced by factors such as bridge skew or curvature, and by the need to accommodate seismic displacements in uncertain directions. For example, finger joints tend to lock up in skewed bridges because some transverse movement inevitably occurs and causes the joint to bind. Skew may also affect gland-based joints, because motion parallel to the joint may occur and increase the total deformation demand on the gland.

2.4.1 Jointless Systems

It has been said that the best joint is no joint. During the 1960s, when much of the freeway system was built, many bridges were constructed using simple spans. Those were subsequently found to give poor ride quality, so many bridges today are built as simply supported for self-weight and continuous for live load. This is achieved by casting a diaphragm between the abutting girder ends, and means that all the superstructure expansion and contraction must be taken at the ends of the bridge. The live load connection can be made in two ways. In the first, (Figure 2.4-1) the girders are connected to each other, but not to the supporting cap beam, or cross-beam. The continuous girder then rests on a bearing, and no moment is transferred through the cap beam to the columns. This approach is prevalent in states with a low level of seismicity.

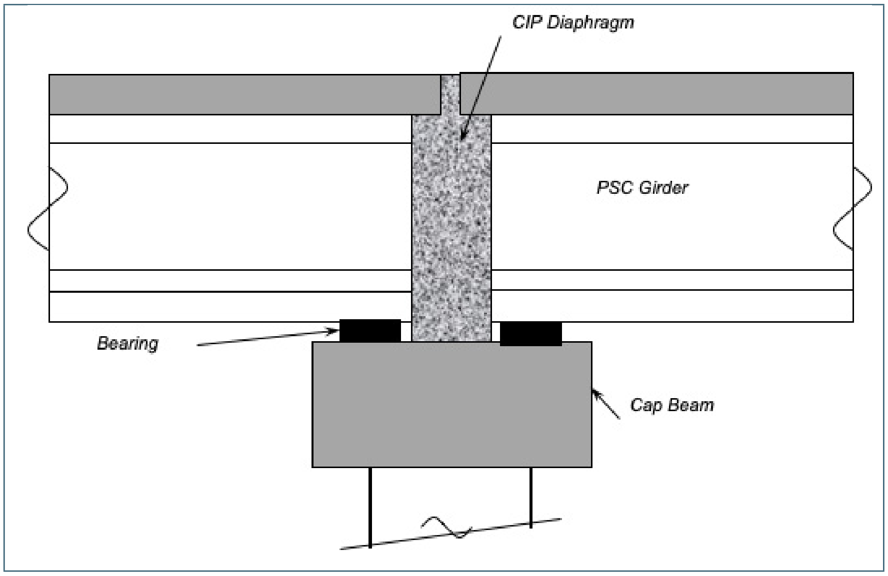

Figure 2.4-1. Girders connected only to each other.

The second approach, shown in Figure 2.4-2 is to join the girders both to each other and to the cap beam. This provides a moment connection between the girders and cap beam, and allows longitudinal forces due to braking or seismic action to be resisted by frame action. That approach is common in seismic states. Details differ among states. California builds most bridges using cast-in-place continuous box girders, in which the cap beam forms part of the box and the soffits of the two are flush. Other West Coast states (e.g., Washington) use a drop cap beam, which allows precast prestressed girders to be set on the cap beam, but results in their soffits not being flush (Figure 2.4-2).

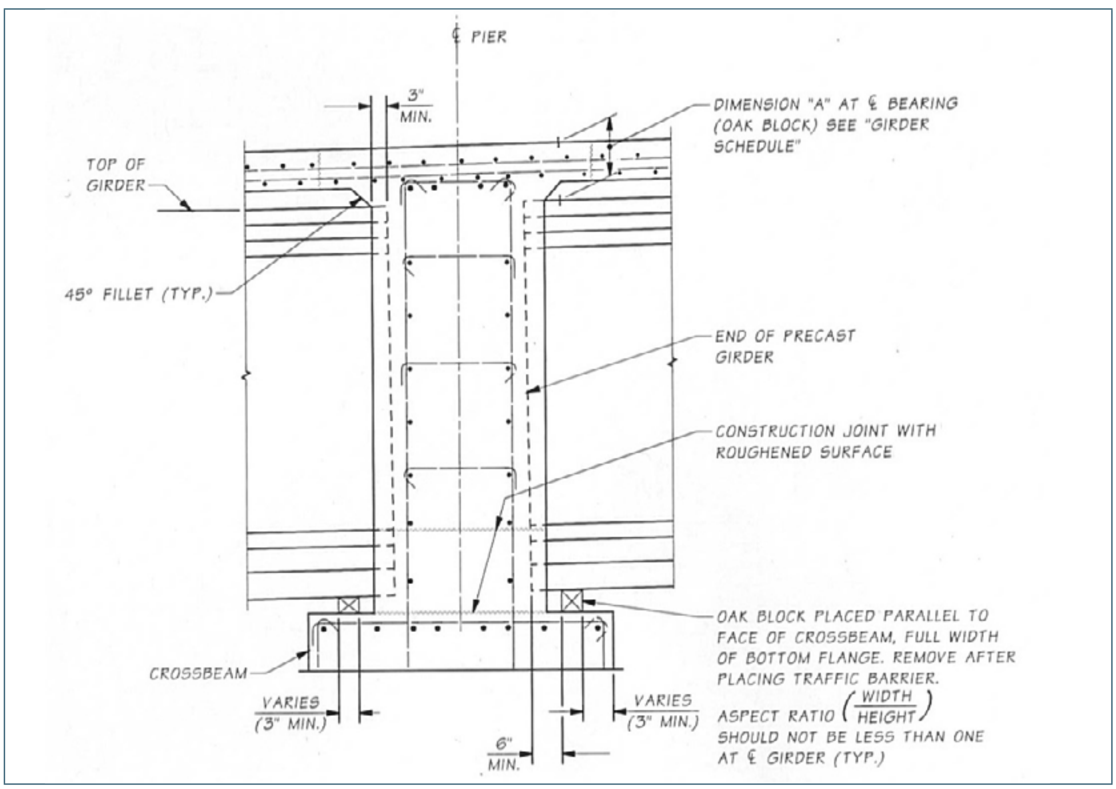

Figure 2.4-2. Pier connection: girders and cap beam connected. (WSDOT)

For existing girders that are simply supported, the ride can be improved by casting a link slab across the open joint (Figure 2.4-3). That makes the deck continuous, and so improves the ride, but the slab is not stiff enough flexurally to provide moment continuity between the girders for live load. Consequently, the girders still behave essentially as simply supported, their ends still rotate under live load, and the link slab must have sufficient flexural deformation capacity to accommodate those rotations. However, the link slab is stiff axially, and will connect the top flanges of the girders, so any girder rotation will cause the bottom flanges to move longitudinally relative to one another. For this free movement to occur, the bearings under at least one of the spans must permit longitudinal movement. This may require replacing, or at least modifying, some bearings. In steel girder bridges, some states (e.g., Virginia) also cut off the shear studs near the ends of the girders to increase the effective length of the link slab and reduce the bending strains in it (Figure 2.4-4).

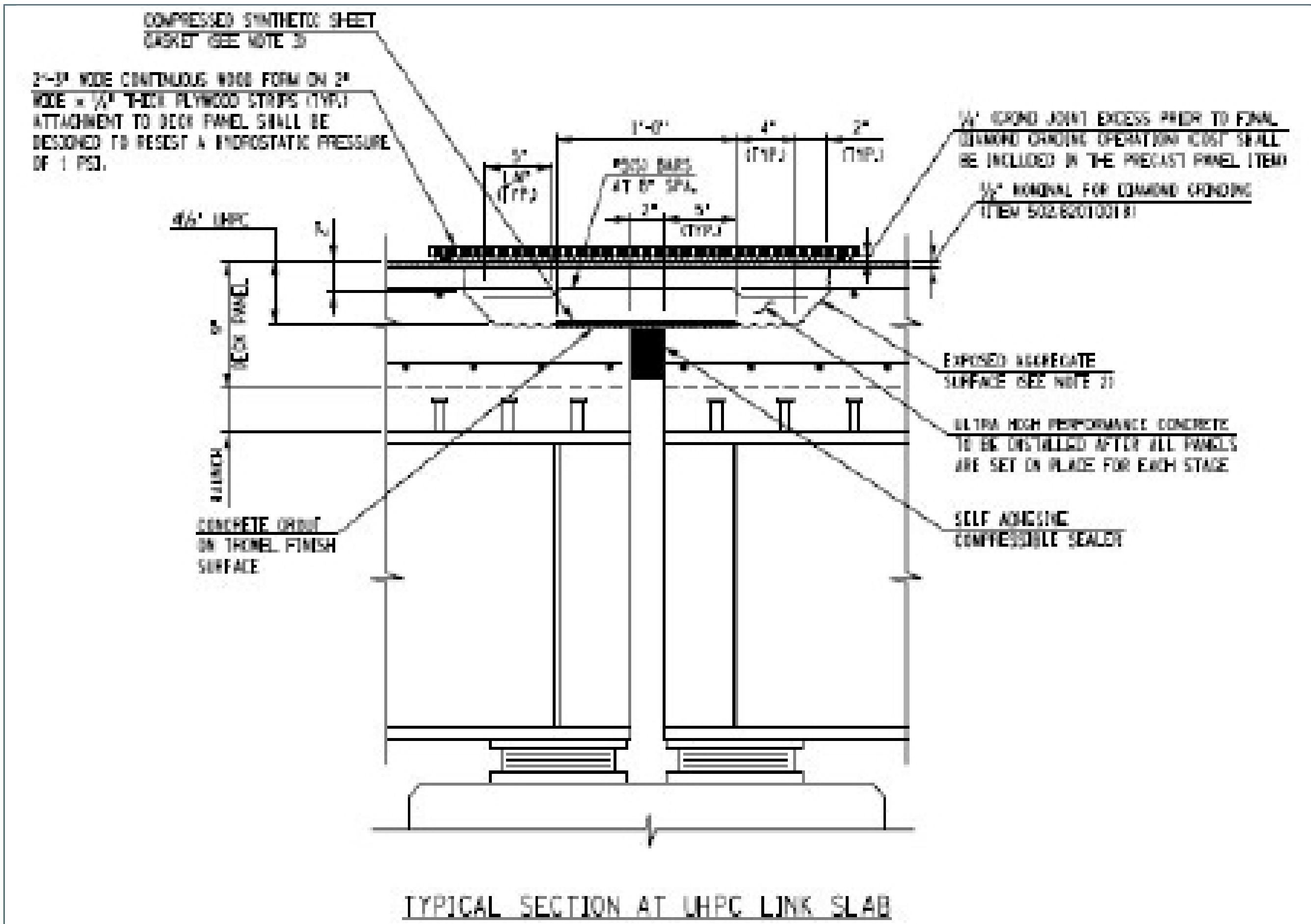

Figure 2.4-3. Link slab

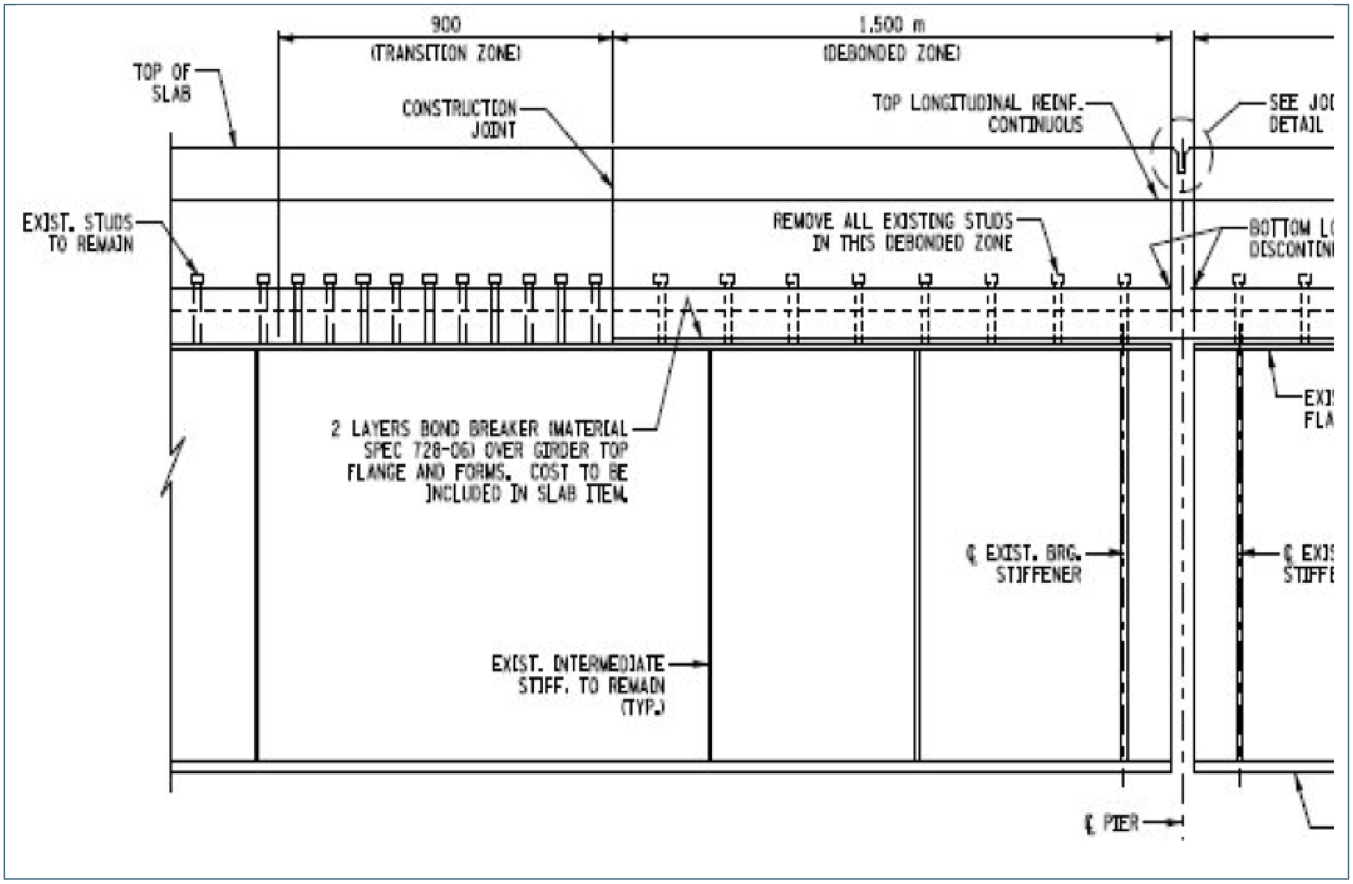

Figure 2.4-4. Shear studs removed from girder end (VA)

Link slabs create axial continuity of the girders at the piers and force all the longitudinal movement to the end of the bridge superstructure.

Link slabs are used for both new construction and retrofit, and a choice of materials exists. Virginia uses High Performance Concrete (HPC) if the whole deck is being replaced, because the deck is anyway made from HPC. They use Ultra-High Performance Concrete (UHPC) if just the joint region is being re-built, and New York is considering doing the same. However, other states believe that UHPC is too stiff and strong to accommodate the required bending deformations, and might lead to cracking or even failure at the ends of the link slab where it is joined to the deck concrete. Another alternative is to use Engineered Cementitious Composites (Li, 2003) which contain a significant amount of fiber and have been used successfully for the purpose.

All of these systems (link slabs and pier diaphragms to create live load continuity) avoid placing a joint over the pier, thereby preventing leakage onto any bearings below.

Louisiana offered some more detailed thoughts on link slabs, summarized here.

At an internal pier, four primary choices exist:

- Install a joint,
- Install a link slab,

- Install a continuity diaphragm,
- Install a seismic diaphragm.

A continuity diaphragm connects the girder ends for bending and thus makes the superstructure continuous for live load. It is not wide enough to provide much torsional resistance, and therefore cannot be relied upon to transmit girder end moments, caused by longitudinal loads such as braking forces, along the cross-beam and down into the columns.

A seismic diaphragm is functionally similar to a continuity diaphragm, but is wide enough to be torsionally stiff and strong, and so is able to transmit girder end moment along the cross-beam as torsional moments, then down into the column as bending moments. The advantage is that the longitudinal (seismic) forces can be resisted by the columns in frame action, rather than as cantilevers. This choice reduces the moments to be carried by the foundation and, thus, foundation costs. However, the diaphragms are larger, and the cross-beam may also be wider, than for a non-seismic continuity diaphragm.

However, if the girders are made continuous, the superstructure becomes statically indeterminate and moments are introduced by thermal gradients and by camber growth due to creep and shrinkage. Those actions typically cause bottom tension stress in the girder and may cause a crack at the bottom of the connection to the diaphragm and some partial debonding either side of the crack. The effectiveness of the intended live load continuity is then in doubt. The negative effects of camber growth could be ameliorated by delaying casting the diaphragm for long enough for the creep and shrinkage to be largely complete (say, six months or a year). However, the construction schedule would usually not allow that, and delay anyway does nothing to lessen the adverse effects of daily thermal gradients.

Use of a link slab avoids these problems, because the link slab is too flexible in rotation to induce a significant end moment in the girders. Camber growth and thermal gradient effects can thus take place relatively freely, and no damage is induced, provided that the link slab can accommodate the relative rotations. However, the link slab is stiff enough axially to force all the longitudinal expansion displacements to the ends of the bridge, and avoids the need for a joint at the internal pier. Louisiana has concluded that link slabs provide the best compromise for the state's conditions, which include large thermal gradients but no seismic demands. In other states with, for example higher seismic but lower thermal loadings, the optimal solution may be different.

