Asphalt Plug Joints: Characterization and Specifications

Final Report

University of Wyoming
Department of Civil and Architectural Engineering
May 1999

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The asphaltic plug joints (APJs) are a type of bridge expansion joint that have been promoted by manufacturers as simple general purpose expansion devices for bridges with less than 50 mm (2 in) total motion. These joints are simple to construct - a flexible segment of asphalt spans between the pavement and the bridge deck. While the joint is physically simple, it requires a complex engineering definition to describe, model, and prescribe for service applications. Unfortunately, APJs have been developed by trial and error and used when little engineering research exists outlining their complex behavior. To help designers to better understand this behavior and be able to properly apply them, the University of Wyoming (UW) has developed engineering-based design guidelines and material specifications. This effort was aided with support of the Wyoming and Colorado Departments of Transportation. The results are presented in this report.

The discussion linearly flows from general topic on the subject to focused and detailed research into the joint behavior and material qualification tests. The research focus is on delivering a concise engineering-based design guideline and corresponding material qualification test. The report is structured to present this body of information through five stand-alone technical papers. The general topics are contained in the second through fourth chapters. The second paper addresses the state of the practice through a survey of State DOTs (*1*). In the third, the material characterization is presented from a battery of material tests using temperature-dependent elastic and elastic-plastic models (3). Within the fourth chapter, the material

characterization is used within the finite element analysis method to develop an understanding of the classical stress and strain fields and to develop design guidelines based on elastic-plastic material models (4). The fifth chapter presents the validation study used to verify the assumptions incorporated into the elastic-plastic material characterization and the finite element analysis (5). The sixth chapter presents the design guidelines and a modified standard AASHTO test to qualify the APJ material (6).

Two appendices provide the supplemental information used to develop the technical papers. Appendix A contains the raw survey data composed of nearly 300 pages, it is the basis for the survey paper. Appendix B is the raw load-deflection curves that serve as basis for the normal bond strength, shear bond strength, and elastic/plastic material characterization.

The validation study highlighted the overwhelming importance of relaxation in the function of the APJs. The geometry of the joint presents problems from a classical elastic engineering perspective with its re-entrant corners, bonded substrates, and theoretically infinite stress concentrations. However, with the material relaxing by viscous flow at about the same rate as the bridge places motion demands on the joint, minimal loads and stresses are created. With these small loads almost no limit on the allowable motion exists from a structural perspective. However, functional limitations exist from a volumetric perspective and the need to maintain a smooth and serviceable transition from the pavement onto the bridge deck. This volumetric limit is the basis for the bridge-motion limits that are proposed.

Two material characteristics fully define the structural applicability of an APJ location/application. These are the material time constant, $t_{75,}$ for the load to relax to 75% of its initial load and the material glass-transition temperature, T_g. Both of these qualifying characteristics may be obtained using a UW-modified AASHTO TSRST test. The UW modification determines the material time constant, which characterizes the relaxation. This test was used in this research and is proposed for future use. It needs to be further refined and qualified. The material may be evaluated and qualified quickly with this test using readily available standard AASHTO TSRST equipment. This relatively simple test should allow suppliers to develop better joint material through careful qualification.

While the relaxation allows for the material to function from a structural perspective, it can be detrimental to roadway roughness. The design trade-off is that a soft viscous material allows bridge motion but also allows the material to flow out of the wheel paths creating rutting. Another problem is that the asphalt binder is highly modified and will lose ductility as it ages, resulting in structural failures. The last major concern is that while relaxation lowers the loads and stresses it does not remove them entirely. The small loads when combined with the stress concentrations still produce stresses that promote fatigue problems. These problems can be mitigated and some concepts are included in the final paper. Maintenance and inspection of APJs will be required. In summary, APJs are good for some applications but should not be considered maintenance-free, long-term solutions. Periodical replacement will be required.

Figure 1.1 shows the proposed design guideline work sheet identified from this research. A design example using this sheet is illustrated and explained in paper five.

Design Guidelines:

Structure Number:________ Designer:__________________ Date: _________

Are there:
High likelihood for thermal shocks? Yes_________No___
Slow moving or stationary traffic? Yes_________No___
Skew angle in excess of 30°? Yes_________No___
If any of the above answers are ***Yes, Stop Here***. APJs are not applicable for this location.

Max anticipated bridge daily motion: ________ mm (______ in)

Bridge t_{B75} time requirement: ___________________ Minute
Anticipated Temperature Range for the Structure: (from Superpave SP1 at bridge location)
High ______°C (______°F) Probability: ____________% to be lower
Low ______°C (______°F) Probability: ____________% to be higher
Anticipated Bridge Total Motion: ______ mm (______ in)
Required Joint Thickness: ______ mm (______ in)

Selection Checklist						
Material Supplier	**T_g**	**t_{75}, min**	**T_g<T_{LOW} Y / N**	**t_{75}<t_{B75} Y / N**	**Acceptable Y / N**	**Thickness**
Watson Bowman Acme	**-27°C, (-16°F)**	**42**				
Pavetech	**-26°C, (-15°F)**	**76,000**				
Koch/LDI	**-43°C, (-45°F)**	**2.8**				

Joint Thickness	Maximum Motion
50 mm (2 in)	57.2 mm (2.25 in)
63 mm (2.5 in)	53.3 mm (2.1 in)
75 mm (3 in)	50.8 mm (2.0 in)
88 mm (3.5 in)	50 mm (1.97 in)
100 mm (4 in)	49.02 mm (1.93 in)

Figure 1.1 Design Guideline Worksheet

2.0 ASPHALT PLUG JOINT USAGE AND PERCEPTIONS IN THE UNITED STATES

2.1 ABSTRACT

The University of Wyoming is developing design guidelines for asphalt plug joint use and application under the sponsorship and guidance of the Wyoming and Colorado Departments of Transportation. As a first step in this research a survey of 50 state departments of transportation was conducted to assess use, perceptions, and installation guidelines. Fifty states responded. The survey results are presented and the trends are summarized.

This paper was originally published in:

Transportation Research Record 1594, Pg. 172-178, 1997 by Brian K. Bramel, Jay A. Puckett, Khaled Ksaibati, and Charles W. Dolan. MINOR REVISIONS ARE INCLUDED HEREIN.

2.2 Introduction

Asphalt plug joints (APJs) are flexible asphalt segments that span between the bridge deck and abutment, serving as the expansion joint. As an expansion joint an APJ not only is required to allow bridge movement caused by expansion and contraction while providing a smooth transition between the approach pavement and the bridge deck, but it also must remain watertight and keep debris from entering the gap between the bridge deck and the abutment.

A common cross section of a plug joint is illustrated in Figure 2.1. This joint consists of a blockout, a modified asphalt plug joint material, and a backer plate. The backer plate is used to prevent the joint material from extruding into the gap separating the bridge deck and the abutment.

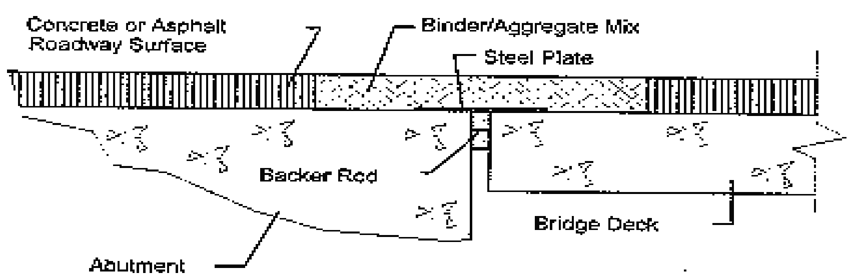


FIGURE 2.1 Typical Asphalt Plug Joint Cross Section.

Advantages of APJs include:

- They are quick and easy to install.

- They are easily repaired.
- They are not as prone to snow plow damage.
- They may be cold milled when a road is resurfaced.
- They are relatively inexpensive.

Because of these benefits the authors were asked by the Wyoming and Colorado Departments of Transportation to investigate APJ applications and determine appropriate design guidelines. Specifically, the authors were asked to evaluate performance to develop design guidelines, and to prepare material and installation guidelines.

APJs have some major disadvantages. The material characteristics are a function of temperature. They are soft and pliable when warm and stiff when cold. They have a glass transition temperature at which they become brittle, lose ductility, and crack, causing leaks and debonding at the plug joint-pavement interface. When warm, they may rut with heavy traffic volumes and heave with low traffic volumes. They may also track out, delaminate, and spall.

Some of these problems are attributed to inadequate blockout preparation or the use of an incorrect binder, whereas others are caused by improper applications. The design guidelines from the manufacturers are somewhat limited and require reliance on the expertise of the supplier. APJs are not for all applications, but where they are applicable they offer advantages that are not offered by other joints.

As a first part of this research, 50 state DOTs were surveyed regarding their use and perceptions of APJs. The goal of the survey was to determine the use of APJs, problems encountered with APJs, and serviceability and relative cost of APJs and to identify current unpublished information.

2.3 SURVEY GOALS

The survey contained questions designed to determine and quantify the following:

1. Design contact person.

2. Usage and cost.
3. Benefits.
4. Problems encountered.
5. Approved manufacturers.
6. Design guidelines.
7. Material specifications.
8. Installation guidelines.
9. Postconstruction joint inspection.
10. Interest in a continued pooled resource research project.
11. Desire to receive survey results and technical papers.

2.4 COST AND USE

As illustrated in Figure 2.2 APJs have been tried, at least experimentally, in 41 states. Of these 41 states, 23 still specify their use for either new construction or retrofits. APJs are used in both warm and cold climates, with no strong geographical preference for their use.

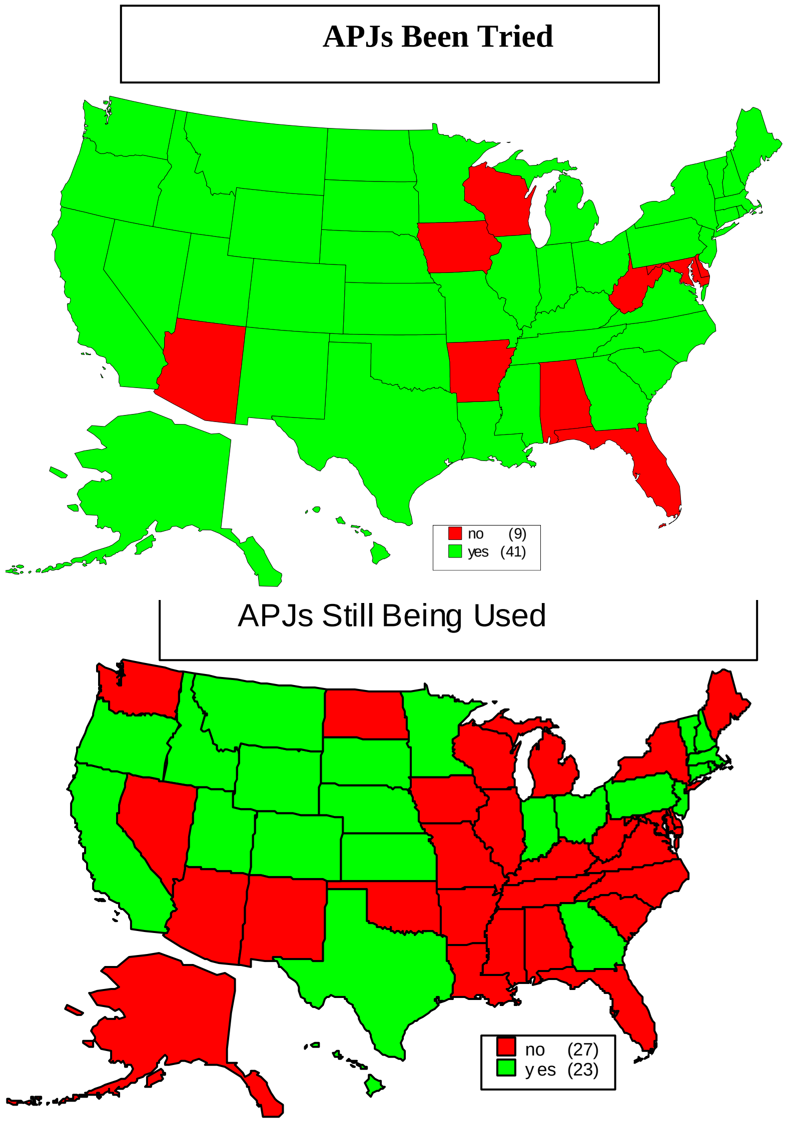


Figure 2.2 APJ Usage

The average number of joints per state for states that had at least tried them was 95. This average is skewed because the number of joints installed ranged from 1 joint (experimental) to 1,370 plug joints. Six states (California, Connecticut, Indiana, Ohio, Massachusetts, and Texas) have installed 88 percent of all APJs.

The cost of installed joints ranged from $60 per linear foot to $325 per linear foot. The average price for installed joints was $135 per linear foot, with a standard deviation of $60 per linear foot.

2.5 APJ BENEFITS AND LIABILITIES

Ten benefits were given as the major benefits for specifying APJs, and they are listed in Table 2.1. The benefits are categorized into three areas: functionality, serviceability, and cost. For functionality the responses mentioned a smooth transition from pavement to bridge deck, asphalt compatibility, reduced leakage, and ease of installation. The stated benefits for serviceability are reduced snow plow snagging, reduced maintenance, timeliness of installation, ease of placement over existing joint, and convenience in making temporary fixes. Two states also responded that the reduced cost was a major benefit.

Table 2.1 Benefits of APJs

Benefits	Number of States	States
Smooth Transition From Pavement to Bridge Deck	9 States	CO, CT, IN, MA,NH,OH,OR,RI,VT
Ease of Installation	6 States	CO, CT, MA, NJ,OH,WY
Asphalt Compatibility	5 States	CA, ID, HI,KS,WY
Reduced Leakage	5 States	CT, MA, MT, NH, RI
Reduced Snow Plow Snagging	4 States	CT, NH, OR, TX
Reduced Maintenance	4 States	CT, MN,OR,SD
Timeliness of Installation	3 States	CO, OH, NJ
Reduced Cost	2 States	MA, NJ
Can be Placed over Existing Joint System	1 State	CO
Temporary Fix	1 State	UT

Along with the benefits, the problems being encountered were requested. The 18 states that have either discontinued or never implemented APJs after their experimental installations reported various concerns. The reasons reported for not using or discontinuing the use of APJs are give in Table 2.2.

Table 2.2   Why APJs Were Discontinued

Reason Discontinued	Number of States	States
Experimental Installation Only	10 States	AK, KY, LA, ME, MI, ND, NM, NV, VA, WA
High Failure Rates	11 States	AK, IL, KY, LA, ME, MI, OK, ND, NY, VA, WA
To Expensive for Allowable Movement	2 States	TN. MI
Sensitive to Improper Construction Practice	2 States	ME, WA
Water Leakage after 1 Year	1 State	NY
Very Soft Pliable Material	1 State	LA

Ten of the states have tried APJs on an experimental basis only. Eight of these experiments did not meet the states' expectations. Nevada is still in the experimental stage of a 5-year experiment, with a 3-year-old joint that is still in good condition.

Two states believed that APJs are too expensive for the allowable movement. New York had a leaking problem with its experimental joint. Louisiana believed that the APJ material was too soft and pliable. In the LA DOT experimental installation, the joint suffered a broad range of problems from track-out and rutting to splitting in tension.

Two states pointed out that the joint was too sensitive for construction practice or installation methods to be practical.

2.6 PROBLEMS ENCOUNTERED

The problems encountered are summarized in Figure 2.3. Forty-five of the 50 states responding are reporting some type of problem with APJs, suggesting the need for improved understanding of the specifications and installation methodology. Of the states reporting problems, seven reported delamination, 12 reported leaking, 19 reported splitting in tension, 20 reported rutting, and 15 reported additional problems.

Rutting is a problem most seen when either the material is warm and becomes soft and pliable or the traffic is heavy. The survey responses indicate rutting that is a problem more often reported in the warmer states, thus supporting the warm-pliable argument. California reported that it prefers to use APJs in high-traffic areas because the traffic prevents the material from bulging, thus supporting the argument that high traffic by itself is not a significant contributor to rutting.

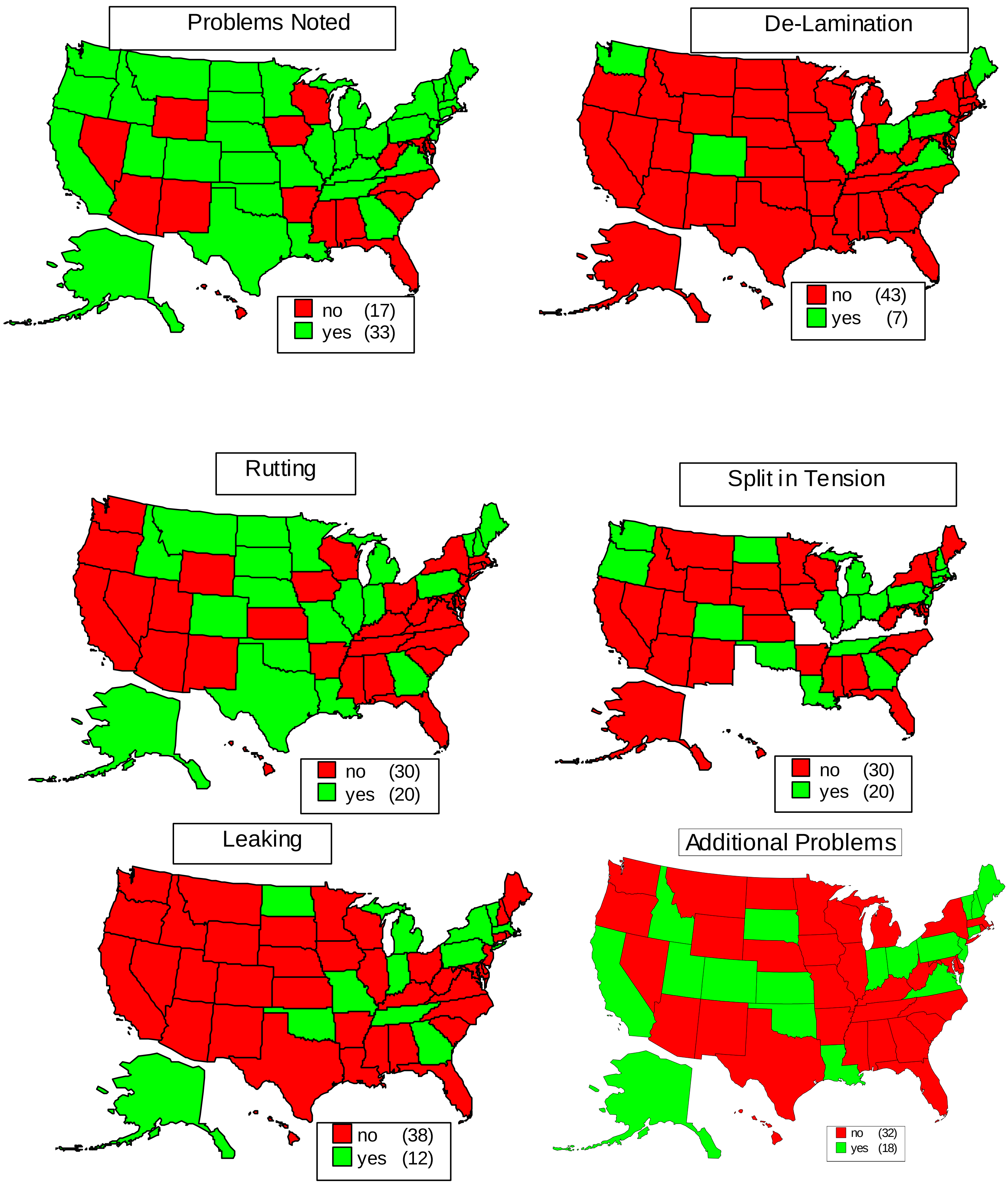


Figure 2.3 Problems Encountered

The condition directly opposed to rutting is splitting in tension. This problem may be induced in three ways. First, the ultimate strength of the material could be reached by either excessive strain/stress or joint motion. Second, at or near the brittle transition temperature, thermal stresses could exceed the material capacity. Third, the material could become fatigued because of a high number of cycles.

Excessive strain or motion is either an application problem or a blockout design problem. It may be an application problem if the joint motion is too large for the temperature range, average daily traffic, or effective length of the joint. It may be a blockout-design problem if the stress concentrations due to the geometry of the blockout exceed the ultimate strength of the material. This failure condition should be relatively insensitive to geographical or climate conditions.

The problem of splitting at or near the glass transition temperature should be a larger problem in the colder climates. As indicated in Figure 2.3, this is strongly correlated by the survey responses. The largest number of states reporting tension-splitting problems are located in the Midwest or the Northeast.

Delamination is a condition in which the APJ material becomes separated from the blockout. Because sliding motion is required at the bridging plate, all joints must delaminate somewhere adjacent to the plate. This problem is noted by either spalling of material or a crack propagating to the joint interface.

Leaking is a problem associated with the splitting in tension, but is can also be created by delamination. The survey responses indicate that leaking was a problem along with either a split in tension or delaminaton for all states but New York.

Additional problems being reported are outlined in Table 2.3. They are bonding problems at the joint interface, light surface cracking, snowplow cutting, heaving and bulging, tracking out, seal coat problems, and basis of payment.

Table 2.3 Additional Problems Being Reported

Bonding Problems	4 States	AK, SD, UT, WA
Cracking	3 States	MN, MT, ID
Snow Plow Cutting	2 States	NH, VT
Heaving and Bulging	2 States	CA, CO
Tracking Out	2 States	LA, VA
Seal Coats Inducing Cracks	1 State	ID
Basis of Payment	1 State	CT

2.7 APPROVED MANUFACTURERS

The commercial manufacturers of APJ material were identified and are as follows:

1. Pavetech (D.S. Brown).
2. Watson Bowman Acme (Harris Specialty Chemicals, Inc.).
3. Linear Dynamics Incorporated (LDI).
4. Koch BJS system (now being produced and supplied by LDI).
5. State of Vermont (which produces and installs its own APJs).

2.8 DESIGN GUIDELINES

Of the 50 states responding to the survey, only the 15 states indicated in Figure 2.4 have their own design guidelines. The other 35 states are relying on the expertise of the suppliers to use APJs.

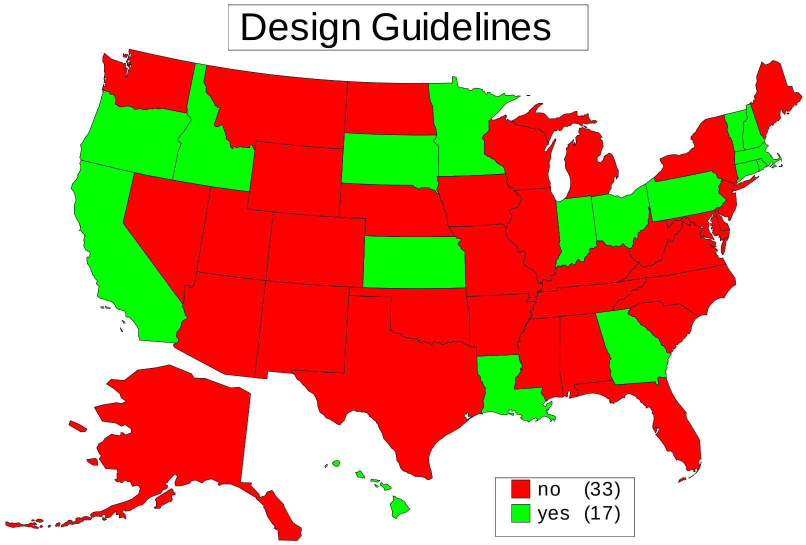


Figure 2.4 Design Guidelines

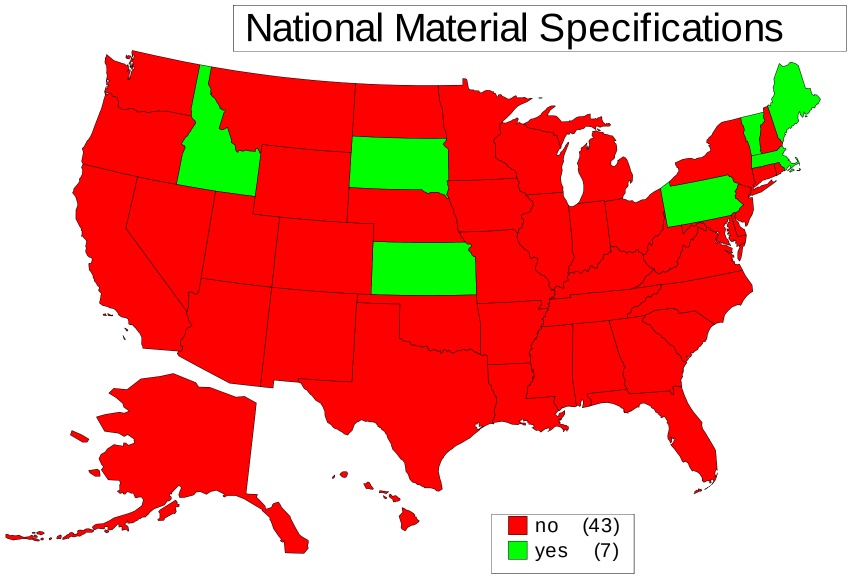
Some of the comments regarding design guidelines were as follows:

- 50 mm (2 in) minimum asphalt concrete overlay and movement rating less than 50 mm. Typically, the joint is 500 mm (20 in) wide with a 250 mm (10 in) bonding zone on either side of the joint opening (California).
- All joints with up to 50 mm of movement are APJs (Connecticut).
- Small joint movement, use with asphalt overlays (Georgia).
- Only place on structures with existing bituminous mats or where a petromat is to be placed. Only consider as a 5- to 10-year solution or as a last resort (Kansas).
- Do not use in moving joints (Louisiana).
- Use spans of between 24.4 m (80 ft) and 42.7 m (140 ft) and less than 25 degrees of skew (New Hampshire).
- Limit use to joints with total movement of less than 38 mm (1.5 in.) and little or no skew angle (Rhode Island).
- Used at joint locations where movements are less than 50 mm (South Dakota).

