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Simplifying Bridge Expansion Joint Design and Maintenance

Project Report

October 19 2011

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16. Abstract

This report presents a study focused on identifying the most durable expansion joints for the South Carolina Department of Transportation. This is performed by proposing a degradation model for the expansion joints and updating it based on bridge inspections.
Open expansion joints and pourable joint seal were found to be the best performing joints based on the proposed degradation models. Assembly joints and compression joint seal have an intermediate performance and strip seal expansion joints have the lowest performance of the type of expansion joints studied. Assembly joints are found to be problematic because of the different moving parts composing the joint. A significant number of bridge joint failures are caused because of incorrect installation, in particular, joints with complex anchor systems between the bridge deck and expansion joint.
The SCDOT standards were found to be up to date and comparable to other DOT standards in terms of the design and installation aspects of bridge joints. A recommendation is made to request a warranty for the installation of the when appropriate. Other general best practices during the installation of the expansion joint include: i) when possible, install joints when the ambient temperature is the average of the range of temperatures in the area. This allows the joint to be installed close to the "undeformed" position of the bridge, ii) the support of the joint should be installed in good quality, cured concrete, iii) avoid spliced of any pre-manufactured material. If splices cannot be avoided, place the splice outside the wheel path.

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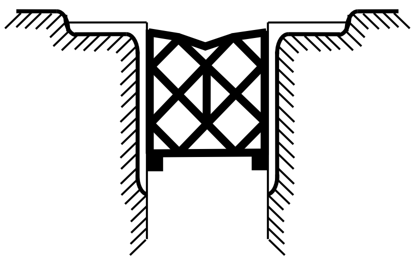
1 Introduction

1.1 Background

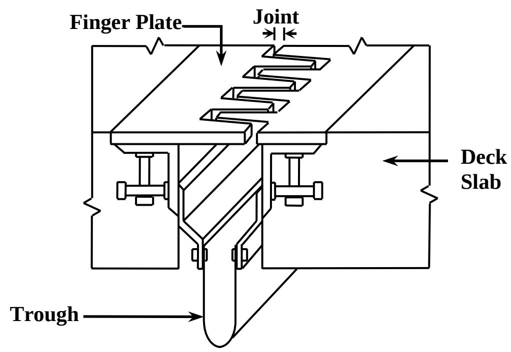
Bridge expansion joints are designed to permit the longitudinal movement and small rotations presented on bridge decks due to changes in environmental condition, live loads, and physical changes on the structural materials such as creep and shrinkage. While early designs did not provide water sealing, current designs require sealing to prevent the damage of support components. Similarly, the material used for bridge joints is constantly evolving due to problems identified with the different materials. Materials such as silicone foam sealants (Malla et al 2007) are now available for use in expansion joints. Some materials are more effective in areas with extreme changes in temperature, while other materials have been found to be more suitable for aggressive environmental conditions.

1.2 Bridge Joint classification

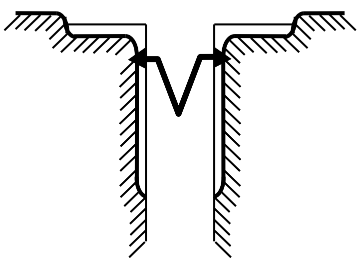
Bridge joints can be classified in open and closed joint systems. Open joint systems allow water and debris into the support components, while closed joint systems are sealed, protecting the supporting components. Examples of open expansion joints include finger, butt and sliding plates. Compression seals, strip seals, silicone foam, and plug seals are typical examples of closed joints. Figure 1 shows a graphical representation of these joint systems. Some open joint systems, such as the finger joint system shown in Figure 1.b have been modified to include a trough under the joint, providing drainage and protection to the support components. Open joints without trough should be in general avoided when possible to provide protection to critical supporting



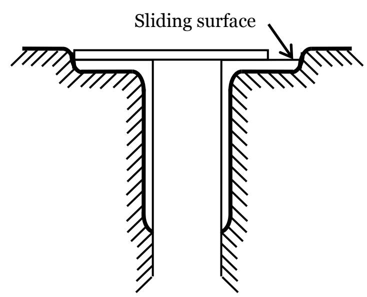
a. Open cell compression seal



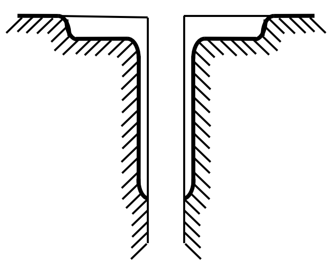
b. Finger joint with trough



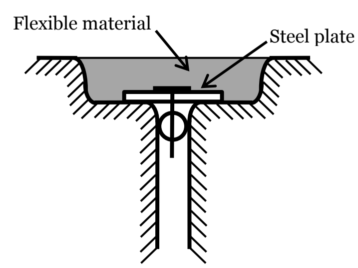
c. Strip seal



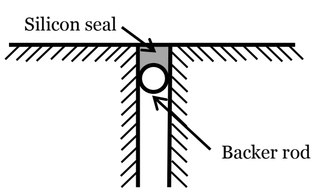
d. Sliding plate



e. Butt joint



f. Asphaltic plug joint



g. Silicon seal

Figure 1. Typical bridge joints

elements (Purvis 2003). Overall, the performance requirements for bridge expansion joints can be summarized in (Dexter et al 2001): i) Protect structural members from surface drainage; ii) support traffic between bridge components; and iii) allow for expected or unexpected bridge motion.

The selection of the type of expansion joint for a specific project is commonly determined by the maximum joint opening. The SCDOT bridge design manual, for example, indicates that asphaltic plug and silicone rubber sealant should be used for maximum joint openings less than 2 inches, while open finger plates and modular expansions should be used for openings of more than 4 inches. Bridge joints can drastically change the performance and lifetime due to a large number of factors, ranging from the amount and type of traffic to the installation procedure. Limited research is available in the identification of these parameters. Within this context Chang and Lee (2002) performed a study to determine the performance of several types of joints being used in Indiana. In their study, strip seal joints were found to outperform compression seal joints.

Arguably, modular expansion joints are the most complex joints. Figure 2 shows a multiple-support-bar modular expansion joint. Other types of modular expansion joints

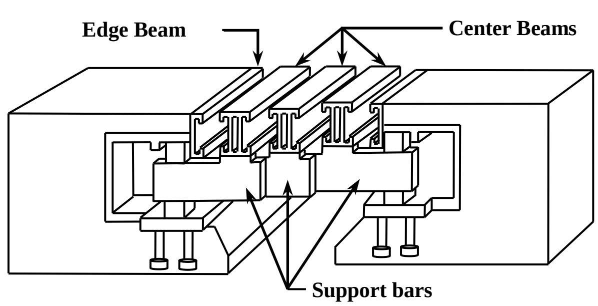


Figure 2. Modular Expansion Joints

include single support bar, swivel-joint, and scissor-type. These joints are used for large movements and have many moving parts which tend to fail over time. Modular joints are highly discouraged by the SCDOT Bridge Design Manual which specified that “modular joints may require significant maintenance” and that “the use of modular expansion joints must be approved by the State Bridge Design Engineer”. Several studies have found fatigue problems on these joints (Chaallal et al 2006, Crocetti and Edlund 2003, Roeder 1998, Dexter et al, 1997) and a high influence on the dynamics characteristics of these joints (i.e. Ancich et al 2006, Steenbergen 2004).

Joint-less bridges are an alternative to the high maintenance and operational costs created by bridge joints on new bridges. Joint-less bridges could be more expensive during construction due to increase detailing on the deck, but the initial cost has the potential to drastically reduce the overall bridge maintenance by eliminating the inspection, maintenance and replacement of expansion joints. However, it should be considered that any movement on joint-less bridges will be transferred to the end of the bridge which may result in other maintenance problems. In addition, joint-less bridges can accommodate a maximum approximate displacement of 2 inches (Wiss, Janney, Elstner Associates, 2002).

1.3 Objectives

The main objective of this research is to perform a probabilistic analysis of the performance of bridge expansion joints. Recommendations about what type of joints have the best performance is given based on the results of this analysis.

1.4 Organization

This report consists of the following sections. Chapter 2 contains a literature review in the area of bridge expansion joints. A description of the numerical tools created to study the performance of bridge expansion joints is presented in Chapter 3. Chapter 4 discusses the methodologies used for the study, including the formulation of a degradation model and its model updating. Chapter 5 shows the results of the updating process. Chapter 6 discusses the results of the project.

2 Literature review

2.1 Joints in general

Chang, L.-M. and Lee, Y-H, (2002) this paper looks at the performance of different joints. The joints looked at were compression seal, strip seal, integral abutment, poured silicone, and polymer modified asphalt joints. Thirty-three surveys were returned for analysis, 126 were sent out, from bridge inspectors and engineers in Illinois, Indiana, Kentucky, Michigan, and Ohio. What the questionnaire asked was for opinions on the serious problems, causes, advantages, and improvements for each joint. The answers to each of these are summarized for each joint. Referenced are the factors to influence joint performance: i) structural movement at joint; ii) traffic loading; iii) joint design; iv) materials used; v) detritus, foreign matter, and corrosion; vi) bond and anchorage; vii) condition of substrate; viii) weather and temperature during installation and service; ix) detritus, debris, and corrosion; x) site preparation and workmanship; and xi) performance of bearings. Also combined into the analysis was data obtained from the Indiana Department of Transportation. The analysis ranked the joints on performance with three factors: age, traffic loading, and settlement. The poured silicone and polymer modified asphalt joints were not included in this analysis because of lack of sufficient data. Additionally, personal interviews were conducted with representatives from all of the aforementioned states about the joints. The conclusions are broad due the fact that personal opinions are asked of engineers in comparison. The stated recommendations are: i) caution on selecting the right joint; ii) have a remedy procedure for the common joint problems (ex. snowplow damage and weather); iii) feasibility of warranty clause, as stated workmanship is a factor in joint performance; iv) testing the joint materials, to make sure they are effective for their desired use; and v) upgrade evaluation methods as currently only three grades can be given to a joint: good, fair, poor. More detailed evaluation criteria could help greatly in current and future evaluation.