At the end of the bridge, the goal of a "jointless" system is to move the joint to a location behind the abutment rather than in front of it, in order to avoid leaks onto the bearings and girder ends. For shorter bridges with modest movement demands, this may be done by using an "integral abutment", illustrated in Figure 2.4-5. The superstructure (deck and girders), the back wall and stem are all cast so they behave as a monolithic unit. Longitudinal movement is accommodated by placing a compressible material behind the back wall, and by bending of the piles supporting the stem. A temporary bearing carries the girder load during construction, but is no longer needed in service.

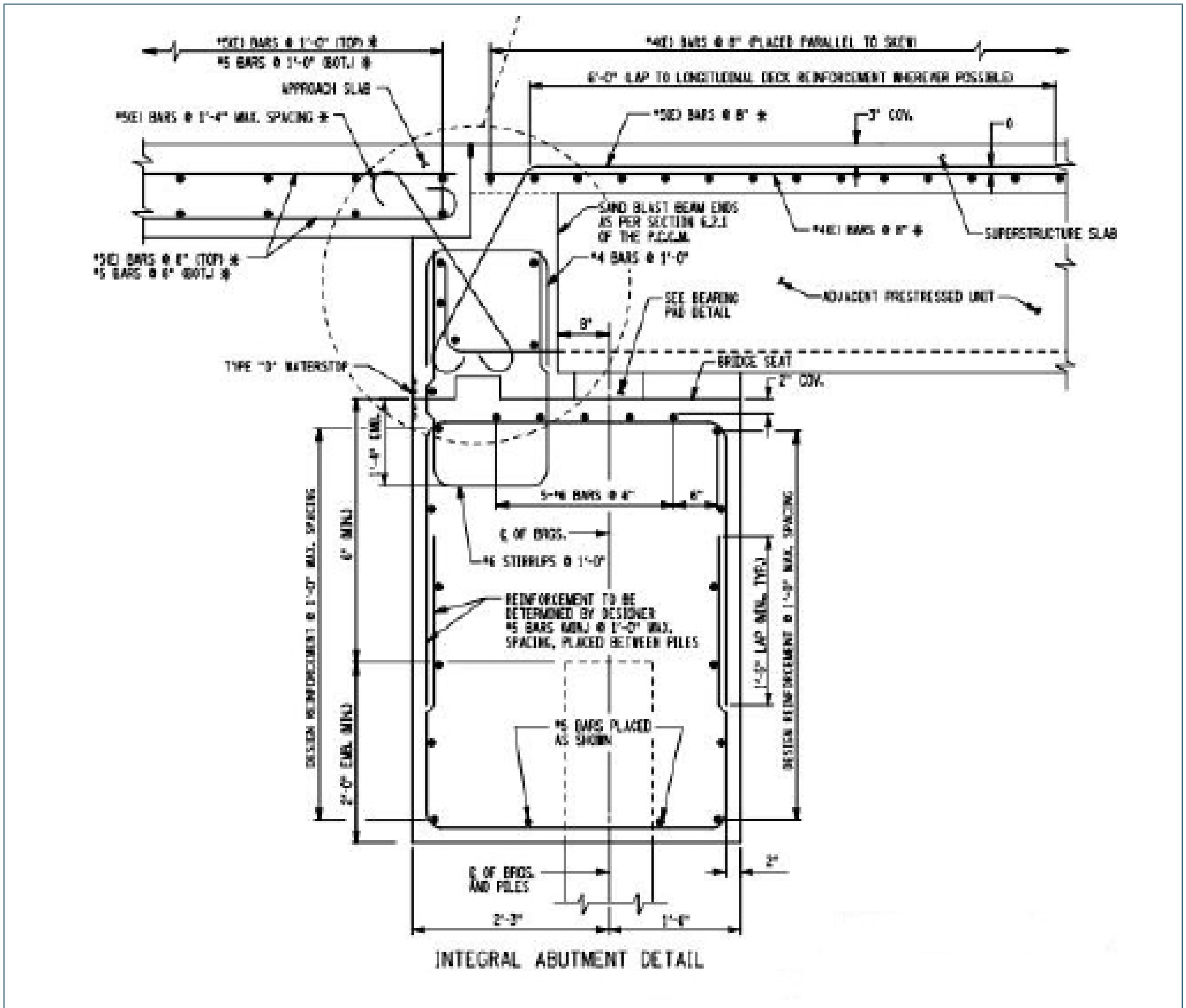

Figure 2.4-5. Integral abutment

The approach slab is usually connected to the back wall by a joint that allows some rotation but little longitudinal movement (Figure 2.4-6). The expansion of the bridge occurs at a joint at the remote end of the approach slab, where it and the pavement both rest on a "sleeper slab" slab (Figure 2.4-7). The approach slab usually slides on the sleeper slab, although in some cases the approach and sleeper slabs are cast together, and both slide relative to the supporting fill and the pavement (Figure 2.4-8). The joint between the sleeper slab and the approach slab is referred to as a "pavement relief joint". Because the joint is located over fill or a sleeper slab, no elements susceptible to corrosion damage lie beneath it, so leakage is less of a problem than with an in-span joint. If an overlay is applied later, it must be discontinuous over the joint. Louisiana has had problems with pavement relief joints blocked by overlays, and has developed a special detail to prevent them.

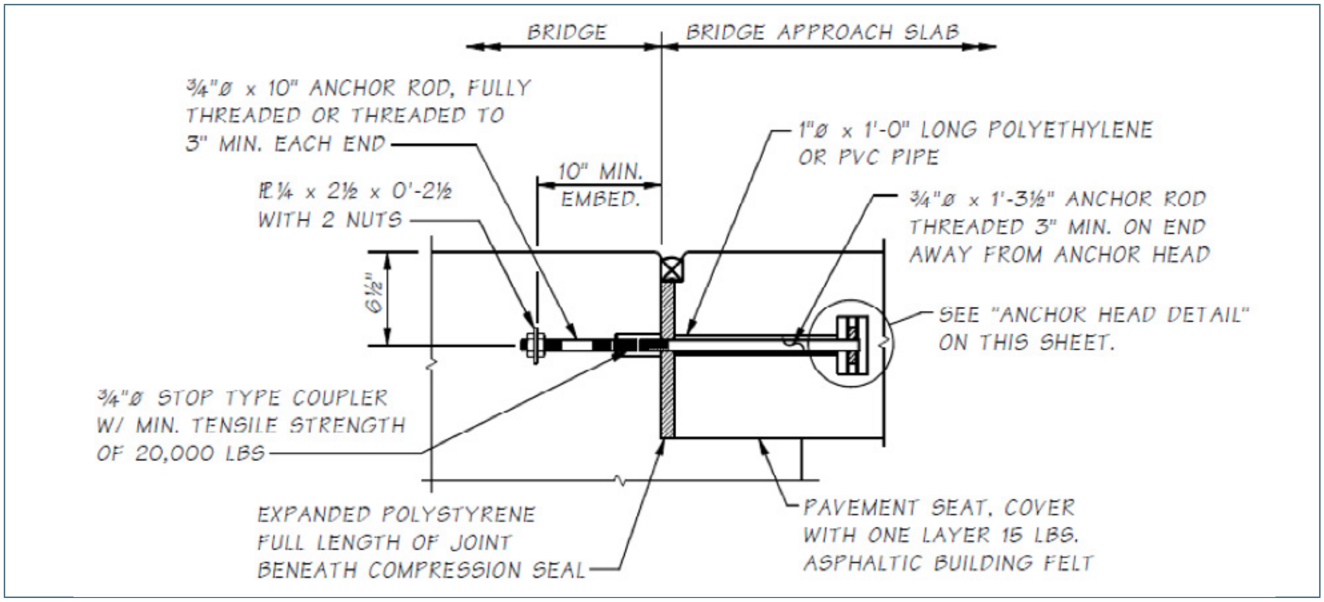

Figure 2.4-6. Approach slab connection detail.

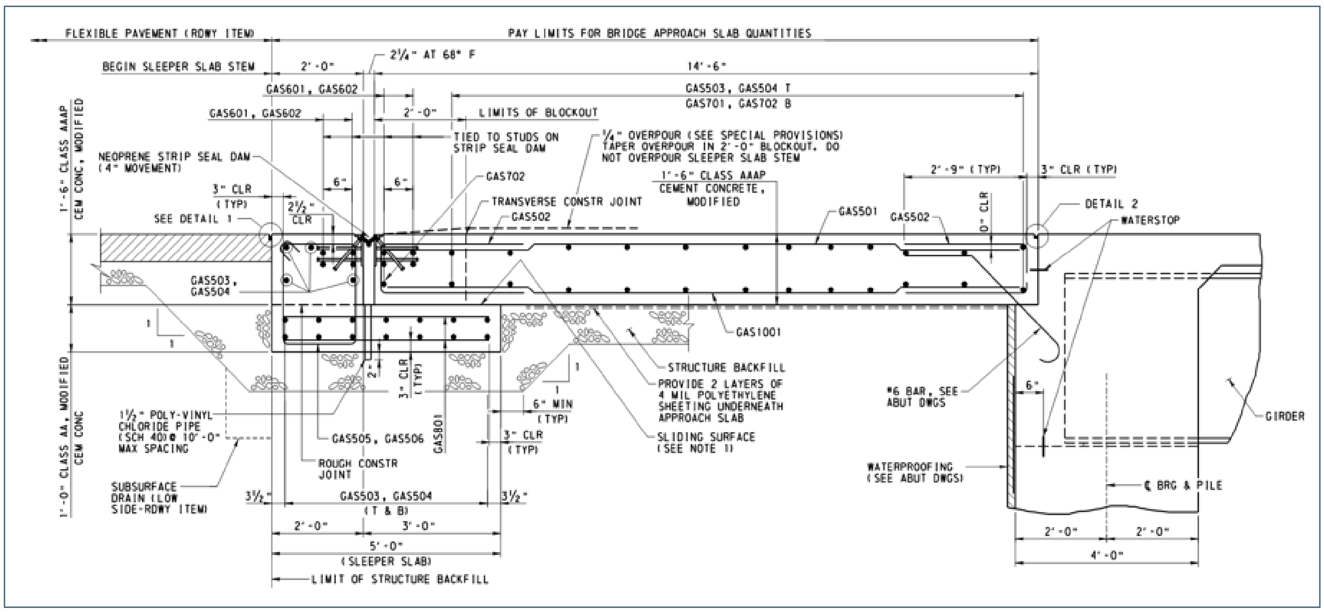

Figure 2.4-7. Sleeper slab detail

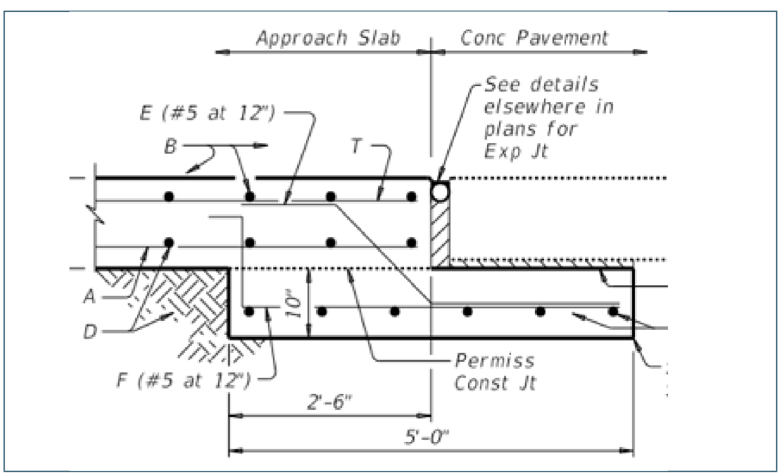

Figure 2.4-8. Alternative sleeper slab detail

The use of an integral abutment is limited to cases in which the expansion is small enough to be tolerated by bending of the piles. For example, Pennsylvania has used micropiles on integral abutments for their greater flexibility, and New York requires a minimum pile length of 10 ft. Minnesota uses integral piers only for slab spans, which are inherently short. Minnesota also does not use integral abutments at stream crossings, because of the possibility of scour.

Expansion of the bridge causes negative moments in the deck and back wall, and requires top reinforcement in the deck, extended for 20% of the span length (New York). Because some components (e.g., the girders and back wall) move, and others (such as the wing walls) do not, provision must be made for relative movements between the affected elements. Figure 2.4-9 shows an example of failure to accommodate differential movement.

Figure 2.4-9.Concrete cracked by failure to account properly for movements. (NY)

For bridges with larger expansion demands, a “semi-integral abutment” can be used (Figure 2.4-10 and Figure 2.4-11). Here, the superstructure and back wall are monolithic, but bearings separate the girders from the abutment stem, and support them. The back wall lies behind the abutment stem, and extends below the top of it. The approach slab and joint details are similar to those of the integral abutment. The main difference is that, in the semi-integral abutment, the stem and foundation do not need to move, and the relative movement between superstructure and foundation is accommodated by the bearing. The bearing is needed both during construction and in service, but is protected from leaks by the fact that the back wall lies between the bearing and joint.

The backfill must be prevented from entering the space between the back wall and the stem, and New York uses a preformed closed cell foam for the purpose. They also recommend not back filling behind the back wall until after the deck has been cast, lest the backfill provide resistance to rotation and lock in the girder camber. This is, of course, a question of the relative rotational stiffnesses of the girders and fill. The backfill has to be in place before the approach slab can be cast, so the time of backfilling affects the contractor’s casting sequence.

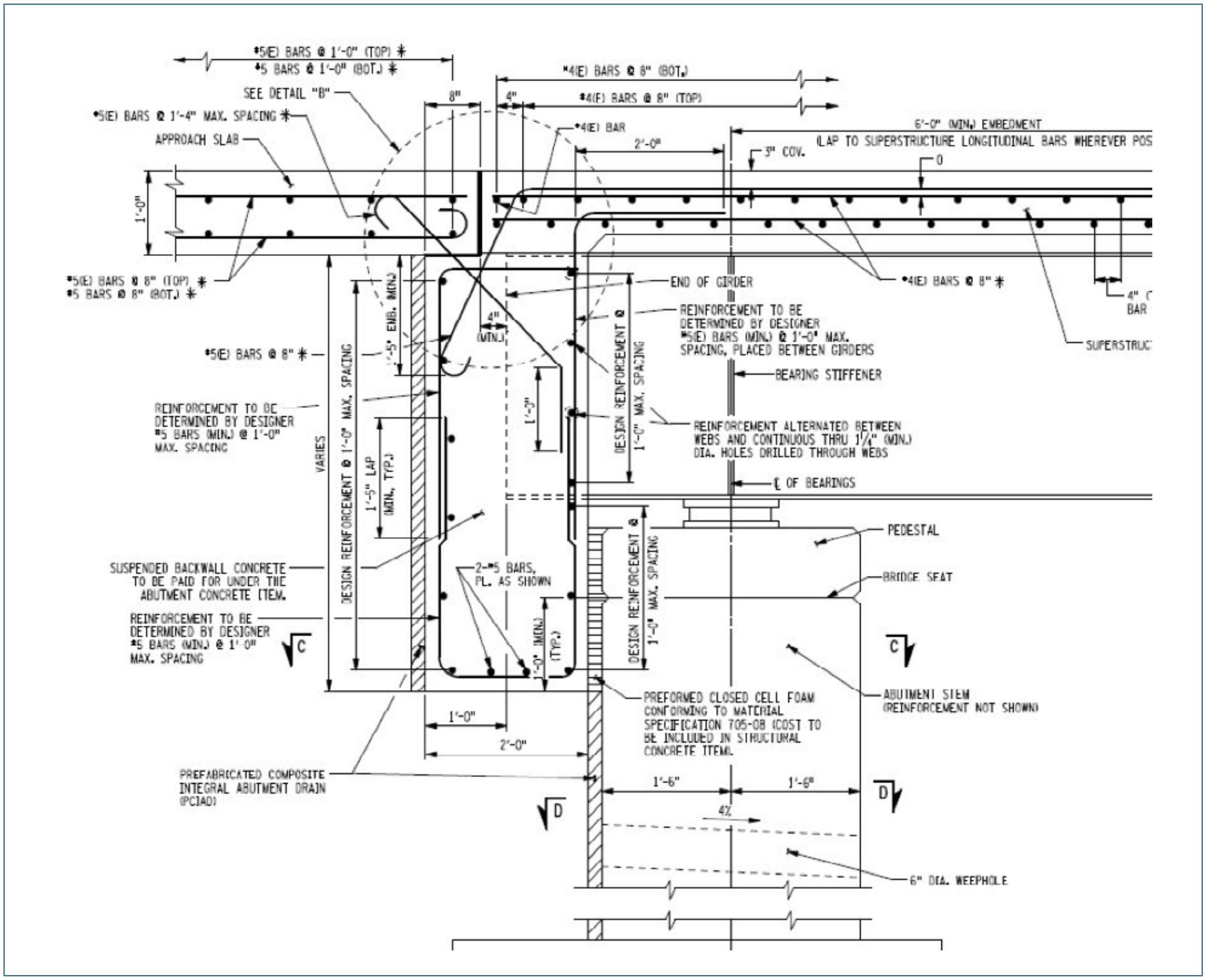

Figure 2.4-10. Semi-integral abutment – cross-section

Figure 2.4-11. Semi-integral abutment - photo.