The reported difficulties in using the existing design guidelines are the desire to extend applications and create appropriate guidelines, the difficulty in removing the old joint, getting proper equipment to the site, and the previously stated problems seen with APJs.

2.9 MATERIAL SPECIFICATIONS

Figure 2.5 indicates both the states that use national material specifications and the states that have state-specific material specifications. Nine states use national material specifications and 12 states have state-specific material specifications. Twenty states that have tried APJs are relying on the suppliers for quality materials and construction support.



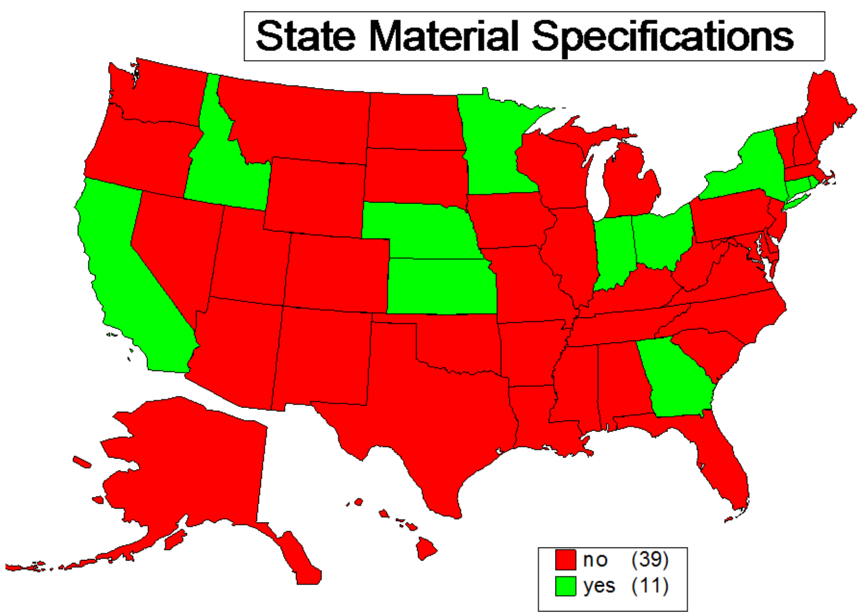


Figure 2.5 Material specifications.

2.10 INSTALLATION GUIDELINES

Figure 2.6 indicates the 19 states that have installation guidelines. The other states that specify APJs use the manufacturers' recommended installation guidelines or allow the installer freedom in the placement.

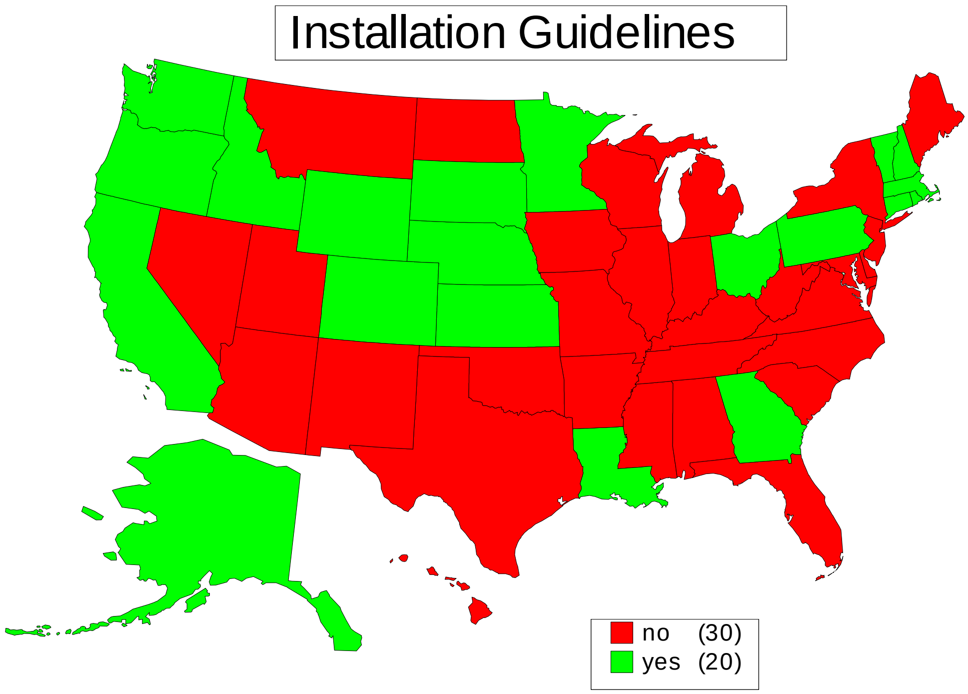


Figure 2.6 Installation Guidelines

2.11 POSTCONSTRUCTION JOINT INSPECTING

Figure 2.7 indicates the 41 states that conduct post construction inspections. All of these states respond that newly installed joints look good and are offering smooth transitions.

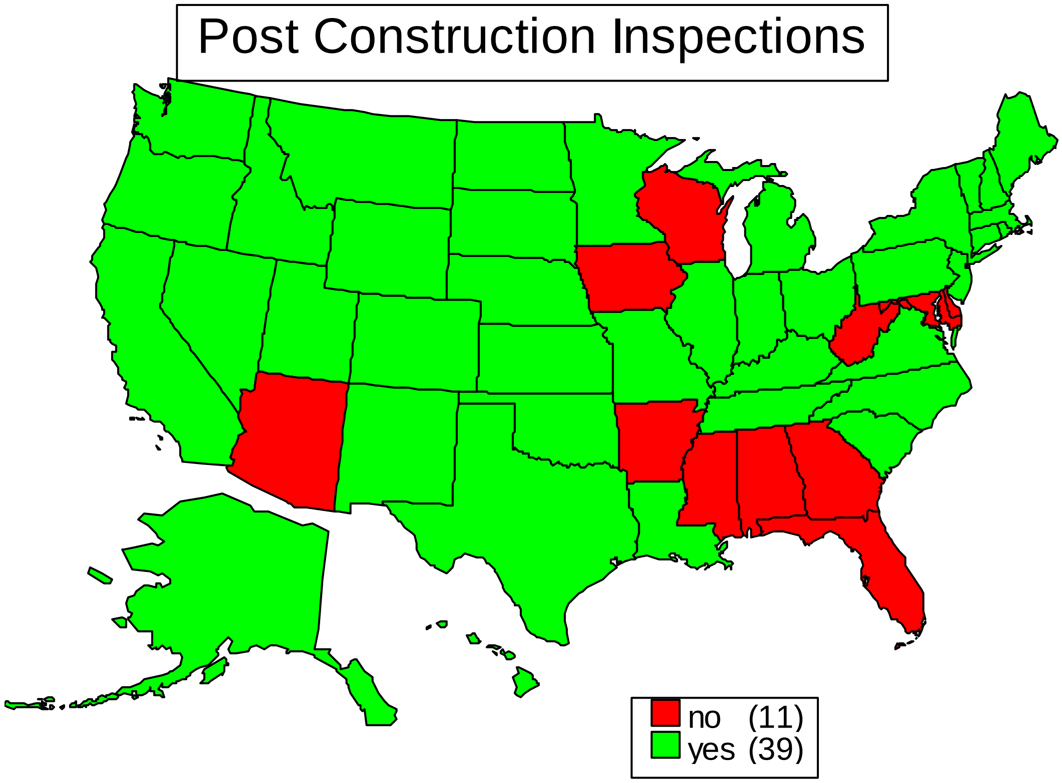


Figure 2.7 Joint Inspection.

2.12 CONCLUSIONS

The survey data suggest that APJs are being used successfully in many states but are being used unsuccessfully in others. Because of splitting in cold weather and track out in warm weather there is some possible climate/temerature dependence. Also, there does not appear to be a geographical preference because the states that are still specifying the use of APJs are geographically dispersed.

Little research into the joint behavior and material characteristics exists. Because of this basic research void, many states have installed experimental joints to evaluate the applicability. These joints have been installed solely on the basis of the manufacturer's design guidelines. There are no standard design guidelines. Reliance is placed on the suppliers for applications and design considerations. The design guidelines should be based on quantifiable parameters such as joint movement, temperature range, average daily traffic, and the material chosen.

Few states use national or state-specific material specifications or installation specifications. This leads to the inconsistent performance. Standard material specifications would help establish consistent results.

2.13 RECOMMENDATIONS

The need for basic research into the nature of APJs exists. With the material characterized as a function of temperature and proper material specifications, the temperature range for allowable motions can be determined. The joint blockout can be optimized to lengthen the effective joint length and reduce stresses and strains. From this basic research, appropriate and rational design guidelines will be developed in the continuing research program.

3.0 ASPHALT PLUG JOINTS - MATERIAL CHARACTERIZATION AND SPECIFICATION

3.1 SUMMARY

Asphalt Plug Joints are being used for bridge expansion joints following the manufacturers' recommendations. These recommendations were developed through application experience and incorporate little engineering assessment. For this reason many joints are being installed in unsuitable sites or are being overlooked for other, more appropriate, sites. To aid bridge engineers in using this type of joint more effectively, research to develop rational design guidelines is outlined. The progression of this research is to clarify the suitable applications, characterize the materials, develop and validate design guidelines. This paper addresses the material characterization using direct tension tests, normal bond test, shear bond tests, Georgia loaded wheel rutting tests, and Thermal Stress Restrained Specimen Tests (TSRST). Analytical modeling, validation and design/specification guidelines are addressed in later publications.

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5th International Conference on Short and Medium Span Bridges, Calgary, Canada 1998. By Brian K. Bramel, Charles W. Dolan, Jay A. Puckett, and Khaled Ksaibati Editorial revisions and updates are included in this version.

3.2 INTRODUCTION

Asphalt Plug Joints (APJs) are one of the simplest bridge expansion joints available. They possess a number of advantages that range from simple in concept, to ease of both installation and repair, with the added benefit of relatively low cost. However, they are not free from maintenance and problems. Asphalt plug joints have been used effectively but performance is varied (*1*). Rational design guidelines for these types of expansion joints are necessary to determine how and where APJs are best used. Moreover, rational test procedures are needed to qualify materials for differing performance levels. To produce design guidelines, it was required to determine failure modes, develop material constitutive models, and investigate the joint strain fields. The focus of this research is to develop rational guidelines for the use of APJs and the methods and results of the material characterization is presented in this paper.

Asphalt Plug Joints are a flexible asphalt segment that spans between the abutment and the bridge deck serving as the bridge expansion joint. A typical cross section is shown in Figure 3.1. The joint typically consists of a backer rod, gap plate, asphalt binder/aggregate mix, and pavement blockout. The backer rod serves as a seal to keep the initial binder between the gap plate and blockout until the binder sets. The gap plate keeps the binder/aggregate mix from extruding into the gap separating the abutment and the bridge deck. The binder/aggregate mix is made with a highly modified binder that remains flexible over large temperature ranges. A gap graded aggregate that yields large voids in mineral aggregate ratio (VMA) allows an asphalt content of between 20% and 40% by volume. The blockout is typically between 500 mm (20 in) and 610 mm (24 in) wide and no thinner than 50 mm (2 in) spans between concrete and/or asphalt sections.

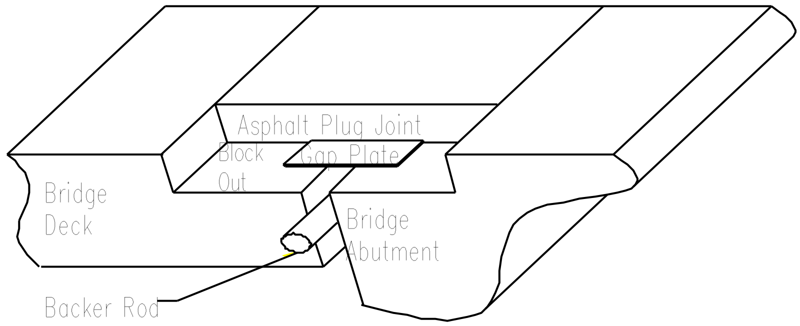


Figure 3.1   Typical Asphalt Plug Joint Cross Section

The joints are constructed by the blockout first being cast into the deck or cut into the asphalt overlay. The blockout is grit blasted to remove foreign materials. The joint is then dried with a heat lance to remove any moisture from the form work or cutting operation. The backer rod is installed. Heated APJ binder is then applied to the bottom of the blockout and filling in the gap above the backer rod. The gap plates are then laid into the APJ binder and secured into the backer rod with galvanized 16d nails. Another coating of heated APJ binder is then applied to the entire block surface. The aggregate binder mix is then heated, mixed and placed into the joint in three lifts with a filler coat of binder applied after each lift. The joint is then compacted, typically using a 2 - ton compactor. Another coating of APJ binder is then added. After this last coating of binder has cooled, a top coat of dry fine aggregate is applied to reduce binder track-out. This final coating of binder levels the joint and acts as the adhesive for the fine aggregate. The final coating makes a binder rich topping that seals the joint and helps it flow due to traffic.

The functional requirements of the joint are that the material remains flexible enough to allow for the bridge movements due to temperature variations, prohibits debris from entering the gap between the abutment and the deck, remains watertight, and provides a smooth transition between the pavement and the bridge deck. These functional requirements and a survey of the state bridge engineers (*1*) formed the basis for establishing appropriate failure modes.

The binder aggregate mix creates a material that behaves as a viscous fluid at warm temperatures and elastic solid at cold temperatures. The material flowing in hot weather and cracking in cold weather demonstrates this behavior. But it is this visco-elastic nature that allows this material to overcome the physical requirements of the joint movement and meet the functional requirements. The material is bonded on three sides of the joint by the same binder material that makes up the APJ, the blockout bottom and two pavement surfaces perpendicular to the traffic are the bonded surfaces. The translation imposed on the joint due to the motion of the bridge is forced to occur on either side of the gap plate, but will generally occur in a small region.

This localized motion in a bonded material creates a condition of movement (translation) occurring over a small length. From basic mechanics of materials, strain is defined as translation divided by the length over which the translation occurs. Because we are forcing a finite displacement to occur in a very localized region we are forcing the material to fail by cracking, debonding, and/or plastically flowing to redistribute the motion over a large enough length to meet the materials abilities. Failure will occur when this motion can not be redistributed to large enough lengths.

3.3 FAILURE MODES

The critical failure modes occur when the expansion joint leaks or ride quality over the joint is poor. The expansion joint can leak due to tension cracks through the joint (cracks propagating from the point of motion through the material), debonding (cracks propagating along the blockout interfaces detaching the plug joint from the blockout), or material spalling out of the blockout (multiple cracks radiating out of the point of motion resulting in loss of material). Poor ride quality can occur due to rutting, material piling up due to compression, material flowing out of the blockout in the traffic lanes, and track-out of the plug joint material by passing traffic.

To evaluate these failure modes, it was necessary to determine the materials ability to resist the service demands. Tensile strength, shear bonding strength, normal bonding strength, modulus of elasticity, and modulus of resilience were evaluated as functions of temperature to develop the allowed motions for standard geometries and temperature ranges. Because the rideability issues of rutting and flowing are a warm temperature phenomenon, the Georgia Loaded Wheel Test an elevated temperature test designed to give an indicative evaluation of asphalt pavements was used.

A Thermal Stress Restrained Specimen Test was used to give an absolute lowest temperature that the material can be used.

3.4 MATERIAL TESTING

The in-situ strength and behavior of the materials were required. To determine in-situ properties, a mold was developed that would allow a one-meter joint segment and material samples to be placed in one operation allowing for a uniform mix. These molds were self contained, which allowed the three suppliers to fill them either in the field or at their plants. The molds were produced and sent to the three US suppliers for placing with their material: Pavetech, Koch/LDI, and Watson Bowman Acme. A diagram of the mold is shown in Figure 3.2. The mold consists of three segments: joint segment, core segment, and specimens segment.

The molds were filled using the same construction methods as regular bridge joints. The mold was cleaned with grit blasting, dried backer rod installed, coated with heated APJ binder, gap plate installed, coated again with APJ binder, aggregate/binder mix applied in multiple lifts, compacted, topped off with APJ binder, and top finish aggregate applied. This similitude with field construction and materials should allow the sample to possess no size effects.

The mold yielded 16 normal-bond specimens, 16 shear-bond specimens, 16 material specimens, 12 -150 mm (6 in) diameter by 75 mm (3 in) thick cores, and one 0.9 m (3 ft) joint segment. The mold was cast of unreinforced 41 MPa (6000 psi) concrete so that the bond characteristics of the material concrete interface could be evaluated.

In summary, this one placement accommodates construction practice and provides for all material and joint test specimens. Size effects are minimized and all tests are derived from the same material.

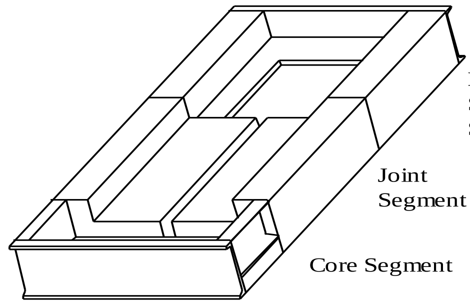


Figure 3.2 Plug Joint Specimen Mold

3.5 SPECIMEN PREPARATION

The materials were prepared by shearing the mold into its three segments, thereby separating the joint from the material samples. The core segment was cored using a 150 mm (6”) diameter diamond core drill in a water bath. This yielded a 150 mm (6 in) diameter by 175 mm (7 in) long core. These cores were then sectioned using a 400 mm (16 in) diamond wet saw into the final 150 mm (6 in) diameter by 75 mm (3 in) samples. This yielded samples from both the binder rich top of the joint and the aggregate heavy bottom portion of the joint.

The tensile specimens all came out of the specimens side of the mold. To conform to our wet saw size limitations the specimens mold was sheared into 4 pieces approximately 375 mm (15 in) by 375 mm (15 in) by 250 mm (10 in). These pieces were then sectioned into the final 50 mm (2 in) by 50 mm (2 in) by 250 mm (10 in) prisms for testing using a 400 mm (16 in) diamond wet saw.

The sawing operation was initially performed at -40°C (-40°F) in an attempt to cut a more rigid solid material. The heat from the sawing operation caused the APJ binder at the cut interface to warm and flow. This local flowing reduced the advantages of cutting the material cold. Samples were then cut from room temperature pieces and no differences were noted in the dimensional control and the saw-cut quality. Since differences were not noticed, the rest of the specimens were sectioned at room temperatures. The prisms were tested in a timely fashion after they were sectioned since they tended to slump, flow, and bond themselves to whatever surface that they were placed against. The material approximately 12 mm (.5 in) on either side of the shearing operation was discarded to insure that shearing damage was not introduced into the test results.

3.6 MATERIAL PROPERTIES

The material properties can be roughly divided into two categories, tensile and indicative. The material properties of the most importance were those needed to apply analytical tools. The finite element method was used in conjunction with the material characterizations to better understand and predict the joint behavior. The final objective is to develop rational design techniques that can be employed for a host of material, geometry, and environmental situations. The primary focus was on the tensile properties since the primary failure modes are associated with tensile phenomenon.

The tensile tests were used to determine the appropriate material constitutive models and the interface bond characteristics. With the assumption of an elastic-perfectly-plastic material, the properties that were required to measure were; modulus of elasticity, yield stress, normal bond strength, and shear bond strength. All of the properties were developed as functions of temperature by measuring them at four discrete temperatures in the possible tensile operating range; 21°C, 4.4°C, -18°C, and -40°C (70°F, 40°F, 0°F, -40°F).

3.7 MATERIAL TEST PROGRAM

Tensile tests were conducted on samples of the APJ material at a rate of 5 mm / minute (0.2 ”/min). The goals of these tests were to validate the linear-elastic-perfectly-plastic constitutive model and quantify the modulus of elasticity and the yield stress.

The specimens were prepared into 50 mm (2 in) x 50 mm (2 in) x 250 mm (10 in) prisms and bonded onto 150 mm diameter aluminum platens using Devcon 3873, a high modulus, low temperature, two-part epoxy. These bonded specimens were then conditioned to the one of the four temperatures, 21°C, 4.4°C, -18°C, and -40°C (70°F, 40°F, 0°F, -40°F), for a minimum of four hours. The specimens were taken out of the conditioning refrigerator and tested immediately in direct tension on an Instron 1332 load frame. An environmental chamber was not used because these specimens were tested relatively quickly with little change in internal temperature.

The elastic-perfectly-plastic material constitutive model fits the material behavior down to the -18° C (0°F) temperature. This behavior consists of a linear stress-strain relationship up to a yield plateau and the large additional strain with no increase in stress. This behavior is demonstrated by Figure 3.3, the stress-strain diagram for a Koch/LDI sample tested at 21° C (77°F). The test load levels are at the lower end of the 27 KN (5 kip) load cell which accounts for the data scatter during the plastic deformation.

At some temperature between -18°C (0°F) and the -40°C (-40°F) the material becomes brittle with little or no plastic deformation before failure. This is due to a glass transition temperature existing between these temperatures. The TSRST test, described later, illuminates this glass transition and at what temperature it occurs.

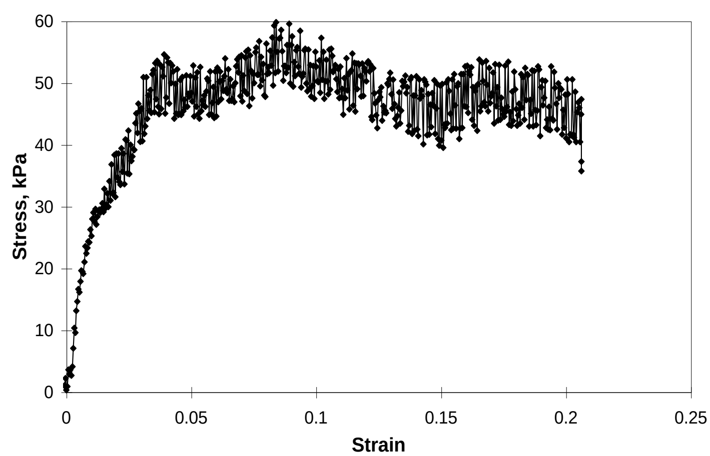


Figure 3.3 Stress Strain Diagram for Koch/LDI Material at 21°C (77° F)

The modulus of elasticity is the linear slope of the elastic portion of the stress-strain curve. An ideal APJ material would have a modulus much lower than the pavement material and nearly constant modulus of elasticity for the operating temperature range. As a point of reference, good modulus of elasticity for asphalt pavement is in the 1,300 to 4,500 MPa (190,000 to 656,000 psi) (*2*) range at room temperatures. While for concrete pavements or bridge decks the modulus of elasticity is commonly between 27,600 to 48,300 MPa (4,000,000 to 7,000,000 psi) (*3*). Table 3.1 illustrates the joint moduli is lower than the surrounding pavement. Even at the -40°C (-40°F) temperature the modulus is less than half of the asphalt pavement thereby attracting the deformation and/or failure by acting as the fusible link for the loads associated with the bridge movements.

Table 3.1 Material Specimen Results.