2.2 Sealant Joints

Malla, R., Montgomery, S., Shrestha, M., and Brijmohan, S. (2007) performed laboratory testing of a silicone foam sealant and a Wabo silicone seal. Wabo silicone seal is a solid sealant and it was chosen as a comparison based on its commercial availability. This seal was the template for the original base for the silicone foam seal. This paper only reports testing in laboratory conditions. Tension, compression, shear, saltwater immersion, bond, stress relaxation, cure-rate, tack-free time, and water tightness tests were performed. The testing leads to the conclusion that the foam seal has a lower modulus than the Wabo silicone seal but in the saltwater test had problems with bonding and significant decrease in elongation capacity. No significant change in relaxation testing between the two sealants. Tack free times of less than 1.5 hours and 21 day curing rate reached 64% in three days and 80% in 7 days. Key note is the foam has a significantly lower modulus than the solid and the unsupported claim of its economical benefits. No long term testing was performed.

French, J. and Wallace Jr., M. (2003) investigates selected joint seals available to Virginia Department of Transportation, not all commercially available seals were selected. Additionally the goal is for developing a procedure to evaluate new joint sealing systems that come on the market. The seals selected for study were field molded, open cell compression seal, closed cell compression, strip, plug, and inflatable neoprene seals. The report emphasizes that all systems test well but also have failed in various situations. Adherence to recommended installation procedures is stressed for optimal results and the assistance of the manufacturer on site during installation is also recommended. The paper concludes that all of the systems evaluated provide satisfactory service but can fail if poorly designed or installed improperly. None of the joints can last indefinitely, and proper procedures need to be maintained when installing.

2.3 Modular Joints

Modular expansion joints are the most studied type based on the literature review performed for this research. One possible reason is because of their complexity and the interaction between the different joint components. Several studies focus on the dynamics of the joint and cracks on the joint components.

Crocetti, R. and Edlund, B. (2003) looks at how deck joints could be one of the most severely loaded components of a bridge. In an expansion joint the impact provided by the wheels is not distributed as effectively as other primary load member such as the asphalt. The joints studied on the paper were modular bridge expansion joints, modeled as single degree of freedom structures. This model is only valid for loads in the vertical direction given that the horizontal direction is not modeled. The conclusions of the paper indicate that an “equivalent axle load” with a “distribution” factor should be used for fatigue analysis. The paper includes field measurements obtained from Lehigh University.

Chaallal et al (2006) performed laboratory testing on modular expansion joints, with strong emphasis on to the full-penetration welds connecting the joints to the support bar.

Two actuators were used with varying loads. Static tests were conducted for obtaining strain measurement, which results were used for verification of a three-dimensional model. The results were used to create an analytical model that can be used for further analysis. The paper references the NCHRP-402 report “fatigue design of modular bridge expansion joints from the transportation board.” The paper indicates that modular joints were preferred over finger joints for their particular environmental conditions because deicing materials could seep through a finger joint and cause corrosion in other bridge elements. Numerical models and laboratory experiments were used to study the number of cycles required to fatigue the first welded connection. After the first crack appears, their numerical model is no longer representative of the joint. A maximum error of 21%, 25%, and 18% between numerical models and experimental results were obtained for the three test specimens.

Roeder, C. (1998) studied the fatigue damage on the center beams of the modular joints through two similar methods. The I-90 3rd Lake Washington Bridge in Seattle, Washington was instrumented for the experiment. A computational analysis was performed and verified with the experimental results. Two types of tests were performed. One test was controlled by closing a lane off to traffic and using a “moderately heavy” truck over the modular expansion joint. The truck was at constant speed braking, accelerating, and stopped over the joint. The second test was uncontrolled using real traffic, the measurements were recorded when a truck, heavy enough to trigger their data acquisition system, passes over the modular expansion joint. The first test was used to help with the effect of wheel position on the modular expansion joint and distribution of load between joints. Their conclusion is that the worst case is when a tire is nearly directly over center beam of a particular joint. The largest center beam movement occurs at slower speeds. Horizontal forces on the joint are believed to be a major contributor to fatigue damage and braking and acceleration of traffic is believed to be the biggest cause of the horizontal force.

Roeder, C. and Van Lund, J. (1993) used the same bridge to conclude that cracks on in-service expansion joints were caused by cyclic loads induced by trucks passing through the bridge. However, no specific information about the exact causes of the joint failure where obtained with the available data. The main goal of the paper is to provide a peer review of Tschemmernegg's fatigue evaluation method and proposed repair methods. Secondary goals include the cration of a computer model to determine the probable causes of the cracking, evaluate the proposed repairs and further recommendations. They concluded that: i) the fatigue problem is most serious at the edge of the beams. Center beams have the weight better distributed than edge beams; ii) large bending stresses are created on modular joints by truck wheel loads; iii) the tubular center beams contribute to the fatigue problem but fatigue would have been a problem with a different type of center beam; iv) The Tschemmernegg test does not accurately reflect the fatigue happening in these specific joints; v) Wheel loads cause multiple stress cycle during a single pass through; vi) The analysis done on the smaller expansion joints shows that they will begin cracking in a similar manner to the large joints after 8 years; vii) The damages on the joint accumulated more rapidly during a particular season, the fall.

Steenbergen, M. (2004).The paper discusses the dynamic response of expansion joints to traffic loading, mainly lamella joints from one specific supplier (Maurer) but state that the results are applicable to different joint-types. The testing locates three resonance frequencies for the different lamella configurations. First frequency is related to the lamella motion, the second is related to the mixed motion of the cross-beams and the lamella, and the third is related to the crossbeam motion. All are carried out with varying lamellas of one through seven with their mathematical model. The numerically calculated results are compared with previously measured results and shown to be within 5% of each other based on the single comparison put forward. The ranges of resonance frequencies are calculated for the different configurations and for the previously mentioned motions. The DAF, dynamic application factor, which is commonly assumed to be 1, was calculated and in all cases was found to be higher than 1. The middle lamella and the middle crossbeams have the largest dynamic effects in its response. As the gaps between the lamellas increase, the dynamic effects of the joint response increase.

Ancich, E. J. et al (2006) discusses the dynamic range factor for the modular bridge expansion joint, by testing through experimental modal analysis, ambient traffic excitation, and dynamic excitation on a nine seal joint on the west abutment of the Anzac Bridge. The initial study focused on the noise created by modular joints and expanded to study the joints fatigue problems. Testing deduced that the dynamic behavior of modular joints should be modeled as two similar but independent structures. This comes about as the odd numbered center beams were affixed to support bar 1 whereas the even numbered center beams were affixed to support bar 2. The experimental modal analysis states that the joint is lightly damped. Strain gauges were attached to the beams and the results, from the dynamic test (truck passing by) revealing the dynamic range factors (DRF). The maximum beam stress DRF was 4.6, derived from a maximum positive beam stress DRF of 2.7 and a negative beam stress DRF of 1.9. The support beams and center beams were acting dynamically as if simply supported.

Ravshanovich, K., Yamaguchi, H., Matsumoto, Y., Tomida, N., and Uno, S. (2007) this paper investigates the cause of the generation of the reported noise of the modular bridge expansion joints, as they produce louder noises than other expansion joints when vehicles pass over them. Finding the cause of the noise generation can lead to counter-measures. Testing was done with a series of car-running experiments with a full-scale model of a joint; this was performed to identify the noise and vibration of the joint when the car runs over it. The car used was a sedan-type two-axle car with a weight of approximately 2000kg, a width of 1.65m, and a controlled speed of 50km/h. The sound pressure was measured above and below the joint, and accelerometers were used to measure vibrations. Additionally an experimental modal analysis was done with impact testing, a 1.5 kg hammer, repeated twice on the third middle beam and the second support beam. Third the joint was modeled in ANSYS. The conclusion from the car test was that the noise below the joint was dominated with frequencies in the range of 500 to 800 Hz. This was attributed to sudden changes in air pressure within the gaps formed by the rubber sealing with the two adjacent middle beams. For the noise generated below, the joint is dominated by frequencies below 200Hz, which is caused by sound radiation. Further conclusion is below the joint noise can be greatly affected by the acoustic characteristic below the joint, as acoustic resonance in that space. The dominant frequency components of the joint in the car experiment can be related to the numerically obtained vibration modes of the ANSYS model. Further study in field measurements of the joint will be performed on a prototype bridge.

2.4 Plugs

Plug seal joints are built of an elastic material (usually polymer modified asphalt) placed on a cutout area of the deck. A steel plate is used to support the elastic material over the joint (Figure 1.f). Plug seals are commonly used for less than 2 in movement and some of the advantages of this type of joint include the easy and relatively inexpensive installation and repair. One of the main disadvantages of plug joints is that they were made for bridges with no curbs, parapets and barriers (Purvis, 2003). The NCHRP-319 report states that asphalt plugs were used to replace compression joints as part of several DOTs as part of deck rehabilitation.