Virginia has developed a variant on the semi-integral abutment, shown in Figure 2.4-12. The joint lies directly behind the back wall, and a trough beneath it diverts any water directly away from the bridge. The state reports that the abutment is working successfully. However, it deposits the water from the bridge to the adjacent ground. Minnesota noted that this would be unacceptable in their state for environmental reasons, especially over waterways.

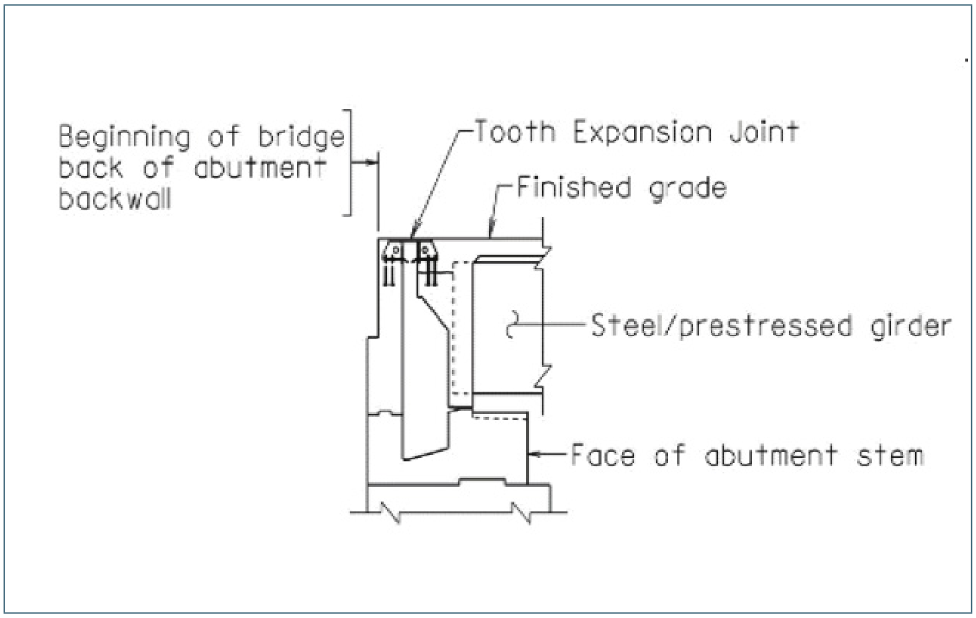

Figure 2.4-12. Virginia abutment: cross-section

Figure 2.4-13. Virginia abutment- photo

75% of the attending states reported either a policy of minimizing the number of joints or a move of some sort towards jointless bridges, albeit with some limitations, such as on span length, expansion distance, girder splay, bridge radius or skew angle. Some states (e.g., Michigan and Oregon) impose requirements on the abutment pile details, such as using steel and orienting them for maximum flexibility in the expansion direction. NCHRP Project 12-100 (Shenton, 2017) also found that more than 50% of all the states responding to their survey preferred jointless bridges. For example, Virginia moved to jointless construction in 1980, and now uses the Virginia abutment and pier. Utah tries to use integral or semi-integral systems wherever possible. The state has been a leader in the use of Accelerated Bridge Construction, and has done more work with SPMTs (Self-Propelled Modular Transporters), and lateral slides than any other. In those cases, the joint details at the end of the bridge are critical and must be considered when planning the ABC features and the state frequently uses semi-integral details for them. Pennsylvania has a primary goal of getting the joints off the bridge, to protect the bearings. To support this policy, the state notes that the troughs (for modular and finger joints) clog up, even if they have a significant slope, and especially in winter when ice contributes to blockages. They use sleeper slabs, where the joint is usually a strip seal, but occasionally a finger joint ("tooth dam") if the movement is large.

However, some conditions do not favor integral abutments. Bridge superstructure expansion in skewed and curved bridges causes some movement parallel to the support, and thus some shearing motion at the joint, which has a negative impact on the use of integral or semi-integral abutments. Minnesota has been using integral systems since 2000, and the state quotes in order of preference integral, semi-integral and then parapet bridge systems, but also provides a graph (Figure 2.4-14) that shows the combinations of span length and skew for which integral abutments are permissible. Other states also impose limits (e.g., 45 degrees skew and 300 ft span in New York.)

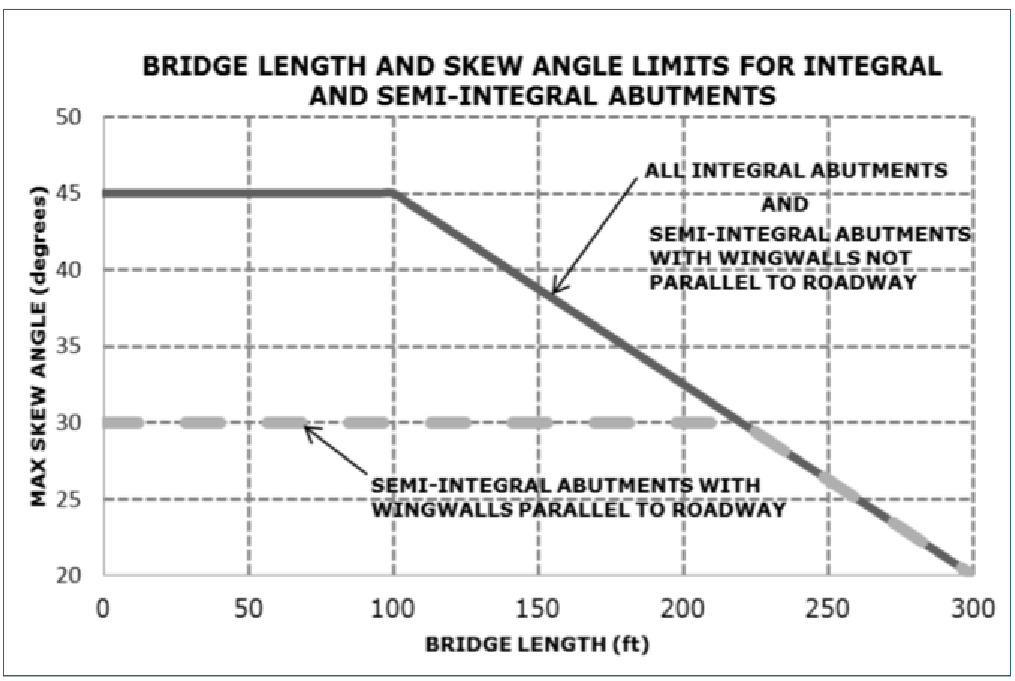

Figure 2.4-14. Skew and span limits on the use of integral abutments (MN)

The distance between expansion joints depends on the movement capacity at the piers and abutments. Virginia reported much the largest distance between joints for a land-based bridge, with 3000 ft between expansion joints. (In Washington State, the joints in the SR 520 floating bridge over Lake Washington are 7500 ft apart, but the conditions there differ significantly from those of a land-based bridge.)

All of the states reported using link slabs in some context. Some (Utah, Oregon) use them primarily for retrofit, as a means of eliminating a joint. The details, including concrete materials, the presence of crack control joints, reinforcing details, etc., varied from state to state.

The design of a jointless system may be different for new construction and rehabilitation. For new construction, the consensus was that integral abutments should be the first choice, provided that the necessary criteria (expansion length, skew angle, soil conditions, etc.) were satisfied. For rehabilitation work, it is necessary to review and understand the articulation of the whole bridge. The possibility exists that, by locking up a previously active joint at one location, a force or displacement would be activated in a different location, where it could cause damage.

2.4.2 Joint Type Selection and Design

2.4.2.1 General

Design of joints differs from that of bearings. A bearing must carry loads, and its design therefore follows the principles of structural mechanics. By contrast, a joint must accommodate movements, and its ability to do so depends mostly on the strain capacity of the constituent materials, usually under cyclic loading conditions, and not on known forces. The movement demands may be difficult to quantify precisely, especially in skewed or curved bridges. Joints are also subject to poorly defined loadings, such as impact by sharp objects (e.g., gravel in a strip seal or a snow-plow blade hitting edge armor), and by the deleterious effects of temperature swings and chemicals such as deicing salts. These features mean that joints are not amenable to formal design procedures, but depend more heavily on field experience.

The result is that the Engineer of Record (EOR) specifies the movement range required, and possibly the type of joint, but the details of the joint are largely decided by the supplier. These details vary with the complexity of the joint. At the simple end of the range, such as a compressions seal, the supplier will provide a table giving the maximum and minimum joint width, and the range of permissible installation widths, for that specific product. The EOR then picks from the supplier table. For more complex joints, such as modular joints, the supplier chooses the element sizes and connection details such as bolts and welds, and delivers the finished product on a turn-key basis, but remains responsible for the design.

One outcome of this arrangement is that the responsibility for joint life is then shared between the supplier, who provides the design, and the owner, who is responsible for cleaning and maintenance. The feedback loop between design and responsibility for field performance is thus broken. The division of responsibility is further muddied if the owner makes modifications to the supplier's joint design. For example, several states have welded snowplow ramps to the armor of strip-seal joints with the goal of preventing the plow blades from catching and damaging the armor

(Figure 2.4-15). These ramps have usually been beneficial, so the question of design responsibility has not arisen. In another example, Oregon modified the stirrup detail in a modular joint, substituting a bolted connection for the original welded one after repeated fatigue failures (Figure 2.4-16).

Figure 2.4-15. Snowplow ramps (OR)

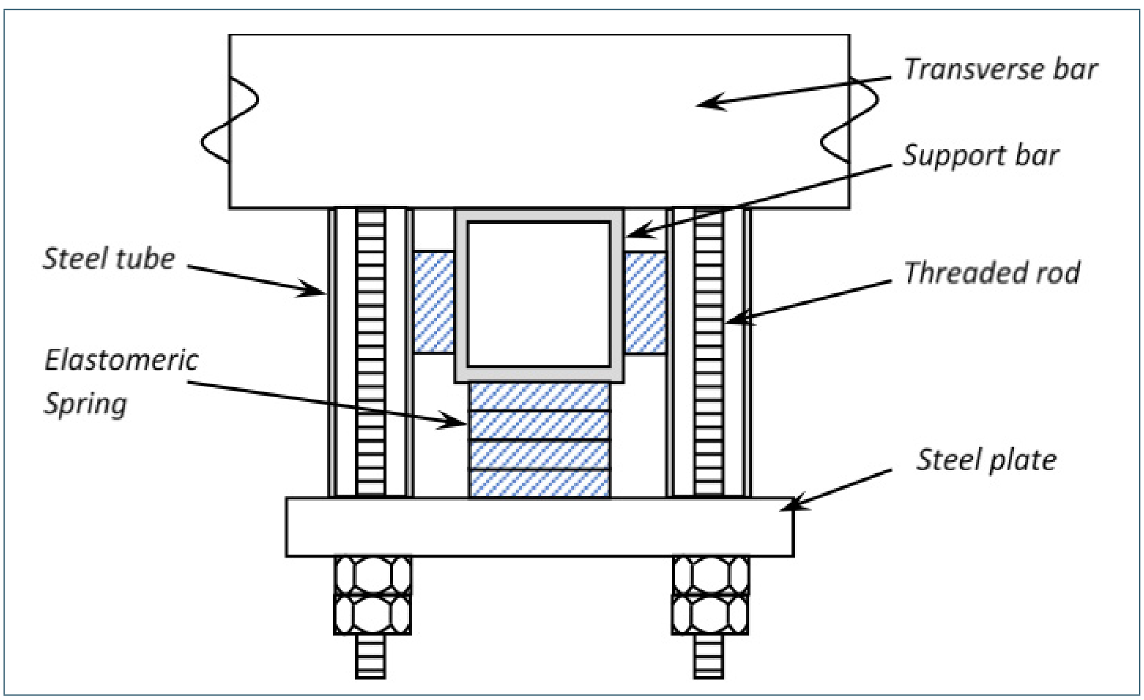

Figure 2.4-16. Bolted fix for failed welded stirrup in modular joint (OR)

Several features of design are common to all joints. For example, movement capability must exist in the barrier as well as the superstructure. While this appears obvious, ensuring that the capacity is provided requires that someone is responsible for doing so (Figure 2.4-17). Skewed abutments can also lead to displacements in unexpected directions, most often a component parallel to the joint caused by a rotation of the whole bridge superstructure about a vertical axis. For strip seals, several participating states and some suppliers recommend increasing the nominal movement capacity in the longitudinal direction to account approximately for the transverse component.

Figure 2.4-17. No joint in barrier (MI)

2.4.2.2 Joint Type Selection

Joints are available in a wide variety of types. Categorization is difficult not only because of the breadth of types, but also because a different name has historically been used for the same product in different regions. For example, what to one person is a “finger joint”, is to another a “tooth dam”. Furthermore, the different types are often referred to by a commercial name, rather than a generic description. This panoply of nomenclature can be confusing. Here, the different joint types are classified here according to the way that they work. They are described briefly in this section. More detail on performance follows in Sections 2.4.2.3 to 2.4.2.11.

Type	Movement	Pref	Section
Open	S	X	2.4.2.3
Plug	S	X	2.4.2.4
Poured Seals	S	Y	2.4.2.5
Compression seals	S	X	2.4.2.6
Gland based	M	Y	2.4.2.7
Bolt-down panels	M	X	2.4.2.8
Steel sliding	M	X	2.4.2.9
Finger	L	X	2.4.2.10
Modular	L	Y	2.4.2.11

Table 2-1. Joint Types and Movement Ranges

The different joint types are classified according to movement ranges, as indicated in Table 2-1. Details for each type are given in Sections 2.4.2.3 to 2.4.2.11, as indicated in the table. Movement capacities are shown in the table, but depend on several factors such as the environmental conditions in the state, the bridge skew, etc., so they are not precise and are presented simply as small, medium or large. Those descriptions correspond approximately to the ranges 0" to 2", 2" to 4", and greater than 4". Note that those values define the movement, i.e., the difference between the minimum and maximum opening widths. The nominal joint size may be controlled by other factors, such as the need to maintain a certain minimum compression strain in the elastomeric element of a compression joint, even when the joint is fully open. That information is usually available from the manufacturer. The "Preferred" designation in the table indicates the most widely used joint type in each movement range, according to the participants written responses to the amplifying questions. However, as discussed below, considerable variation was found among states' preferences.

In the small movement range, many options exist. Poured silicone seals with a foam backer rod enjoyed the widest use, but preformed silicone-on-foam joints), neoprene compression seals and asphalt plugs were all quite widely used. The small movement range offers the greatest array of products, and new ones are also being developed, so the findings expressed here may change in the not-too-distant future. However, it is also possible that the move towards jointless bridge will lead to bridges that contain fewer joints, each with larger movements, in which case the need for all types of small movement joints will diminish.

In the medium movement range, mechanically bonded strip seals with neoprene glands were the most widely used. They have generally performed well. The glands can be patched as a first level of repair (perhaps every five years), new glands can be installed reasonably simply (perhaps every ten years), and, after perhaps 25 years, the whole joint, including the steel extrusion, can be broken out and replaced. The first two types of repair are relatively non-invasive and economical. The gland can be anchored either mechanically (in a slot in the metal extrusion) or adhesively. Mechanical bonding was found to be the more common, on the basis that replacement is easier.

In the large movement range, the only options up to now have been finger joints and modular joints. Both finger joints and modular joints have drawbacks, but those of the modular joints, which were mostly associated with fatigue of the welded elements, were largely resolved by a 2004 change in the fatigue requirements of the AASHTO Specifications. Most states expressed a preference for modular joints, although finger joints continue to be used in cases where their vulnerabilities are low (e.g., in a straight bridge with low skew). Preferences differed among states; California no longer allows finger joints, Louisiana uses them routinely, and other states (e.g., Oregon) use both finger and modular joints for large movements. The California seismic joint (Section 2.4.2.9), based on sliding plate technology, can also accommodate large movements both longitudinally and transversely.

Expected life is an important criterion in selecting a joint type. Almost all states reported that longevity depended strongly on the quality of the original installation and the level of subsequent maintenance, so it is not surprising that the life expectations offered by the different states varied widely for each joint type. The following common failure modes were mentioned by several states:

- Strip seals. If the joint is too narrow, replacing the gland may be impossible. Gland damage due to debris, such as anti-skid materials, and other sharp objects (e.g., Pennsylvania). Failure of welded studs supporting end dam (e.g., Texas)
- Pourable joints. Bond failures (e.g., Utah). Traffic impact if poured too high.
- Tooth dams. Cleaning the trough (and teeth) is difficult, so the trough fabric experiences damage.

2.4.2.3 Open Joints

Open Joints. (Figure 2.4-18). These consist of an opening between the ends of two slab segments, which are usually armored with steel angles to protect the corners of the deck. They contain no sealing element, so they leak. Consequently, they are almost never used in new construction. A few remain in older bridges, but are for the most part scheduled to be eliminated, by a link slab, a diaphragm, or some sealed joint type.