	Pavetech			**Watson, Bowman, Acme**			**Koch/LDI**		
Temperature	**E, kPa**	**σ_y, kPa**	**ε_y**	**E, kPa**	**σ_y, kPa**	**ε_y**	**E, kPa**	**σ_y, kPa**	**ε_y**
21°C	**1,300**	**27**	**0.020**	**4,600**	**130**	**0.029**	**1,500**	**44**	**0.030**
4.4°C	**9,300**	**130**	**0.014**	**14,000**	**180**	**0.015**	**7,800**	**150**	**0.019**
-18°C	**205,000**	**830**	**0.006**	**150,000**	**750**	**0.006**	**87,000**	**520**	**0.006**
-40°C	**607,000**	**1,500**	**0.003**	**929,000**	**1,700**	**0.002**	**342,000**	**500**	**0.001**

Unfortunately, the joint material does not begin to meet the second ideal behavior of remaining constant over the operating temperatures. As can be seen in Figure 3.4, the modulus of elasticity increases almost exponentially as a function of temperature. Unfortunately, this makes the material the stiffest when the tensile requirements are most critical--at cold temperatures. This leads to cold temperature cracking and the functional failure of the joint due to leaking.

The modulus of elasticity gives an indication of the stiffness of the material but to fully characterize as an elastic-perfectly-plastic material the yield stress, $\sigma_{y,}$ is needed. These stresses were determined from the material specimens in the same tests that modulus of elasticity was derived and are given as the average for three samples in Table 3.1. The yield stresses were determined as the stress at which additional (plastic) deformation started to occur without additional stress. For example, using Figure 3.3 the yield stress was determined to be 49 kPa. As shown in Figure 3.5, the yield stress increases as the temperature decreases giving the highest yield stresses at the lowest temperatures.

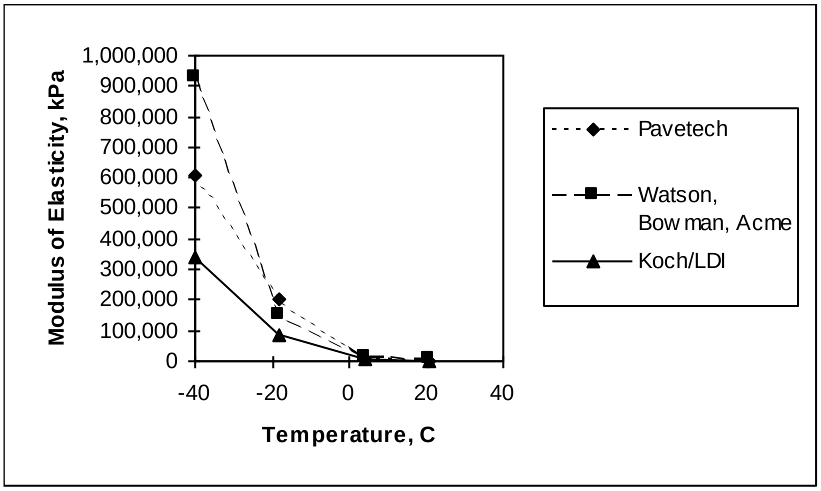


Figure 3.4 Modulus of Elasticity as a Function of Temperature

This material model does not show the degree of plastic deformation between yielding and failure. The model assumes that plastic strain capacity is infinite. This was not the case, with final failure occurring from 0.5 to 7 times the yield strain. The 0.5 times occurs after the material had gone through its glass transition temperature, T_g, and the 7 times occurs when the material is warm and acting as a viscous solid.

Since the main requirement is for this material to accommodate the translation of the bridge, the relationship of the yield strain becomes important. The yield strain is the yield stress divided by the modulus of elasticity and indicates the strain at which the material starts plastic behavior. The average yield strains for specimens are given in Table 3.1. Figure 3.6 shows why these joints work well at temperatures above zero degrees C (32 °F), where the material remains elastic well above 1% strain. The joints are susceptible to cold weather failure because the strain capacity is between 0.1% and 0.3%.

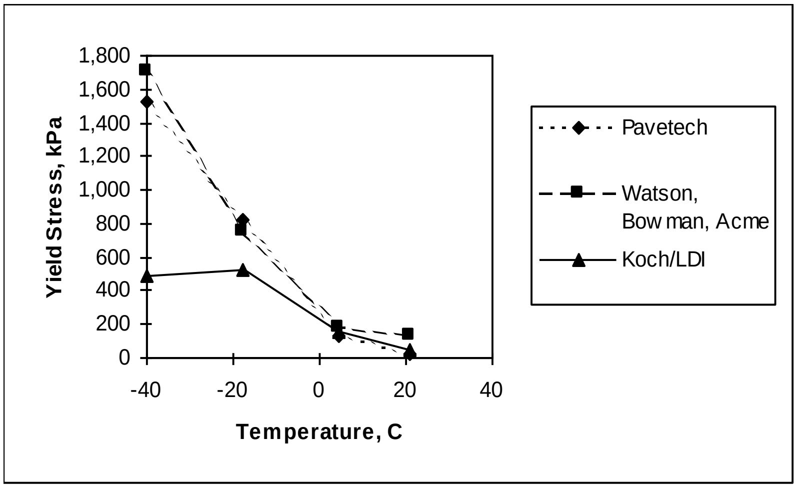
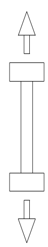


Figure 3.5 Yield Stress as a Function of Temperature



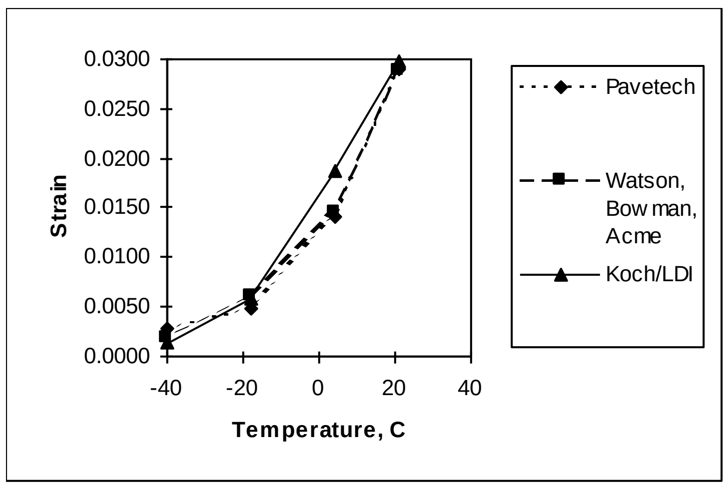


Figure 3.6 Yield Strain as a Function of Temperature

3.8 NORMAL BOND TEST

The normal bond tests evaluate the bonding capacity of the APJ binder when applied to the concrete deck and/or pavement when the load is applied perpendicular to the bond plane. This occurs in the APJ joint at the abutment/APJ and the deck/APJ interfaces normal to the traffic direction. This test was a modification of ASTM D897.

The evaluation was done in direct tension on a 50 mm (2 in) by 50 mm (2 in) by 250 mm (10 in) sample that was 125mm (5 in) of APJ and 125 (5 in) mm of concrete. These samples were prepared using diamond saw sectioning as previously described. The samples were bonded onto 150 mm (6 in) diameter aluminum platens using Devcon 3873 epoxy. These bonded specimens were then conditioned to the one of the four temperatures, 21°C, 4.4°C, -18°C, and -40°C (70°F, 40°F, 0°F, -40°F) for a minimum of 4 hours.

The tests were conducted in an Instron 1332 load frame at a loading rate of 5mm/min (.2 in) with the data being recorded using a Strawberry Tree data acquisition system. Three specimens were tested for each of the four temperatures and the averaged results are given in Table 3.2.

Table 3.2 Normal and Shear Bond Stress

	Pavetech		**Watson, Bowman, ACME**		**Koch/LDI**	
Temperature	**Normal σ_u, kPa**	**Shear σ_u, kPa**	**Normal σ_u, kPa**	**Shear σ_u, kPa**	**Normal σ_u, kPa**	**Shear σ_u, kPa**
21.1°C	**67**	**110**	**90**	**130**	**98**	**76**
4.4°C	**220**	**210**	**260**	**330**	**190**	**220**
-17.8°C	**330**	**260**	**770**	**690**	**330**	**710**
-40°C	**340**	**97**	**640**	**1200**	**170**	**650**

The ultimate strengths are reported for these tests because the material demonstrated a brittle bond behavior. This information is needed in the softened contact models for the finite element modeling of the joint. The softened model will allow the boundary gap elements to remain in contact and transmit the load induced by the bridge motion with no separation up to the ultimate bond stress at which time they will separate as in the actual joint.

The ultimate normal bond strength as a function of temperatures is shown in Figure 3.7. The normal bond strength is also dependent on temperature, however the strength increases up to -18 °C (0°F) then decreases. This suggests that the normal bond demonstrates a more clearly defined glass transition temperature than the material itself. This can be explained by realizing that the material has the aggregate and binder interface in an interlocking maze where any failure will be a combination of normal bond, shear bond, and APJ binder failure. While at the normal bond interface the binder is attaching the plug joint to a clean normal surface that appears as a large single aggregate with the probability for initial bond flaws high. This can also help explain why the normal bond ultimate stress is lower than the material yield stress.

One of the consequences of the normal bond strength being lower than the material strength is that even if the joint were to be totally unbonded along the bottom (no shear bond stress) then the joint would fail at the normal bond surfaces prior to the material failing. Bond failure in the field leads to leakage in the joint

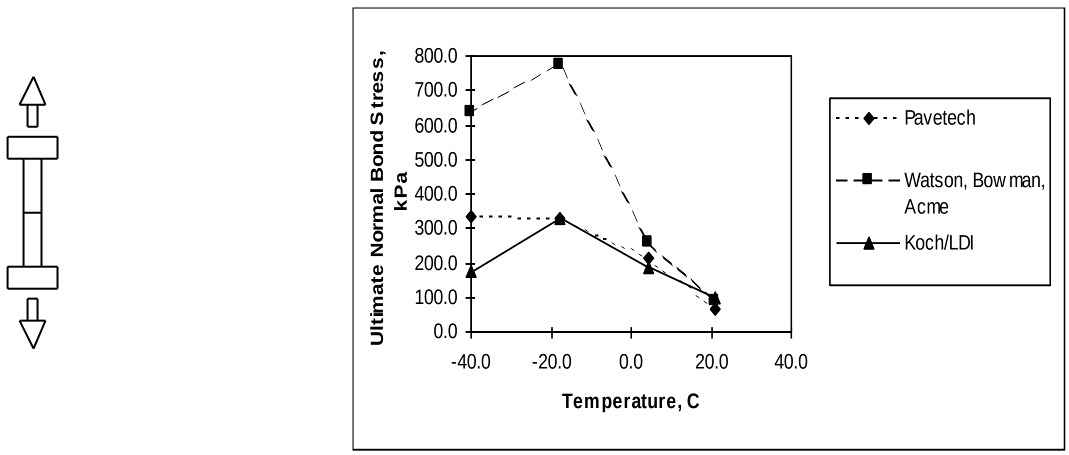


Figure 3.7   Ultimate Normal Bond Stress as a Function of Temperature

3.9 SHEAR BOND TEST

The shear bond tests evaluate the bonding capacity of the APJ binder when applied to the concrete deck and abutment blockout where the load is applied parallel to the bond plane. This occurs in the APJ joint on the blockout bottom interface parallel to the traffic direction. This test was a modification of ASTM D3165.

The test was a thick adherent test where a 50 mm (2 in) by 50 mm (2 in) by 250 mm (10 in) sample with approximately a 19 mm shear zone between a concrete and APJ segment evaluated in pure shear. These samples were prepared using diamond saw sectioning as previously described. The samples were bonded onto 150 mm (6 in) diameter aluminum platens using Devcon 3873. After the specimens were bonded the relief cuts for the shear zones were cut using the diamond saw. These specimens were then very easily broken in handling and required great care. These bonded specimens were then conditioned to the one of the four temperatures, 21°C, 4.4°C, -18°C, and -40°C (70°F, 40°F, 0°F, -40°F) for a minimum of four hours.

The tests were conducted in an Instron 1332 load frame at a loading rate of 5mm/min with the data being recorded using a Strawberry Tree data acquisition system. Three specimens were tested per each of the 4 temperatures and the averaged results are given in Table 3.2.

As with the normal bond stresses, the behavior of the bond was brittle with a relatively high modulus of elasticity until failure and then no ductility. Due to this measured behavior only the ultimate stresses are reported. The ultimate shear stress was above the material yield stress for temperatures above -18 ° C (0° F), which will force the bridge translation to be seen as deformation of the APJ material. At temperatures near and below -18° C (0° F), the shear bonding strength is lower than the material and will force the energy from the bridge motion to go into shearing the joint from its support. The ultimate shear bond strengths are summarized in Figure 3.8. These ultimate shear stresses will be used in analytical models to release the shear bond interface to simulate the cracks that will develop in the actual joint.

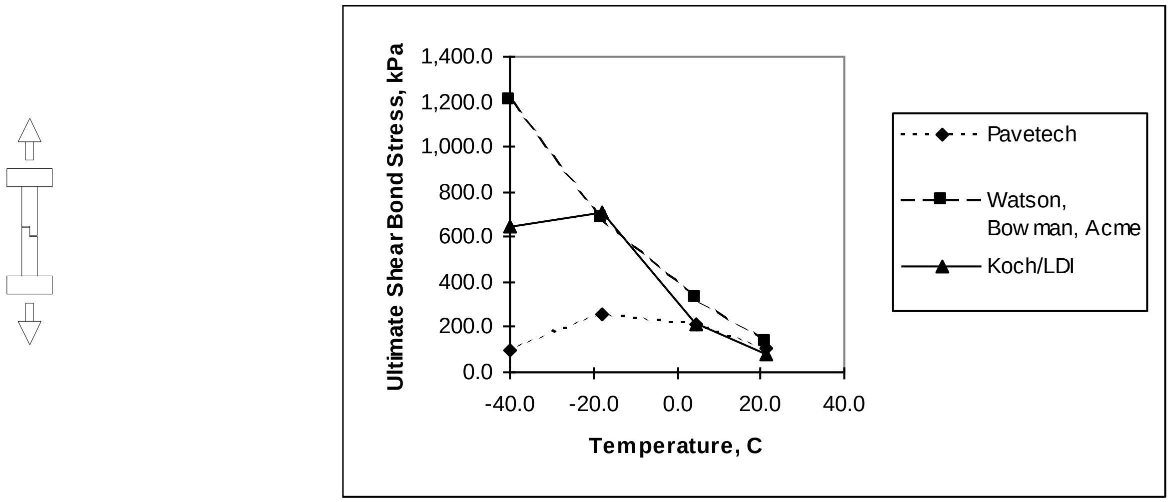


Figure 3.8 Ultimate Shear Bond Stress as a Function of Temperature

3.10 MODULUS RESILIENCE TEST

The resilience modulus test (M_r), ASTM D-4123, is a standard test for asphalt materials. It measures the modulus of the elastic or visco-elastic rebound by using a dynamic compressive loading of 1,2, or 3 Hz with a haversine loading wave form of one-tenth cycle duration. Since it produces a rapid loading rate, the results will demonstrate some of the visco-elastic properties of the material.

The resilience modulus is an indirect tensile test where a 150 mm (6 in) diameter by 75 mm (3 in) thick core is placed in compression and the tensile deformations are measured diametrically normal to the compressive load. From an assumed Poison's ratio of 0.35, the apparent tensile loads are calculated and the indirect tensile stress-strain diagram can be constructed. From this diagram, the modulus of resilience is defined as the slope of the unloading cycle, with the two points for computing the long-term modulus of resilience being the point at which unloading begins and where the next loading cycle starts.

The tests were conducted according to the ASTM standard with one exception. The temperature 40°C (102°F) was disregarded and the temperature of -40°C(-40°F) was added. The 3-Hz loading with a 0.3 sec duration was selected. The average results for three samples per temperature are shown in Table 3.3.

Table 3.3 Modulus of Resilience Results

	Pavetech	Watson, Bowman, Acme	Koch/LDI
Temperature	**M_R, kPa**	**M_R, kPa**	**M_R, kPa**
21°C	**16,000**	**8,900**	**15,000**
4.4°C	**94,000**	**28,000**	**56,000**
-18°C	**1,200,000**	**880,000**	**330,000**
-40°C	**1,400,000**	**980,000**	**410,000**

This test represents the loading due to traffic where a wheel load will be traveling down the pavement and is on the joint for only a short time. However, it is not a good simulation of the loading condition on the bridge joint which is not as rapid. The M_r results were consistently higher than the modulus of elasticity and were converging as the temperature decreased. The convergence suggests that the APJ material was approaching an elastic material as the temperature decreased.

3.11 Georgia Loaded Wheel Test

The Georgia Loaded Wheel Test (GLW) is a rutting evaluation test for asphalt pavement (*4*). It is an accelerated test where a 150 mm (6 in) diameter by 75 mm (3 in) thick core is placed in a machine that exposes it to a set number of cycles from a standard "wheel" and the resulting rut depth is measured. The test is run at an elevated temperature of 46°C (110° F) with a 45-kg steel wheel running on top of a pneumatic hose inflated to 690 kPa (100 psi) for finite increments up to 8000 total cycles. For asphalt pavement, acceptable rutting performance has been correlated with a total rutting depth of less than 7 mm (0.3 in) at 8000 cycles of the GLW.

This test is only useful as a comparative test for APJ since they are softer than asphalt pavements. All of the APJ specimens failed to meet the GLW criteria and we would expect these joints to rut. The measured rut depths are shown in Figure 3.9. Since all of these samples failed by 4000 cycles, data was not collected for the 8000 cycles. As expected rut depth approximately followed the same trend as the modulus of elasticity, being the lowest for the highest modulus of elasticity.

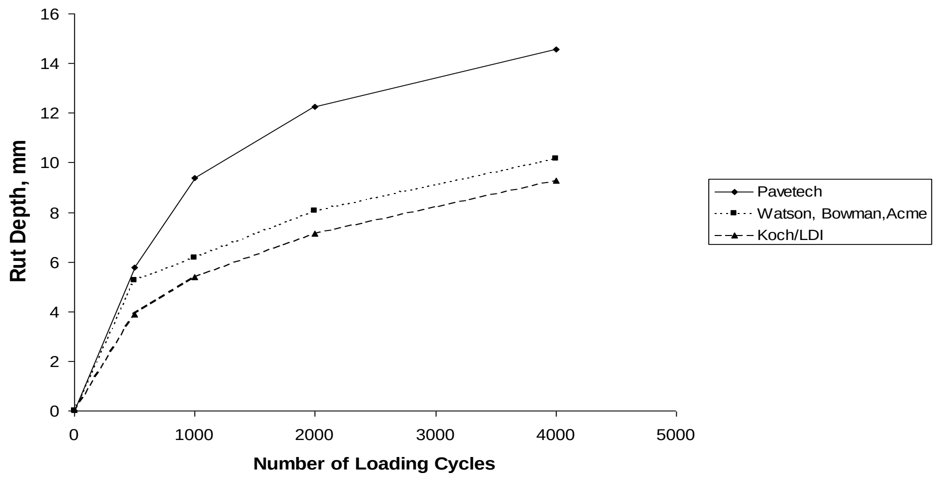


Figure 3.9 Georgia Loaded Wheel Results at 46°C

3.12 THERMAL STRESS RESTRAINED SPECIMEN TEST (TSRST)

The TSRST is a low-temperature test to evaluate the ability of an asphalt pavement to resist the internal stresses developed through cooling (*5*). The test apparatus actively maintains the original length of the specimen and measures the force induced by the thermal contraction as the temperature drops. At some temperature the internal stress will equal the materials resistance and a brittle failure will occur. This has become a standard SHRP test to evaluate cold weather performance of asphalt pavement (*5*).

This test applies to APJs by giving the lowest-temperature bound for their application. This test is equivalent to a locked (zero motion) bridge joint undergoing a temperature drop. Therefore, the TRSRT results indicate the temperature at which joint failure should occur even with zero movement. This is a non-conservative bound because joint movement puts additional demands on the material.

The TSRST also indicates the glass transition temperature. The glass transition temperature, T_g is an important property of the highly modified APJ binder. The glass transition temperature is a temperature where a change in mechanical behavior occurs.

Above T_g the APJ binder behaves like a ductile solid or highly viscous liquid. Below T_g, the material behaves as a brittle solid. (*6*)

The TSRST results for the Watson Bowman Acme Material given Figure 3.9. The fracture temperatures were -27°C, -43°C and -26°C (-16.6°F, -45°F, and -14.8°F) for Watson Bowman Acme, Koch/LDI, and Pavetech, respectively. The TSRST Temperatures were repeatable with a variation of less than 2°C (3.6°F) for each material. For all three materials the glass transition temperature closely corresponded to the fracture temperature.

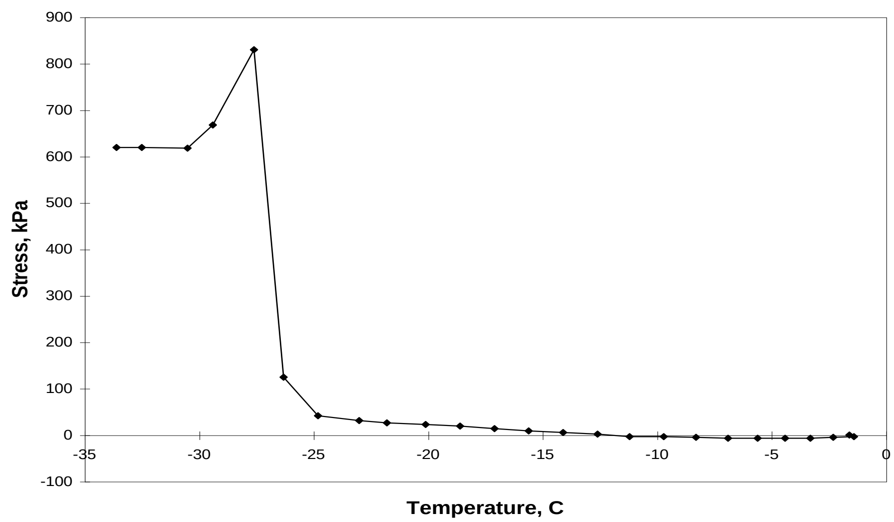


Figure 3.10 Watson Bowman Acme TSRST Results

3.13 CONCLUSIONS

The conclusions from the material characterizations of APJ materials are:

- Material is reasonably well defined by an elastic perfectly plastic constitutive model.
- Adhesion is well described as a brittle material and may be modeled as a "soft" interface.
- Material properties have been determined by direct tension tests and are given in Tables 1 and 2.

- Modulus of elasticity is consistently lower than the modulus of resilience and should be used for analytical studies.
- The GLW rutting requirement will not be met.
- The TSRST is a reasonable non-conservative lowest-temperature bound for APJ application.
- These tests are appropriate to qualify APJ materials and establish parameters for modeling joint behavior.
- Viscoelastic effects were not directly assessed in this series of tests.

3.14 ACKNOWLEDGMENTS

Any information in this paper is the sole opinion of the authors and should not be misconstrued as the opinion of the funding agencies.

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4.0 ASPHALT PLUG JOINTS: ANALYTICAL DEVELOPMENT OF DESIGN GUIDELINES

4.1 ABSTRACT

Highway builders and rehabilitators throughout the United States are using asphalt plug joints (APJs) in bridge expansion joints following manufacturer recommendations. The manufacturers generated these recommendations strictly through application experience with little to no engineering basis. U.S. departments of transportation are installing joints in unsuitable sites or are overlooking other sites where APJs would work well. To aid bridge engineers in using this class of joints more effectively the University of Wyoming (UW) has established a research program to develop rational design guidelines. The research should clarify suitable applications, characterize materials, develop design guidelines, and validate them. This paper addresses developing design guidelines based on material characterizations and failure modes already determined in previous studies. These design guidelines were derived using finite elements as a tool to aid understanding the joint displacement fields and to uniformly impose failure criteria over broad temperature and stiffness ranges. Future work will address validation and finalized design/specification guidelines.

4.2 INTRODUCTION

Asphalt Plug Joints (APJs) are expansion joints placed between pavement and bridge deck, see Figure 4.1. They are conceptually simple, easy to install, and readily maintained. APJ usage in the United States has been hindered by inconsistent performance primarily related to the lack of:

- Fundamental understanding of the materials and their performance over operational temperature ranges.
- Understanding of the failure modes and their associated causes.
- Information about the application of the material in engineering design.

- Mathematical models incorporating material characterizations to predict APJ behavior including failure modes, geometric effects, and
- Consistent and clear installation methods.

Installers currently follow manufacturers' application guidelines in the "design" and installation. These guidelines often state that APJs may be used where the anticipated bridge motion is ± 25 mm (1 in.) relative to the joint-setting temperature. APJ performance varies widely across the United States as seen in a survey of US state departments of transportation (*1*). Though APJs perform well in many installations, in other cases they do not. The data seem to indicate that APJs comprise a worthwhile system but the methods upon which to design and install joints must be refined. The research reported here focuses on refinement through numerical simulations. The analysis assumptions, material and geometric modeling, and load conditions are outlined and form the basis for a possible design approach.