Johnson and McAndrew (1993) performed an evaluation of 250 joints over a period of two years, including 125 plug joints. The recommendations of their study are summarized by Yuen (2005) for design installation and maintenance. The design recommendations include: "i) Joints for roads with significant cross-sectional or profile gradients should be designed using relatively stiff binders to reduce debonding and binder flow; ii) Joints should be linear with uniform widths of at least 20 in; iii) Localized widening should be avoided especially on heavily trafficked roads; iv) If widening is unavoidable, stiffer binders should be used to minimize deformation and binder flow." For installation the recommendations include "i) Before the joint is installed, all loose material should be removed from the deck, and the deck substrate should be thoroughly dry; ii) Bridging plates should be installed across the expansion gap to prevent extrusion of the joint material in the gap under traffic loads; iii) Joints should be continued straight through the curb. The depth of the curb over the joint should be reduced to ensure that the full joint depth can be maintained under the curb; iv) The joint and transition strips should be approximately level with the adjacent deck surfaces to provide good ride quality." The maintenance recommendations include: "If the surface adjacent to a failed joint deteriorates, both the joint and the deteriorated surfacing should be replaced to improve ride quality and overall durability".

French and Wallace (2003) performed a series of case studies for different type of bridge joints. They study found that the overall performance of asphaltic plugs was good but expressed some concerns about the VDOT specifications. The specifications might not provide enough alternatives of the binder material to support the required deck expansion due to the expected temperature changes.

Some research has been performed in the use of semi-rigid joints. Similar to plug joints, these joints use some elastomeric material (Ure-Fast PF-60). Avila, D., Sharp, B., and Stewart, R. (2003) describe the use of an elastomeric concrete as a semi-rigid joint for the San Rafael Bridge on State Route 24 in Utah. The current standard procedure to replace joints at the UDOT is to saw-cut the current joint out and replace it with a new one. The purpose of this study was to study a quick alternative for quick joint replacement. The elastomeric concrete was found to be stronger in compression than the surrounding concrete creating lift of the concrete and causing possible problem with snowplows. The cure time for the Ure-Fast PF-60 material is approximately 10 minutes bringing the total time for replacement of the joint to approximate one hour. The Oregon, Washington, and Colorado DOT's also reported using this material. The preliminary conclusion, due to the high compression strength, is that it should not be used for joints with large displacements because it can cause the surrounding concrete to crack.

Coeslli, C.J., Griffith, E.M., Ryan, J.L., Bayrak, O., Jirsa, J.O., Breen, J.E., and Klingner, R.E. (2005) this paper discusses the behavior of bridge slab ends used currently by the Texas Department of Transportation, and additional ends behavior as well. The standard detail used is "IBTS." The additional details are "UTSE" which is a smaller version of the "IBTS," which can be observed in their figures, and the "PCPE" which is used of stay-in-place precast prestressed concrete panels minimal to no further description of the details is given past the figures. The testing done was making three full scale bridge deck specimens and tested. The testing for the IBTS and UTSE were performed on two of the decks with skews of 0° and 45°, while the third deck was used with PCPE. The first test of IBTS and UTSE had all areas failed in shear, with UTSE failing in slightly lower load levels, believed to be because of its smaller section depth. Both of them had ultimate capacities at loads that well exceed the design load levels. The second test exhibited tremendous capacity during the extreme portion of the testing at 45° skew with 10-ft spacing. All of the three options performed well under the AASHTO LRFD design tandem load. Even when subjected to overloading, the cracking was observed to be minimal, which was defined as less than 24 inches in length and 0.01 inches in width. The cracking remained minimal until approximately 1.5 times the AASHTO loads. In conclusion, the last two methods were recommended as they were smaller, more cost efficient, had lower construction times, and similar testing results. The findings of the report were implemented into the Galveston causeway expansion project.

3 Numerical tools

Two main numerical tools, a data repository and an analysis tool, were created to implement the methodology shown in the previous chapter. A relational database was created to store information about inspection data and store the results from the analysis. Matlab scripts were created to connect to the database, download the data required for the analysis, perform the analysis and upload the processed data back to the database for storage.

An online database was created to organize the data collected from bridge joints. The database was created in MySQL, a free database that can be easily connected to programs such as Matlab for data processing. MySQL supports a relational model that allows the grouping of information using specific relationships. Data having the same characteristics are organized in a particular table. Relationships between the different tables can be performed, allowing the organization of the data in different ways. For example, a relationship between a table containing expansion joint definition and a second table containing the inspection of the expansion joints can be used to identify the inspection rating of a particular type of expansion joint.

The database was designed to contain the information from the result of the analysis, to be queried from an online tool, as well as information from the SCDOT Pontis database. The data needed from the Pontis database consists of the element definition and the inspection history. The element definition table contains a numeric identifier for each type of expansion joint on the Pontis database as shown in Table 1. The data required on the inspection history consists of the inspection year, and the percentage of the length of the joint that is at a particular stage (i.e. PCTSTATE1, PCTSTATE2, PCTSTATE3, PCTSTATE4 and PCTSTATE5). The description of the different fields of the relational database and the respective fields is shown in Figure 3.

Table 1. Element definition and corresponding element key

Element Key	Joint type
300	Strip Seal Expansion Joint
301	Pourable Joint Seal
302	Compression Joint Seal
303	Assembly Joint (Modular expansion joint)
304	Open expansion joint

A Matlab program was created to query the database and perform calculations. The Matlab program calculates an expansion joint performance metric and updates a degradation model for the expansion joints. Both, the performance metric and the degradation model are discussed in detail in the next chapter.

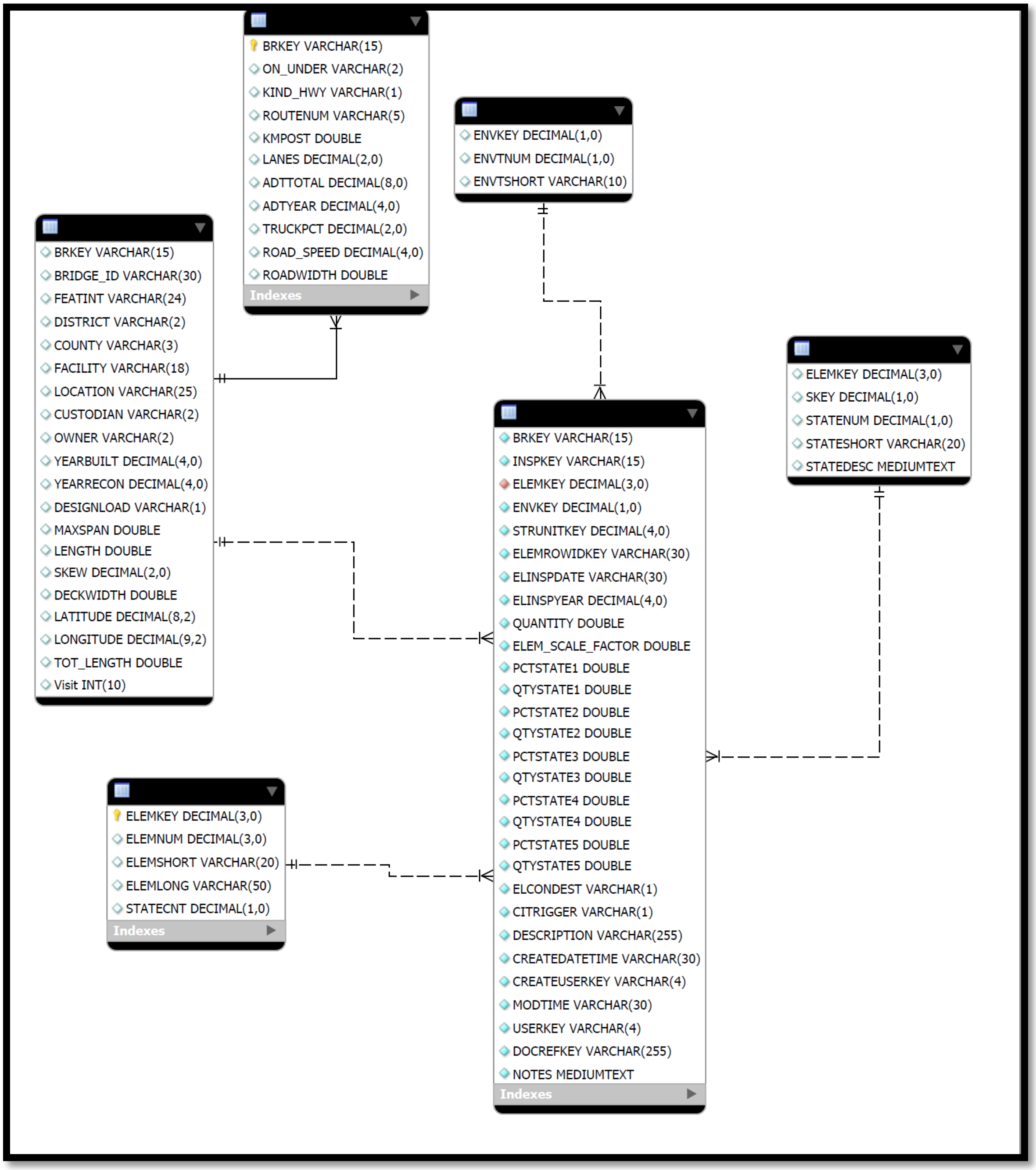


Figure 3. Database tables, fields and relationships

4 Methodology

This chapter contains information about the methodology used to investigate the robustness of different types of bridge expansion joints. First, a metric is proposed to identify the performance of the joints based on the information provided from Pontis by SCDOT. Then, a degradation model is proposed to model the degradation of the joints as a function of time. Finally, the methodology used to estimate the model parameters for each joint is described. This procedure allows the identification of a model that mimics the performance of the joints, which can be later used to investigate which type of joints last longer. Even though the results presented in this report are specific for SCDOT, the methodology has the potential to be used for other environments. Overall the methodology can be summarized in the following steps:

1. Measure the performance of expansion joints in state1 (no damage) using a performance metric.
2. Calculate the parameters of a degradation model for each of the expansion joint.
3. Use the developed degradation models to estimate the time required for an expansion joint to need repairs.