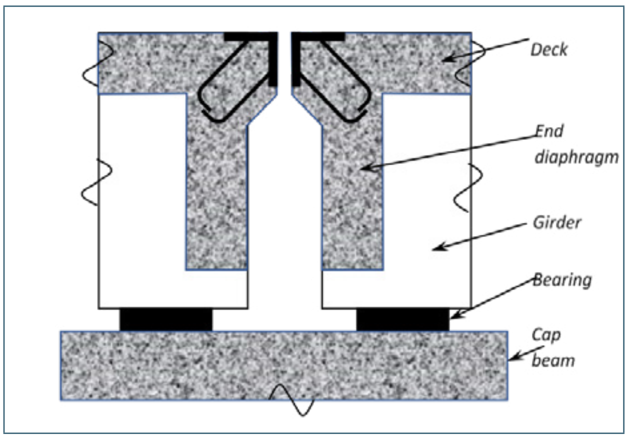

Figure 2.4-18. Open Joint

2.4.2.4 Plug Joints

In plug joints (Figure 2.4-19), a section, or plug, of deformable material, such as asphalt, fills the space between the ends of two slab segments, and bonds to the slab concrete. A sliding plate beneath the plug prevents it from falling through the open gap between slab ends and allows the plug to stretch as the joint opens. A plug joint typically has a low first cost, but is not very durable.

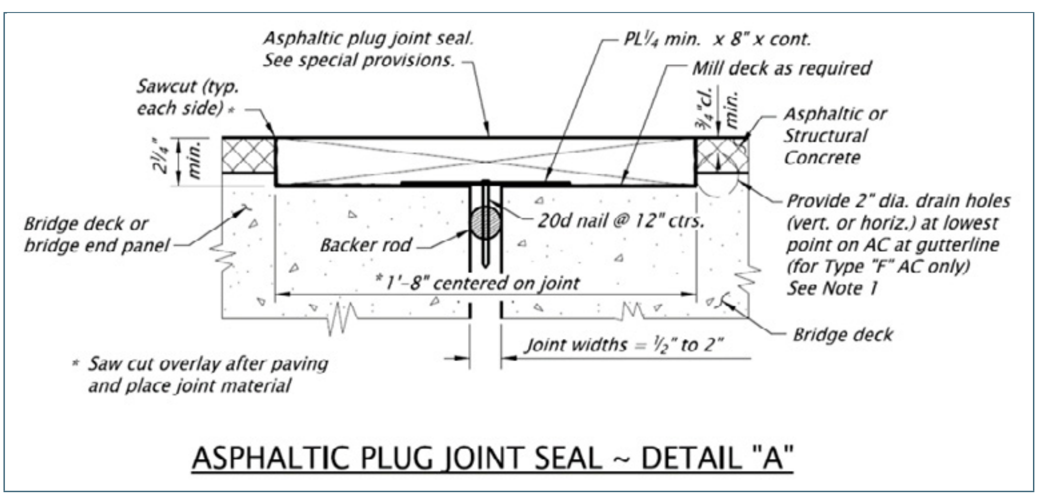

Figure 2.4-19. Asphaltic Plug Joint

A plug joint is typically used as a retrofit measure, in a case where a previous joint has failed. If an asphalt overlay is being placed at the same time, the plug is usually made from asphalt as well. Otherwise, the plug is typically made from an elastomeric concrete material. Both asphalt and elastomeric concrete display time-dependent properties, which allow them to behave as flexible materials under slowly applied loads, such as thermal, but to remain stiffer under short-term load such as traffic.

The plug material is anchored to the substrate at the ends of the joint, and is free to stretch in the center, over the sliding plate. States have reported different levels of success with them. In snowy states, they offer the advantage of no exposed corners and a lower likelihood of snow-plow hits. Virginia regularly uses an elastomeric plug joint for spans less than 100 ft, but Pennsylvania reported successive failures of three different joint types, including one asphaltic plug joint, before finally achieving success with a strip seal. Texas has found that asphalt joints are not very durable, and that they suffer in hot weather (when they suffer from rutting) and under heavy truck loading. Oregon found that asphalt plugs, coupled with an ACWS, were working well for short spans with low skew, but polymer concrete plugs, used with longer spans and no overlay, were deteriorating. Where polymeric concrete is used, its surface may be inherently slippery, in which case some fine aggregate may need to be applied to the surface after pouring, to increase friction.

2.4.2.5 Poured Seals

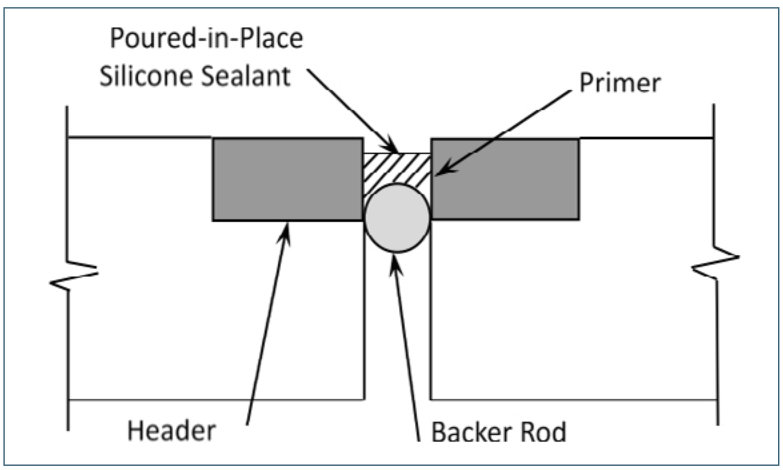

Figure 2.4-20. Poured Seal (with backing rod)

In poured Seals (Figure 2.4-20), a foam backer rod is placed between the two slab ends, which are then primed, and a pourable elastomeric material (usually silicone) is poured to fill the space. The foam provides the support during pouring, and the silicone provides the waterproof seal. The success of the joint depends on the workmanship of the installer, particularly in cleaning and preparing the faces of the concrete and in installing the seal the appropriate distance down form the top of roadway, to avoid tire damage.

Poured seals are suitable for narrow joints with small movements, and were reported to have given good performance in that range. The most common seal material was silicone. For example, Michigan now uses only silicone and polyurethane, and has discontinued their previous use of hot rubber-based products. Louisiana is also looking into the use of polyurethane.

2.4.2.6 Compression Seals

In compression-type seals (Figure 2.4-21), a pre-formed elastomeric element is pre-compressed and secured between the slab ends. If it is correctly installed, it will still be in modest pre-compression even (in cold weather) when the joint is at its widest. Several elastomeric element types are possible. Although the difference types of pre-formed seal are functionally similar, they have different performance characteristics.

The traditional one is an extruded elastomer, as shown in Figure 2.4-21. This joint type achieves the necessary movement by distortion of the web of elastomeric elements. Compression seals originally relied on the pre-compression alone to keep them in place. Most manufacturers' literature now recommends an adhesive as well.

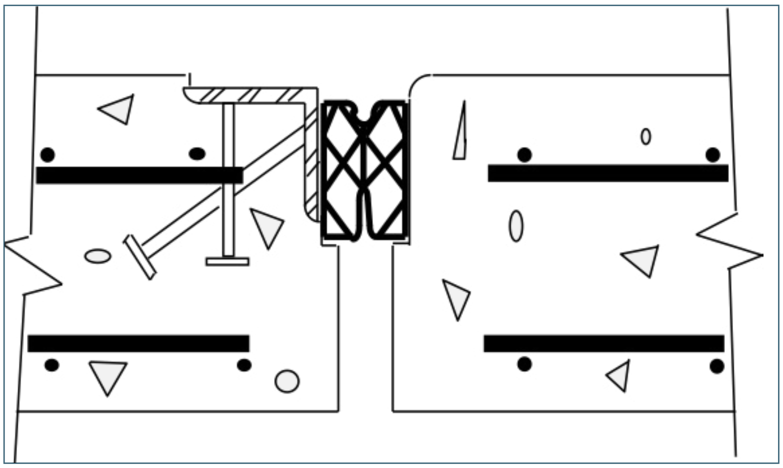

Figure 2.4-21. Compression seal, with armor (left), without armor (right). [WA]

Other, foam-based seals are also available, (Figure 2.4-22 and) that are functionally similar in that they are preformed, fill the space, are installed with some level of pre-compression, and rely in part on adhesive to keep them in place. Foam-based seals are included here with compression seals because of their functional similarity.

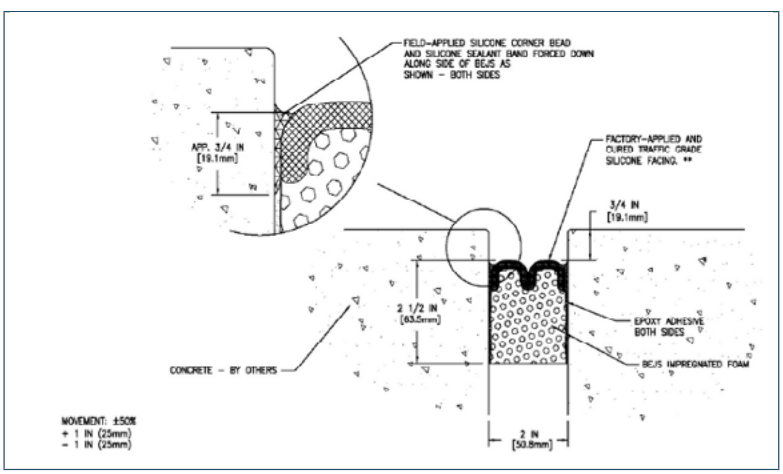

Figure 2.4-22. Preformed (Impregnated open cell) foam seal with silicone facing

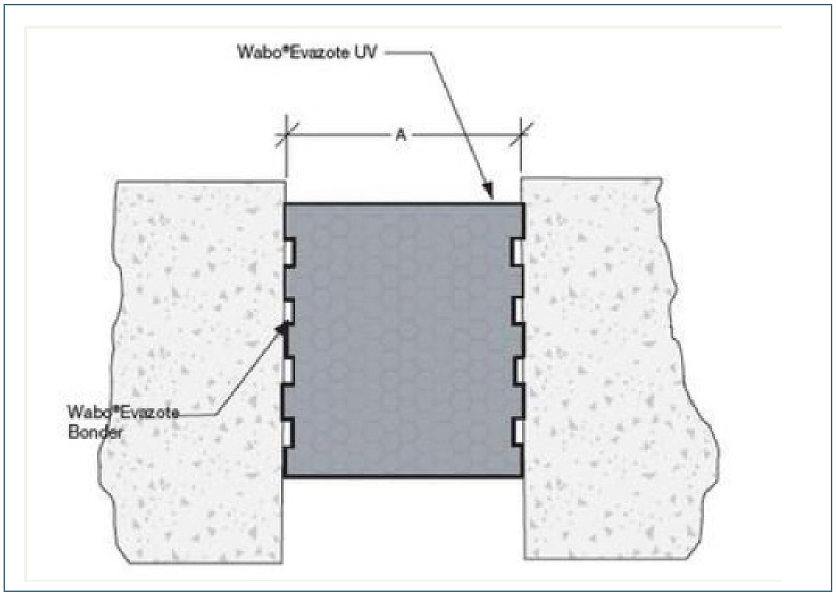

Figure 2.4-23. Pre-formed compressible closed cell foam seal. [WA]

Pre-formed rubber elements (Figure 2.4-21), which are what most agencies mean by "compression seals" are relatively easy to pre-compress and install. The rubber allows the seal to expand instantaneously and therefore to be in good contact with the concrete while the adhesive sets up. However, the rubber deteriorates over time due to UV exposure. The seals are also quite heavy, and Texas reports that the combination of weight and any imperfections in the adhesive bond eventually cause them to fall out. Several states (e.g., Minnesota and Louisiana) have now stopped using them because of their poor performance.

Silicone-coated foam seals (Figure 2.4-22) also require pre-compression and are consequently quite easy to install. However, the foam expands only slowly, sometimes so slowly that the adhesive has partly set up before it even makes contact with both sides of the opening. This detracts from the bond quality and means that the seal may eventually fail by squeezing out of the opening. Traffic can also damage the silicone, as shown in Figure 2.4-24.

Figure 2.4-24. Failure of silicone-coated foam seal (NY)

Other foam seals have no silicone barrier on the top and are made from closed cell foam to make them watertight. They are typically specified to be slightly (e.g., 25%) wider than the joint at the neutral positions (Figure 2.4-25), and to be secured in place with an epoxy adhesive. Because the foam element is wider than the opening, Texas reports that installing it after applying the adhesive is difficult and messy. However, New York finds them easy to install. The material can accept some tension strain, but any such net tension strain in the foam necessarily stresses the adhesive connection as well, and risks detaching it if it was not well installed. An earlier report (Henley et al 1994) recorded unsatisfactory performance from an overcrossing on I-5. New York has also found that foam-based seals have not held up well (Figure 2.4-26).

Figure 2.4-25. Installation of foam joint

Figure 2.4-26. Foam joint pulling away. Gap too wide (NY)

2.4.2.7 Gland-based Seals

Two common examples are "Strip seals", also known as "Mechanically Bonded Strip Seals", (Figure 2.4-27) and "Pre-formed Silicon Seals", or "Adhesive Bonded Strip Seals" (Figure 2.4-28 and Figure 2.4-29). In both types, the functional element is an elastomeric gland that folds and unfolds as the joint closes and opens. The two types are distinguished by the fact that the Mechanically Bonded Strip Seal is secured by slots in the metal extrusions (the "end dams") embedded in the opposing slab faces, while the Preformed Silicon Seal is secured with adhesive, either directly to the concrete face or to steel armor. Both types are widely used.

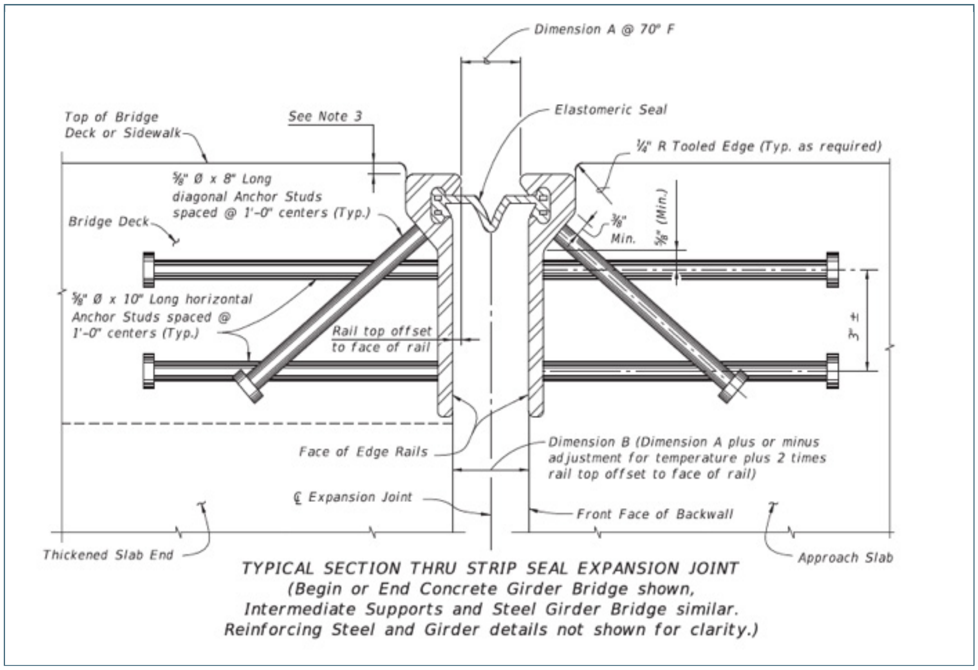

Figure 2.4-27. Gland-based seal: mechanically bonded strip seal (WABO)