4.3 RESEARCH APPROACH

The research started with surveys and field-based inspection to determine the performance, failure modes, and possible refinements required for proper APJ design and installation. Performance observations have established possible failure modes (*1*). The public can obtain little information on the engineering properties of the materials used and their relationship their range of operational temperatures. Thus, previous research had to provide physical tests to establish material properties (*2*). Experimental data helped develop constitutive models; a linearly elastic-perfectly plastic model may reasonably characterize the material behavior. Research also employed numerical analysis models to aid in predicting APJ behavior and their performance with the surrounding structures. To this end, the study used ANSYS, a finite element package (*4*) to model APJs. The ANSYS plane-strain model included material nonlinearities, large deflection theory, and the effects of debond failure along the boundary reactions, making it a comprehensive analysis tool. The model load consisted of an imposed translation that modeled bridge movement. Material properties due to changes in temperatures were also included. The joint was assumed to be in "thermal equilibrium" eliminating the need to model the heat transfer. Additionally, small strain ratings rendered viscoelastic effects for the daily movements inconsequential in the model. The failure criteria, described later, were derived through physical tests outlined elsewhere (*2*), including normal bond, shear bond, tension, thermal stress restrained specimen test, Georgia Wheel Load, and modulus of resilience. These tests were conducted at four discrete temperatures on materials supplied by the three manufacturers.

Finite element models helped establish the design guidelines and capacity curve for various materials. These models permit the extrapolation of material tests/properties to the physical joint. Because this extrapolation/modeling is being validated with near full-scale tests, the guidelines reported herein are preliminary.

4.4 JOINT DESCRIPTION

An APJ, a blockout filled with a binder-rich asphaltic material, provides a smooth transition between the pavement and the bridge deck, Figure 4.1. The joint typically contains a backer rod, gap plate, and asphalt binder/aggregate mix. The backer rod serves as a construction dam to keep the initial binder in place between the gap plate and the blockout. In turn, the gap plate keeps the binder/aggregate mix from extruding into the gap separating the abutment and bridge deck. The asphalt material, generally composed of a highly modified binder and a gap-graded aggregate, creates an asphalt segment with a voids in mineral aggregate (VMA) of between 20 and 40 percent. This binder-rich asphalt segment behaves as a viscous fluid at high temperatures and as an elastic solid at cold temperatures.

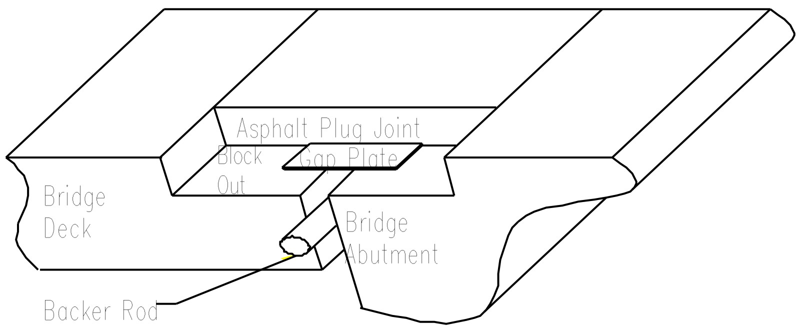

Figure 4.1 Typical Asphalt Plug Joint Cross Section.

It is this high viscosity and low elasticity that allows APJs to span the constantly narrowing and widening gap between abutment and bridge deck while meeting serviceability demands. As state bridge engineers put it, APJs primary serviceability requirements are to remain flexible enough to allow bridge movements with temperature variations, prohibit debris from entering the gap between the abutment and the deck, remain water tight, and provide a smooth transition between the pavement and the bridge. These functional requirements formed the basis for establishing failure modes.

The typical blockout ranges from 500 mm (20in) to 610 mm (24in) wide and is at least 50 mm (2in) thick. These joints can span between concrete and/or asphalt sections. Installers begin by casting the blockout into the deck and abutment or cutting the blockout into the asphalt overlying. They then sandblast the blockout to remove foreign material. The joints are then dried with heat lances to remove any residual moisture from the concrete formwork or cutting operation. Once the backer rod is installed, installers coat the blockout with heated APJ binder. Next, they lay gap plates into this hot binder coating and secure them with nails. Next, another coating of heated APJ binder is applied to the entire blockout surface. Installation requires that the heated aggregate/binder mix is placed in three lifts that each sandwich a filler coat of binder. The joint is then compacted, typically using a 2-ton compactor. A final coating of APJ binder is then topped by a topcoat of dry aggregate to help reduce binder track out. This final coating of binder levels the joint and acts as an adhesive for the fine aggregate, creating a binder-rich topping that helps seal the joint. Unfortunately, this layer also allows joint material to flow easier under traffic loads and contributes to observable rutting.

4.5 MODELING ASSUMPTIONS

The numerical models include reasonable and conservative simplifying assumptions based upon experience, observation, and material tests . Beyond the assumptions associated with plane strain finite element models this research further required the following assumptions:

- APJs act as linearly elastic-perfectly plastic material.
- Large-deflection theory applies.
- Poisson ratio is a constant 0.35 and not effected by temperature.
- Normal bond failure is brittle.
- Shear bond failure is brittle.
- The total strain used to define failure equals 5 ε_y.
- The bridge motion is considered quasi-static.
- Shear bonds detached with movement re-attaches under traffic pressures at temperatures above 0°C (32°F).
- The APJs material "resets" its initial setting strain over time due to its viscoelastic nature.
- The blockout-surfaces are considered as rigid boundaries.

4.6 MODELING ASSUMPTION RATIONALES

The material characteristics were determined experimentally at four discrete temperatures 21°C, 4.4°C, -18°C, and -40°C (70°F, 40°F, 0°F, -40°F) as summarized in Table 4.1. Because a perfectly plastic material model is unstable, this study used a bilinear material model that incorporated a hardening modulus of one percent of the modulus of elasticity. The material model used for Pavetech at 21°C (70 F) is shown in Figure 2. By limiting the total strain to five times the yield strain, the maximum stress increase due to this modeling assumption is 4%. This slight increase in stress was felt to be acceptable.

Table 4.1 Material Specimen Results

	Pavetech			Watson, Bowman, Acme			Koch/LDI		
Temperature	**E, kPa**	**σ_y, kPa**	**ε_y**	**E, kPa**	**σ_y, kPa**	**ε_y**	**E, kPa**	**σ_y, kPa**	**ε_y**
21°C	**1,300**	**27**	**0.020**	**4,600**	**130**	**0.029**	**1,500**	**44**	**0.030**
4.4°C	**9,300**	**130**	**0.014**	**14,000**	**180**	**0.015**	**7,800**	**150**	**0.019**
-18°C	**205,000**	**830**	**0.006**	**150,000**	**750**	**0.006**	**87,000**	**520**	**0.006**
-40°C	**607,000**	**1,500**	**0.003**	**929,000**	**1,700**	**0.002**	**342,000**	**500**	**0.001**

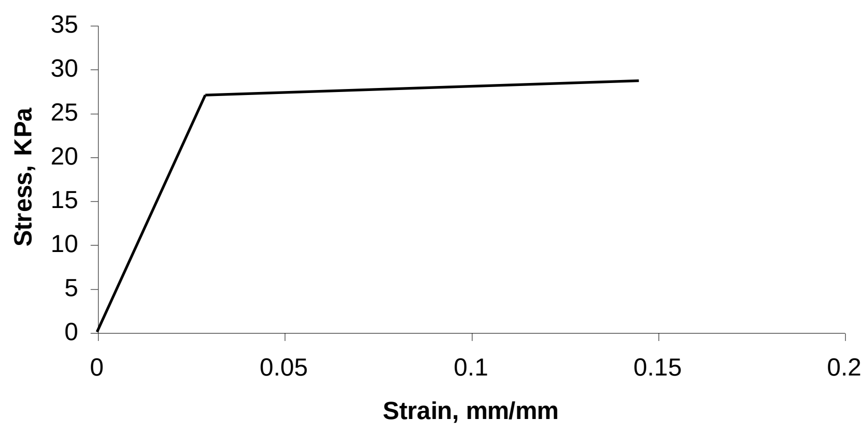

Figure 4.2 Pavetech Material Model at 21°C (70°F)

The material surrounding the APJ is concrete and/or asphalt. These materials would have modulus of elasticity in from 27,600 to 48,300 MPa (4,000,000 to 7,000,000 psi) for the concrete and 1,300 to 4,500 MPa (190,000 to 656,000 psi) for the asphalt. Because both of the blockout materials have moduli of elasticity at least an order of magnitude greater than the APJ material factor, it is appropriate to model the blockout surfaces as rigid. The models used represent a perfectly rigid bonded surface around the APJ by restraining the translation of the nodes on the blockout surface. Previous material tests confirm that this is a valid assumption (*2*). Additionally, models incorporated a bond interface stiffness approximately equal to the APJ modulus of elasticity until the APJ reached its stress limit, when the joint experienced brittle failure. Table 4.2 contains the failure stresses used to control this bond behavior.

Material tests demonstrated that the material remains plastic well past the yield point (*2*). For temperatures greater than 4.4°C (40°F) specimen failure did not occur until approximately seven times the yield strain. Limiting the total strain to five times the yield strain allows the analysis to take advantage of the plastic behavior of the material while invoking a safety factor of 1.4 against failure.

Table 4.2 Normal and Shear Bond Stress

	Pavetech		**Watson, Bowman, ACME**		**Koch/LDI**	
Temperature	**Normal σ_u, kPa**	**Shear σ_u, kPa**	**Normal σ_u, kPa**	**Shear σ_u, kPa**	**Normal σ_u, kPa**	**Shear σ_u, kPa**
21.1°C	**67**	**110**	**90**	**130**	**98**	**76**
4.4°C	**220**	**210**	**260**	**330**	**190**	**220**
-17.8°C	**330**	**260**	**770**	**690**	**330**	**710**
-40°C	**340**	**97**	**640**	**1200**	**170**	**650**

The bridge motions under investigation are due to daily and seasonal movement. These motions are due to the temperature changes in the bridge. Since these temperature-driven movements generally take hours due to the thermal inertia of the bridge, the motions seen by the joint are slow enough to be considered quasi-static.

During material testing, gravity loads provided enough pressure to bond material samples to a horizontal surface. This happened within a few hours for temperatures greater than -18°C (0°F). The conservative implication is that the shear bond separated in movements is reattached to the blockout as the joint relaxes to a seasonal norm. The limiting temperature of 0°C (32°F) is imposed to account for the possibility of ice forming at the crack interface after moisture infiltrates the joint from below the bridge and prevents re-adhesion. The implication of this assumption is that the de-bonded length is two times the length of the motion, which accounts for the cyclical temperature variation about a seasonal norm.

The APJ material is viscoelastic (flows over time). Therefore, unlike a bridge that has a distinct "setting" temperature, or neutral temperature about which no temperature-induced stresses or strains are present, APJs constantly relieve their internal stresses. A resulting <u>conservative</u> assumption states that the long-term seasonal temperature norms are the "set" temperatures, and the daily temperature variations are rapid enough to be considered strain-inducing events. This allows the seasonal normal length (and temperature) to be the neutral length about which strain is induced. Also, this temperature becomes the temperature at which the joint's motion capacity is checked.

Near-full-scale tests, and material tests reported in other papers (*3,6*), indicate that the stress relaxation occurs very rapidly, i.e. in a few minutes for most materials. Henceforth, the analytical work performed in this section is conservative when such extreme (and typical) relaxation is present.

4.7 FAILURE MODES

The study assumed failure criteria based upon tensile problems seen in the field, including splitting in tension, leaking, de-bonding, and, to some degree spalling. Once any model exceeded the following failure criteria they were deemed to have failed and the analysis was terminated.

- Exceeding the normal bond strength at the joint boundary conditions.
- Exceeding the shear bond strength at the joint boundary conditions.
- Exceeding a total strain of 5 ε_y.

The FEA analysis was only concerned with tensile failures, which allowed one set of boundary conditions to be used on all models. The boundary conditions are fixed in horizontal and vertical to the left of the gap plate and positive horizontal translation and fixed vertical translation for the gap plate and to the left of the gap plate. Figure 4.3 illustrates a typical 500 mm x 100 mm (20 in x 4 in) plug joint model. This study used solid Quad 82, an eight-node quadrilateral element, to model the joint, (*4*). This element type assumes a quadratic strain field in each element, allowing for a coarse mesh, and decreased solution times.

Material nonlinearities require the path taken to get to the final joint displacement. For these analyses, all models were loaded in a ramped displacement until one of the failure criteria was reached. The loading displacement at which one of the failure criteria was met can be considered the maximum allowable motion for the modeled joint geometry and material characteristics at that joint temperature.

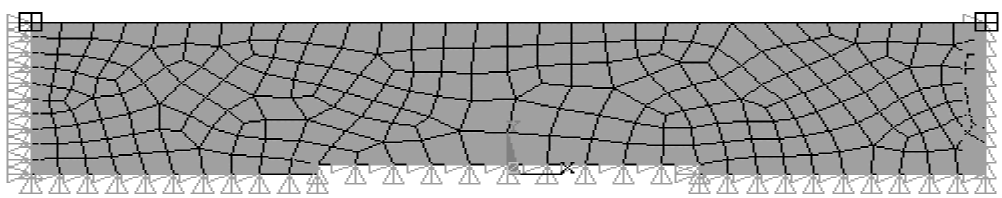

Figure 4.3 500 mm x 100 mm (20 in x 4 in) Plug Joint

No models terminated due to debonding of the shear interface. At temperatures greater than 4.4°C (40°F), the total strain ($\varepsilon > 5\varepsilon_y$) condition controlled. At temperatures less than -18°C (0°F), the normal bond criterion controlled. This change in the controlling failure criterion came because at temperatures below –18°C (0°F) for Koch/LDI and Pavetech and –40°C (-40°F) for Watson Bowman Acme, the normal bond strength was lower than the material elastic yield and therefore controls failure.

One additional failure mode needs to be imposed outside of these analytical modes. Low temperature bounds provide the failure temperatures generated by the thermal stress restrained specimen test (TSRST), the temperature at which the material fails due to its own thermally-induced strains without any bridge movements. This temperature is also generally equivalent to the glass transition temperature, T_g, at which the material transforms from being a ductile material to a brittle material. These low temperature limits are discussed more fully in previous work (*2*). The low temperature limits are -27°C, -43°C and -26°C (-16.6°F, -45°F, and -14.8°F) for Watson Bowman Acme, Koch/LDI, and Pavetech, respectively. The importance of this test is that it allows these manufacturers to reformulate their products to achieve a lower low temperature bound.

4.8 ANALYSIS RESULTS

Figure 4.4 shows a typical strain field for a 500 mm x 100 mm (20 in x 4 in) plug joint. Stresses and strains developed an inverted arch with the legs on the surface of the plug joint and its top at the sliding edge of the gap plate. Joint motion strains the APJ material enveloped within this arch, but little is being shared with the material outside of this arch. Thus, although we have a 500 or 600 mm (20 or 24 in) long joint, only a small length is effectively developing the strain field with bridge motion. This ***effective length*** is a function of the joint geometry, modulus of elasticity, and yield strength. Because the material properties are a function of temperature, this makes the effective length a function temperature as well.

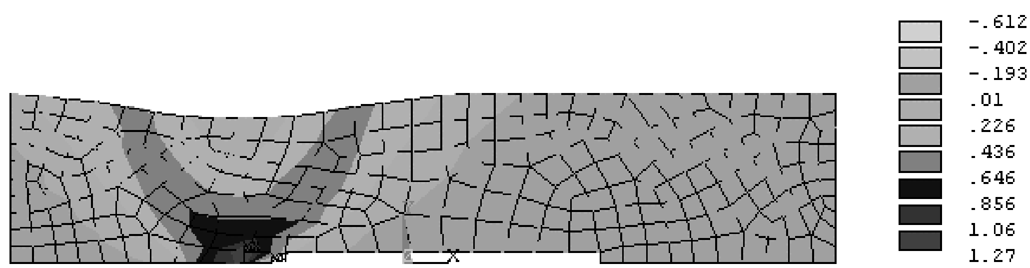

Figure 4.4 Pavetech Horizontal Strain at 21°C (70°F)

The effective length of the 500 mm x 100 mm (20 in x 4 in) is seen by examining the horizontal translations across the top of the joint, Figure 4.5. The non-linear curve inside the effective length is generated by a slightly higher strain distribution in the arc legs having than in the material in the middle of the arch. This nonlinearity becomes more distinct as the thickness of the plug joint increases and the legs of the arch spread further apart.

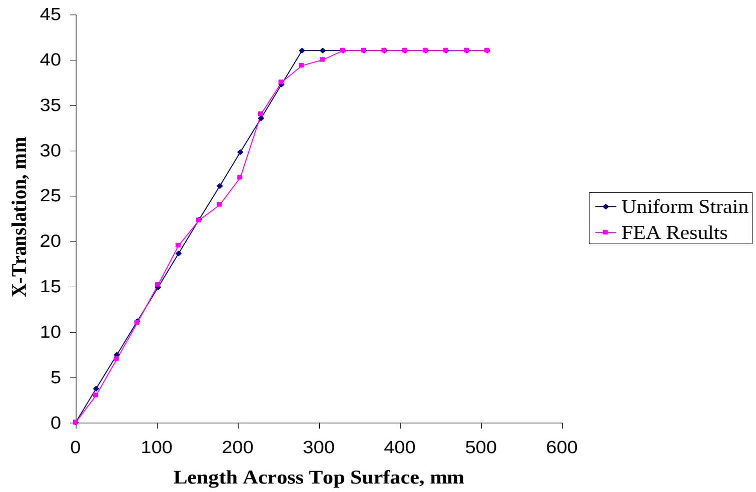

Figure 4.5 Horizontal Translations Across the Top of a Koch/LDI Joint at 21°C (70°F)

Because the material outside of the arch does not contribute to the effectiveness of the joint, it is not needed for the joint to meet its functional demands. This extra material adds to the expense of the joint and increases the opportunities for serviceability failures, especially rutting. This extra material could be eliminated with a revised joint geometry.

The dashed line on Figure 4.5 represents the assumed total strain limit of $5\varepsilon_y$. When the material total strain controlled the failure criteria, $5\varepsilon_y$, this constant strain over the entire effective gage length gives the allowable movement for a joint configuration. When normal bond strength controls the joint failure, the values for the allowable movement come from the finite element models. The allowable motion is the joint displacement at which a failure criterion is first reached. Figures. 4.6, 4.7, and 4.8

provide the allowable movements as functions of temperature for Pavetech, Watson Bowman Acme, and Koch/LDI.

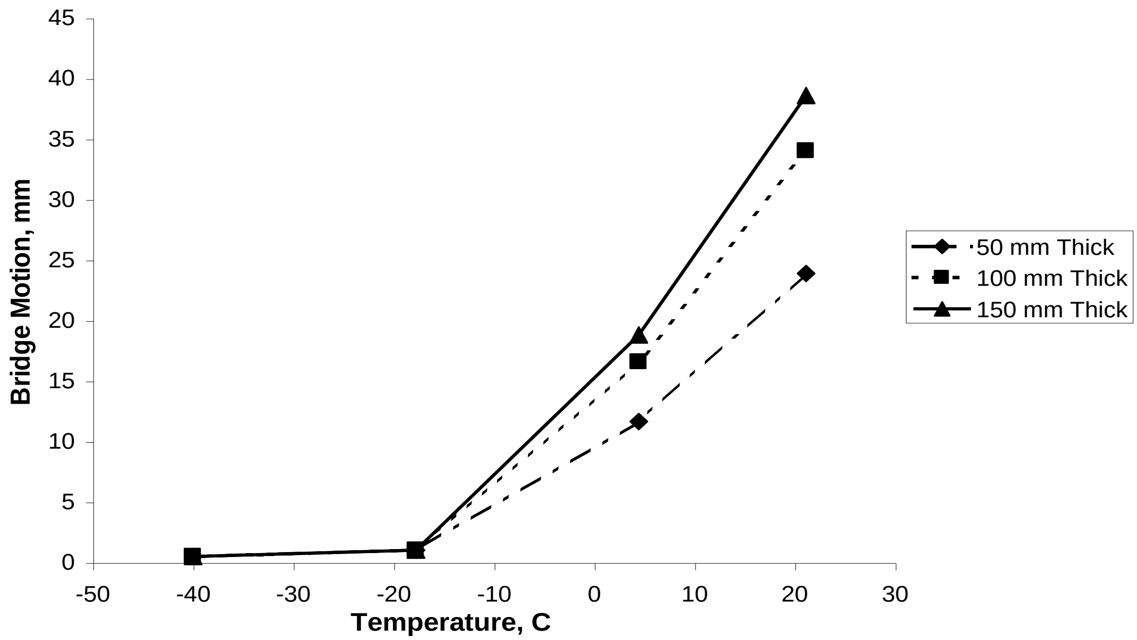

Figure 4.6   Pavetech Allowable Motions

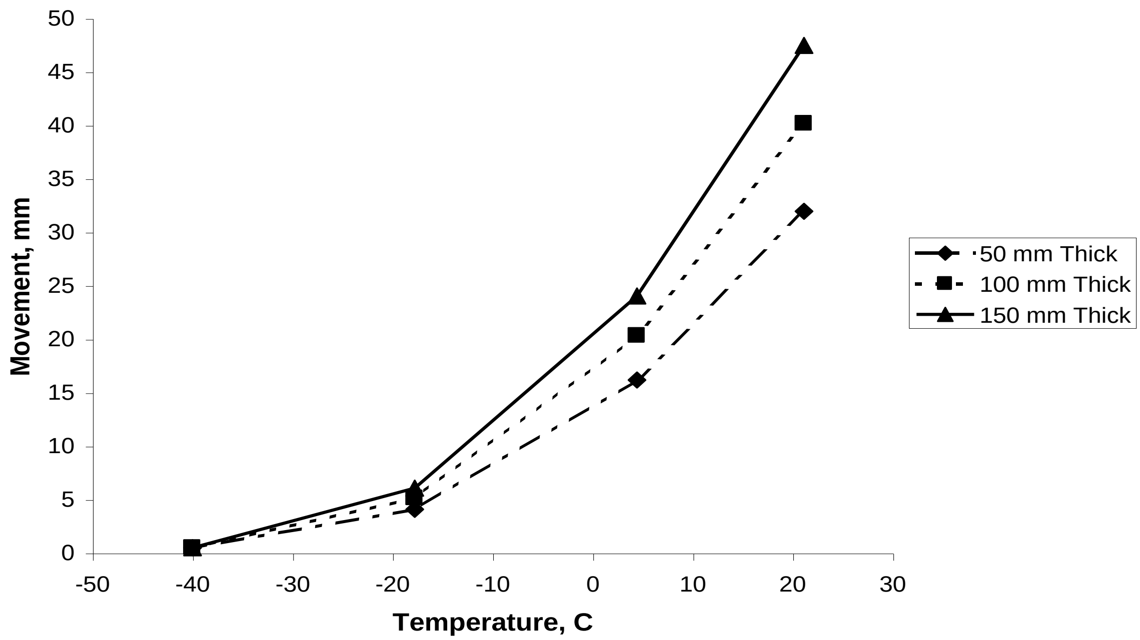

Figure 4.7 Watson Bowman Acme Allowable Motions.

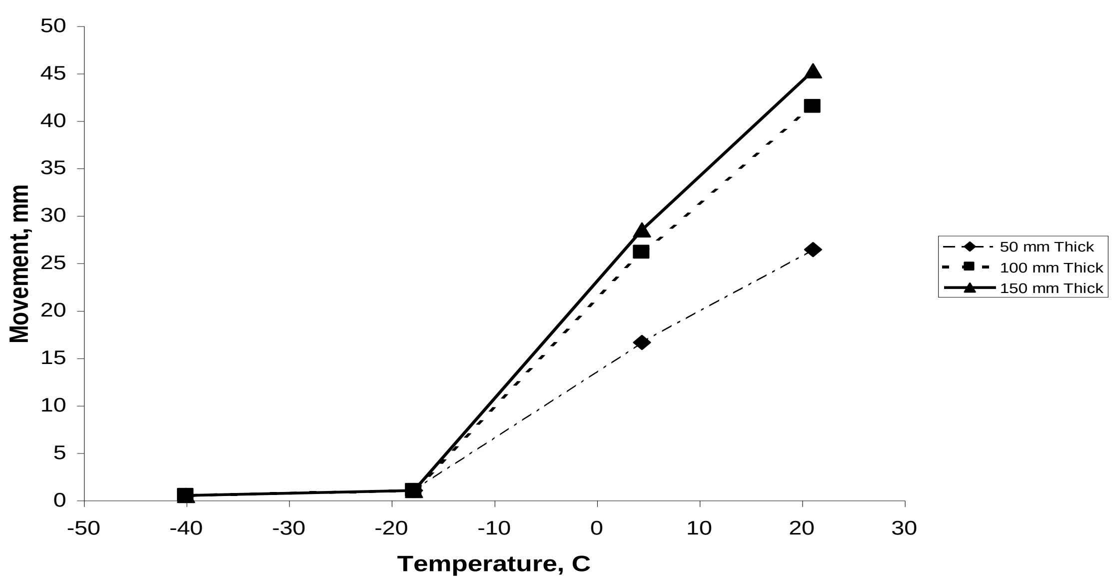

Figure 4.8 Koch/LDI Allowable Motions.