The results of the analysis is a degradation model that indicates the time needed for the expansion joint to need repairs, which can be used to give recommendations about what joints require less maintenance.

4.1 Performance metric

A performance metric is proposed to measure how quickly an expansion joint degrades based on the inspections performed by SCDOT. The SCDOT Pontis database fields imported into the relational database developed for this project are considered for the metric definition. The metric is used to combine the information on the different states in a particular inspection. The proposed performance metric is defined with the equation

$$m_{inspections} = \frac{500 - \sum_{s=1}^{5} s * p(s)}{5}$$

where m is the particular metric for a particular inspection, *s* is the state and *p(s)* is the percentage of the length of the expansion joint that is in state *s*. The highest value for the metric is 100 and it indicates a completely undamaged joint. The smallest value is 0 corresponding to 100% of the joint being in state 5. To describe the calculation of the metric, consider that the entry for a particular inspection having the values shown in Table 2.

PCTSTATE1 p(1)	PCTSTATE2 p(2)	PCTSTATE3 p(3)	PCTSTATE4 p(4)	PCTSTATE5 p(5)
80%	0%	10%	10%	0%

Table 2. Metric calculation example

The value of the metric would be calculated as

$$m_{inspections} = \frac{500 - (80 * 1 + 0 * 2 + 10 * 3 + 10 * 4 + 0 * 5)}{5} = 70$$

Given that inspections are performed every 2 years, the metric change can be used as an indication on the degradation of the expansion joint during that interval. In this particular case the change in metric is defined as

$$\Delta m_{inspections} = m(t_2) - m(t_1)$$

where $\Delta m_{inspections}$ is the performance metric change between the inspections at time t_1 and t_2. Only negative changes of $\Delta m_{inspections}$ are included in the calculations reported here. Negative values indicate the case when an expansion joint is fixed and the analysis focuses in the degradation of the joints.

Figure 4 shows a typical histogram showing the performance metric change $\Delta m_{inspections}$ using the information from Pontis. Even though the results for each joint type are discussed in detail in the following chapter, this plot is shown here to indicate that most of the change observed on the Pontis database is in fractions of 25 points. In this particular figure, only 5 inspections from over 1380 indicated a change in performance different to 0 and 25 points. The change of performance of 25 points corresponds to changes of 100% state 1 during one inspection and 100% state 2 in the following inspection two years later. This indicates that once an expansion joint starts failing, the failure mechanism is very likely presented along the whole expansion joint. This information is important when proposing a degradation model as discussed in the following section because the change in performance metric can be defined at particular discrete points. For the particular case of Figure 4 the change of performance metric can be described by the values of

$n\Delta m_{inspections}(0) = 1320 = 95\%,$

$n\Delta m_{inspections}(-25) = 60 = 5\%$

$n\Delta m_{inspections}(-50) = n\Delta m_{inspections}(-75) = n\Delta m_{inspections}(-100) = 0$

where $n\Delta m_{inspections}(k)$ is the percentage of inspections with a change of performance metric equal to k.

4.2 Proposed degradation model

Degradation models or degradation functions have been proposed in the past to model the loss in strength for structural members (Enright et al 1996, Enright and Frangopol, 1998). For a reinforced concrete beam subjected to flexure the resistance degradation function was defined by Enright and Frangopol (1998) as

$$g(t) = 1 - \theta_1 * t - \theta_2 * t^2$$

where $g(t)$ is the resistance degradation function at time *t*, and θ_1 and θ_2 are random variables. The same degradation function is proposed to be used here for the bridge expansion joints such that the performance metric at time *t* can be calculated using

$$m_{model}(t) = 100 * g(t)$$

The parameters θ_1 and θ_2 are assumed to be random variables that can be tuned to reproduce the metric of the different type of joints. The process of tuning the parameters is usually called model updating and it is explained in detail in the following section. Given than the change in performance metric, identified from the SCDOT inspections, is in steps of -25 points, the values of Δm are approximated to the next appropriate value. This is:

$$\Delta m_{model} = 0 \quad for \quad m_{model}(t_2) - m_{model(t_1)} > -12.5$$

$$\Delta m_{model} = -25 \quad for \quad -12.5 \geq m_{model}(t_2) - m_{model(t_1)} > -37.5$$

$$\Delta m_{model} = -50 \quad for \quad -37.5 \geq m_{model}(t_2) - m_{model(t_1)} > -62.5$$

$$\Delta m_{model} = -75 \quad for \quad -62.5 \geq m_{model}(t_2) - m_{model(t_1)} > -87.5$$

$$\Delta m_{model} = -100 \quad for \quad -87.5 \geq m_{model}(t_2) - m_{model(t_1)}$$

Similar to the definition of $n\Delta m_{inspection}(k)$, a percentage of change of performance metric is defined for the degradation model ($n\Delta m_{model}(k)$). This value is calculated after performing a simulation in which the degradation of several expansion joints is simulated. Given that the time in which the joint was constructed is not known, the simulated expansion joints are assumed to be constructed based on a uniform random number from 0 to 10 years before the first simulated inspection.

4.3 Updating of degradation models

Bayesian approaches on model updating have become more popular in recent years because it accounts for all sources of uncertainty. For instance, Beck and Katafygiotis (1998) proposed a Bayesian approach for solving the problem of model updating including uncertainties. Katafygiotis et al. (1998) proposed a Bayesian model updating technique that allows obtaining the uncertainty of the updating parameters and accounting for the issue of identifiability. Lam et al. (2004) successfully applied a statistical methodology for model updating that accounts for uncertainties in measurements and modeling, and possible nonuniqueness on the ASCE structural health monitoring benchmark study.

Let, *M*, be a chosen model (*e.g.* a degradation model), which is a function of some parameters (Θ), and *D* represents the experimental information obtained from the inspections (*i.e.* change of performance metric). The Bayes' theorem can be written in model updating context, as

$$P(\Theta|D,M) \propto P(D|\Theta,M)\,P(\Theta|M)$$

where $P(\Theta|D,M)$ corresponds to the probability density function (PDF) of Θ for the chosen model *M* after being updated with the observation *D*, or posterior PDF. $P(\Theta|M)$ is the PDF of the parameters Θ for the chosen model *M* before updating, or prior PDF. $P(D|\Theta,M)$ is the likelihood of occurrence of the measurement *D* given the vector of parameters Θ and the model *M*. For convenience Bayes' theorem is rewritten as:

$$f_p(\Theta;D) = c\, g_p(D;\Theta)\, h_p(\Theta)$$

where f_p, g_p and h_p are the posterior, likelihood and prior probability density functions respectively and *c* is normalization constant given by:

$$c^{-1} = \int_{S(\Theta)} g_p(D;\Theta)\, h_p(\Theta)\, d\Theta$$

The likelihood is defined as a Gaussian distribution of the difference between the response of the model and the experimental values, corresponding in this case to the change on performance metric based on the inspections. The likelihood can be calculated using the expression

$$g_p(D;\Theta) = \frac{1}{Z_g} exp\left(-\frac{1}{2}\sum_{j=0,-25,-50,-75,-100}\left\|\frac{n\Delta m_{inspections}(j) - n\Delta m_{model}(j,\Theta)}{\sigma_j}\right\|\right)$$

where σ_j is the standard deviation of error between the measured and performance metric calculated with the model and

$$Z_g = \int_{s(\theta)} g_p(D;\Theta)d\Theta = \left(\sqrt{2\pi}\right)^{2n} \prod_{j=1}^{n} \sigma_j$$

The evaluation of the likelihood is usually computationally expensive depending upon the complexity of the model.

The prior PDF ($h_p(\Theta)$) is a probability representation of the engineering judgment on the parameters describing the expansion joint degradation. In this research, the prior is defined as a uniform distribution that covers the feasible region of the parameters to update. In other words, any value inside of the feasible region has equal probability and any value outside of this region has probability zero, as presented in the following:

$$\begin{aligned} h_p(\Theta) &= cp \qquad \Theta_l < \Theta < \Theta_u \\ &= 0 \qquad\quad \textit{otherwise} \end{aligned}$$

The posterior PDF is finally calculated using the prior and likelihood functions proposed above. This posterior PDF corresponds to the probability of the degradation parameters $\Theta = [\theta_1\ \theta_2]$ representing the change in performance metric calculated from the Pontis database.