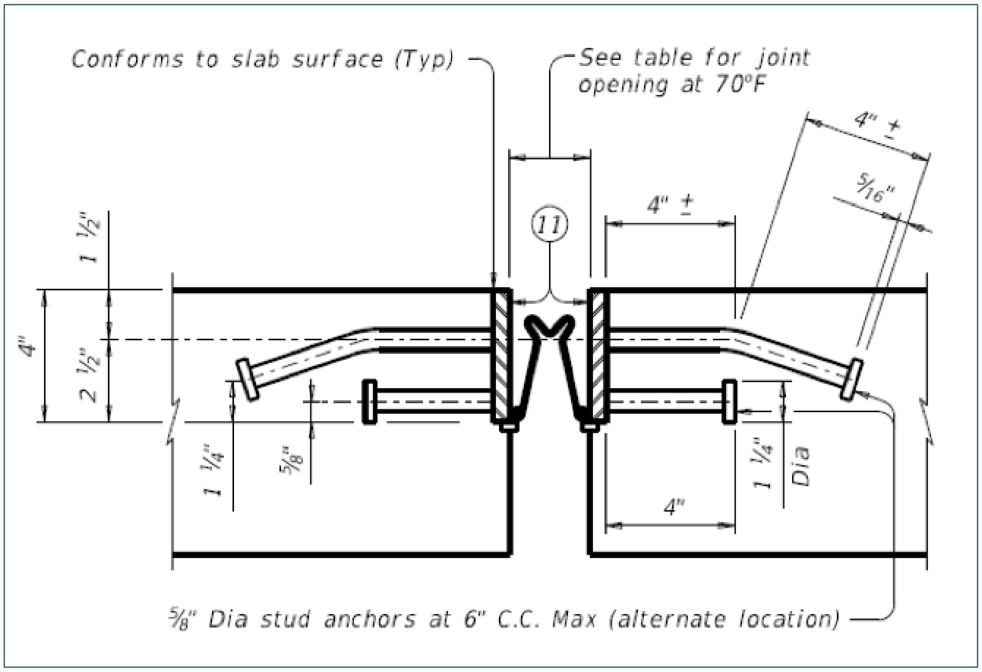

Figure 2.4-28. Gland-based seal: adhesive bonded strip seal, with armor

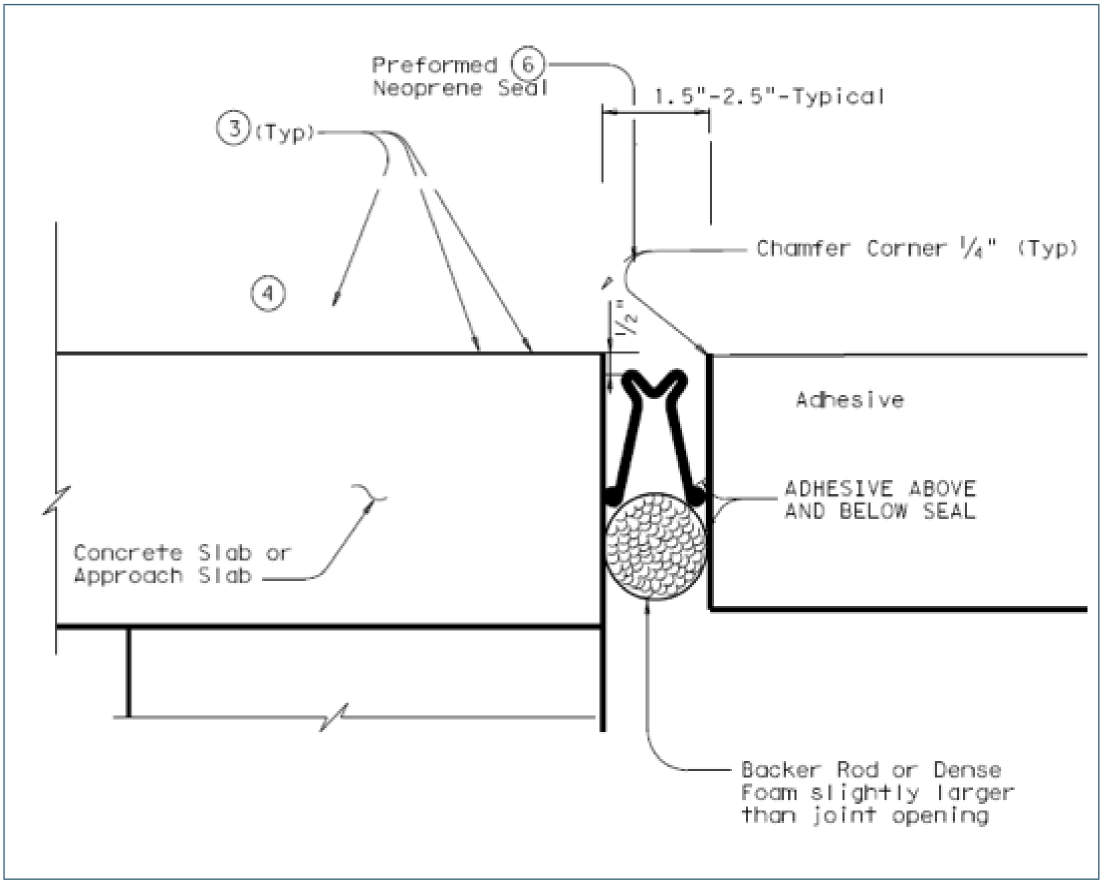

Figure 2.4-29. Gland-based seal: adhesive bonded strip seal, without armor

Gland-based seals, either mechanically bonded or adhesive bonded, were reported by many states to be their bread-and-butter seals for medium movement. No strong preference was expressed for one or the other.

Mechanically bonded strip seals offer three levels of repair, and thus provide options. First, if the gland suffers a small puncture, it can be repaired with a patch, cold-bonded to the original seal. At least one manufacturer sells a suitable repair kit, consisting of repair sheet material and adhesive. Second, if the gland is too badly damaged, but the steel extrusion is still intact, the gland can be replaced. The ease with which this can be done depends on whether the gland was bonded into the end dam and on the level of corrosion of the steel. Bonding helps to prevent the gland from escaping from the extrusion in service, and so helps to prevent leaks, but it makes removal more difficult. Opinions differed among states about whether bonding was beneficial, and DS Brown suggested that it would be beneficial to conduct a study to compare the performance of the two approaches with respect to pulling out and water leakage. Last, if the extrusion is damaged (Figure 2.4 30), perhaps by a snow-plow hit, then it must be replaced. This requires breaking out and replacing the header concrete.

Figure 2.4-30. Mechanically bonded strip seal failure: Chino Creek, CA

The expected life of the seal varies with the local environmental conditions (weather and traffic), but average ranges given were 5-10 years for gland repair, 10-20 years for gland replacement, and 15 – 25 years for total replacement. Some states (e.g., Minnesota) are considering using stainless steel extrusions to increase their life, because rust on the carbon steel extrusions in use today prevents re-glanding. They also use a minimum gland thickness of ¼". Note that re-glanding requires a minimum joint opening of 1.5 inches.

Pennsylvania reported on a curved, skewed, bridge on I-70 in which they tried successively a foam seal, an elastomeric concrete joint and a compression seal, all of which failed. The (mechanically bonded) strip seal that they installed after that has now lasted 14 years and is still performing well.

Glands should never be in tension (just varying degrees of compression). They should also be ordered to length so as to avoid a splice, which would be prone to damage. Joints in skew bridges pose additional problems, because movements parallel to the joint impose shear deformations on the gland. These can be accounted for approximately by increasing the nominal opening capacity of the gland.

Pre-formed silicone seals ("adhesive based strip seals") tavoid the difficulty of removing and replacing the gland from the steel extrusion. They use a gland similar to that of a mechanically bonded strip seal, but it is attached to the vertical face of the opening by adhesive rather than mechanically. The gland is also inverted; the cross section is an inverted V rather than the upright V typical of strip seals, and this appears to be less susceptible to damage by grit and gravel. However, several instances were reported of the seal debonding from the deck (Figure 2.4-31)

Figure 2.4-31. Adhesive-bonded strip seal failure.

2.4.2.8 Bolted-down Elastomeric Panels

In bolted-down elastomeric panels, (Figure 2.4-32), a slab of elastomer, equipped with suitable slots in the top and bottom and reinforced with internal steel plates, is bolted down to the two sides of the opening. It stretches and shortens by means of shear deformations in the elastomer, somewhat akin to the shear deformation of an SREB.

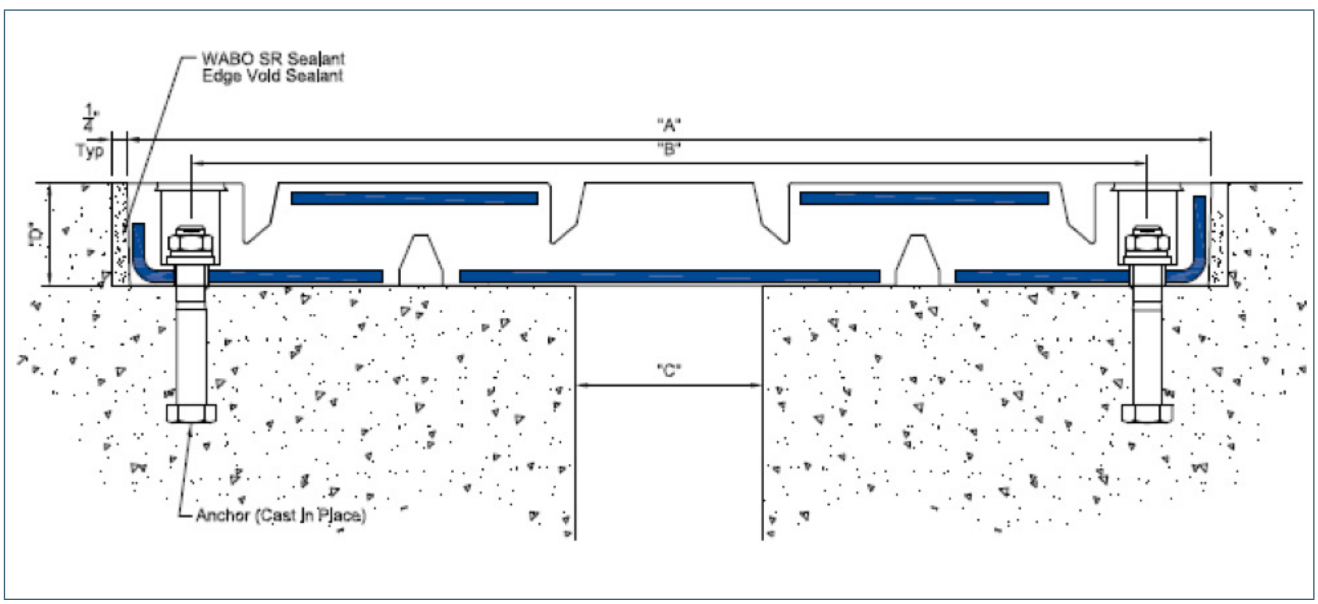

Figure 2.4-32. Bolted-on elastomeric panel

Bolted-down elastomeric panels have enjoyed less widespread use than other joint types. The elastomeric components themselves have proven relatively robust, but some states (e.g., Washington) have found that the anchor bolts fail under traffic loading. The broken bolts constitute a hazard to drivers (Figure 2.4-33), and consequently the joints are being phased out. However, California is using such panels as elements in their plate joint seal assembly (Figure 2.4-36). Those are relatively new installations, and no panel failures have yet been reported.

Figure 2.4-33. Bolt-down elastomeric panel, showing failed bolts. (LA)

2.4.2.9 Steel Sliding Joints

The two sides of a steel sliding joint (Figure 2.4-34) are armored with steel angles. A steel field plate is attached to the angle on one side, and slides on the other angle. While many of these joints were installed in the past, they are seldom used today because the connection of the sliding field plate has proved unreliable. A primary problem is leakage and consequent corrosion (Michigan).

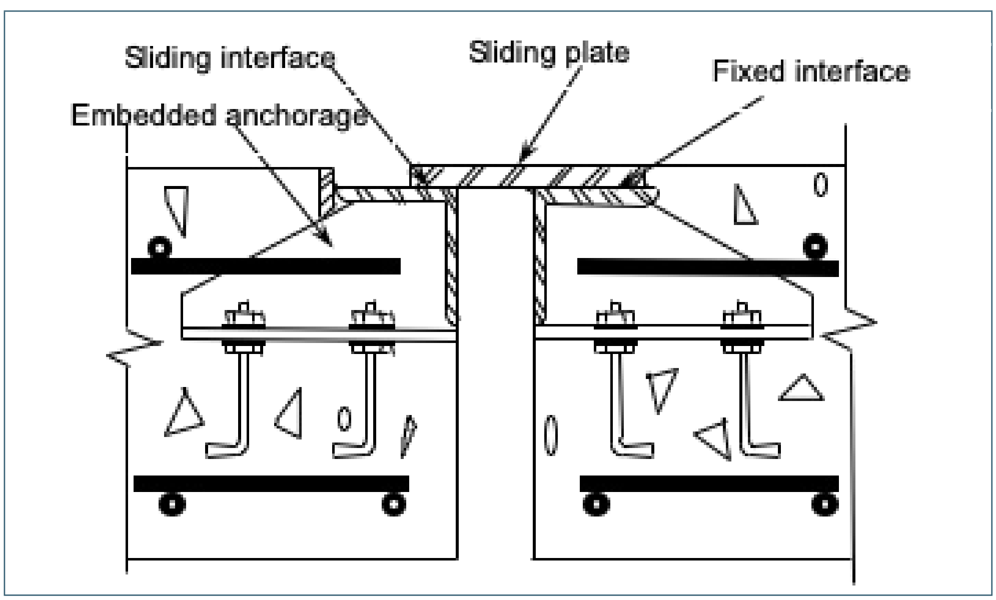

Figure 2.4-34. Steel Sliding Joint

Steel-on-steel sliding plate assemblies have largely been phased out now on the basis of poor performance. Figure 2.4-35 shows a joint in the I-5 freeway in Washington State that failed after bolts fractured. The failure caused a hazard and stopped traffic on the freeway.

Figure 2.4-35. Sliding plate failure (WA)

California has developed a new sliding plate system for seismic joints: the California's Plate Joint Seal Assembly (Figure 2.4-36). It uses the same basic principle as conventional sliding steel plate joints, but is detailed differently and is appropriate for the large displacements that are expected during a severe earthquake.

The joint was developed as a large-movement alternative to finger and modular joints, with seismic motions in mind. It allows primarily longitudinal movement, but can also accommodate some transverse movement. The two conventional large-movement joints (finger and modular) have proved to be neither robust nor able to accommodate the transverse movements that might occur in an earthquake.

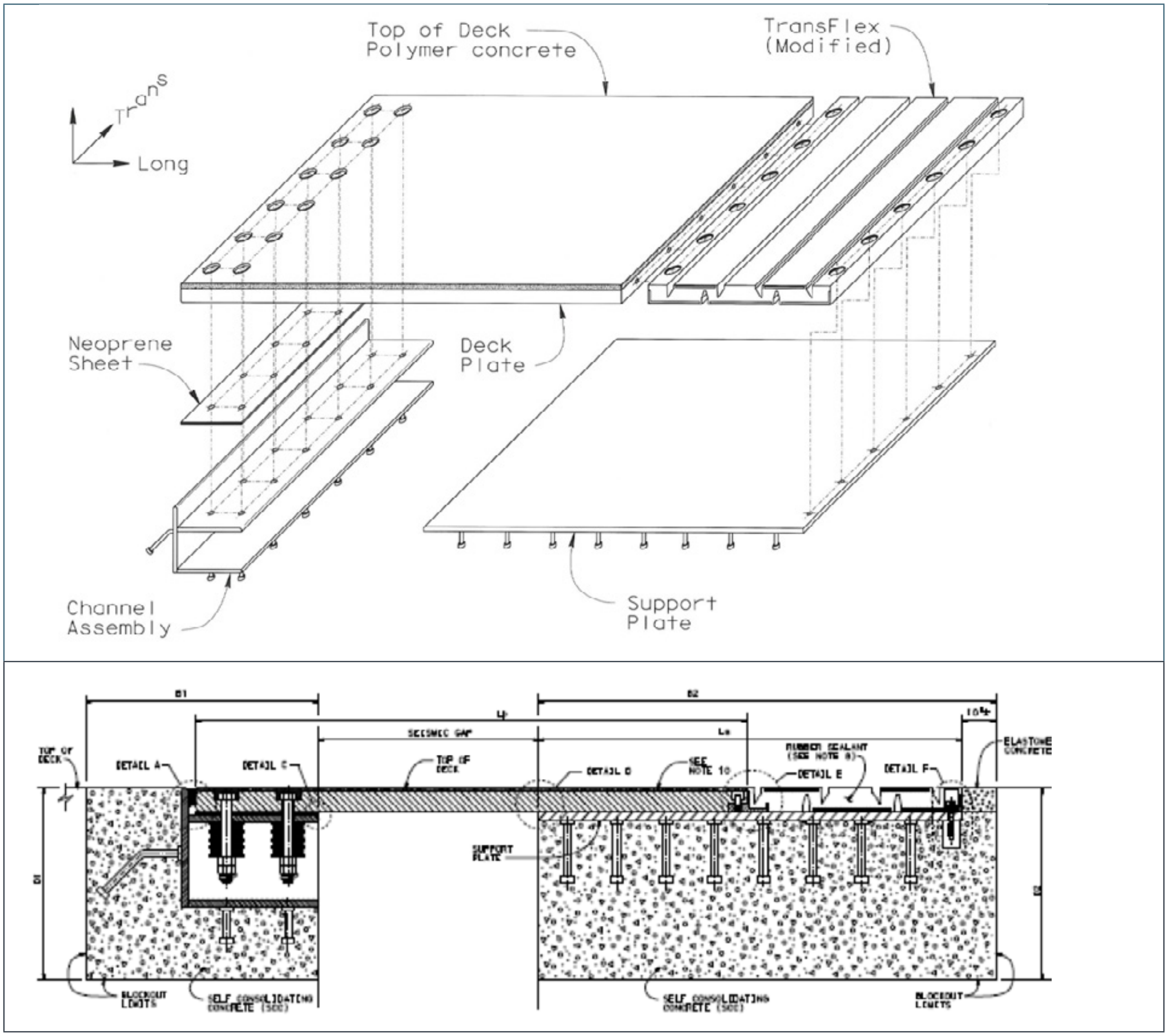

Figure 2.4-36. California Plate Joint Seal Assembly (Type II). Isometric and Section

Figure 2.4-37. Bolted-down elastomeric panel

The California Plate Joint Seal Assembly is a steel plate sliding system, and comes in a modular form. Each module covers a half traffic lane and is made of i) a Channel Assembly, ii) the Support Plate, iii) the Deck Plate and iv) the "elastomeric sealant", which is a bolt-down panel dimensioned to suit the needs. The elastomeric sealant accommodates service and moderate seismic demands elastically and is considered a sacrificial element in large seismic events (Figure 2.4-37). The Deck Plate is pinned at one end and free to slide at the other. Its top surface is covered with polymeric concrete for skid resistance. The Deck Plate spans the joint opening, slides over the Support Plate and, unlike other joints systems, shifts potential joint damage away from the critical joint opening, therefore preventing joint collapse and traffic disruption even after a major seismic event. It is a simple joint solution suitable for both small and large longitudinal and transverse movements.