Though the analyses were conducted on 500 mm (20 in) wide joints, the allowable movements are directly applicable for both the 500 mm (20 in) and 600 mm (25 in) joints because the blockout boundary or joint length only becomes the controlling factor at low temperatures, where the limiting failure criterion is the bond strength. The failure criterion of normal bond is a strength or stress limit. These stress criteria are relatively insensitive to whether the effective gage length begins at the blockout boundary or elsewhere. At these cold temperatures, the normal bond criterion is severe because the bond strength is much less than the yield stress. This forces the normal bond to become the "weak link", and likely the greatest probability of failure.

Joints less than 500 mm (20 in) wide have shorter effective lengths for the 100 mm and 150 mm (4 in and 6 in) thick joints because the effective gage length arch would be restricted from fully developing.

The FEA models are providing qualitative results similar to observed field performance. They indicate that thin joints fail directly over the gap plate while thick joints fail at the blockout interface, or further away from the edge of the gap plate. The deformed shape is also of the form observed in the field. It is due to this failure directly above the edge of the plate that the joints are not recommended for installation where their thickness would be less than 50 mm (2 in).

4.9 APPLICATION OF ANALYTICAL RESULTS

With the allowable temperature-movement relationship established, it is possible to determine if a joint is appropriate for a particular application or bridge site. The limiting condition for design will be the winter condition because this time is the most severe for tensile-type failures. Assessment requires knowledge of the winter normal pavement temperature and the lowest expected pavement temperature to be known. The guidelines outlined in Superpave SP 1 (5),which are based on readily available air temperatures are recommended to obtain this information. A possible procedure for determining the appropriate joint is as follows:

1. Determine the winter seasonal average pavement temperature.

2. Calculate the lowest expected pavement temperature.
3. Using the bridge's temperature movement factors, calculate the motion associated with the temperature difference between the seasonal average and the lowest expected bridge temperature.
4. Using this maximum seasonal motion, check the material capability at the winter seasonal average temperature for joint adequacy.
5. Check the material low temperature limit.

This finite element method study is based on quasi-static loading. It does not directly include fatigue effects, especially at the stress concentration locations. Full-scale or near-full-scale tests are needed to confirm these recommendations.

4.10 EXAMPLE

To illustrate the FEA design approach

Given: A Bridge in Cheyenne Wyoming with a temperature motion of 0.55 mm/°C (.04 in/°F)

Solution: Pavement temperature range: High 58°C (142°F) 99.9 % reliability (*5*)

Low -28°C (-21 ° F) 94.8 % reliability (*5*)

Seasonal Average Temperature: –7°C (19°F)

Maximum Seasonal Movement: = 0.55 mm/°C × (-7°C – [-28°C]) = 11.55 mm (.5 in) movement.

From Allowable Movement Charts with 12 mm (0.5 in) movement and –7 °C (19°F), four choices are possible: Watson Bowman Acme 100 mm (4 in) or 150 mm (6 in) thick, or Koch/LDI 100 mm (4 in) or 150 mm (6 in) thick. Pavetech is eliminated due to limited movement range at seasonal average temperature.

From Low Temperature Limit Watson Bowman Acme is eliminated and Koch/LDI is specified.

4.11 CONCLUSIONS

- Joint behavior has been investigated using the finite element method and has yielded reasonable and qualitative results.
- A simple, preliminary design procedure is outlined. This procedure provided valuable guidance for the development of full-scale joint tests.
- The effective gage length of an APJ is less than the actual plug joint length.
- An opportunity exists to optimize APJs to reduce their cost without affecting the functionality and may improve service characteristics.

4.12 RECOMMENDATIONS

- Joint geometric optimization should be conducted to remove non-contributing material from the joint.
- Finite element model results, need to be compared with full-scale joint segments in a controlled environment to validate their performance

4.13 ACKNOWLEDGEMENTS

The support of the Wyoming Department of Transportation, Colorado Department of Transportation, Koch/LDI, Watson Bowman Acme, and Pavetech is appreciated.

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5.0 ASPHALT PLUG JOINTS: NEAR FULL-SCALE JOINT VALIDATIONS

5.1 ABSTRACT

Highway builders and rehabilitators throughout the United States are using asphalt plug joints (APJs) in bridge expansion joints following manufacturer recommendations. The manufacturers developed these recommendations through experience and they include little engineering basis in their design. For this reason, U.S. departments of transportation are installing joints in unsuitable sites or are overlooking other sites where APJs may work well. To aid bridge engineers in effectively using APJs, the University of Wyoming (UW) established a research program to clarify suitable applications, characterize materials, develop design guidelines, and validate the design process. This paper addresses the near full-scale joint performance validations. The validation program is a series of laboratory tests that simulate a 50mm (2 in) motion on a bridge located in Cheyenne, Wyoming. The data obtained from these tests are compared to the previously-determined material characterizations, failure modes, finite element analysis, and proposed design guidelines. The testing highlights the importance of material relaxation to joint performance. Refined design guidelines include the relaxation characteristics in the joint design and the material specifications and this is presented later.

5.2 INTRODUCTION

Asphalt Plug Joints (APJs) are bridge expansion joints composed of highly modified binder-aggregate mix placed over a gap plate in a blockout spanning between the approach slab and the bridge deck, Figure 5.1. The joints are used in bridges with joint movements less than ± 50 mm (2 in). This paper is the fourth in a series to explore the possible advantages and potential of APJs. This program began with a laboratory investigation of a single joint (*1*). The performance of that joint was the basis of more detailed investigations that consisted of a survey of the state departments of transportation (*2*) to establish in-service performance. Material characterization (*3*) and preliminary development of design guidelines (*4*) led to this near full-scale joint validation program. Each research stage builds upon the foundations established by the preceding work. The survey amplified the knowledge gained from the initial sample joint test. The material characterization determined experimental methods to predict some of the failure modes identified in the survey. The initial design guidelines are based on both the failure modes identified in the survey and on the material characterizations. The near full-scale joint validations presented in this paper validate, disprove, or augment the design assumptions and provide a basis for refinement of these guidelines. The near full-scale joint validations are a series of tests that were conducted on 0.9-meter (36-inch) wide APJ samples. The samples are tested in an environmental chamber under cyclic loading simulating five years of daily bridge motion on a ±50mm (2 in) motion bridge in Cheyenne, WY. The environmental chamber provides the thermal excursions associated with the bridge movements.

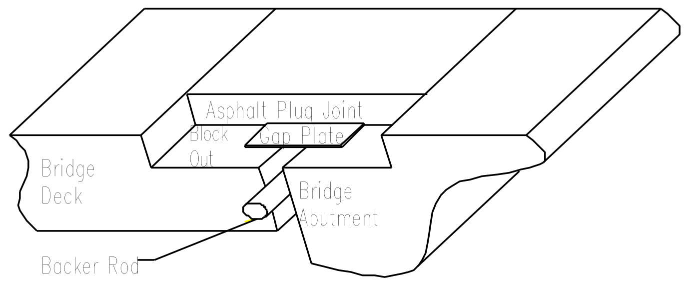

Figure 5.1 Typical Asphalt Plug Joint Cross Section

The basis for the design of APJs is that the relaxation of the plug joint material results in sufficient stress reduction that a tensile failure or a debonding failure is precluded. Relaxation is defined as a reduction in load/stress for a given translation or strain and is due to plastic flow of the material. The load may be related to displacement as the material stiffness times the applied displacement (the difference between the current imposed displacement and the setting displacement). For bridge joints, the translations are directly related to the temperatures. If the stiffness remains constant, which is a valid assumption at a nearly constant temperature, then the reduction in the force can be viewed as either a free flow of the joint material or conceptually as an effective reduction in the total applied displacement. If the relaxation continues to relieve stress as fast as the applied displacement induces it, then there is no effective residual stress in the joint when the next thermal cycle is applied. The process is repeated after every bridge movement. If this relaxation occurs faster than the structure can displace, then the loads/stresses present in the joint are minimal. The importance of this relaxation (or re-setting) behavior is essential to the APJ joint design and performance.

5.3 TEST PROGRAM AND OBJECTIVES

The validation tests in this program are designed to confirm the following hypotheses:

1. Material tensile behavior controls the joint design.

2. Joints may have a five-year minimum life based on 50 mm (2-inch) bridge motion under temperatures experienced in Wyoming and Colorado.

3. APJs re-set their initial strain to the seasonal average temperature through relaxation.

4. The relaxation time is less than the time to induce a thermal gradient, thereby allowing relaxation time to decrease the stress in the joint.

5. Non-linear finite element analysis is a valid tool to predict allowable motion.

6. APJs experience motion on only one side of the gap plate.

7. Joints autogenously 'heal' under traffic load and warm temperatures.

To validate these hypotheses, the APJ samples were subjected to translations that represented movement ranges for 1825 maximum daily (5 years) temperature cycles. The range of motion ranges was determined from the following scenario:

Given: A Bridge in Cheyenne Wyoming with a joint motion of 0.55 mm/°C (0.039 in/^{o}F). This motion is based on the maximum expansion and contraction of the bridge and its apparent point of thermal fixity.

Solution: Pavement temperature range: High 58°C (142°F) 99.9 % reliability (5) Low -28°C (-21 ° F) 94.8 % reliability (5). The maximum seasonal motion is determined by calculating the maximum seasonal temperature range and multiplying it by the bridge thermal expansion coefficient. The average temperatures, number of cycles, and displacement ranges for the bridge are given in Table 5.1.

Table 5.1 Seasonal Movements and Number of Cycles.

Season	Average Temperatures	Number of Cycles	Maximum Daily Movement
Summer:	25°C (77 °F)	456	19.2mm(.759")
Spring/Fall:	4.4°C (40 °F)	912	19.2mm(.759")
Winter:	–7°C (19°F)	456	25.4mm(1.0")

Use of the maximum daily translations in Table 5.1 is conservative and represents a severe in-service condition. By using the full daily temperature translation range, the five-year simulation has every single day experiencing a full seasonal temperature variation.

The ability of the plug joint to reset its initial strain through relaxation is paramount for the structural integrity of the joint. If the joint is not able to reset its initial strain, then the translation inducing the strain due to the temperature difference is cumulative from its installation temperature to its low/high service temperature. For our hypothetical bridge, the maximum joint translation is 27.8 mm (1.09 in) from the installation temperature (or a 50 mm (2 in) total motion). This compares to a maximum daily translation of 11.6 mm (0.46 in) assuming that the relaxation resets to the seasonal residual strain. Measuring the relaxation of the sample allows determination of the time period for the load to relax to below five percent its initial load.

The finite element analysis (FEA) is correlated with the load-deflection curve and the measured surface deflections. The surface deflections show a reflection of the bonding condition on the bottom of the APJ. Due to the thermally-induced movements, the joint either expands or contracts. These movements force the gap plate to move relative to the block-out bottom and create debonding below and adjacent to the plate. Debonding may occur on either end of the gap plate, however, experience and logic suggest that only one side of the plate slips. All motion is forced into the free-edge zone of the gap plate. The debonded region provides a larger "effective" gage length over which the bridge motions are distributed. The surface measurements and examination of the failed joint confirm debonded region behavior and whether debonding occurs on one or two sides of the gap plate.

Field investigations indicate that the APJs typically show no signs of tensile cracks in the traffic areas while tensile cracks are apparent on the shoulders (*2*). It is therefore believed that the traffic has a "healing" effect. This condition may be simulated by placing a partially failed joint at an elevated temperate of 32°C (90°F) and precompressing the joint. After the sample is allowed to sit undisturbed it is then loaded. The measured load-deflection curve is compared to the non-failed joint to determine if healing has occurred.

5.4 JOINT SAMPLES AND FABRICATION

The joint samples consisted of an APJ specimen placed in a concrete mold. The concrete mold is designed to provide material test samples, and a joint comparable to that found in the field (3). The joint portion is designed to attach to a load frame to simulate a moving bridge deck and a fixed abutment. One 500 mm (20") wide and 100mm (4") thick APJ sample was obtained from each manufacturer. Figure 5.2 shows a joint sample in the load frame and environmental chamber.

Figure 5.2 Joint Sample Test Setup

Manufacturers provided sample APJs either from their factory locations (Pavetech, Watson Bowman Acme) or from a field (Koch/LDI) installation. The field-placed joint (Koch/LDI) was installed in less than optimum conditions due to snowfall. This joint was also not as well compacted as actual bridge joints due to equipment limitation on the site and the size of the test mold. An experienced crew superintendent felt that the joint sample was "good" but not above average. This joint is considered as a conservative representation of a Koch/LDI service installation.

The APJ was installed roughly following the manufacturer's normal procedures. The following general procedure installed the joint material in the mold:

- Sandblast the mold to remove foreign material and roughen the surface.
- Dry the sample with a heat lance to remove residual moisture.
- Place the backer rod between the concrete faces in the mold.
- Coat the bottom of the blockout with heated APJ binder.
- Place gap plates into the hot binder coating and temporarily secure them with nails into the backer rod.
- Apply additional coating of heated APJ binder over the entire blockout surface.
- Place the heated aggregate/binder mix in three lifts sandwiching each layer with a filler coat of binder.
- Compact the material using methods to as close to field conditions as possible, generally with a 2-ton compactor.
- Place a final coating of APJ binder over the top of the sample to level the joint and act as an adhesive for the fine aggregate topcoat. The topcoat of dry aggregates helps reduce binder track out.

This final coating of binder-rich topping seals the joint, and as testing demonstrated, helped to increase the normal adhesion of the joint by adhering onto the top of the sample.

5.5 TEST SET UP

Samples were installed into a load frame encased in an insulated environmental chamber bolted to a structural floor in the Kester Structural Lab at the UW, Figure 5.2. The joint samples were cycled using an MTS 407 Controller and a 22.2 kN (5,500 lb) Hydraulic Cylinder at a rate of one cycle per minute. This represented a translation rate of between 37 and 50 mm/min (1.5 and 2 in/min). This rate is faster than the 5 mm/min (.2 in/min) rate used to determine the material characteristics, however, through testing, it was shown to give similar load deflection curves without the introduction of additional viscous loads.

Data were recorded from four channels electronically using Labveiw data acquisition software and SCXI data acquisition hardware in a 486 50MHz computer. The four channels recorded were cylinder stroke, cylinder load, environmental chamber air temperature, and plug joint temperature. The temperatures were recorded using type K thermocouples.

The samples were tested at three temperatures that approximately correspond to seasonal average temperatures in Cheyenne Wyoming; 25°C, 4.4°C and –7°C (77°F, 40°F, and 19°F). The temperatures were obtained by placing dry ice directly on top of the plug joint samples and allowing heat to transfer through conduction. Temperatures were controlled by manually monitoring the plug joint temperature being recorded from a thermocouple embedded one half of the way through the joint thickness and adding or removing dry ice. The embedded thermocouple represents the average temperature of the APJ material. Since the dominant heat transfer mechanism was conduction, the environmental chamber ambient air temperature was warmer than the joint.

5.6 TEST RESULTS

Figure 5.3 shows the first cycle of tensile load-deflection curve for Koch/LDI at 25°C (77°F). The load frame and data acquisition systems are configured so tensile loads and deflections are negative and compressive loads are positive. Therefore, the tensile load-deflection curve has been transformed to the first quadrant for clarity.

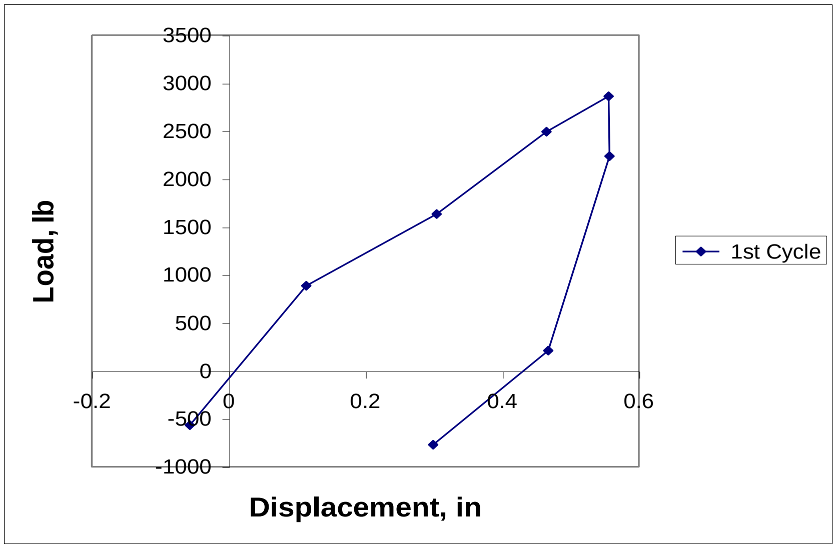

Figure 5.3 Koch/LDI 25°C (77°F) Tensile Load-Deflection Diagram

For this cycle, the "yield" point occurred at 3400 N (770 lb) whereas the equivalent joint reaction from finite element analysis (FEA) is 2710 N (605 lb) or a difference of 27%. The yield point reactions ranged from 7% to 31% for the various tests, Table 5.2. The lower percentage differences are all at the cooler temperatures. In all cases, the FEA predicted loads are lower than the test results for the same deflections. The reason for this difference is that all samples show a higher yield stress in the joint than was predicted by the small sample tests. The material confinement in the joint raised the effective modulus of elasticity beyond what was predicted from uni-axial materials tests. The behavior of the analytical model thus adds to the conservatism in the predictions of joint performance. Moreover, the low yield stresses indicate potential service problems if the traffic is moving slowly or stopped on the joint. Confinement is critical to the support of vertical loads such as truck tires.

Table 5.2 Comparison of Material Test and Joint Yield Stresses.

Joint Material	Material Test σ_y	Joint Test σ_y	% Difference
Temperature =25°C			
Koch/LDI	7.7 psi	10 psi	27 %
Pavetech	4.1 psi	5.6 psi	30 %
Watson Bowman Acme	19 psi	22 psi	16 %
Temperature = 4°C			
Koch/LDI	22 psi	24 psi	10 %
Pavetech	19 psi	14 psi	31 %
Watson Bowman Acme	26 psi	22 psi	18 %
Temperature = -7°C			
Koch/LDI	49 psi	43 psi	13 %
Pavetech	70 psi	54 psi	23 %
Watson Bowman Acme	68 psi	63 psi	7.8 %

The surface translations were measured using nails driven into the joint approximately 3-mm (1/8 in) deep, 25 mm (1 in) apart along the 500 mm (20 in) width of the joint. Two rows nails were used each spaced approximately 150 mm (6 in) from the edge of the joint. These surface translations were measured by stopping the test at the maximum tensile translation and measuring the translation of the nails with respect to the joint casting. These measurements were taken at 1, 5, 20, and 100 cycles. As can be seen in Figure 5.4, the translations along the surface shows significant scatter. The testing and finite element models correlate reasonably well. The joint translation is concentrated in slightly over half of the joint. This means that almost half of the joint is simply "unused." On the three tests, the joint motion was always concentrated on the fixed (abutment) side of the model. Field investigation does not support this finding. The gap plate may slip randomly on either end depending on the plate geometric placement and the binder adhesion. Designing the plate attachment for a more predictable movement is important and is addressed in later work.

Figure 5.5 illustrates the load-deflection hysteresis curve for 456 loading cycles of Koch/LDI at 25°C (77°F). The slopes of the loading curves are decreasing as the number of cycle's increases. This behavior is even more apparent as individual cycles are isolated as shown in Figure 5.6.

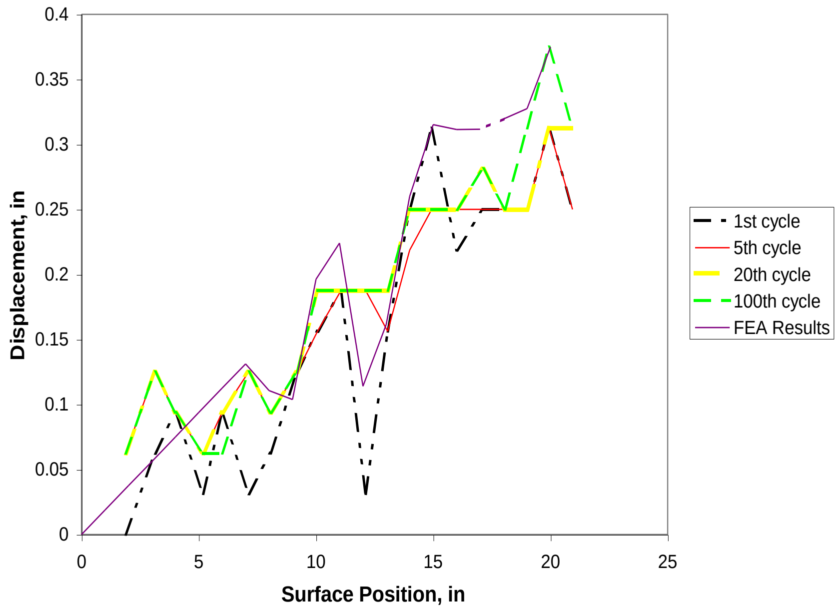

Figure 5.4 Pavetech 25°C(77°F) Surface Displacements.

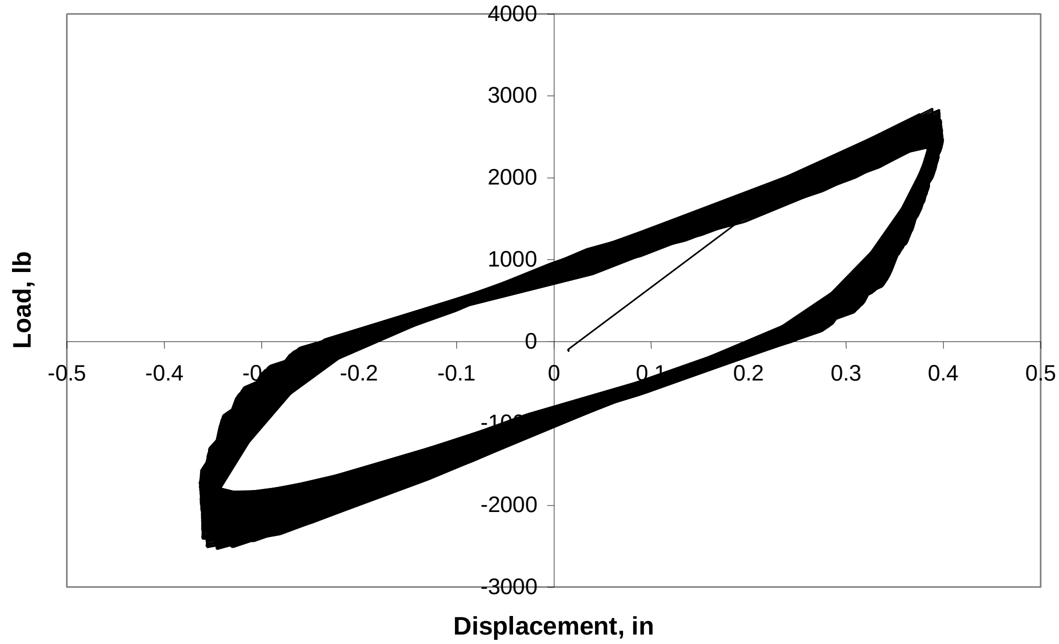

Figure 5.5 Koch/LDI 25°C(77°F) Load-Deflection Hysteresis

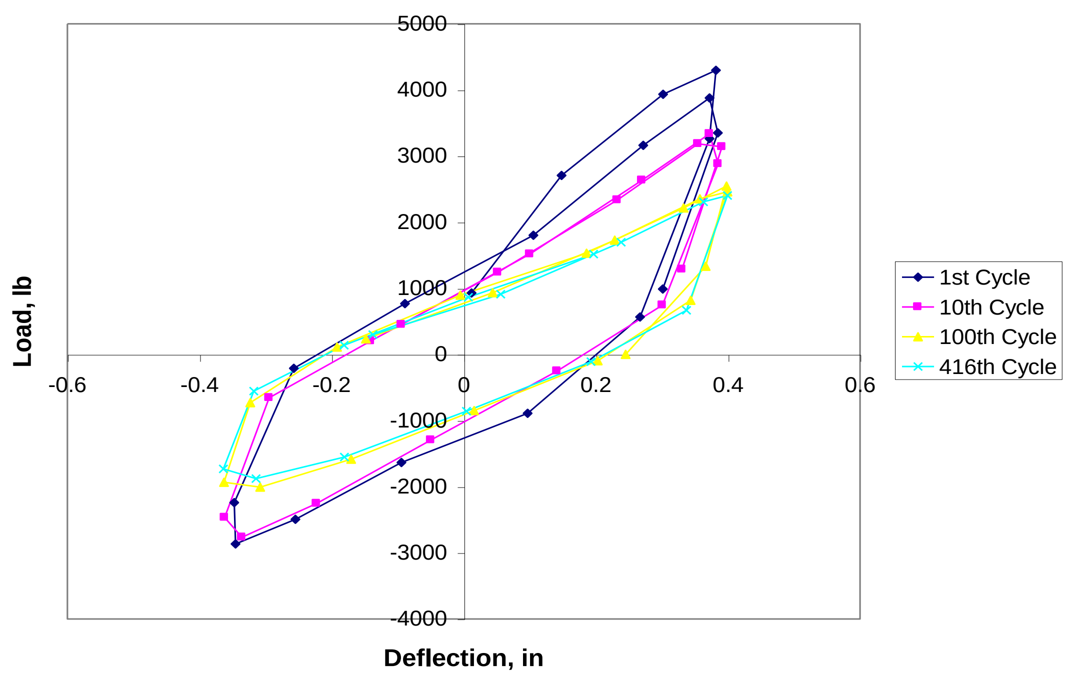

Figure 5.6 Koch/LDI 25°C (77°F) Hysteresis Migration.