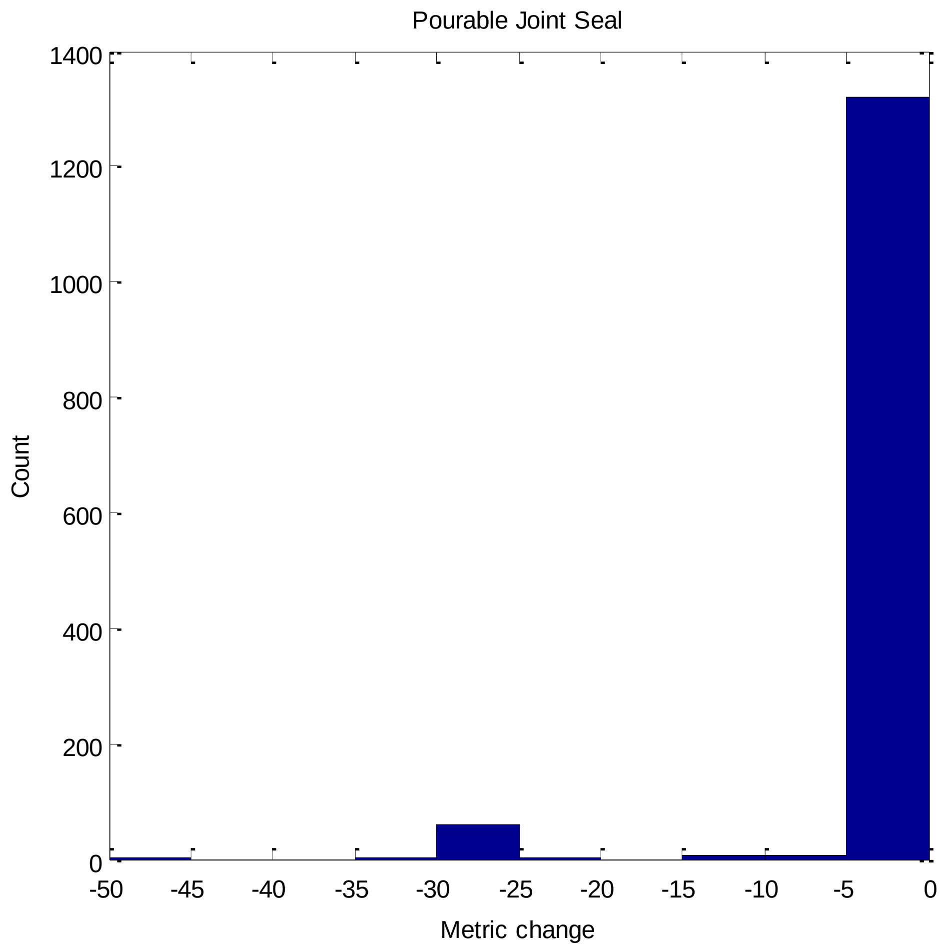


Figure 4. Metric change for pourable joint seal for two consecutive inspections (2 years)

5 Results

The data of 3061 bridges inspected by the SCDOT from 2001 to 2007 where considered for this research. The distribution of the number of inspections per type of expansion joint is shown on Table 3 and the distribution of the total number of inspections per year is shown in Figure 5. The number of available inspections for pourable joint seals and compression joint seals is significantly larger than strip seal, assembly and open expansion joints. Therefore, the methodology implemented in this research is expected to provide more statistically meaningful results from pourable and compression joints. Of special interest are consecutive inspections that show a decrease in the change performance metric (Δm), (see section 4.1 for the definition of the performance metric).

Expansion Joint Type	Number of inspections considered
Strip Seal Expansion Joint	114
Pourable Joint Seal	6049
Compression Joint Seal	2092
Assembly Joint	139
Open Expansion Joint	170

Table 3. Inspections available from Pontis database between 2001 and 2007

5.1 Performance metric

The change in performance metric (Δm) was calculated for two consecutive inspections that started with a completely healthy expansion joint for each joint type. Joints that were repaired, showing as a gain in the performance metric were not considered because the objective of this study is to model the degradation of the expansion joints. The results of this analysis can be seen in Figure 6 to Figure 10 for the different expansion joints considered. Close to ¾ of the strip seal expansion joint found did not have a change of status, while nearly ¼ had a decrease in performance of 25 points. This decrease in performance corresponds to changing from 100% from state 1 to 100% to state 2.

5.2 Posterior PDF

The posterior joint probability density function for θ_1 and θ_2 for each of the types of expansion joints studied are shown in Figure 11 to Figure 15. Darker color indicates low probability and lighter color indicate high probability. One common finding between all the joints is that the value of θ_2 is most likely equal to zero, indicating that the degradation of the expansion joints could be modeled as a linear relationship. This is confirmed with the marginal probability density function of θ_2 shown in Figure 16. With this finding, we can assume that the value of θ_1 can be used as an indicator of the degradation of the expansion joints. The marginal probability density function for θ_1 is shown in Figure 17. This plot shows clear differences between the different types of expansion joints. The range of values for this probability density function varies from 0 to 0.65. The highest values of probability are found close to $\theta_1 = 0.1$ and $\theta_1 = 0.4$.

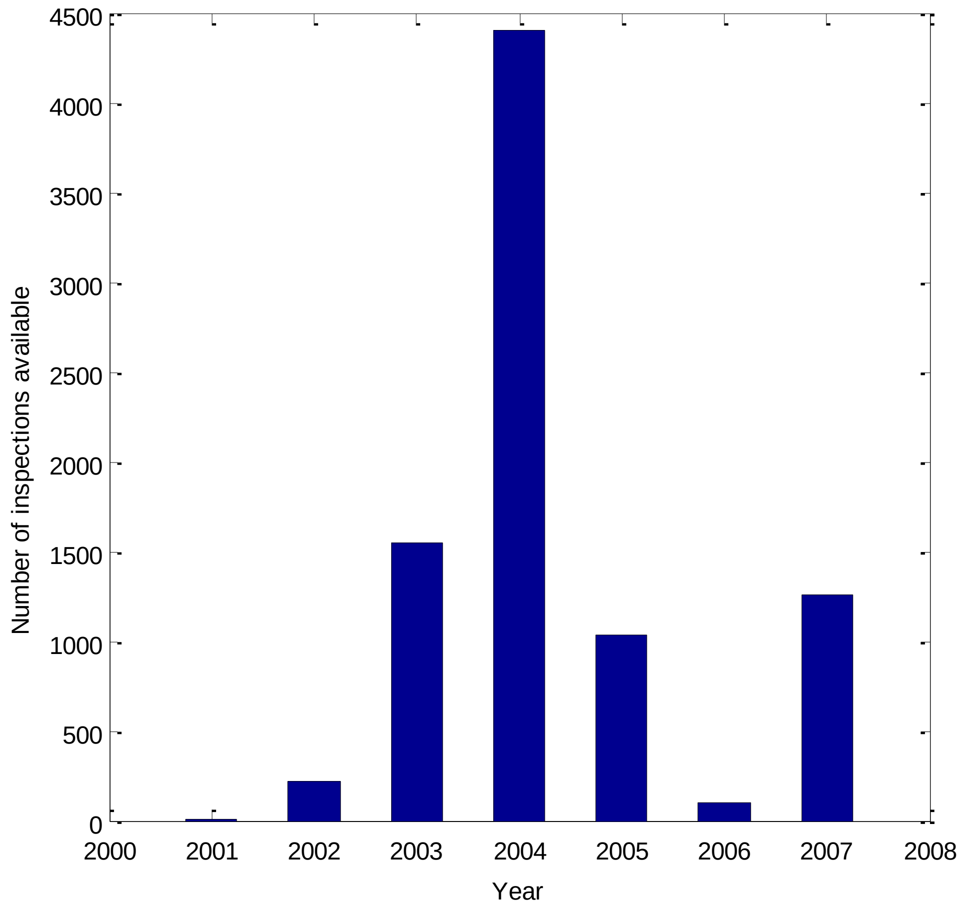


Figure 5. Number of inspections available for analysis per year

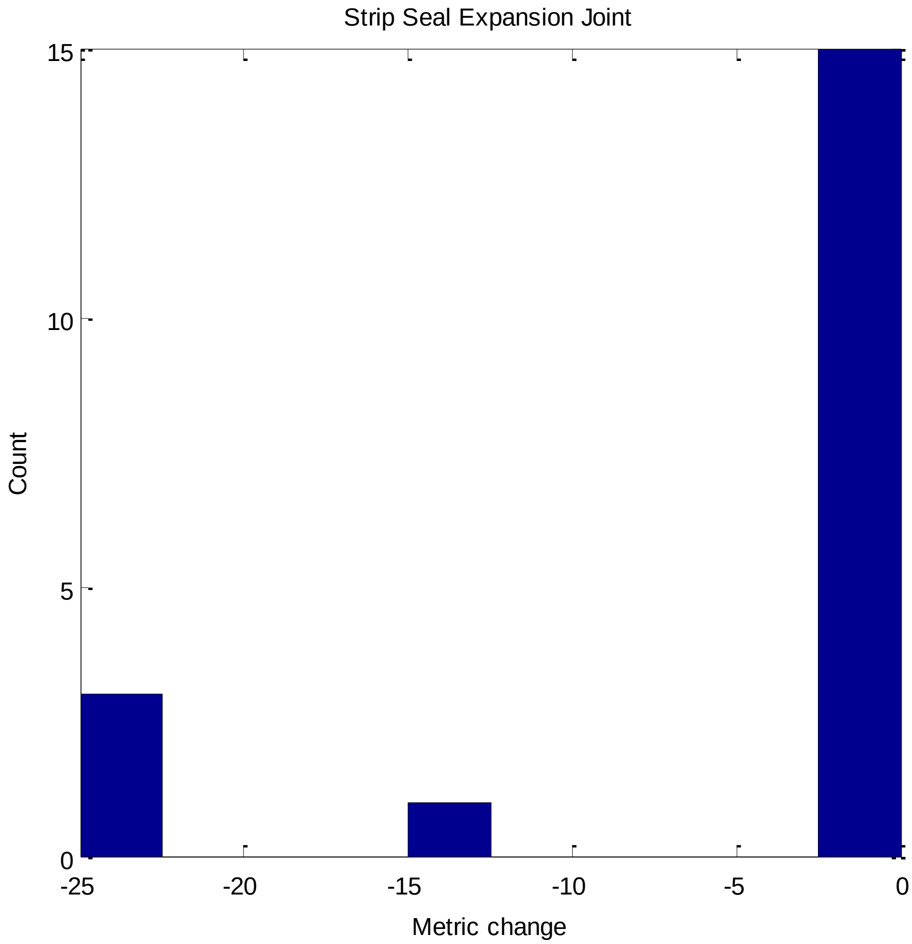


Figure 6. Metric change for strip seal expansion joint for two consecutive inspections (2 years)