Excessive displacement in an unexpectedly large earthquake is likely to damage only the Elastomeric Panel, which can be replaced relatively easily. Various versions of the joint have been deployed in California and have so far performed well under thermal movements. The joint has yet to be field-tested in a severe earthquake. Its primary advantages are that the design principles are relatively simple, and the joint is not proprietary and so can be fabricated by any large fabrication shop. The initial costs are reported to be competitive, and the life-cycle costs are expected to be low.

2.4.2.10 Finger Joints

In finger Joints, also known as "Tooth Dams", (Figure 2.4-38), heavy steel plates with projecting fingers are attached on each side of the joint. They provide vertical support for vehicles and are able to move longitudinally to allow the joint to open and close. Finger joints are not inherently sealed, and a trough beneath the joint is needed to direct water and debris away from the bearings below. They are one of two systems suitable for large movements and work well under limited, ideal, circumstances. However, they have given problems under conditions that are not ideal (i.e., skewed bridges, lateral seismic displacements, etc.) Furthermore, the trough needs to be cleaned under all circumstances, which imposes an additional maintenance requirement. Consequently, the use of finger joints in new construction is declining. Their continued use appears to be more common in states, such as Pennsylvania, that are traditionally associated with the steel industry.

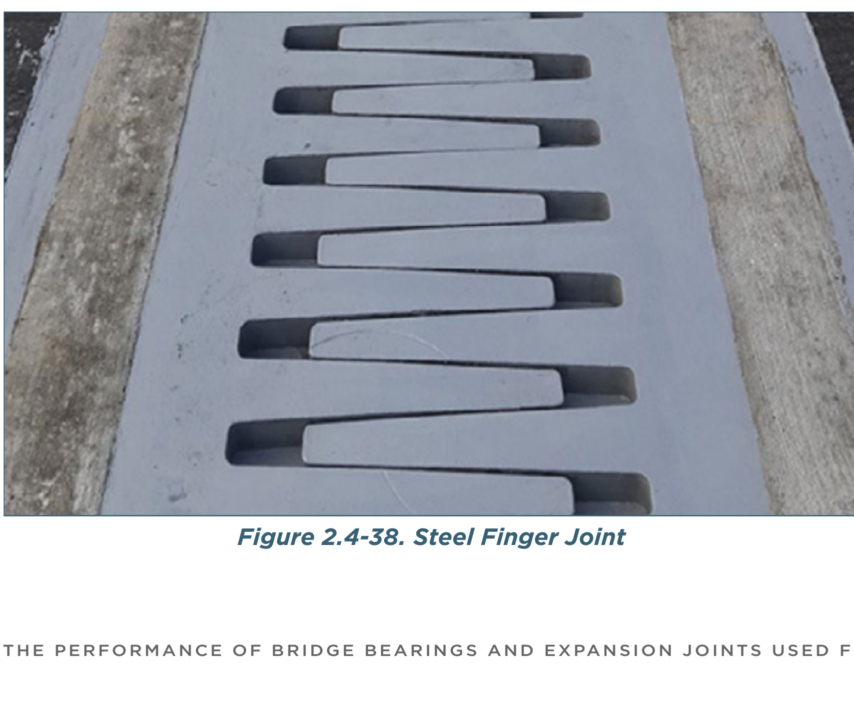

Figure 2.4-38. Steel Finger Joint

Finger joints are generally less expensive than modular joints (Section 2.4.2.11), but modern modular joints generally perform better and so are preferred; (earlier modular joints suffered from fatigue problems.) Finger and modular joints both require significant space, both longitudinally (where the space needed greatly exceeds the movement capacity) and vertically, because of the structure of the joints.

Finger joints can be made in several styles. In the one shown in Figure 2.4-39, the fingers are supported at the tip, as well as the root, of the cantilever, and are consequently quite slender. Nonetheless, several have fractured. Note that only a small differential vertical displacement is needed between the two sides of the joint for one set of fingers to lose their tip support and for the set, on the other side, to yield. That may explain the fractures seen in the figure. The design is not robust.

Figure 2.4-39. Finger Joint: Fractured fingers (LA). Note support both ends

A more traditional approach is for the fingers to act as cantilevers, as shown in Figure 2.4-40. The deep stiffener welded beneath the finger strengthens it considerably so there is no need for tip support by the opposite side of the joint. However, this arrangement allows the two sides to become mis-aligned vertically. In the case shown in the figure, the joint had been distorted by the supporting girder, and the vertical discontinuity at the joint caused a traffic hazard. The superstructure was a curved prestressed concrete box girder, which had been designed using methods for straight girders. Failure to account in design for the curved geometry led in practice to unexpected distortions, including twisting, of the girder. The twist caused one end of the finger joint to rise and the other to fall. The joint was repaired by grinding down the upstanding fingers and coating them with a polymer-based grout to improve skid resistance.

Figure 2.4-40. Finger Joint: vertical displacement due to girder twist. (Note also support for fingers.)

2.4.2.11 Modular Joints

Modular Joints (Figure 2.4-41) are also known as "Assembly Joints". They function like an assembly of strip seal joints in series.

Like strip seals, modular joints contain glands to prevent leakage. However, because the joints have multiple openings, Center Beams are required between each glanded opening. The Center Beams are supported vertically on "Support Bars". The space between the Center Beams opens and closes with the movement imposed on the whole joint. The Center Beams slide on the Support Bars, and a control mechanism keeps the Center Beams at equal spacings. Special shackles, or stirrups, hold them down onto the Support Beams while allowing them to slide, riding on spring-mounted sliding surfaces. Last, the Support Bars, with their axes parallel to that of the bridge, can slide at the Support Boxes in which their ends are contained. This sliding allows them to span the opening without inducing any resisting force.

The mechanical arrangements that control the support and movement of the Center Beams are quite complicated. Consequently, the first modular joints, built in the 1960s, suffered fatigue problems. Stirrups welded to the Support Bars held down the Center Beams and a special mechanism maintained equal spacing between Center Beams. The fatigue problems were largely associated with the welds between those components. Changes to the AASHTO Specifications in 2004, which altered the fatigue design requirements, improved the situation significantly.

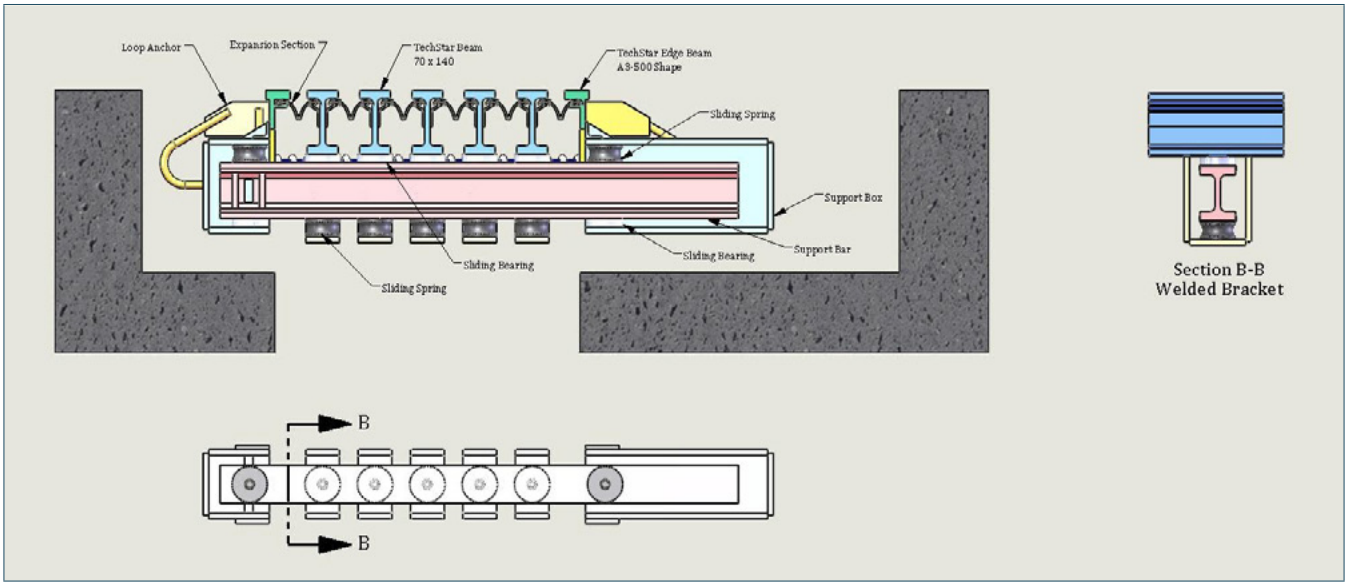

Figure 2.4-41. Modular Joint

Wheeled traffic traveling over the multiple center beams inevitably causes vibrations that in turn cause noise. One manufacturer has developed a system of "sinus plates", shown in Figure 2.4-42, to form the upper surface of the joint for the purpose of reducing noise. They are claimed to reduce noise by 80%.

Modular joints are relatively compltex mechanical devices. They are also deeper than most other joint systems, because the Center Beams ride on the Support Bars. They are also typically wide, because they are intended for large movements. Consequently, they are relative expensive.

The depth required to accommodate a modular joint means that it cannot easily be installed as a retrofit; essentially all modular joints are designed to fit in new construction. The Support Bear Boxes (Figure 2.4-41) are the lowest component. If they coincide with a girder line, the top flange of the girder may have to be coped, in the case of steel. The Support Bar Boxes can also cause problems during installation; because they are typically set quite close to the concrete diaphragm beneath, that space is limited. It is inaccessible to vibrators, so consolidating the concrete there may prove challenging. It was suggested that the use of Self-Consolidating Concrete (SCC) in the header might improve the consolidation.

Modular joints are subject to a variety of failure modes:

- Overall: The joint may be placed too high or low relative to the deck profile, resulting in traffic vibration, noise and fatigue damage.
- Steel related: Fatigue of welded connections. Welded splices (in the shop and field) of the center beams. Failure of the control spring system that maintains equal spacing of the center beams.
- Concrete related: The details of the reinforcement may not be sufficiently well thought out and may not tie the joint adequately in to the deck. (The transverse bars are likely to be discontinuous where they meet the Support Bar boxes). The header concrete may not be properly placed and consolidated, leading to spalling and failure, as shown in Figure 2-4 43. Concrete may fall between the Center Beams during casting, thereby preventing their proper movement, etc.

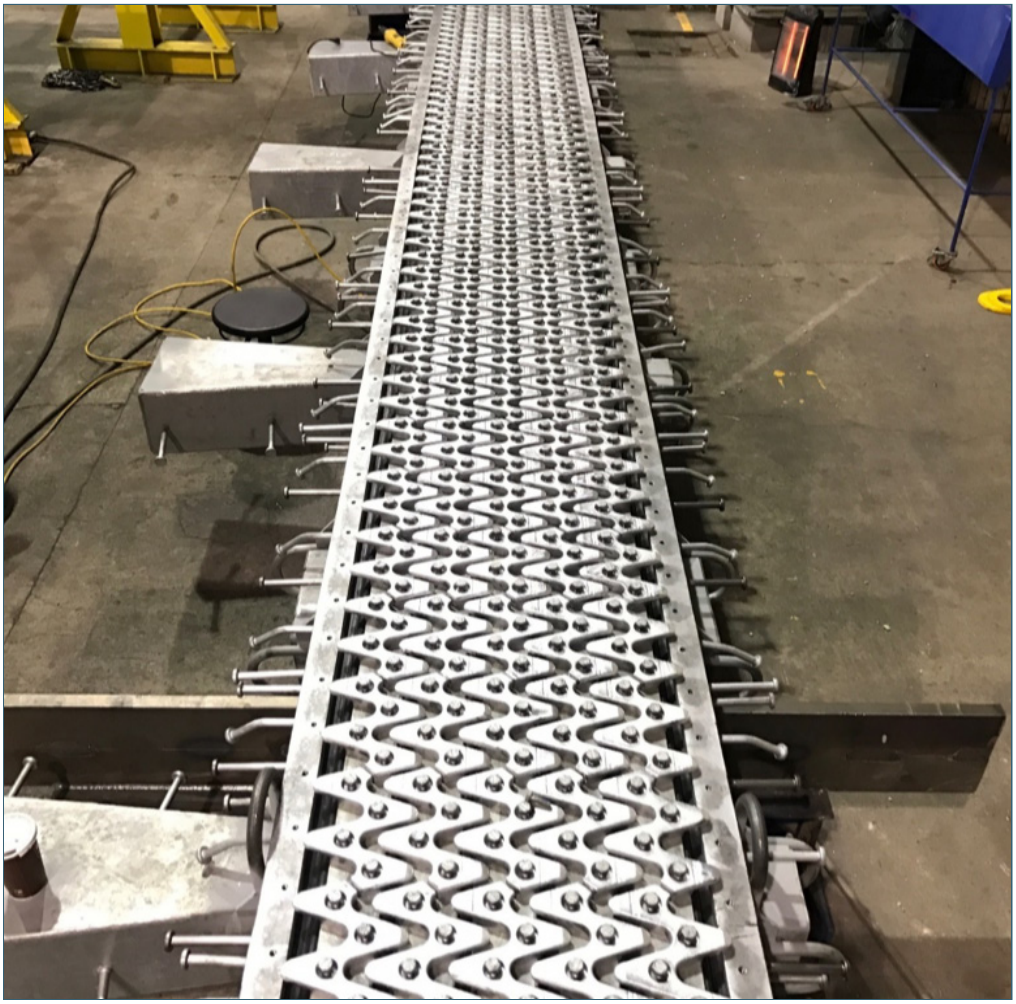

Figure 2.4-42. Noise-reducing "Sinus Plates" (Mageba).

Figure 2.4-43. Modular Joint: header failure (CA)

One manufacturer offers a Swivel Joint that resembles a modular joint (Figure 2.4-44), but also allows for some transverse displacements. It also contains many moving parts.

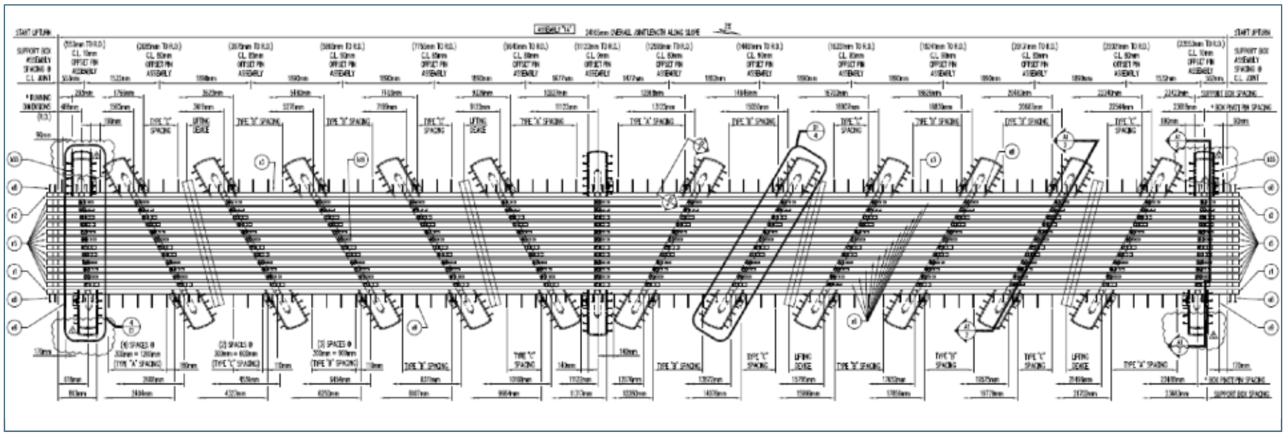

Figure 2.4-44. Swivel Joint. (D.S. Brown).

2.4.2.12 Use of Armor

Joint armor was originally developed to protect the projecting corners of concrete elements, but it presents dangers of its own. In snowy states, the snowplow blades can catch in the armor and dislodge it, causing a significant traffic hazard. Consequently, states such as New York are moving away from the use of armor.

2.4.3 Fabrication, Construction and Installation of Joints

Fabrication of joints raised no widely occurring problems. Modular joints are the most complex of the joint types, and the welding details need to be carefully planned and executed. These were addressed under design, in Section 2.4.2.11. Several states reported that it was beneficial to have an Approved Supplier List (ASL) at least for complex devices like modular joints.

Installation of the joint is generally the responsibility of the general contractor, and that is where most of the problems were reported. For all joint types, pre-submission of installation procedures, and a pre-construction meeting, are essential. For large or critical joints, having a manufacturer's representative on site is important, and the presence of an experienced engineer is also valuable.

2.4.3.1 Retrofit and Replacement of Joints

For a multi-lane highway, it is likely that only one lane can be closed at a time, so any new joint must be supplied in sections and connected after installation. For modular joints, this affects the Center Beams, which need to be spliced by welding. For strip seals, the end dams need to be spliced. Both bolting the sections with a splice plate and welding have been tried, but bolting was generally found to provide unsatisfactory performance because the bolts eventually corroded or worked loose.

2.4.3.2 Setting of Joint Hardware

Several issues arose with respect to installing the joint hardware at the right height or in the right location.

New York install all seals at low temperature, if possible, because the joint opening is then greatest, allowing the largest working space and ensuring the most compression across the joint in service.