A flattening of the curve appears as a lowering of the apparent yield load and an earlier entrance onto the kinematic strain-hardening portion of the loading cycle as seen in Figures 5.6 and 5.7. This kinematic strain hardening is important since the material testing showed virtually no strain hardening. The finite element models all used an elastic-perfectly plastic strain hardening model that would produce a hysteresis curve that is parallel to the translation axis with a total plastic width of two times yield load. This higher apparent kinematic hardening is the probable reason maximum loads for the cycles are approximately 9000 N (2000 lb) while FEA predicted 3000 N (670 lb). This strain hardening is producing larger loads for the translations being imposed on the bridge but are still low enough to be an inconsequential load to the bridge. However, the strain hardening is increasing the cyclic stress range and possibly exasperating the problem of fatigue failure.

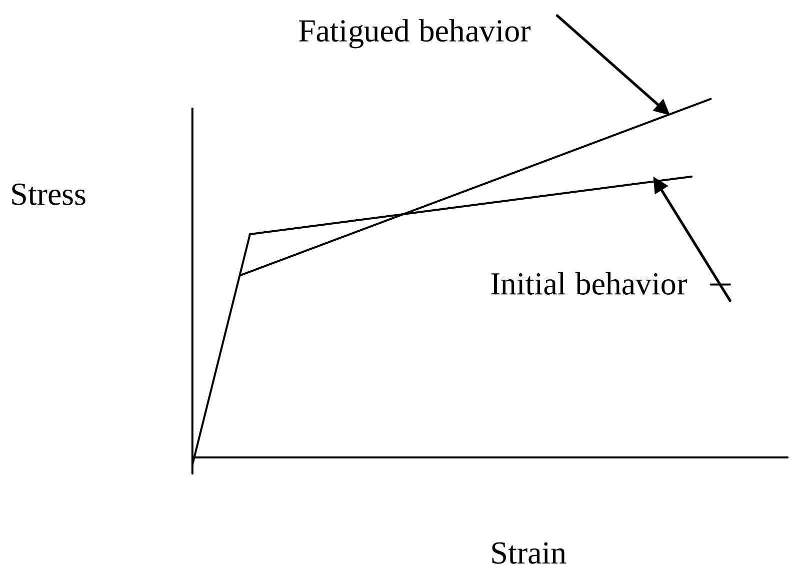

Figure 5.7 Schematic Kinematic Strain Hardening with Lower Yield.

Figure 5.8 shows the 100^{th} cycle for three temperatures, 25°C (77°F), 4.4°C (40°F) and -7°C (19°F), and demonstrates the ramifications of this increased fatigue sensitivity. Between 20 and 50 cycles at 4.4°C (40°F), the normal bond along the fixed blockout started to release. This behavior is illustrated in the load-deflection curve as the discontinuity in the loading cycles. At the end of the 4.4°C (40°F) cycles approximately 25% of the normal bond was debonded and after the -7°C (19°F) cycles approximately

60% of the normal bond area was debonded. The debonding action is shown schematically in Figure 5.9.

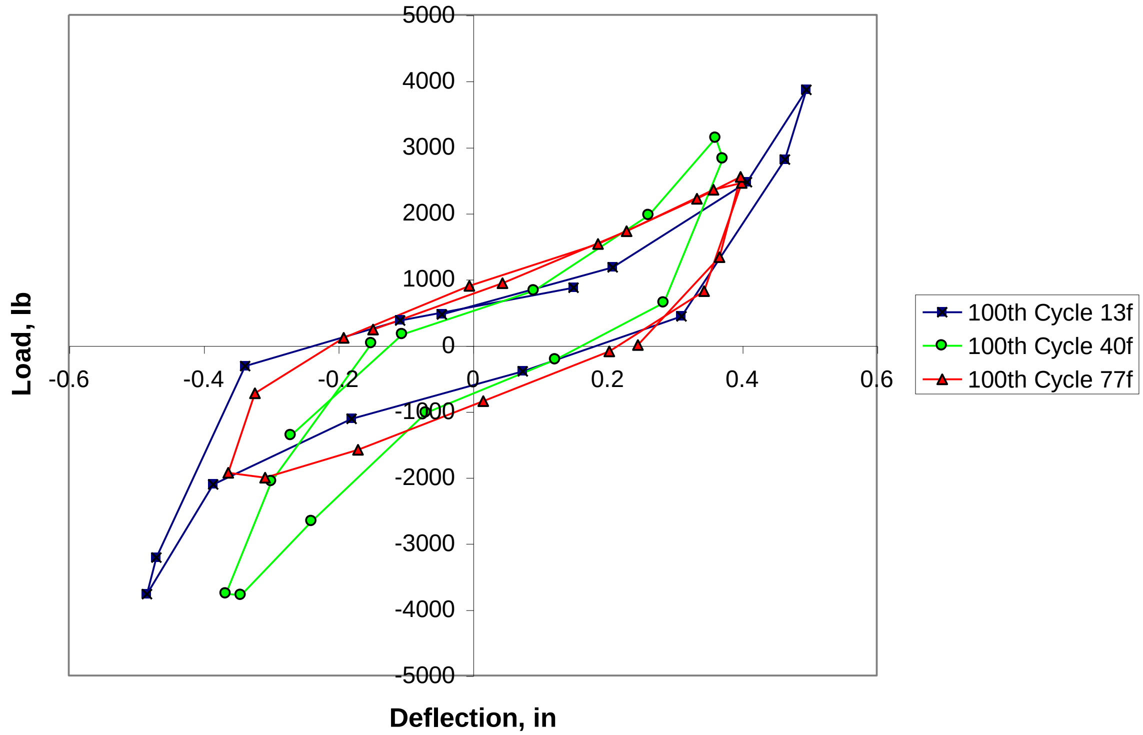

Figure 5.8   Koch/LDI Temperature Comparison

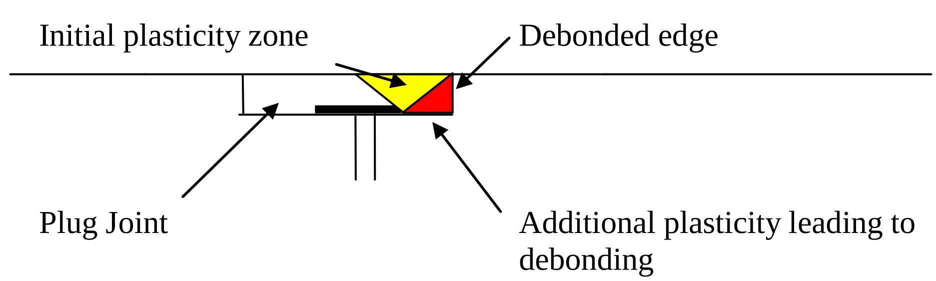

Figure 5.9   Schematic of Debonding Actions

The shear bond at the bottom of the blockout remained intact throughout the entire test and the joint would have remained watertight in service. Further, if the joint was placed such that the direction of traffic was focused at the de-bonding surface the traffic could have help to ‘anneal’ and re-seal the joint. Figure 5.9 illustrates various regions of concern.

Figure 5.10 shows a typical relaxation graph for Watson Bowman Acme at 25°C (77°F). The relaxation for the three materials is summarized in Table 5.3. This table also gives the time for the joint to relax to 5% of an initial load. Koch/LDI and Watson Bowman Acme require far less than a day to relax 95% of their initial loads. By relaxing this rapidly, the material is effectively re-setting to its strain during a very short time.

Table 5.3 Material Specimen Results

	Curve Fit	5% load	Time @ 5% load	r^2
		Lb	Hours	
Watson Bowman Acme	400 ln(t) - 3800	-190	3	0.99
Pavetech	92 ln(t) - 1600	-80	3094	0.99
Koch/LDI	270 ln(t) - 2400	-120	1	0.97

The relaxation time for the Pavetech sample is three orders of magnitude higher than the other two samples. The slow relaxation may be due to excessive heat when placing the sample material. Heat removes volatile materials and effectively stiffens the joint.

Additionally, Pavetech joints are consistently stiffer than the other two materials. The modulus of elasticity tests indicated that the Pavetech modulus is three times Koch/Ldi and almost double Watson Bowman Acme. Thus for our samples, a consistent pattern of higher stiffness of Pavetech samples at lower temperatures was observed. Therefore, the relaxation time, while inordinately long in our tests, is still expected to be greater than the other samples.

The second reason for the long relaxation time for the Pavetech sample is that the stress plateaus at approximately 75% of the initial stress at around five minutes. Therefore, extrapolation to a 95% reduction provides a high apparent relaxation time.

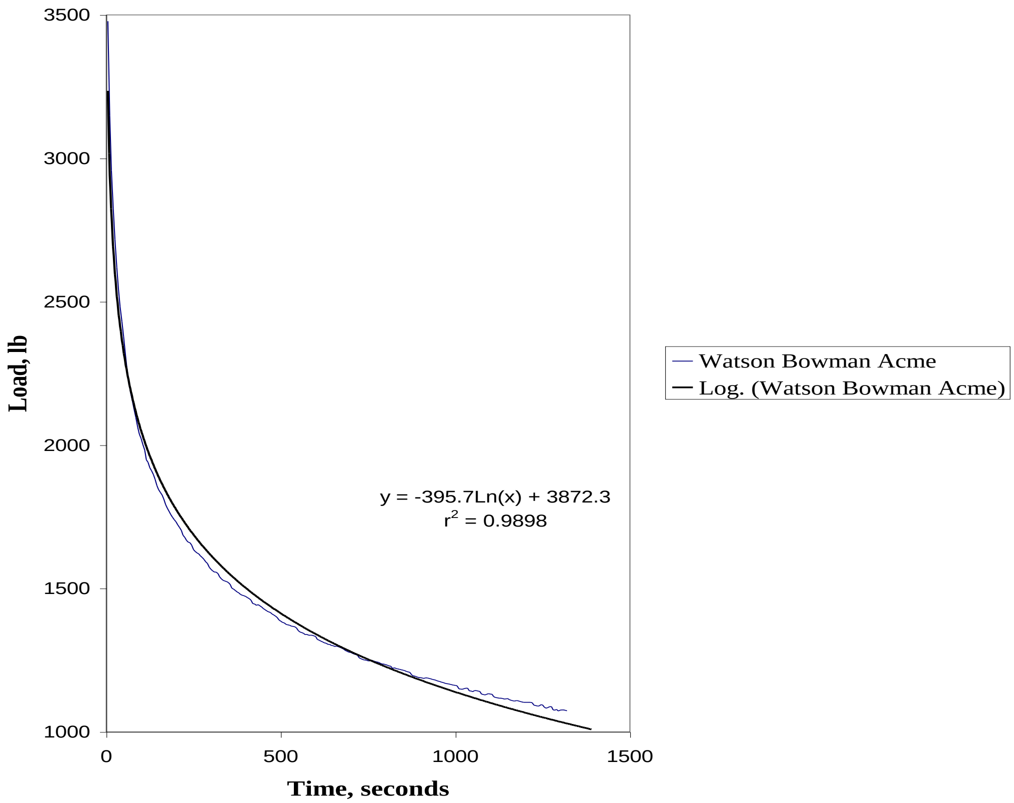

Figure 5.10 Watson Bowman Acme 25°C (77°F) Relaxation.

5.7 JOINT FAILURES

5.7.1 WATSON BOWMAN ACME

Figure 5.11 shows the Watson Bowman Acme joint sample after all of the cycles were conducted. This joint was initially cracked at only 10 cycles by an errant 37-mm (1.5 in) displacement. Following this larger than planned displacement, a crack was visible on the surface. The crack was approximately 228-mm (9 in) long. The joint was cycled for the full 456 cycles at 25°C (77°F) with the crack visibly growing less than 25 mm (1 in).

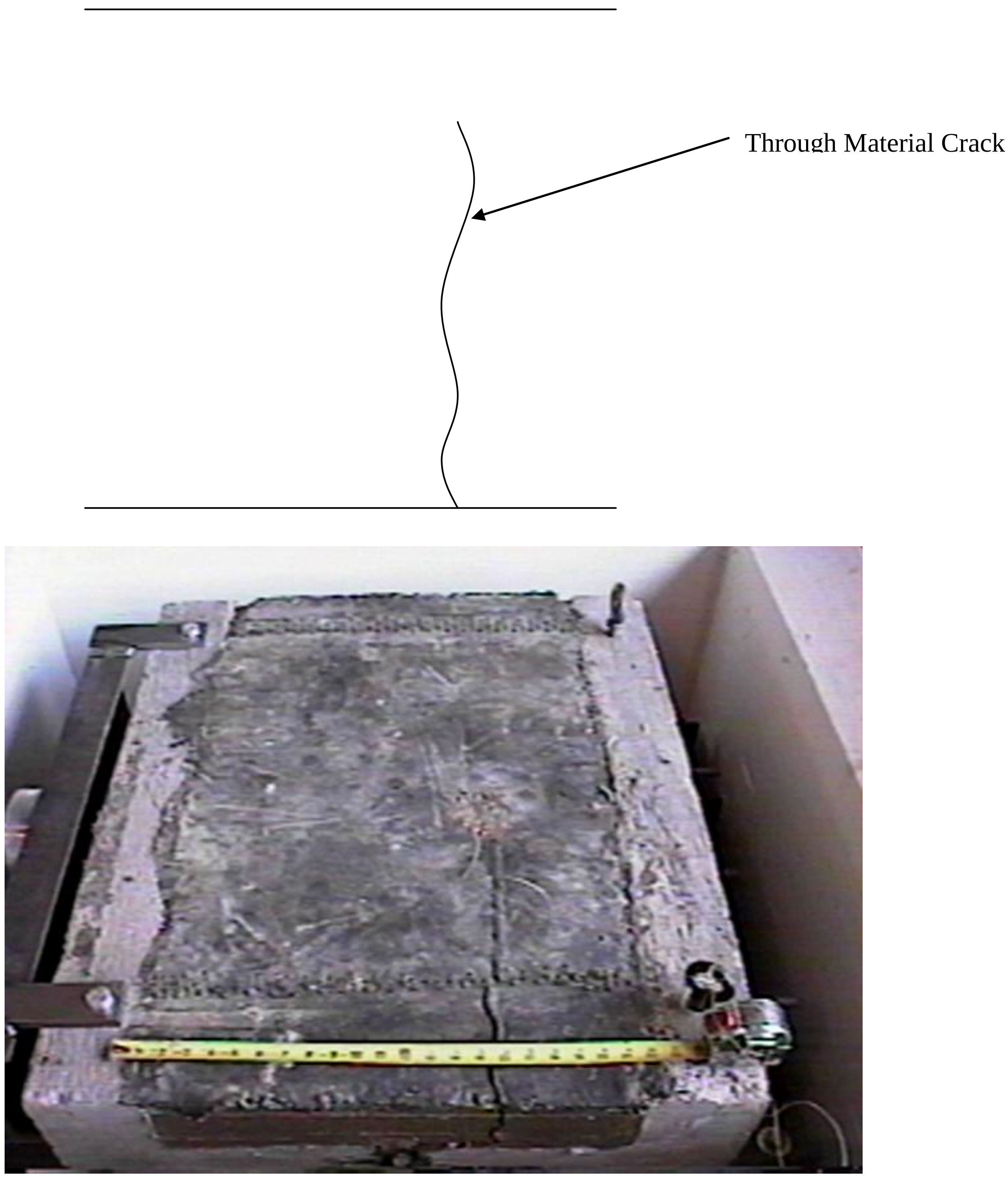

Figure 5.11 Watson Bowman Acme Failed Joint Sample.

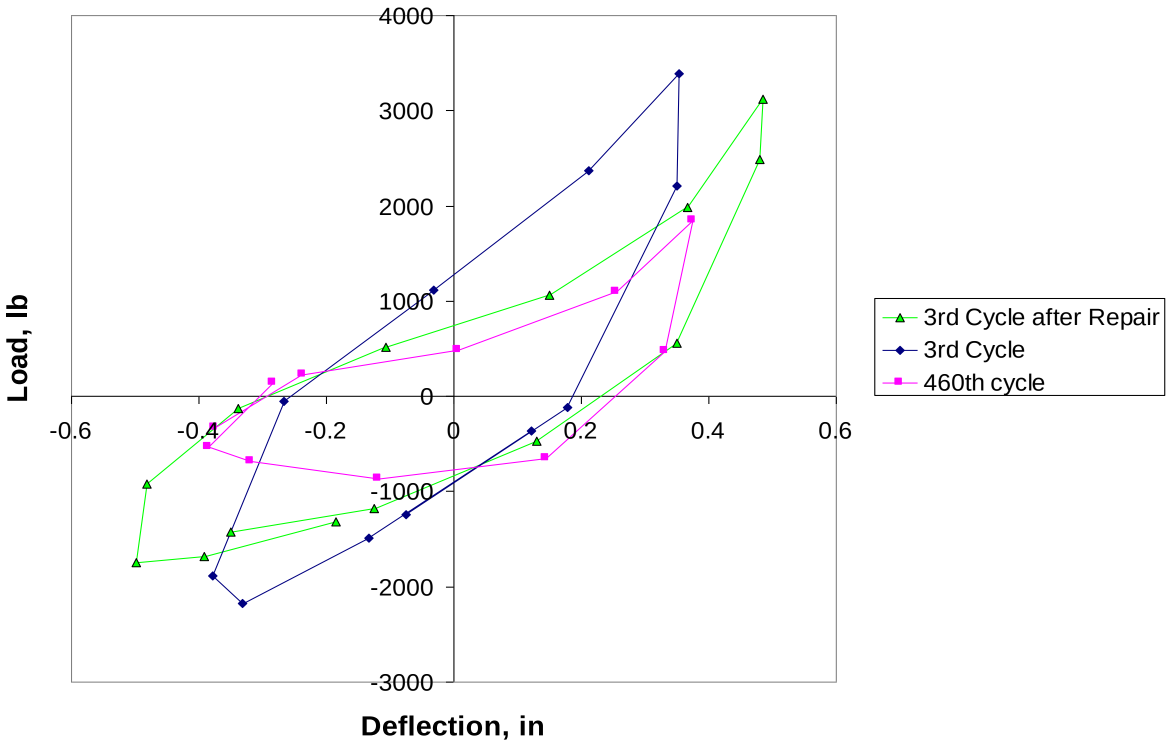

Figure 5.12 Watson Bowman Acme 25 °C (77°F) Autogenous Healing.

After these summer cycles, the joint was removed to evaluate the "autogenous" healing performance. The partially failed joint was placed at an elevated temperature of 32°C (90°F) and a compressive pressure of approximately 275 kPa (40 psi) was applied. Three 5/8 – 11 UNC all-threads torqued to 27.2 N-m (20 ft-lb) induced the compressive pressure. The stressed sample sat undisturbed for seven days. Following the healing period, the joint was placed back in the test frame and displaced. The load-displacement curve follows the cracked load displacement curve (460^{th} Cycle), Figure 5.12. The lack of healing is due to two factors. First, kneading action of the traffic is not present. Second, the material relaxation removed the initial compressive load before healing could occur. In retrospect, the compressive force should have been applied with a spring mechanism to counteract the relaxation characteristics of the plug joint.

The joint was then cooled and cycled through the remaining cycles. The joint remained intact through the entire test with the visible crack growing to a final length of approximately 610 mm (24 in). The crack opened and closed throughout the testing cycles and would have failed the watertight functional requirement. However, the joint would have remained intact and still served to keep debris from entering the expansion joint. The joint would continue to provide a smooth ride and traffic may have annealed the surface of the cracked area.

Upon completion of the cyclic loading the joint was pulled apart in tension. The joint separated by failing through the material directly above the sharp edge of the gap plate. The joint failed in tensile fracture initiated at the free edge of the gap plate re-entrant corner and grew directly along the edge of the gap plate. Material remained adhered to the blockout and the gap plate, respectively. The failure was a material failure with no signs of a bond failure occurring.

5.7.2 PAVETECH

Figure 5.13 shows the Pavetech joint sample after 200 winter cycles. At the completion of the summer seasonal cycles an initial crack approximately 39 mm (1.56 in) long was noted on the surface. This crack initiated due to cyclical loading only, no errant translations were introduced. This fatigue crack continued to grow through the spring/fall cycles to 550 mm (21.63 in) at 912 seasonal cycles. During the winter cycles, the crack grew to the full length of the joint at 310 cycles at which time no load was transferred through the joint and failure was considered to have occurred.

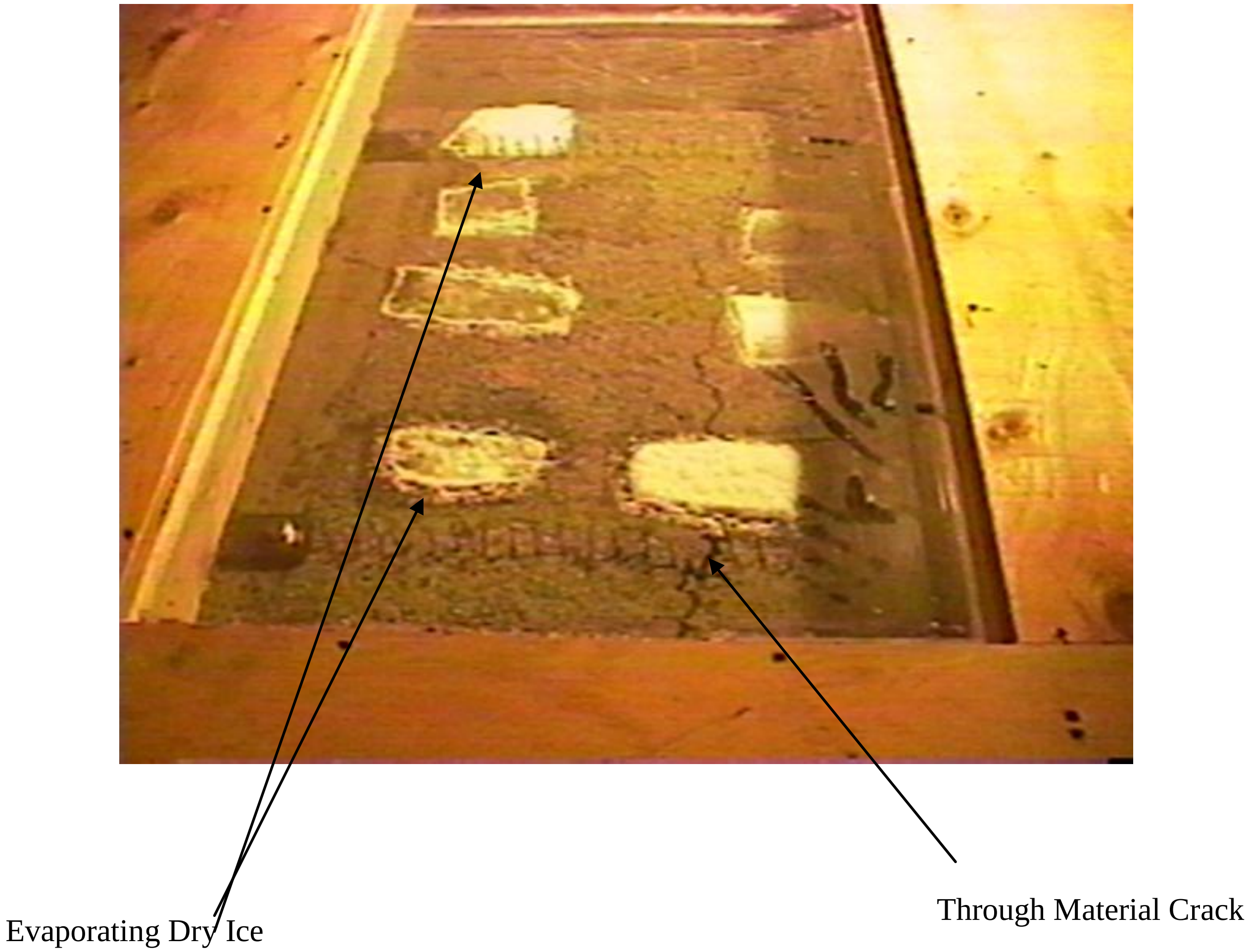

Figure 5.13 Pavetech Failed Joint Sample

After the joint failed, it was separated and inspected. The failure was similar to the Watson Bowman Acme joint. The joint failed in tension through the material directly above the sharp edge of the gap plate. The crack propagated along the high stress gradient at the re-entrant edge. The remaining material adhered to the blockout and the gap plate, respectively. The joint showed no signs of a bond failure. The tensile failure surface showed a considerable amount of “clean” aggregate where poor adhesion of the binder existed or the binder-aggregate bond had failed cleanly. This “clean” aggregate was not seen in the other joints.