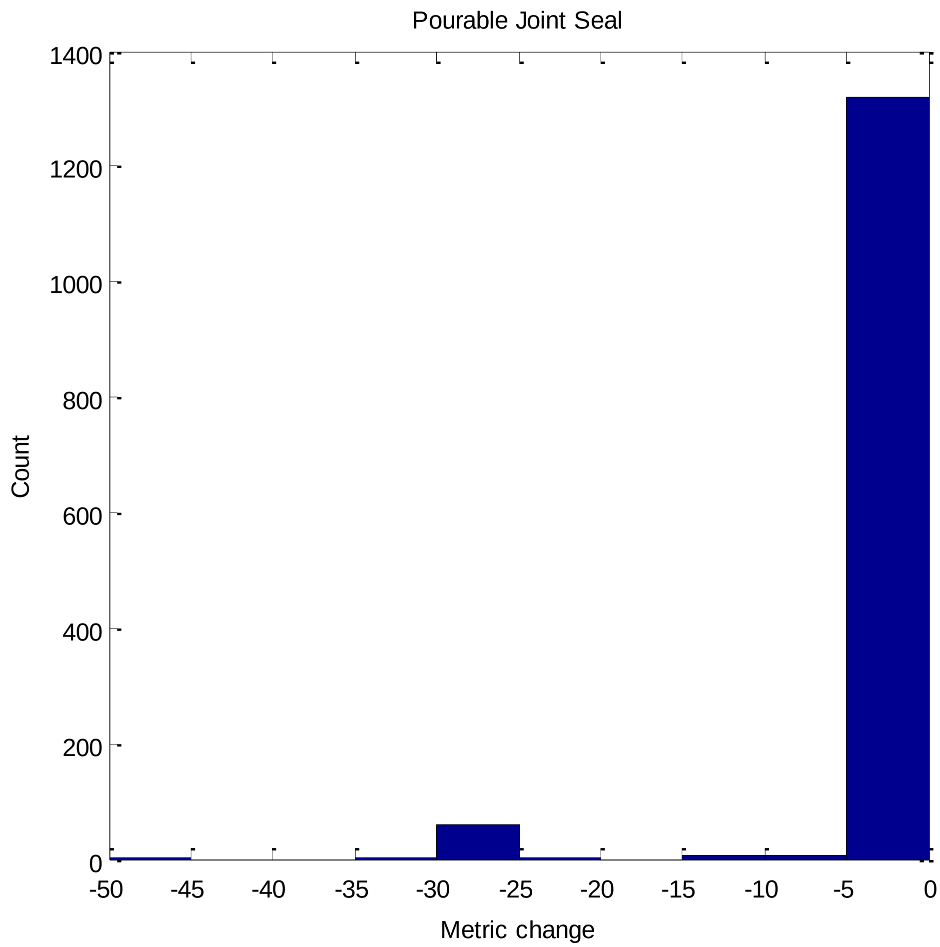


Figure 7. Metric change for pourable joint seal for two consecutive inspections (2 years)

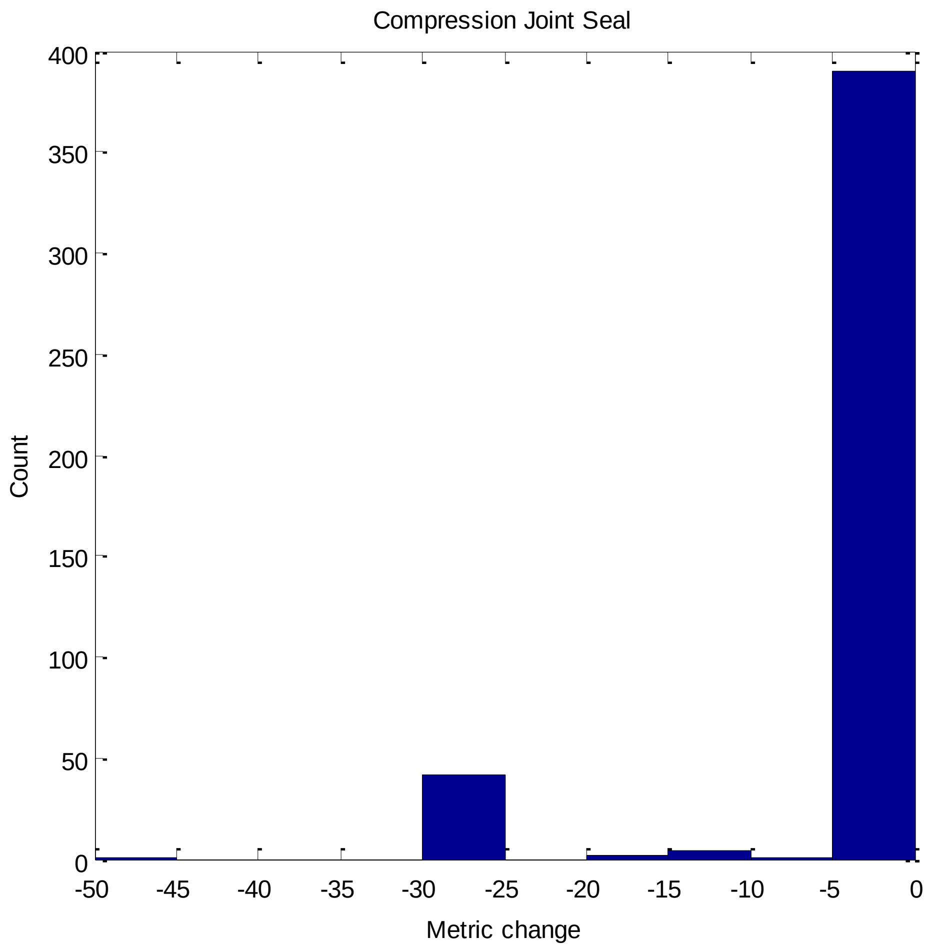


Figure 8. Metric change for compression joint seal for two consecutive inspections (2 years)

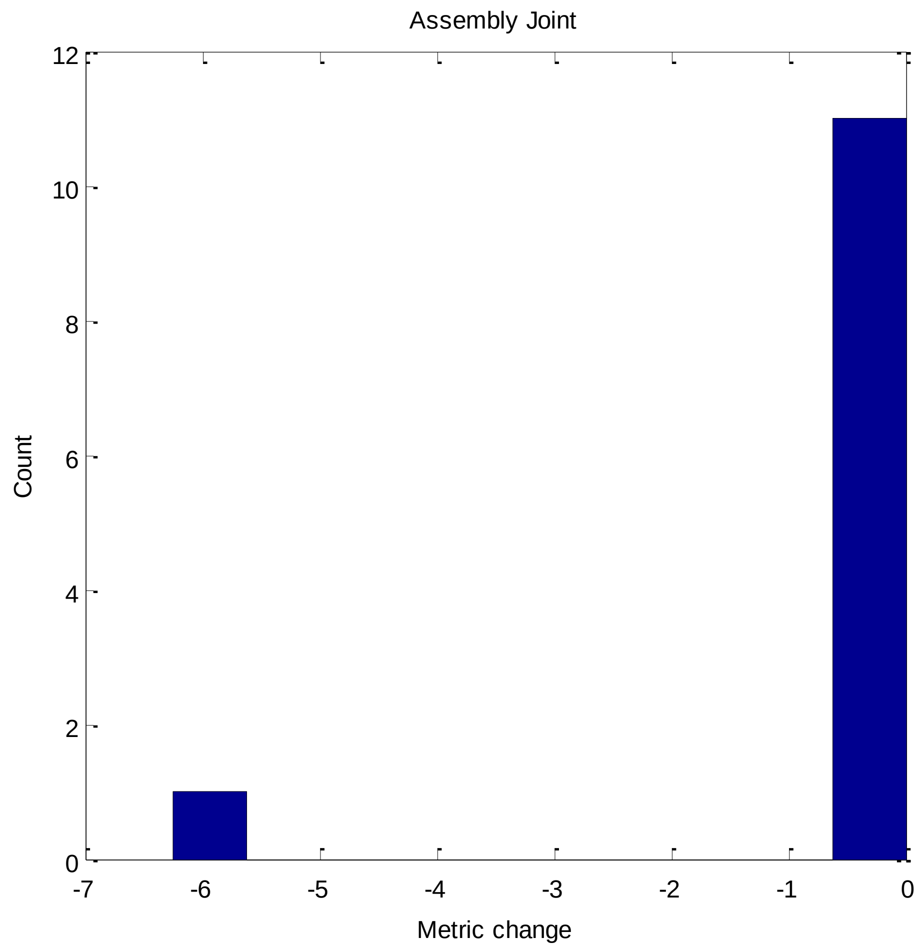


Figure 9. Metric change for assembly joint for two consecutive inspections (2 years)