The elevation of the joint hardware, or armor, relative to the road surface affects noise and vibration. Vibration influences the joint's life. Minnesota reported a recent case in which the Center Beams of a modular joint were mistakenly set 3/8" too high, and this led to increased fatigue loading and damage. (Their standard tolerance is 1/8"). In snowy states, the height is also important in avoiding snowplow hits and, for that reason, Pennsylvania typically sets strip seal armor 1/8" below the road surface. The setting height should also take into account the possibility of grinding the deck surface.

In all cases, but particularly for modular joints, it is important to ensure that the joint opening at the time the concrete is cast is appropriate for the temperature at the time and is consistent with the values assumed in design and stated on the plans. (It is also important to remove any shipping bolts from the joint hardware before installation. Michigan and others reported cases when the contractor failed to do that, with adverse consequences). There was some discussion among the team about the temperature to be used; should it be ambient temperature, deck temperature, or something else? And if deck temperature is used, how and where should it be measured? The consensus was that ambient temperature was the most practical, even though it is related only indirectly to the deck temperature and the amplitude of the bridge expansion. Because the temperature may change while the joint is set in place, it is useful to have some means of easily adjusting the joint opening so that last-minute corrections can be made prior to pouring the header concrete.

Large modular joints are heavy, and lifting them on site needs careful consideration, as does the means used to support and level them prior to casting. In some cases they are hung from temporary supporting beams that span the rough opening, while in others the ends of the joints' support beams rest on the girders beneath the deck. The latter procedure may lead to difficulties in consolidating the fresh concrete that connects the joint to the existing deck.

Continuation of the joint into the barrier, and details of ways to turn it upwards there, were also the subject of debate. If the joint is wide (e.g., modular), the opening in the barrier needs to be protected by a cover plate to avoid being caught by a snow-plow blade. The joint also needs to be set back from the face of the barrier, which makes narrower joints (e.g., strip seals) harder to install. Installation procedures need to be considered when the geometry of that region is selected.

2.4.3.3 Headers

Damage to headers was one of the most widely reported problems, and much of it was attributed to poor materials or construction procedures (Figure 2.4-45).

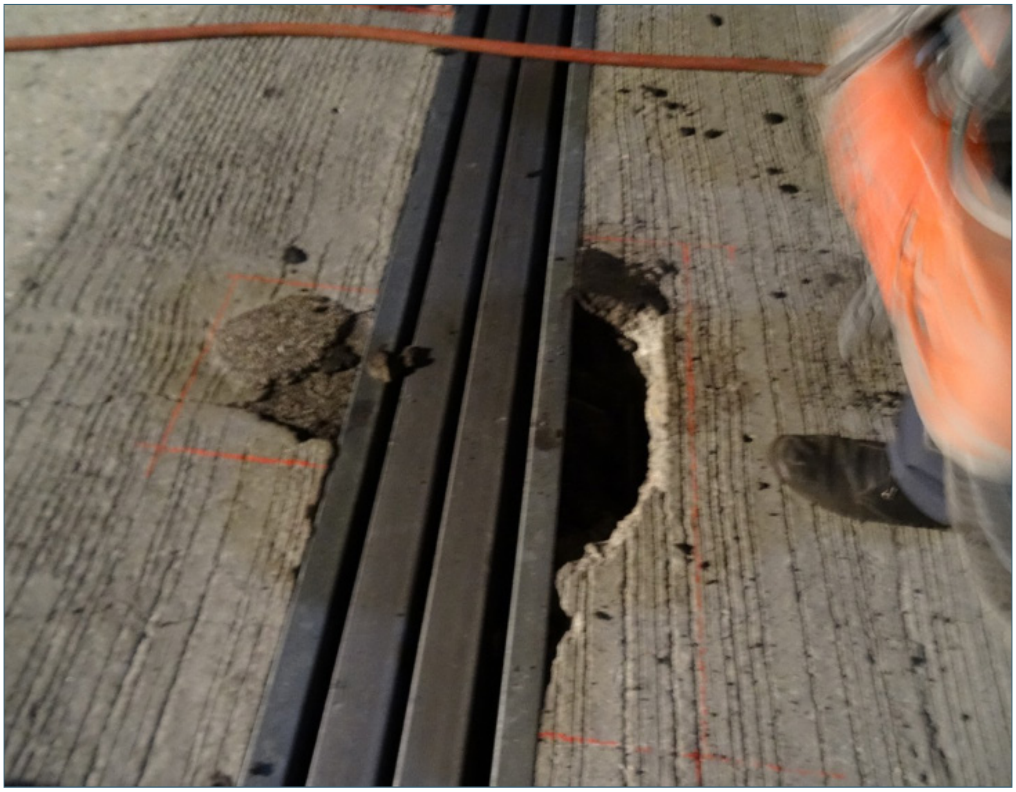

Figure 2.4-45. Modular joint failure (CA)

Several states use special reinforcement (other than epoxy coated bars). For example, Virginia uses stainless steel and MMFX bars (rather than epoxy coated) in critical areas, especially on roads with high traffic volumes. Pennsylvania has used galvanized bars in decks and substructure units on a case-by-case basis, and several states, including Minnesota, reported using GFRP bars. All of these bars have corrosion resistance that is better than that of uncoated bars (CRSI 2013, FHWA 2004, Battaglia 2008) but there is yet no consensus on their relative merits. Some types also have restrictions on cutting and bending, which might limit field adjustments.

The concrete used in the headers attracted much discussion. First, consolidation of the concrete in the header is a problem for almost all joint types, but particularly for modular joints, where the space below the Support Bar Boxes must be filled, but cannot easily be accessed with a vibrator.

Second, the participating states reported having tried several different types of concrete, with varying degrees of success. The choices can be categorized as: cementitious concretes; cementitious concretes with fibers, and polymeric concretes.

Cementitious concretes have the advantages that they are flowable (especially SCC), which is important if the joint contains armor or an end dam, and economical. However, they have poor impact resistance and are prone to spalling near exposed corners. Their permeability depends on the mix design. If a pre-formed silicone seal is used, the concrete mix design should be checked to ensure that it is chemically compatible with the adhesive used to secure the seal; outgassing from the concrete admixtures has sometimes caused the adhesive to fail.

Cementitious concretes with fibers include Engineering Cementitious Composites (ECC), Hybrid Fiber Reinforced Concrete (HyFRC) and Ultra-high Performance Concrete (UHPC). All of these materials have much higher impact resistance than conventional cementitious concretes, but are more expensive. They also require more care in mixing. At the time of writing, UHPC is almost always supplied as a commercial product; then the supplier may be asked to come to site to mix the material. UHPC uses steel fibers, which make it extremely tough. If the joint should ever need to be re-built, breaking out the UHPC would be very difficult. Most of these materials, and particularly UHPC, have the benefit of low permeability.

Polymer-based concretes use a polymer, rather than a cementitious product, for the binder. They have high impact resistance, so are suitable for unarmored joints, and good bond properties. They can be batched on site in small quantities, and set rapidly, which is useful if rapid construction is necessary. Their primary drawbacks are that they are more expensive, they are less flowable than cementitious products during construction, and they are less hard, and therefore more prone to rutting, in service, particularly if the header is deep. They are also somewhat moisture-sensitive. Rutting just before the joint causes impact loading on the joint. The poor flow properties exacerbate the problem of consolidation around joint armor and anchors.

States reported mixed results with polymeric concretes. While the rapid cure during construction was beneficial, several cases of failure in service were reported (e.g., New York and Pennsylvania.) Other states (e.g., Oregon) use it regularly for repair of damaged headers and broken armor. They report that such materials are sensitive to installation conditions and procedures. For example, adverse weather, inadequate surface preparation or improper mixing procedures can compromise the final result.

The ideal material would be economical, have rapid strength gain, good flowability, high impact resistance, low creep (to avoid rutting) and good bond properties. None of the materials considered combined all of those properties, so the material has to be selected on the basis of the properties that are most important for the application in question.

2.4.4 Inspection, Maintenance, Repair and Replacement of Joints.

2.4.4.1 Inspection

Joints typically require more maintenance than bearings, and an active maintenance program can significantly extend the life of a joint. Maintenance may be done by state personnel (e.g., Minnesota) or contracted out (e.g., Florida), using maintenance crews or “push-button” contracts. Both approaches appear to have proven satisfactory. Virginia has already instituted relatively detailed maintenance practices, defining specific elements such as joints and girder ends, for which the condition can be recorded and used as input to a bridge deterioration model. They are also preparing maintenance guidelines for joints. Condition monitoring and inventory management play important roles in a successful maintenance strategy. Minnesota has developed a “Bridge Maintenance Academy” to train their maintenance staff.

Documentation of joint repairs is also important. Many joint types exist and it is important to know which type is installed in which location so that the correct part, such as a new gland, can be ordered when replacement is needed. Documentation is also important at replacement; some states reported that emergency repairs were occasionally developed on site by the repair crews but not recorded, with the consequence that the as-built drawings no longer matched the details on site.

Most states have adopted digital record keeping at some level, and a variety of software platforms are used. For example, Michigan uses tablet computers for real time reporting and is in the process of placing work-orders and crew assignments with the tablets as well. The state uses in-house software for record-keeping.

2.4.4.2 Maintenance

Several states reported that an active maintenance program of inspection and maintenance was the most cost-effective way of keeping bridges in good condition. Many also divide the activities into Preventative and Reactive Maintenance. The former consists of actions that can be scheduled ahead of time, such as joint flushing (every year) and crack sealing (every 3 – 5 years), while the latter are responses to less predictable events, such as fractured bolts or torn glands that require patching.

Cleaning is the most important activity, especially for joints with glands, such as strip seals and modulars, because sharp grit or gravel in the joint (Figure 2.4-46) can eventually puncture the gland. However, the reported frequency of cleaning depended on the weather conditions prevalent in the state; in snowy states, joints were cleaned more often. Furthermore, cleaning has to be scheduled around traffic. Joints are usually cleaned using a water jet and vacuum truck (Figure 2.4-47), assisted as needed by hand tools for loosening hard-packed debris.

Figure 2.4-46. Debris in joint

Figure 2.4-47. Joint cleaning (MI)

Most states (e.g., Minnesota) reported cleaning joints in the spring, and in some cases in the fall as well. Others (e.g., Florida) cleaned them as needed, without a fixed schedule. Joints are typically inspected when they are cleaned. For finger joints, not only should dirt and debris be removed from the steel fingers, but the trough should also be cleaned.

Common joint maintenance items include

- Debris,
- Leaking joints,
- Glands that were torn or pulled out,
- Troughs (e.g., under finger joints) plugged or torn,
- Adhesion failure (e.g., pre-formed silicone joints),
- Damaged armor (e.g., due to snowplow damage).

2.4.4.3 Repair and Replacement

Joint repair and replacement can occur at several levels.

- Poured joints can be removed and replaced.
- Compression and foam-based joints can be removed and a new joint installed.
- In strip seals, the gland can be patched to correct small tears, or replaced if damage is more serious.

Joints can also be modified during replacement. For example, when an overlay is applied next to a strip seal, the joint may be built up as shown in Figure 2.4-48, where the header is raised using polymer concrete (or mortar) and the joint is raised by adding a poured silicone joint over the existing strip seal. The polymer concrete provides impact resistance at the corner, without having to change the armor detail.

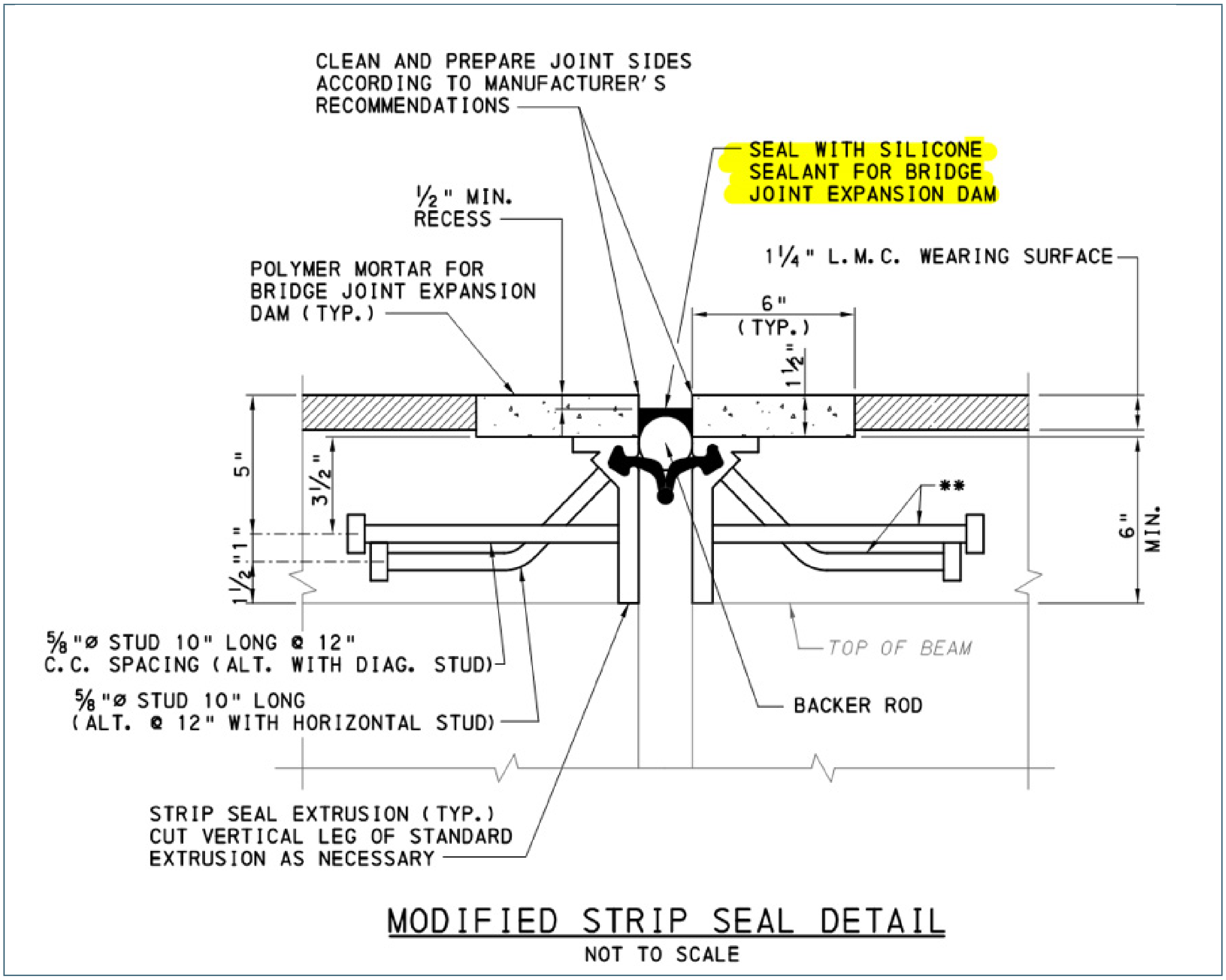

Figure 2.4-48. Strip seal modification to accommodate overlay

Replacement by a different joint type is also possible, but the movement capacity and other conditions must be consistent. For example many finger joints are being replaced by strip seals. Replacing a finger joint with a modular joint (to provide large movement capacity) is likely not possible because the modular joint requires too deep a blockout. Replacement by a strip seal avoids that problem, but is feasible only if its movement capacity is adequate.

A drainage trough has also been added in some cases. For joints such as finger joints, this is essential to prevent leakage. For other joints such as strip seals, a trough adds a second line of protection against leaks, and thereby provides better protection for the bearings in the event that damage to the strip seal goes undetected for a year between inspections.

CHAPTER 3

Conclusions

The following primary conclusions were drawn from the study.

- Practices vary widely among states for design, construction, maintenance and even terminology for joints and bearings. The differences in practices arise partly from the different climates, and partly from the policies instituted by the different states. Sharing of knowledge and successful practices is likely to provide real benefits.

- Joints and bearings have the potential for incurring ongoing costs, such as for maintenance and replacement, that are significantly higher than the initial costs. Therefore expected life-cycle costs should be considered when selecting them.

- Almost all states face problems of continuity of operations, exemplified by difficulties in attracting and retaining staff, and of knowledge transfer from experienced to newer staff members.

- Eliminating the joints on the bridge itself, by using some form of jointless construction, provides advantages. If joints are included on the bridge, failed joints in themselves constitute a problem, but leakage of failed joints has further consequences, such as damage to bearings, supporting members, and girder ends.

- For large movements, modular joints are the most common choice in new construction, although finger joints are still used. For medium movements (2" to 4") (5" in Washington State) strip seals and pre-formed silicone seals are the most widely used. For small movements (less than 2") a wide variety of joint systems is commercially available, and no one type dominates the market. Poured silicone joints are quite widely used.

- Elastomeric bearings are the bearing of choice for short and medium span bridges. They are economical, durable and perform well in practice. The most often quoted problem is moving from their installed location.

- For longer spans and higher loads, California uses spherical bearings almost exclusively, while others use both disc and pot bearings. Some states ban pot bearings, largely because of the elastomer leaks experienced in some early designs. Disc bearings are gaining an increasing share of the US market, because, compared to pots, they have lower initial costs and easier to inspect. However, pot bearings are widely used in Europe, and disc bearings are not used at all there.