5.7.3 Koch/LDI

Figure 5.13 shows the Koch/LDI joint sample after completion of all cycles. This joint met the functional requirements of the test program in the most favorable manner. An adhesion failure at the edge of the joint was first noticed after 200 cycles of the spring/fall cycles. It started at the free edge and propagated across the joint. After the completion of the winter cycles, the separation had grown to a length of 508 mm (20 in). The crack growth is a fatigue-induced phenomena. The Koch/LDI material is the stiffest of the three samples and possesses the lowest normal adhesion strength. So the failure of the normal bond was not surprising.

Observations at the free end of the sample indicated that the shear bond remained intact and carrying load. The material being attached to the bottom of the blockout and the semi-parabolic deformation of the debonded edge demonstrates this adhesion. This joint would have remained watertight due to the shear adhesion.

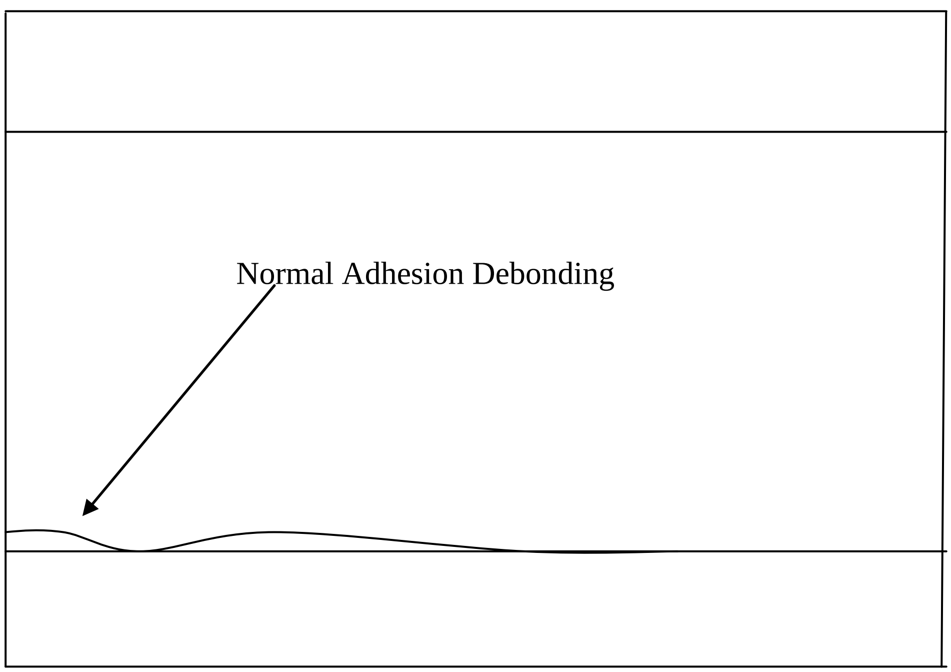

Figure 5.14 Koch/LDI Failed Joint Sample

The joint was separated in tension after completion of the test cycles to inspect the base material. The joint still required a sustained load of 20 kN (4500 lb) and over 37 mm (1.5 in) of displacement to separate. The normal bond surface on the blockout was virtually clean of binder. No visible fatigue striations were apparent on the material or the blockout to indicate where the failure initiated. The most probable location for the failure to have begun is the high stress region at the top corner of the normal bond surface at the free surface.

5.8 COMPARISON WITH ANALYTICALLY PREDICTED CAPACITY

The analytical method proposed earlier is based on elastic/perfectly plastic material properties, experimentally determined bond characteristics, and glass transition temperature (*3*). This method predicted that both the Pavetech and the Watson Bowman Acme samples would fail at the winter cycles and the Koch/LDI sample would survive. The test program confirmed these predictions, but for somewhat different reasons. The analytical prediction did not account for relaxation except as seasonal adjustments and it neglected the fatigue behavior.

The model includes conservative assumptions that reduce the allowable translations such that the relaxation and fatigue are effectively acting within the factors of safety inherent in the material models. The method is yielding a conservative and rational quantifiable allowable joint motion that accurately predicted the failures found in in the test program.

The actual APJ relaxation, which is much faster than the seasonal re-adjustment assumption is rapid enough to induce little load and/or stress in the joint under normal conditions. However, even with a low stress being introduced, the bonding condition combined with the geometry constraints of the joint blockout creates high-localized stresses, which promote crack growth under fatigue conditions.

5.9 CONCLUSIONS

All of the samples exhibited significant fatigue cracking. The materials with higher adhesion and lower tensile strength failed in a tensile mode with a crack forming through the APJ material while stronger APJs showed bond cracking. Both situations indicated fatigue failure in zones of high stress concentrations. The fatigue cracking is a localized phenomenon due to the theoretically infinite point strains at the location of motion, typically at the edge of the gap plate or the top corner of the APJ and the base material. The possibility for crack initiators is high at these discontinuities. The stress riser is a characteristic structural mechanics problem with these types of joints that may be lessened but will not be eliminated. The ramification of this behavior is that APJs will eventually fail by tensile cracking. Autogenous healing, traffic flow, and seasonal joint compression at elevated temperatures extend the joint service life. Nonetheless, the joint will require periodic inspection, maintenance, and/or replacement. In short, these are not maintenance-free joints.

This test program confirms the following conclusions:

- Analytical methods to predict APJ behavior are consistent with the joint behavior.
- The analytically determined design capacity is conservative with respect to a five-year design life.
- Re-setting of the neutral deflection point occurs rapidly, possibly hourly, due to the relaxation.
- Tension failures control the joint design and performance. The stress concentrations at a gap plate corner initiate this fatigue failure through the joint.
- FEA boundary conditions are effectively modeled and predict the elasto-plastic behavior of the joint.
- Experimentally determined material properties did not show the level of kinematic strain hardening that was observed in the joint samples.
- Autogenous healing was not duplicated in the lab due to joint relaxation.

5.10 RECOMMENDATIONS

The following recommendations are developed from this test program.

- An opportunity exists to optimize APJs to reduce their cost without affecting the APJ functionality and may improve in-service performance characteristics.
- Joint geometric optimization exercise should be conducted to identify and remove non-contributing material from the joint. Establishing a preferred movement on the gap plate would accomplish this.
- The analytical design method should be refined to incorporate the data from this test program to improve the prediction of APJ behavior respect to a five-year design life.

5.11 ACKNOWLEDGEMENTS

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6.0 ASPHALT PLUG JOINTS: REFINED MATERIAL TESTS AND DESIGN GUIDELINES

6.1 ABSTRACT

Highway builders and rehabilitators throughout the United States use asphalt plug joints (APJs) in bridge expansion joints following manufacturers recommendations. The joint performance varies widely as indicated in recent surveys. State departments of transportation are installing joints in unsuitable sites and/or are overlooking other sites where APJs might work well. To aid bridge engineers, the University of Wyoming (UW) has developed rational design guidelines for APJs. This research suggests suitable applications, materials characterization, design guidelines, and validation. This paper concludes the research program by providing updated material tests, design guidelines, and improved joint geometry.

Two critical material properties are required to qualify APJ material: relaxation and glass transition temperature. Both properties may be obtained using a slight modification of the standard TSRST asphalt test. This modified standard test was conducted and compared with the near-full-scale test results. Design guidelines are based on field observations, material tests, near full-scale testing, analytical evaluations, and a survey of DOT experience. Joint design changes are proposed to help mitigate the present shortcomings.

6.2 INTRODUCTION

The primary objective of this research is to better understand the performance of Asphalt Plug Joints (APJs) and propose tests, designs, and construction methods to yield satisfactory performance. APJs are bridge expansion joints that use a modified binder-aggregate mix to span between the approach slab and the bridge deck. This mix is placed in a blockout in the roadway and is bonded to the substrate on three sides as shown in Figure 6.1. The bridge motions create displacements where the gap plate slides on the bottom of the blockout. These displacements concentrate deformations in a localized region creating high strain concentrations. Theoretically, these strain (and apparent stress) concentrations should frequently fail these joints. Although failures are common, many joints perform well where general linear elastic theory predicts that they should not, i.e., simple theories do not adequately describe field observations. In summary, these joints are performing well in certain applications, and unsatisfactorily in others, and the impetus of this work is to understand why.

The research initially employed classical engineering material characterizations of elastic and/or elastic perfectly plastic while assuming the visco-elastic and/or visco-plastic characteristics were secondary (*4*). Temperature dependence was considered a primary indicator for these characteristics and was determined to be an important characteristic. The time-dependence material behavior was <u>initially</u> considered a secondary contributor to the joint performance. The near full-scale joint validation (*5*) clearly illustrated the overwhelming importance of time-dependent characteristics. The test program identified a mismatch between the material and component testing, particularly in the effective modulus of elasticity and in the relaxation of stresses. Based on these findings, a second series of material tests was conducted to evaluate the material relaxation and demonstrate that a modification of the standard AASHTO Thermal Specimen Restrained Specimen Test (TSRST) (*7*) can be used to determine the primary material characteristics and qualify the material for application in APJs.

A simple one-dimensional model using the modified test data demonstrates the time-dependent effects in an APJ. The model aids in the understanding and in the presentation of relaxation as it affects joint performance. Test results are compared to the simple model, and finally, refined design criteria are presented.

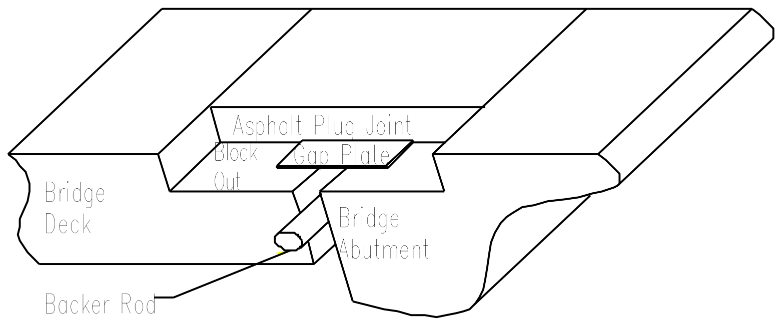

Figure 6.1 Typical Asphalt Plug Joint Cross Section

6.3 IMPORTANCE OF RELAXATION

Bridge joints are exposed to deformations as bridges expand and contract. The load/stress in the joint are displacement-induced effects that occur relatively slowly as a function of ambient conditions as well as the heat-transfer characteristics of the structure. If the material relaxes as rapidly than the temperature change demands, no stress is induced. Hence, relaxation is very important.

The fundamental APJ behavior is the joint stress decreasing/relaxing at a rate nearly equal to the rate that temperature-induced motion that imposes/creates the stress. Hence, no stresses build up in the joint and the material flow accommodates the deformation. As the temperature drops, the ability of the material to flow gradually decreases until an abrupt ductility transition occurs. This transition is referred to as the glass transition temperature, T_g. At this temperature, the material viscosity decreases to virtually zero. At the same time the material becomes brittle; a small joint movement or additional temperature drop will create a fracture that will likely propagate through the joint. Such cracks create serviceability problems and constitute failure.

6.4 MODIFIED MATERIAL TEST

The AASHTO TSRST employs a prismatic sample of asphaltic material that is bonded to two platens. The specimen length is held constant while the sample temperature is lowered. By holding the sample length constant, the temperature strain is equal and opposite the strain due to the tensile restraining stress. The induced stress creates a material failure when the stress exceeds the material capacity at the specimen temperature. The APJ is highly plastic at room temperature but become very brittle at low temperature. When the APJ material transcends below the glass transition temperature and becomes brittle, the sample fails in tension. The temperature at failure is the "TSRST temperature" and may be considered as a lower bound temperature for the application of APJs. This lower bound is a non-conservative estimate because the sample will fail at a temperature above T_g with a very small displacement. In the TSRST, the imposed mechanical strain (bridge movement) is considered to be zero. That is, the entire load affect is due to temperature. Hence, tensile failures are ensured at or below this temperature with or without any bridge motion. Joint failures due to bridge motion can occur at temperatures above the "TSRST temperature", which may be taken as T_g for joint evaluation and design.

The TSRST procedure may be slightly modified to evaluate relaxation. Relaxation is a reduction in load/stress while a constant strain is maintained over time. Relaxation is a time-dependent material characteristic that can be quantified and utilized for this application. The relaxation is determined by inducing a small displacement, holding it constant, and measuring the load decrease with time. The standard TSRST test was modified to induce an initial displacement and hold the temperature constant. One shortcoming of the TSRST is that it can not record this imposed initial displacement. This issue was addressed by using the previously determined modulus of elasticity (*3*) and $\Delta l = PL/AE$. Evaluations were all conducted at a constant temperature of 2°C (34° F), a reasonable mid-range temperature for bridges in cool or cold climates. This research did not examine the sensitivity of relaxation as the temperature approaches T_g.

Samples were prepared for evaluation from the near-full-scale joint (*5*) following completion of the cyclic loadings. High strains and possible degradation of the bond between the aggregate and binder are induced and concentrated on the motion side of the joint. The finite element analysis models conducted in previous research (*4*) illustrated these high strain gradients. Prisms were cut from the "non-moving" side of the joint. Samples were sectioned transversely to the direction of loading in an attempt to obtain specimens that were not exposed to significant strains, and therefore, possible degradation of the binder-aggregate adhesion. This preconditioning was a characteristic of the research due to limited material samples and is not generally recommended for a standard process. Samples of virgin material are preferred for conducting the UW-modified TSRST evaluation.

The samples were placed in the TSRST equipment and cooled to a temperature of 2°C (34° F) where the specimens were loaded to approximately 356 N (80 lbs) and their length was held constant. The specimen load was then recorded with respect to time while the temperature was held constant. The recording was stopped when the load appeared to level to a relatively constant value. Typical plots are given in Figure 6.2. All samples stabilized within 15 minutes of the initial displacement being imposed, which illustrates the rapid rate of relaxation. This is critical to performance and understanding of joint behavior. In summary, the TSRST and this UW-modified TSRST are simple to conduct, yield useful information, and may work well for qualifying APJ materials. Further work is required to determine relaxation characteristics as the temperature approaches T_g.

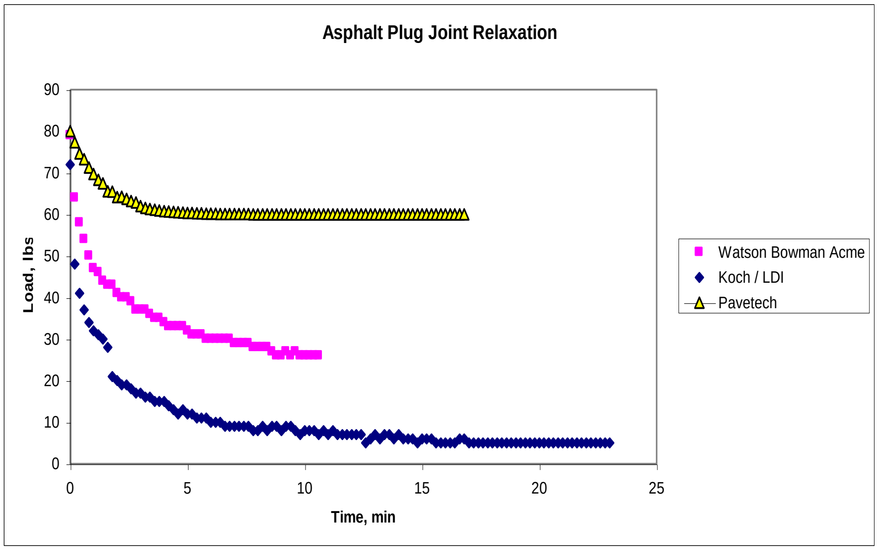

Figure 6.2 Plug Joint Relaxation Data.

6.5 MODELING RELAXATION BEHAVIOR

A design trade-off exists between proportioning the mix, blending the binder to be viscous and accommodating deformation, and overly viscous creating unwanted flow, aggregate separation and rutting. This trade-off is discussed later after a simple illustrative model is described which parallels test data.

In order to understand the significance of relaxation, a simple and intuitive one-dimensional rheological model is developed. Using this model, the loads/stresses in an APJ are compared for a visco-plastic material, a linear elastic material, and a linear elastic-perfectly plastic material. A typical bridge in Cheyenne, Wyoming is used for illustration.

One of the simplest relaxation models is a single Maxwell body. A Maxwell body is represented as a spring and damper positioned in series. This model consists of an elastic element (spring) that can store the energy and a plastic element (damper) that dissipates the energy with time. The characteristic equation for this model is:

$$P(t) = U_o K * e^{-t/\tau}$$

Where:

U_o is the initial imposed displacement (constant),

K is the spring constant from the Maxwell body,

τ is a normalized variable defined as r/K, where r is the damping constant, and

t = time after application of the load.

The Maxwell body gives an initial instantaneous load of $P(t=0) = U_o K$ and the load decays over time as the damper deforms and the spring is unloaded. A single Maxwell body is a simple approximation of relaxation but it often does not represent the true material because the load is relaxed too rapidly. Our relaxation tests showed that the load relaxed rapidly to an intermediate level, at which point the decay rate was significantly lower. To better represent this type of relaxation, or load decay over time, a slightly more refined model consisting of two Maxwell Bodies in parallel was used. Using two bodies in parallel allows one of the bodies to represent the relatively fast initial relaxation while the second body represents the slower rate associated with the secondary relaxation. The schematic of the model used is shown in Figure 6.3. This model is also analogous to the Burger rheological model that is often used to describe creep, a phenomena where a material gains additional displacement under a constantly applied load (*6*).

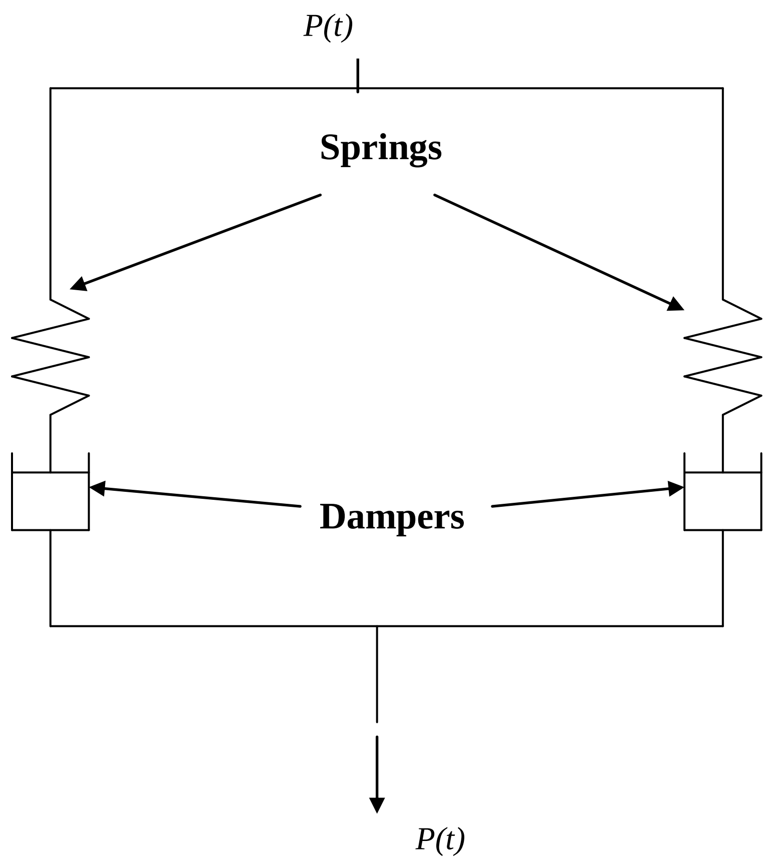

Figure 6.3 Two Element Relaxation Rheological Model

The characteristic equation for the four-element model shown in Figure 6.3 is:

$$P(t) = U_o\left(K_1 * e^{-t/\tau_1} + K_2 * e^{-t/\tau_2}\right)$$

Where the forces in the two elements add to balance the reactions.

The spring constants K_1 and K_2 and the normalized variables τ_1 and τ_2 were developed from the relaxation data recorded from the UW-Modified TSRST test plus the modulus of elasticity determined in previous research (*3*) and the long-term decay time from the near-full-scale joint validation program (*5*). Utilizing these and the following initial conditions and assumptions, the models characteristic equations were solved:

Initial Condition:

Time $t = 0$

$P_o = U_o (K_1 + K_2)$ (1)

$U_o = PL/AE$ Back Calculated

Intermediate Condition 1:

99% decay of the short-term relaxation completed:

Because the time t_1 is much less than t_2 the difference in load is approximately due only to the short-term relaxation, therefore:

Time $t = t_1$

$$e^{-t_1/\tau_2} = 0.01$$

$\Delta P = (P_o - P_1) \cong U_o * K_2$ (2)

Intermediate condition 2:

95 % of the long-term decay is completed

Time $t = t_2$ is much greater than t_1

$$e^{-t_2/\tau_1} = 0.05$$

$$e^{-t_2/\tau_2} \approx 0.0$$

$$P(t_2) = U_o(K_1 * 0.05) \qquad (3)$$

Solving these equations using the curves determined from the UW-Modified TSRST test, the equation parameters shown in Table 6.1 were obtained. When these models are superimposed with the initial data as shown in Figure 6.4, the degree of correlation is acceptable.

Table 6.1 Relaxation Equations

	P_o, lb	E_o, psi	U_o, in	P_1, lb	t_1, min	t_2, min	K_1, lb/in	K_2, lb/in	τ_1	τ_2
Watson Bowman Acme	79	2000	0.09	32	6	180	540	370	1.3	86
Pavetech	80	1300	0.13	61	5	190,000	140	460	1.1	68,000
Koch/LDI	72	1200	0.14	8	7	60	460	57	1.5	75

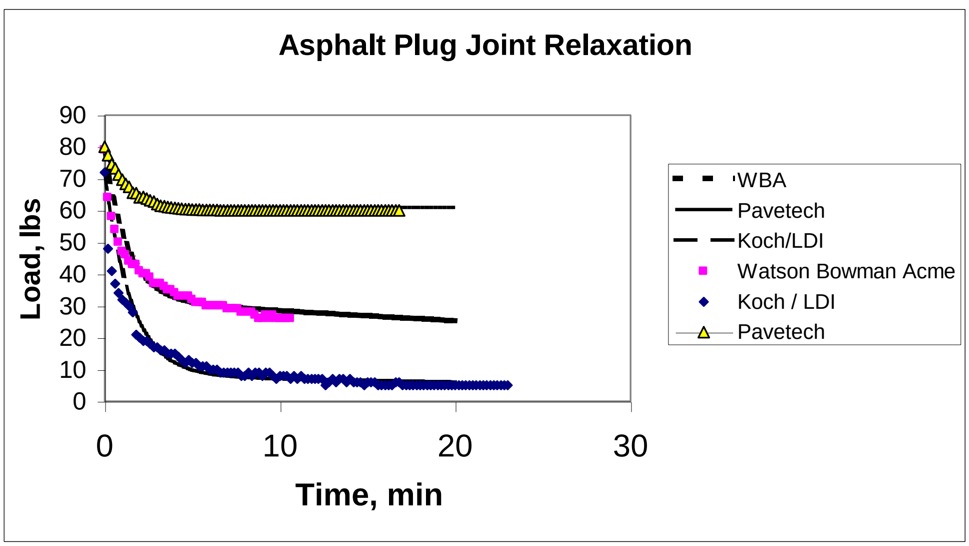

Figure 6.4 Plug Joint Relaxation Model vs Experimental Data at 2°C (34° F)

6.6 APPLICATION TO BRIDGE MOVEMENT

Bridge motion due to temperature is either continuous for bridges on elastomeric pads or discontinuous for bridges on slider bearings. In the later case a stick/slip phenomenon may represent the bridge motion better than a continuous function.