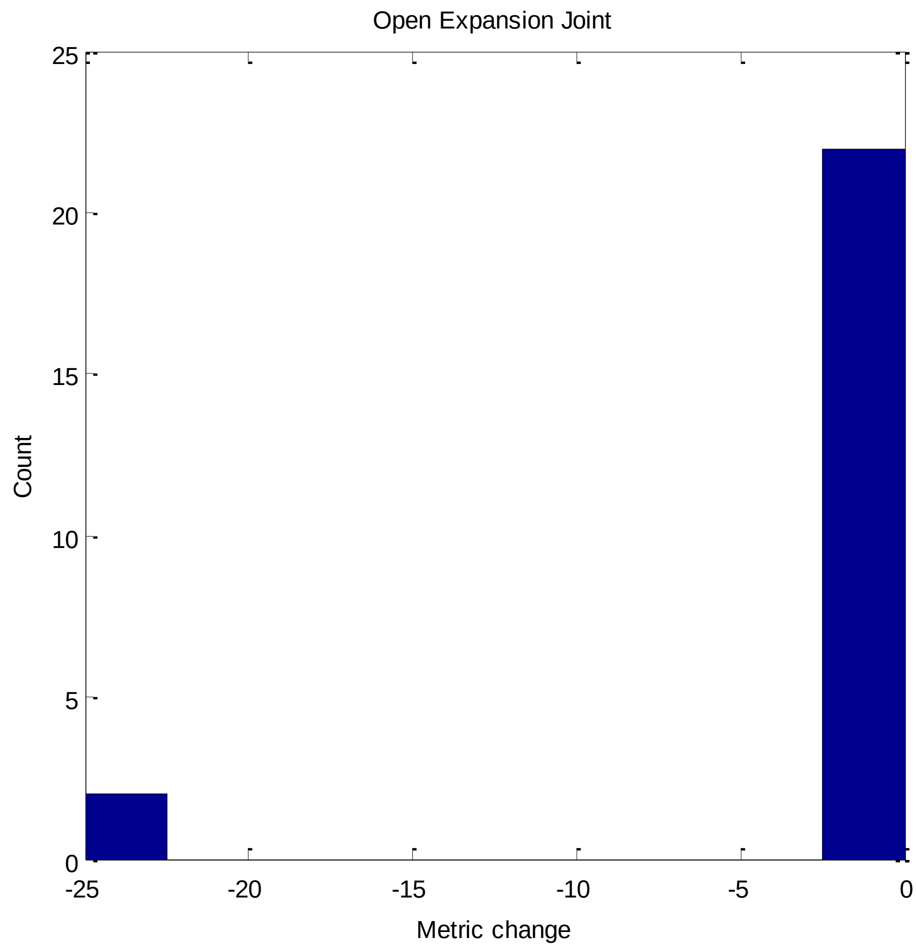


Figure 10. Metric change for open expansion joint for two consecutive inspections (2 years).