CHAPTER 4

Recommendations

Many individual recommendations were discussed. The major ones are:

- A selection matrix should be developed for joints, and another for bearings, to assist designers in selecting the most appropriate joint or bearing for their circumstances.
- Internet-based tools should be developed for gathering information from bridge owners on in-service performance of joints and bearings.
- Additional knowledge gaps beyond those disclosed as a part of this peer exchange should be identified with the goal of developing NCHRP research topics. Examples identified by this scan team include the need for disc bearing design specifications, a change from prescriptive to performance-based specifications, the possibility of creating a national approval and testing agency, etc.
- Training tools should be developed to help transfer knowledge from experienced to newer employees within agencies. These could take the form of webinars, design examples, documented/instituted succession plans and mentorship programs.
- AASHTO LRFD Specifications for design of disc bearings should be expanded. Now that the original patent has expired and disc bearings are no longer a sole-source item, there is a need for comprehensive specifications based on a rational design procedure.
- The inconsistencies between the LRFD Design and M251 Material Specifications for elastomeric bearings need to be resolved. The documents are maintained by separate committees, which should coordinate efforts to address the conflicts.

CHAPTER 5

Implementation Plan

The scan team initially presented its findings and recommendations to the AASHTO COBS T-2 Technical Committee for Bearings and Expansion Devices during the June 2018 Meeting. To disseminate information from the scan, the team is giving technical presentations at national meetings and conferences sponsored by the TRB, ASBI and other organizations and is planning to write papers for various publications. The team will also develop a recommended plan of action based on its findings to submit to T-2 to consider as a committee work plan for further enhancement and improvements to the AASHTO LRFD Bridge Design Specifications. Finally, the team will develop research problem statements to address several of the needs identified in this document that will be submitted for consideration as NCHRP projects.

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BIJAN KHALEGHI (SCAN TEAM CHAIR) is the State Bridge Design Engineer with the Washington State Department of Transportation (WSDOT) Bridge and Structures Office. He received his Master and Doctor of Engineering degrees from the National Institute of Applied Sciences, Lyon, France. He is a member of many committees and task forces, including the American Association of State Highway and Transportation Officials (AASHTO) Technical Committees on Concrete Bridges T-10, Research T-11, Tunnels T-20, ASBI and PCI Bridge Technical Committees. Before his current position, Bijan was the Concrete Specialist for WSDOT B ridge and Structures office. Among his other responsibilities, Bijan is the designated structural engineer for the Alaskan Way Viaduct Tunnel project in Seattle. He is a registered Civil and Structural Engineer in State of Washington.

ZHENGZHENG "JENNY" FU received a MS degree in Civil Engineering from Purdue Univ. in 1991 and BS degree in Mech. Engineering from Huazhong Univ. of Science and Tech. of China in 1984. Ms. Fu joined LADOTD Bridge Design Section in 2003 after working for consultants for 10 plus years. She is now an Assistant Bridge Design Administrator in LADOTD Bridge Design Section, and her responsibilities include consultant management, Bridge Design and Evaluation Manual, standards, and bridge scour program. Ms. Fu is a member of AASHTO Subcommittee on Bridges and Structures and serves in T-10 (Concrete) Technical Committee. She also serves in TRB Long-Term Infrastructure Program Expert Task Group for Bridges.

ED KESTORY is the Structures Maintenance Engineer for the fifth district of the Florida Department of Transportation. In this role, Ed oversees the bridge inspection and rehabilitation program for all structures within the nine counties of Central Florida. Ed has been with FDOT for 18 years holding positions in the District's Structures Design and Program Management Offices prior to his appointment to this position in 2014. Ed received his bachelor's and master's degree in civil engineering both from the University of Florida. He is a licensed professional engineer in the state of Florida.

AHMED N. MONGI is the QA/QC Unit Leader with the West Virginia Department of Transportation (WVDOT), Division of Highways (DOH), Engineering Division in Charleston, West Virginia. He graduated with a Master of Science in Civil Engineering Degree from the West Virginia University and began his career with WVDOT-DOH in the Structures Division as a Design and Evaluation Engineer in 1991. During his 27-year tenure with WVDOT-DOH, Mr. Mongi has held various other positions such as Structures Project Manager, Negotiation and Agreement Unit Leader, Project Construction Engineer of the New Kanawha River Bridge which holds United States record for the longest span of Cast-In-Place concrete box girder using balanced cantilever construction method, and Assistant State Bridge Engineer. Mr. Mongi is a member of the AASHTO Technical Committee for Bearings and Expansion Devices (T-2) and AASHTO Technical Committee for Metals Fabrication (T-17) for the Committee on Bridges and Structures. Mr. Mongi is a licensed Civil Engineer in the States of West Virginia and Ohio.

REBECCA NIX is the Bridge Management Engineer of the Utah Department of Transportation in Salt Lake City, Utah. She oversees bridge planning, programming, inspection, and load rating statewide. The UDOT Bridge Management Division oversees in-service activities for approximately 3,000 bridges, both state and locally owned. Rebecca is a member of the AASHTO Technical Committee for Bearings and Expansion Devices for the Committee on Bridges and Structures. She is a graduate of the University of Utah with a bachelor's degree in civil engineering and a licensed Structural Engineer in the State of Utah.

LINH WARREN is the structural engineer in the Federal Highway Administration's (FHWA) Office of Bridges and Structures in Washington, DC. In this position, she provides technical support to structural engineering program areas including Load Rating, Inspection, Design, and Security of Bridges and Tunnels. Warren supports the development, acceptance and deployment of new and improved construction techniques as well as support the administration of the National Bridge and Tunnel Inspection Programs. Before joining the Office of Bridges and Structures, Warren served as a senior bridge design engineer for the FHWA Eastern Federal Highway Division in Sterling, VA. She holds a master's degree in civil engineering focusing in structural engineering from The George Washington University in Washington, DC. She is a licensed professional engineer in Virginia.

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JOHN STANTON (SUBJECT MATTER EXPERT) is a Professor of Civil Engineering at the University of Washington, where he has taught since 1978. Prior to entering academia, he worked as a structural Engineer in England, France and Canada. His research interests include reinforced and prestressed concrete, earthquake engineering and bridge engineering, including Accelerated Bridge Construction and bridge bearings. He performed the research, and drafted the provisions, for almost all of the bearings requirements in the AASHTO LRFD Design Specifications. He holds a PE license in Washington State.

JILL WALSH (TECHNICAL CONSULTANT) is an Assistant Professor of Structural Engineering in the Civil Engineering Department at St. Martin's University in Lacey, Washington. She received her Master and Doctor of Engineering degrees from the University of California, San Diego. Jill was a Senior Bridge Engineer at T.Y. Lin International for 13 years prior to joining St. Martin's in 2015.

Appendix E: Amplifying Questions

Domestic SCAN 17-03 Experiences in the Performance of Bridge

Bearings and Expansion Joints Used For Highway Bridges

Survey and Amplifying Questions

Below is a listing of the Amplifying Questions (AQs) as sent to the host agencies.

1 DESIGN

1.1 Design of Joints and Bearings – Common questions.

1.1.1 Do you have an in–house specialist for joints and bearings?

1.1.2 Do you design joints and bearings in house, leave the design to the supplier, or use some other approach?

1.1.3 Are there any types of expansion joint or bearing that are not permitted in your state? Please give reasons.

1.1.4 Do you have written procedures for choosing the controlling parameters for your bearing and expansion joints projects, e.g., reasons for choosing one type or design of joint or bearing over another? What is the supporting rationale for the choices?

1.1.5 Do you have a mechanism for transmitting feedback from the field (e.g., inspectors) to designers?

1.1.6 Do your policies for bearing and expansion joints differ when using different project delivery methods, such as design-build, design-bid-build projects or consultant-design?

1.1.7 Does your state have an explicit policy to eliminate expansion joints? If so, please provide it.

1.1.8 Do you use integral piers or integral abutments for your bridges? What are the criteria (e.g., span length) for using them?

1.1.9 Do you use jointless bridges and, if so, what are the criteria for using them?

1.1.10 Do you use link slabs, or other such methods?

1.1.11 What girder types do you use in common bridges such as overpasses? (e.g., steel plate girders, precast prestressed concrete girders, etc.?

1.1.12 What design and other standards do you commonly use for bridge bearing and expansion joints, e.g., AASHTO, others?

1.1.13 Have your designers (either in-house or consultants) identified shortcomings in the AASHTO Design and/or Construction (and Testing) specs?

1.1.14 Does your State develop standard details for bearings and expansion joints?

1.1.15 How do you write specifications for the long-term performance of devices that are susceptible to long-term deterioration, such as fatigue?

1.1.16 How do you differentiate between wide bridges over narrow bridges for designing bearing and expansion joints?

1.1.17 What is your policy with regard to bridge skew angle and curvature for selecting bearings and expansion joints? Please include any limitations on skew angle or radius.

1.1.18 What seismic and other extreme events criteria do you use for design of bearings and expansion joints? For example, do you use a 2500/100 dual level performance evaluation or single 1000-year event?

1.1.19 Are you considering other, more advanced, technologies for use in bridge bearing and expansion joints? If so, please elaborate, including how you see these new technologies advancing "state of the practice" for bearing and expansion joints owners?

1.2 Design of Joints

1.2.1 What specialized technologies for expansion joints do you recommend on the basis of past experience?

1.2.2 Are you considering other, more advanced, technologies for use in expansion joints? If so, please elaborate, including how you see these new technologies advancing the "state of the practice" for expansion joints?

1.2.3 What is the expected life of your expansion joints at the time of construction? Do they achieve this life in practice?

1.2.4 How are maintenance and repair considerations worked into the design of expansion joints?

1.2.5 What waterproofing systems have you employed in expansion joint assemblies?

1.3 Design of Bearings

1.3.1 What is the life expectancy your bearings at the time of construction? Do they achieve this life in practice?

1.3.2 How are maintenance and repair considerations worked into the design of a bearing? (e.g., do you require space for lifting jacks, etc.)?

1.3.3 Do you recommend any particular bearing types on the basis of past experience?

1.3.4 Are you considering other, more advanced, technologies for use in expansion joints? If so, please elaborate.

1.3.5 What bearing types do you use in your structures, (e.g., elastomeric, Teflon, disc, spherical, pot, pin, roller, seismic isolation, etc.)? Do you have any limitations or restrictions for any bearing types? Please give reasons.

1.3.6 What are your criteria for selecting a bearing type for your structures? Are they different for "standard" bridges (e.g., overpasses) and "special bridges" (e,g., cable stayed) or for different types of bridge (e.g., steel vs concrete girders, skewed vs right, etc.)?

1.3.7 For supports that must allow free movement in one direction (e.g., longitudinal) but resist movement in the other (e.g., transverse), How do you design the system? For example, do you use guided bearings, or floating bearings and a separate guide at the center of the bridge?

1.3.8 In skewed or curved bridges, do you orient the bearings parallel to the support line, perpendicular to the girder axis, or some other way (e.g., use circular bearings)?

1.3.9 Do you design bearings for the loading conditions both at installation and in service? (e.g., light load and large rotation at installation, heavy load and smaller rotation in service.)

1.3.10 Do you require anchor bolts for some bearings but not others? If so please provide details of the associated bearing types and the reasons for the requirement. What is the basis of the anchor bolt design?

1.3.11 What waterproofing systems have you employed for the bearing? Do you provide drainage under the joints?

1.3.12 Do you use seismic isolation bearings or other specialized bearings for extreme events?

1.3.13 Do you replace existing bearings with the same types as the original bearings?

1.3.14 Have you ever replaced any conventional bearings with seismic isolation bearings?

2 FABRICATION, TESTING AND CONSTRUCTION

2.1 Fabrication of Joints and Bearings – Common Questions

2.1.1 Are the bearings and joints included in a lump sum cost for the bridge, or bid as separate items? Do you know how much he joints and bearings cost?

2.1.2 What limitations, if any, are there to your purchasing joint or bearing devices from foreign manufacturers (a) if the devices if fabricated in the US, (b) if fabricated abroad?

2.1.3 Do you accept proprietary, sole-source, products?

2.1.4 Do you require the bearing fabricator to provide shop drawings?

2.1.5 Have you identified recurring problems with fabrication or installation procedures for joints and bearings? Please provide details.

2.1.6 Do you require the fabricator of joints and bearings to have a representative on site during installation? If so, for which types?

2.1.7 Are you satisfied with the support during construction from the bearing and expansion joints manufacturer? If not, please provide details.

2.1.8 Have you used or considered technologies that accelerate the installation of bearing and expansion joints?

2.1.9 Have you designed joints and bearings for bridges that were constructed from prefabricated elements? If so, how did their design criteria, installation methods and subsequent performance differ from those associated with bridges constructed conventionally?

2.2 Fabrication, Testing and Installation of Joints

2.2.1 What test methods do you use for expansion joints? In particular, please identify methods that are particular to your agency, and are not based on federal standards, and describe how the test methods vary from one joint type to another.

2.3 Fabrication, Testing and Installation of Bearings

2.3.1 What test methods do you use for testing non-seismic (i.e., expansion) bearings? Please identify any shortcomings, or places where the test requirements are unclear or ineffective for the purpose of ensuring a high quality product.

2.3.2 What test methods do you use for testing seismic isolation bearings? Please identify any shortcomings, or places where the test requirements are unclear or ineffective for the purpose of ensuring a high quality product.

2.3.3 For installation of bearings, do you specify offsets that are related to environmental effects, such as installation temperature or concrete girder age (for creep and shrinkage)?

2.3.4 For installation of bearings, do you specify the accuracy, particularly with respect to rotation, assumed in design? How do you verify installation accuracy?

2.3.5 Do you have special procedures for installing bearings in skew or curved bridges?

3 INSPECTION AND MAINTENANCE

3.1 Inspection and Maintenance of Joints and Bearings – Common Questions

3.1.1 Does your state perform preventive maintenance of expansion joints and bearings?

3.1.2 What routine maintenance, repair, and rehabilitation activities are you doing now that you would consider a best practice for other states to follow, e.g., methods of cleaning, cycles for preventive maintenance activities? Do these methods vary across different joint/bearing types?

3.1.3 How do you handle the workforce for maintenance activities, e.g., with in- house forces, outside on-call contractors, dedicated contractors? What types of on-call contracts do you have and how are their operations addressed through their contracts?

3.1.4 Do you have in-house bridge maintenance personnel? If so, how many? What are the qualifications of the maintenance personnel?

3.1.5 Are your facilities adequate to inspect, maintain and repair urban bearing and expansion joints systems?

3.1.6 How do you allocate money and resources for maintenance and repair?

3.1.7 What are the ages of your bearing and expansion joints, and what maintenance issues do you encounter on a regular basis that can be attributed to the age of your bearing and expansion joints? Are they also related to the local environmental conditions?

3.1.8 How successful have the maintenance strategies that you employ been in decreasing the age-related maintenance problems?

3.1.9 How do you define "good repair" as it relates to your bearing and expansion joints and bearings? Do you defer maintenance due to budget or other issues, and if so, how do you determine what is and is not to be deferred?

3.1.10 Do you have a policy for replacing older joint and bearing types that have been found in principle to perform poorly, even though the particular devices have not shown problems? (e.g., steel rocker bearings in seismic regions.)?

3.1.11 To keep track of the condition, inspection, repair, and funds needed for your bearing and expansion joints, what management systems, such as the FHWA inspection and maintenance manuals, your own manuals, database software, etc. do you use?

3.1.12 Have you used or considered technologies, such as electronic, that improve efficiency in the inspection of bearing and expansion joints? If so, how have they worked, and how could they be improved?

3.1.13 How many bearing and expansion joints types do you have in your inventory?

3.1.14 How do you inventory bearing and expansion joints in your state, i.e., structure/bearing and expansion joints number? What is the basis?

3.1.15 Do you divide your bearing and expansion joints into groups/types for inspection and reporting purposes? If so, how do you do it?

3.1.16 Do you have separate inspection frequencies for each type of bearing and expansion joint? If so, what are the frequencies and how did you determine them?

3.1.17 How do you record and keep track of inspections (including deficiencies) of bearing and expansion joints?

3.1.18 How do you inventory the system characteristics (bridge width, length, type, skew, curvature, existence of bearing guiding systems, etc.) of the bearing and expansion joints?

3.1.19 How do you inventory modifications related to repair and maintenance in bearings and expansion joints?

3.1.20 Do you have limitations on when bearing and expansion joints maintenance can occur, e.g., time of day, season, etc.?

3.1.21 Is access a problem for inspection? If so, how do you overcome it?

3.1.22 Has special accessing equipment been used to inspect your bearing and expansion joints (e.g., for high ADT roads)? What is the equipment, and how did it perform?

3.2 Inspection and Maintenance of Joints

3.2.1 What is the biggest maintenance problem in your expansion joints? What are the most common types of damage and deterioration that you see?

3.2.2 Do you require that your expansion joints be cleaned regularly?

3.2.3 Have you completed any significant repairs or rehabilitation to expansion joints? If so, what types of repairs were made, and how were road closures, if any, handled?

3.3 Inspection and Maintenance of Bearings

3.3.1 What regular maintenance procedures, such as cleaning, do you perform on your bearings?

3.3.2 Do you replace bearings after a certain time, regardless of their condition, or do you wait until you see signs of problems?

3.3.3 For elastomeric bearings, what criteria do you use to determine failure and the need for replacement?

3.3.4 Have you completed any significant repairs to, rehabilitation to, or replacement of bearings? If so, please provide details, including any methods used for lifting the bridge superstructure. Did you close the roadway or did you do it under traffic or with off-hour closures?

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