Conceptually a relaxation time less than 15 minutes implies that the joint will relax faster than the thermal inertia allows the bridge to deform. Therefore, the more critical stick/slip condition is examined. The analogous behavior is “silly putty” being pulled slowly, it will deform indefinitely and no critical stress develop. Pulled quickly, it will fracture.

Many in-service bridges do not generally move smoothly, but rather expand or contract and store energy before motion occurs as friction in the bearings is overcome. The stored energy is released at discrete times creating movement, generally referred to as the stick/slip phenomenon. For comparative analysis in this study, these discrete movements are assumed to occur in equal displacements of 1.27 mm (0.05 in) increments. If the APJ relaxes between movements then the stress in this relaxed state is independent of the next 1.27 mm (0.05 in) movement. If full relaxation has not occurred, then there is some cumulating effect. Figure 6.5 shows the load history for six of these discrete movements.

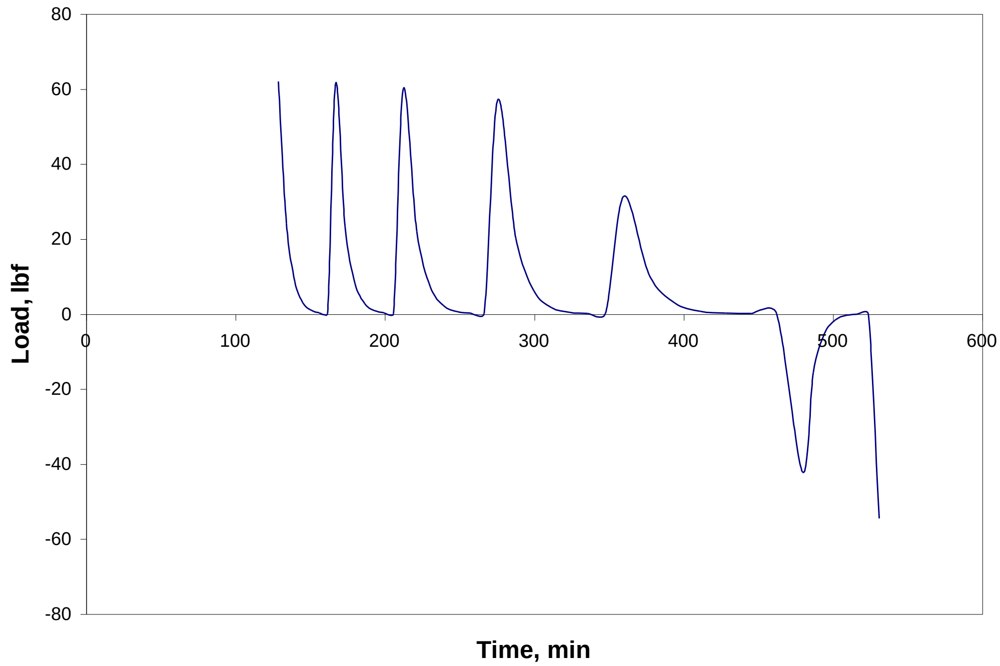

Figure 6.5 Zoom Plot of Relaxation Model

For design comparative purposes let's define a two time constants t_{75} and t_{B75}. t_{B75} is the amount of time between discrete bridge motions. It is approximated by linearizing the maximum daily motion over a 12-hour period, then dividing it by the assumed stick/slip incremental movement per motion. t_{75} is the time required for the material to relax 75% of its initial load. This constant has been calculated from the UW-Modified TSRST test and is given in Table 6.2. These time constants allow direct comparison of the bridge requirement to the material capacity.

The high t_{75} for Pavetech reflects the characteristic of the material to stabilize at a stress level approximately 75% of the initial stress. Therefore, unlike the other two materials, Pavetech joints will oscillate between 75% and 100% of the induced stress. Kinematic strain hardening may lead to an accumulation of stresses in this joint material.

For this comparison approximations on the discreet increments due to stick/slip need to be made. Increments should be larger for steel roller bearings and smaller for elostomeric bearings. For this example a 1.2 mm (0.05 in) increment has been used. The actual selection of the increment is left to the discretion of the designer, however a methodology is indicated in the example.

Table 6.2   Determined Material Time Constant, t_{75}

Material Supplier	t_{75}
Koch/LDI	2.8 min
Watson Bowman Acme	41.5 min
Pavetech	75,980 min

The plasticity model assumes an elastic-perfectly-plastic model that allows infinite stain in the plasticity range (*3*). Like the relaxation model, this model also requires that the previous load history of the material be known. Figure 6.6 shows the assumed load diagram for a one-day movement of the bridge located in Cheyenne, Wyoming during the fall or spring season. The overall daily bridge motion is assumed to be 19 mm (0.75 in) and the motion is assumed to follow a sinusoidal waveform for daily variations. The following assumptions are used for this comparison:

- Stick/Slip produces uniform 1.27 mm (0.05 in) displacement steps.
- Bridge displacement follows a sinusoidal form.
- Material has no residual load at the start of the cycle.
- The APJ is represented as prismatic joint with no stress concentrations.
- The previously calculated elastic-perfectly-plastic model, an elastic model, or the relaxation model represent material behavior.

As can be seen in Figure 6.6, the sine wave represents the elastic behavior. The relaxation model produces lower loads, and therefore stresses, than the plasticity or the elastic models. These models demonstrate the lower loads that occur when relaxation occurs more rapidly than the bridge motions. The joint is not able to build up the large loads/stresses required for failure. Therefore, APJs should perform well structurally down to the glass transition temperature, T_g. At temperatures below $T_{g,}$ the material becomes brittle and tensile failures are assured -- an obvious condition to avoid. The exception to this is thermal shock. In unusual cases where the temperature may abruptly drop several degrees in a very short time, the material would not have time to relax the stress. The temperature drop would then impose large stresses can could create cracking.

For bridge joints and bearings, the set temperature is often used as the temperature and length to which all temperature-induced load effect is referenced. Mathematically, this represents the “zero” of initial temperature deflection. However, considering relaxation this setting reference is no longer fixed but it continually changes as the bridge moves. This change of the set temperature and configuration decreases the apparent imposed strain, and therefore, the associated stress as well. The traditional setting temperature has little relevance for the APJ system. It is the rate of temperature decrease rather than the total joint movement that affects stress.

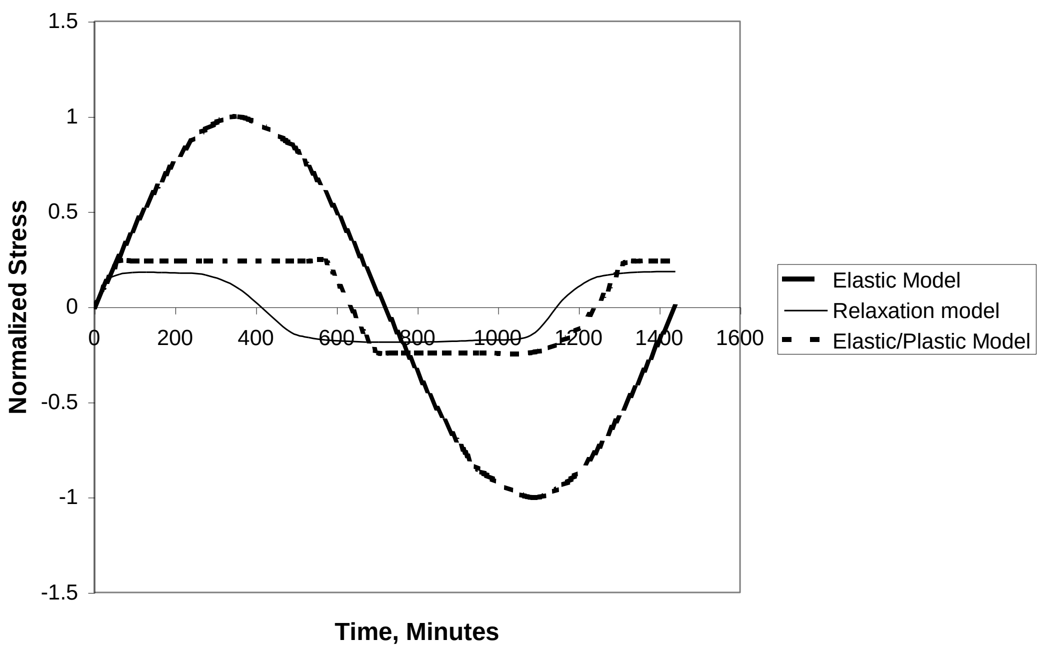

Figure 6.6 Elastic, Elastic/Plastic, and Relaxation Relative Stress Comparison

6.7 JOINT GEOMETRY CONSIDERATIONS

The finite element analysis (*4*) showed that the strain was concentrated within a triangular cross section radiating out from the edge of the gap plate with slip planes at approximately 60° from the blockout bottom. The blockout is also constructed symmetrically simply because we can not predict on which side of the gap plate the motion occurs. By constraining the gap plate to be fixed to one side of the blockout, the motion is forced to occur on one side. Based on these two criteria, the blockout can be constructed as shown in Figure 6.7 with a corresponding reduction in the required joint material.

Some debonding will occur at the gap plate. The minimum dimension for the plug joint is a wedge extending upward at 60° from the edge of the gap plate and the debond. A 25 mm (1 in) debond zone is shown for illustrative purposes. The fixed side of the gap plate must be secured to force the joint motion into the preferred side. This may be accomplished with APJ binder or with physical fasteners.

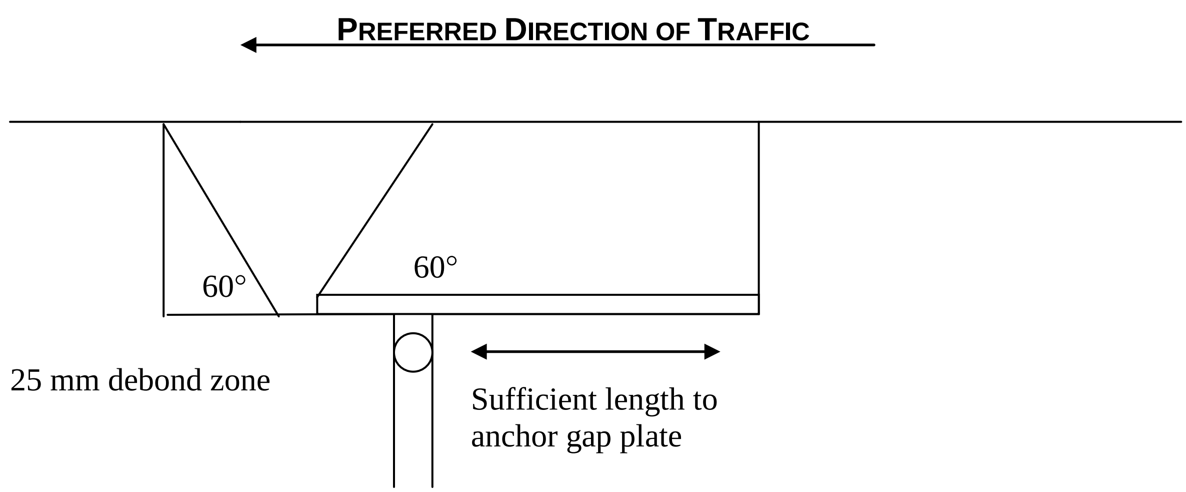

Figure 6.7 Optimized Joint Geometry

It is the viscosity of the material in these joints that is paramount for successful application. If the structures are moving at or about the same rate as the relaxation then the joint loads are very low and the joint structurally has no theoretical limit of motion. Unfortunately, and obviously, a functional limit exists regarding the smooth transition from the pavement to the bridge deck (*2*). This smooth transition onto the bridge requires that a volumetric limit on the motion be imposed. By limiting the depression (bump) to 19 mm (3/4 in) or less, a volumetric relationship based on the failure zone defined using the finite element analysis is established. This geometric relationship is simply that the material inside the defined failure envelope does not depress by more than the specified limit under maximum joint displacement. Table 6.2 gives these motion limits based on a 19mm (3/4 in) limit and various plug joint thicknesses.

Table 6.3 Volumetric Based Motion Limits

Joint Thickness	Maximum Motion
50 mm (2 in)	57 mm (2.3 in)
63 mm (2.5 in)	53 mm (2.1 in)
75 mm (3 in)	50 mm (2.0 in)
88 mm (3.5 in)	50 mm (2.0 in)
100 mm (4 in)	49 mm (1.9 in)

There is 8 mm (.35 in) joint motion difference between the 50 mm (2 in) and the 100 mm (4 in) joint. Since the joint capacity decreases with additional thickness the designer should consider using a 50 mm (2 in) thick joint first.

6.8 OTHER SERVICE ISSUES

One major disadvantage of viscosity/relaxation is that the joint can never be free from rutting. The traffic induces a load that forces the material to flow out of the wheel path. These ruts can be reduced by limiting joint application to locations where the traffic is moving relatively fast (as on highways). These joints should not be installed in locations where the likelihood exists of slow moving or stationary traffic (such as at intersections). The reason for this is two fold. The slow, or stationary, traffic is placing the loading on the joint for a longer time interval thereby increasing the flow of the joint material. Additionally the possibility of steering and wheel scrub accelerates the rutting and track out conditions.

The issue of skew angle is also a matter of pushing the material out of the joint due to traffic loading. APJs are isotropic so it doesn’t matter if the loading from the bridge is perpendicular to the joint cross section or at an oblique angle, the joint should function the same. The joint could have a larger effective cross-section as the skew angle increases. The problem lies in the impact due to traffic. Rutting will increase due to skew and a reasonable limit to place on the skew is 30° because this would effectively limit the load along the joint to one-half that across the joint. This recommendation is consistent with the limits imposed by Oregon and Connecticut (*2*).

This research focused on the structural aspects, specifically the serviceability issues that are directly attributable to structural failures. Researchers intentionally did not address the material chemistry and the long-term issues such as fatigue and age embrittlement due to the loss of volatile material used to plasticize the joint materials. The information from this research can be used to focus the developments of better binders that can produce better joints. The glass transition temperature and the relaxation should be qualified. The UW-Modified TSRST illustrates a possible approach. Tests for long-term aging might be pursued since the binder is asphalt based and it will lose some of its ductility over time. The loss of ductility accelerates fatigue fractures that will likely produce tensile failures over time.

The combinations of the long-term fatigue susceptibility and the serviceability issues of rutting imply that these joints should not be viewed as service-free joints. The maintenance required is relatively easy when a joint exhibits either cracking or rutting, but it will be required. Frequent repairs by adding binder seem to be a reasonable maintenance approach but may damage the base material due to the required heat driving out the volatile materials providing the basic plasticity.

6.9 DESIGN GUIDELINES EXAMPLE

Figure 6.8 illustrates a design sheet for determining applicability and size of an APJ. The basic logic is to answer the gross design questions of whether or not the site is suitable for APJs. Based on the anticipated temperature range for the structure, a reasonable maximum bridge daily motion is estimated. From this maximum daily bridge motion determine the bridge time requirement (t_{B75}). This is illustrated in the example. Obtain the required material characteristics, either from this paper, Figure 6.5, testing, or the manufacturer. Check this material time capacity against the bridge time requirement to ensure that the relaxation is adequate for the location. Check the lowest anticipated temperature against the glass transition temperature, T_g. If all checks are satisfied then determine the maximum joint thickness.

6.9.1 EXAMPLE 1, CHEYENNE WYOMING

This procedure is illustrated using an hypothetical bridge located in Cheyenne, WY.

Given: A Bridge in Cheyenne Wyoming with a temperature motion of 0.55 mm/°C (.04 in/°F).

Solution: Pavement temperature range from Superpave SP1 (*8*):

High 58°C (142°F) 99.9 % reliability .

Low -28°C (-21 °F) 94.8 % reliability .

Seasonal Average Temperature: –7°C (19°F).

Maximum Seasonal Movement: = 0.55 mm/°C × (-7°C – [-28°C]) = 11.55 mm (0.5 in) movement.

Maximum Daily Motion is 2 times maximum seasonal motion = 2*11.5 mm =23.0 mm (0.9 in)

To determine the bridge time requirement, assume that the maximum daily motion occurs over a 12-hour period and that movements are discrete 1.27 mm (0.05 in) movements. The bridge time requirement t_{B75} is:

$$t_{B75} = \left(\frac{12hr}{23mm / 1.27mm/increment}\right)\left(60\min / 1hr\right) = 40 \text{ minutes/increment}$$

This information is summarized in Figure 6.8.

Table 6.4 Material Constant Summary

Supplier	T_g, [1]	t_{75} [2]
Koch/LDI	-43°C, (-45°F)	2.8 min
Watson Bowman Acme	-27°C, (-16.6°F)	41.5 min
Pavetech	-26°C, (-14.8°F)	75,980 min

1 From (*5*)

2 From Table 6.2

Design Guidelines:

Structure Number: *BRA-01* Designer: *B.K. Bramel* Date: *3/24/99*

Are there:

High likelihood for thermal shocks? Yes_________ No *X*

Slow moving or stationary traffic? Yes_________ No *X*

Skew angle in excess of 30°? Yes_________ No *X*

If any of the above answers are ***Yes, Stop Here***. APJs are not applicable for this location.

Max anticipated bridge daily motion: 23.0 mm (0.9 in)

Bridge t_{B75} time requirement: 40 Minutes/Increment

Anticipated temperature range for the structure: (From Superpave SP1 at bridge)

High 58 °C (142 °F) Probability: 99.9 % to be lower

Low -28 °C (-21 °F) Probability: 94.5 % to be higher

Anticipated Bridge Total Motion: 50 mm (2 in)

Required Joint Thickness: 75 mm (3 in)

Selection Checklist						
Material Supplier	**T_g**	**t_{75}, min**	**$T_g<T_{LOW}$ Y / N**	**$t_{75}<t_{B75}$ Y / N**	**Acceptable Y / N**	**Thickness**
Watson Bowman Acme	**-27°C, (-16°F)**	**42**	**No**	**No**	**No**	
Pavetech	**-26°C, (-15°F)**	**76,000**	**No**	**No**	**No**	
Koch/LDI	**-43°C, (-45°F)**	**2.8**	**Yes**	**Yes**	**Yes**	**50 mm(2 in)**

Joint Thickness	Maximum Motion
50 mm (2 in)	57 mm (2.3 in)
63 mm (2.5 in)	53 mm (2.1 in)
75 mm (3 in)	50 mm (2.0 in)
88 mm (3.5 in)	50 mm (2.0 in)
100 mm (4 in)	49 mm (1.9 in)

Figure 6.8 Example 1 Design Guideline Worksheet.

6.9.2 Example 2, Denver Colorado

This example uses a similar bridge located in Denver, CO.

Given: A Bridge in Denver, Colorado with a temperature motion of 0.40 mm/°C (.03in/°F).

Solution: Pavement temperature range:

High 58°C (142°F) 99.9 % reliability (*8*).

Low -22°C (-6° F) 78 % reliability (*8*).

Seasonal Average Temperature: 0°C (32°F).

Maximum Seasonal Movement: = 0.40 mm/°C × (0°C - [-22°C]) = 8.8 mm (0.35 in) movement.

Maximum Daily Motion is 2 times maximum seasonal motion = 2*8.8 mm =18.0 mm (0.69 in).

To determine the bridge time requirement, assume that the maximum daily motion occurs over a 12-hour period and that movements are discrete 1.27 mm (0.05 in) movements. Then the bridge time requirement is:

$$t_{B75} = \left(\frac{12hr}{23mm / 1.27mm/increment}\right)\left(60\min / 1hr\right) = 32 \text{ minutes/increment}$$

This information is summarized in Figure 6.9.

Design Guidelines:

Structure Number: *BRA-02* Designer: *B.K. Bramel* Date: *3/24/99*

Are there:

High likelihood for thermal shocks? Yes______ No *X*

Slow moving or stationary traffic? Yes______ No *X*

Skew angle in excess of 30°? Yes______ No *X*

If any of the above answers are ***Yes, Stop Here***. APJs are not applicable for this Location.

Max anticipated bridge daily motion: 18.0 mm (0.9 in)

Bridge t_{B75} time requirement: 31 Minutes/Increment

Anticipated temperature range for the structure: (From Superpave SP1 at bridge)

High 58 °C (142 °F) Probability: 99.9 % to be lower

Low -22 °C (-21 °F) Probability: 74 % to be higher

Anticipated Bridge Total Motion: 32 mm (1.3 in)

Required Joint Thickness: 100 mm (4 in)

Selection Checklist

Material Supplier	**T_g**	**t_{75}, min**	**T_g<T_{LOW} Y / N**	**t_{75}<t_{B75} Y / N**	**Acceptable Y / N**	**Thickness**
Watson Bowman Acme	**-27°C, (-16°F)**	**42**	**Yes**	**Yes**	**Yes**	**50 mm(2 in)**
Pavetech	**-26°C, (-15°F)**	**76,000**	**No**	**No**	**No**	
Koch/LDI	**-43°C, (-45°F)**	**2.8**	**Yes**	**Yes**	**Yes**	**50 mm(2 in)**

Joint Thickness	Maximum Motion
50 mm (2 in)	57 mm (2.3 in)
63 mm (2.5 in)	53 mm (2.1 in)
75 mm (3 in)	50 mm (2.0 in)
88 mm (3.5 in)	50 mm (2.0 in)
100 mm (4 in)	49 mm (1.9 in)

Figure 6.9 Example 2 Design Guideline Worksheet.

6.10 CONCLUSIONS

This research provides guidelines for the qualification of APJ materials and the selection of sites suitable for APJ installation. APJ installations will involve some maintenance and locations where additional joint fractures may occur are identified.

1. APJ material can be qualified using the TSRST and a modified TSRST procedures. The equipment for these tests were developed under the SHRP program and are readily available to DOTs
2. APJs should not be installed where the lowest anticipated temperature is below the glass transition temperature, T_g
3. Time-dependent material properties are extremely important to overall joint performance. The relaxation of the APJ material should be lower than the applied thermal displacement.
4. The total deformation available in an APJ is based on volumetric considerations of the joint material. Recommendations are provided for thickness and movement limits in Table 6.3.
5. APJs should only be installed where traffic moves at highway speeds.
6. APJs exposed to rapid/large thermal shocks will experience more fractures than in structures with gradual joint movements.
7. APJs should not be placed with a thickness of less than 50 mm (2 in).
8. Securing the gap plate on one side of the joint and forcing the displacement to only one direction may reduce the quantity of material.

6.11 RECOMMENDATIONS

APJs are a viable joint if properly qualified and used where appropriate. All bridges are not candidates for application. The lowest anticipated temperature should be above the glass transition temperature. Thermal shocking should be avoided. Bridge joint motions should be less than those shown in Table 6.2, and service free life expectancy is likely to be less than five years due to fatigue considerations and gradual stiffening of the APJ due to loss of volatile plasticizers. Traffic helps to "heal" joints. Geometric modifications are illustrated.

6.12 FUTURE WORK

Future APJ work should be focused on the following topics:

- Qualifying the UW TSRST with respect to temperature range.
- Develop the continuum relaxation relationship for elastomeric bridge joints.
- Laboratory verify the optimized joint geometry.
- Investigate the long term fatigue and aging characteristics of the joint.

6.13 ACKNOWLEDGEMENTS

The authors thank the Wyoming Department of Transportation and the Colorado Department of Transportation for their financial support, and Koch/LDI, Watson Bowman Acme, and Pavetech for providing materials.

6.14 NOTATION

U_o is the initial imposed displacement (constant).

K is the spring constant from the Maxwell body.

τ is a normalized variable defined as r/K, where r is the damping constant.

t is time after application of the load.

$P(t)$ is the joint load as a function of time.

t_{B75} is the amount of time between discrete bridge motions.

t_{75} is the time required for the material to relax 75% of its initial load.

6.15 REFERENCES

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7.0 APPENDICES TO: ASPHALT PLUG JOINTS: CHARACTERIZATION AND SPECIFICATIONS

These two appendices contain the supplemental data used to develop the survey results and the initial material tests reported in Chapter 3. Appendix A contains the raw survey data and is arranged by state. State material and installation specifications are included if they were supplied. Page A1 summarizes which state provided supplemental information.

Appendix B contains the raw data plots from the material studies. Page B1 provides the code for each test series. Page B2 summarizes the sample dimensions and provides lab contents.