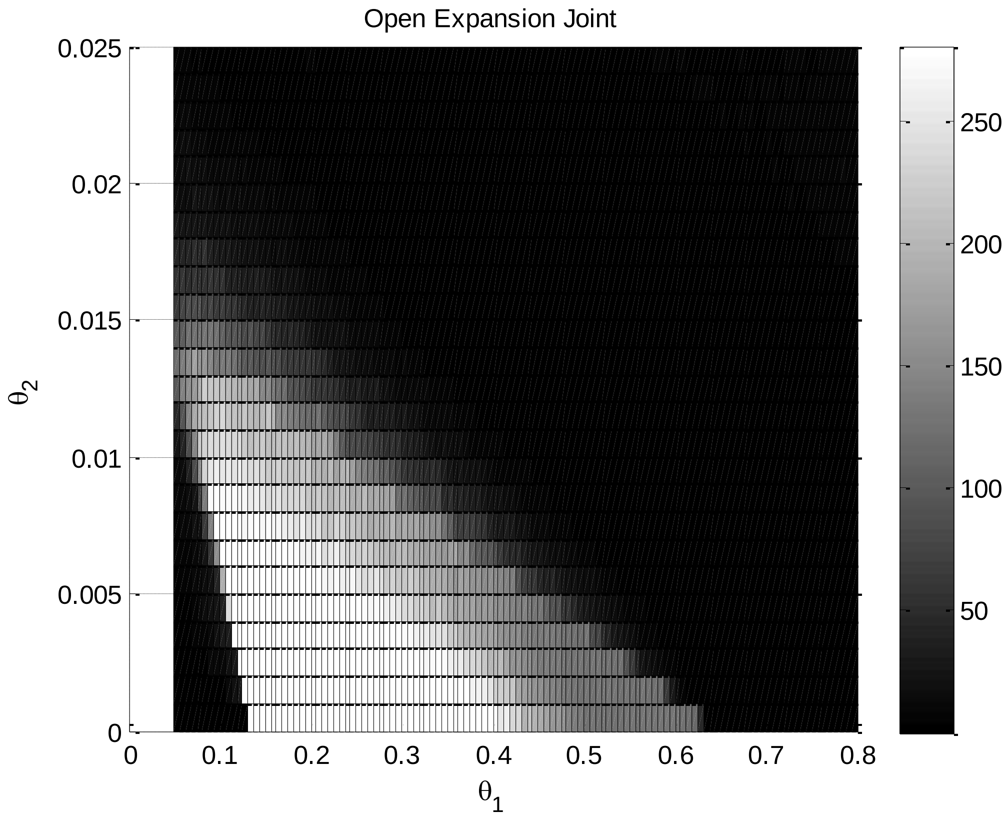


Figure 11. Posterior PDF - Open Expansion Joint

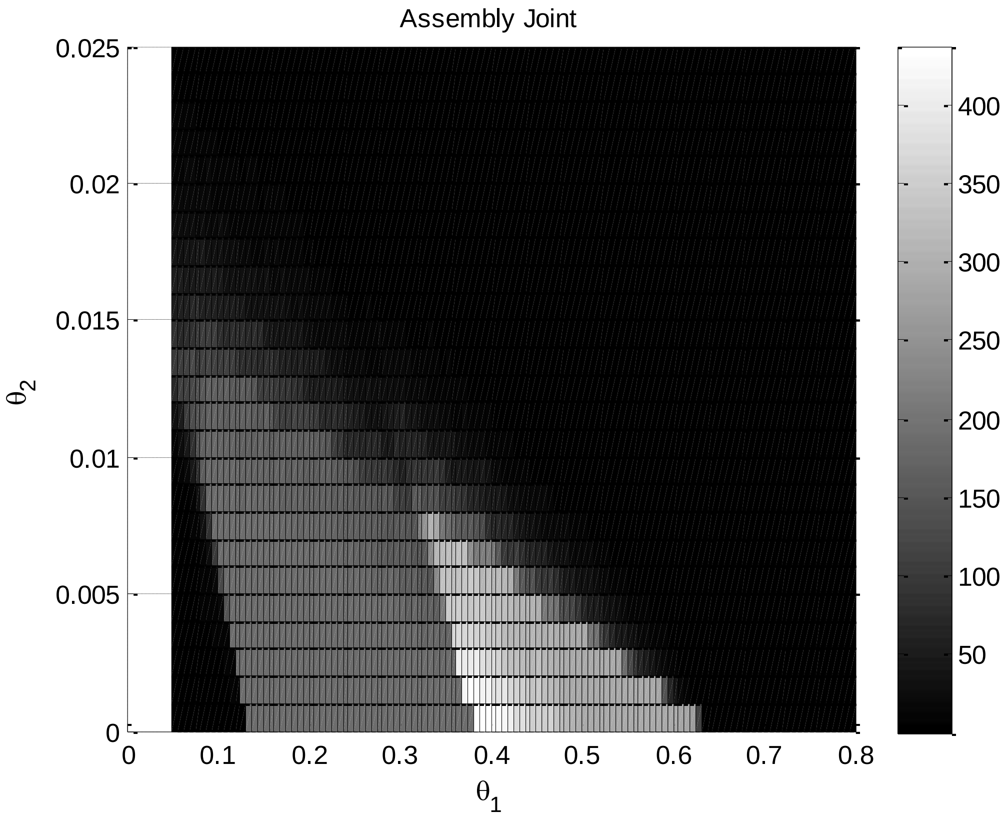


Figure 12. Posterior PDF - Assembly Joint

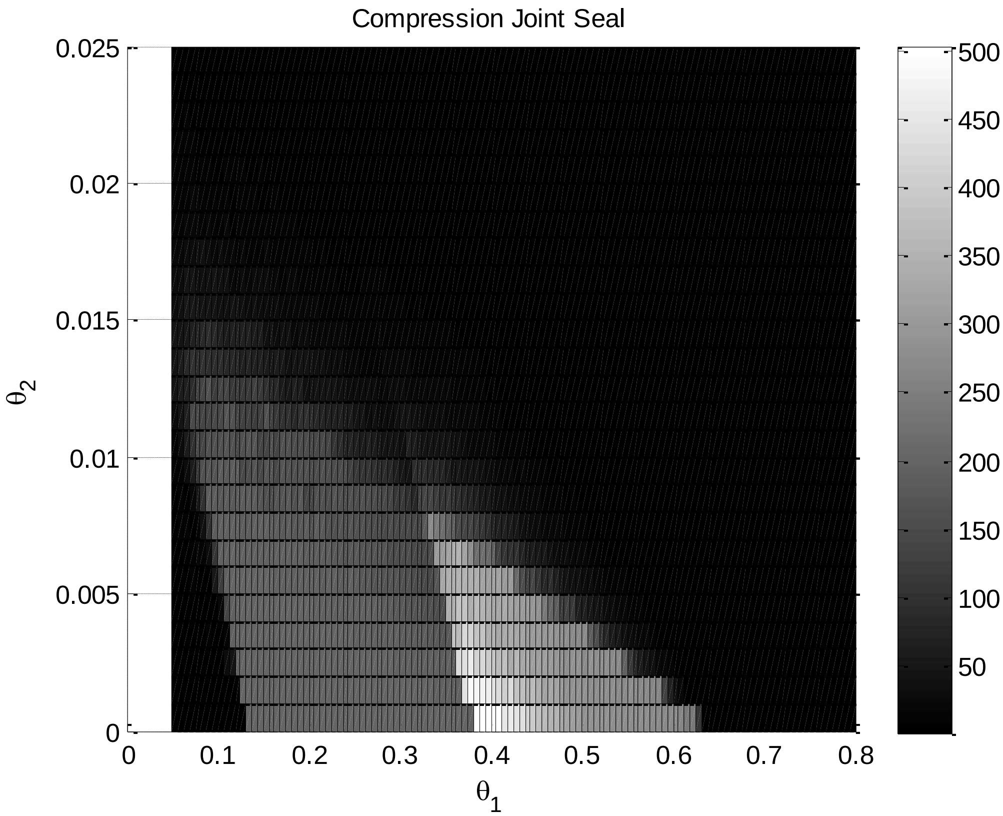


Figure 13. Posterior PDF - Compression Joint Seal

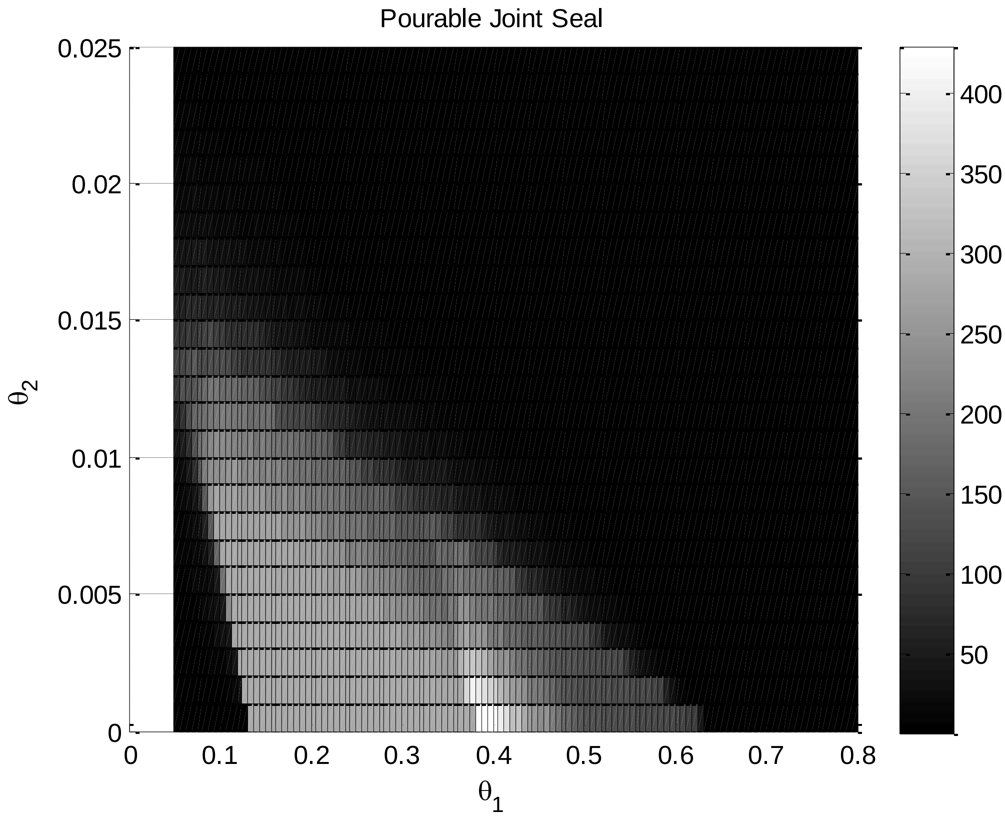


Figure 14. Posterior PDF - Pourable Joint Seal

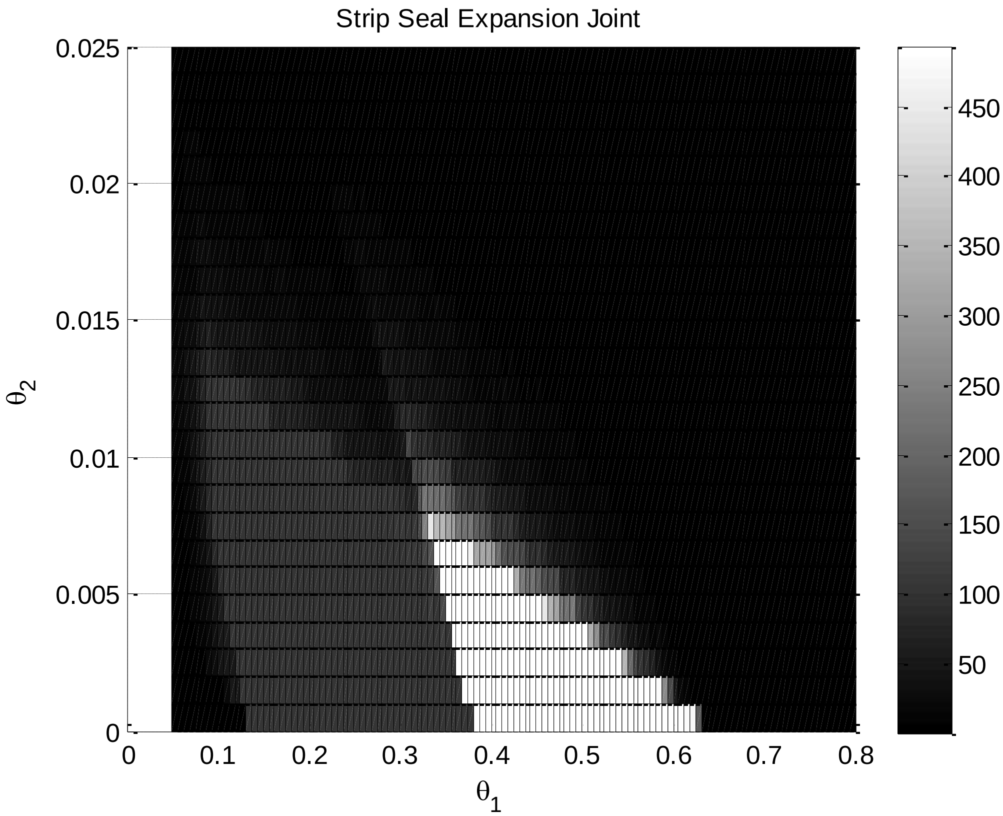


Figure 15. Posterior PDF - Strip Seal Expansion Joint

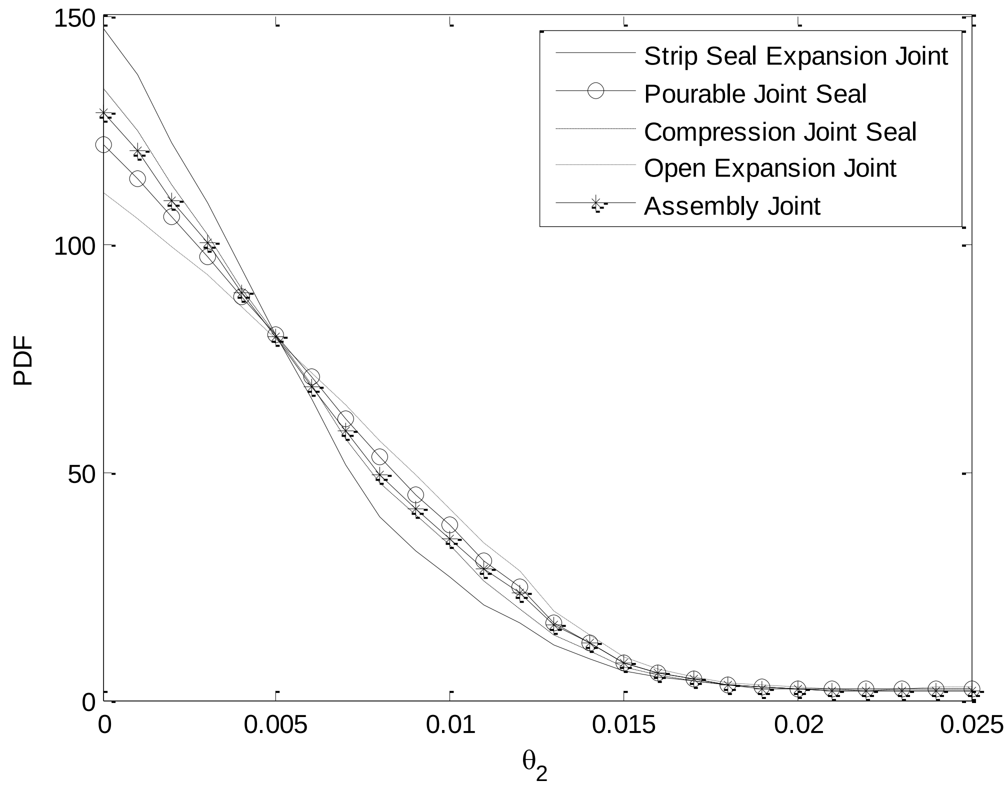


Figure 16. Marginal Probability Density Function θ_2

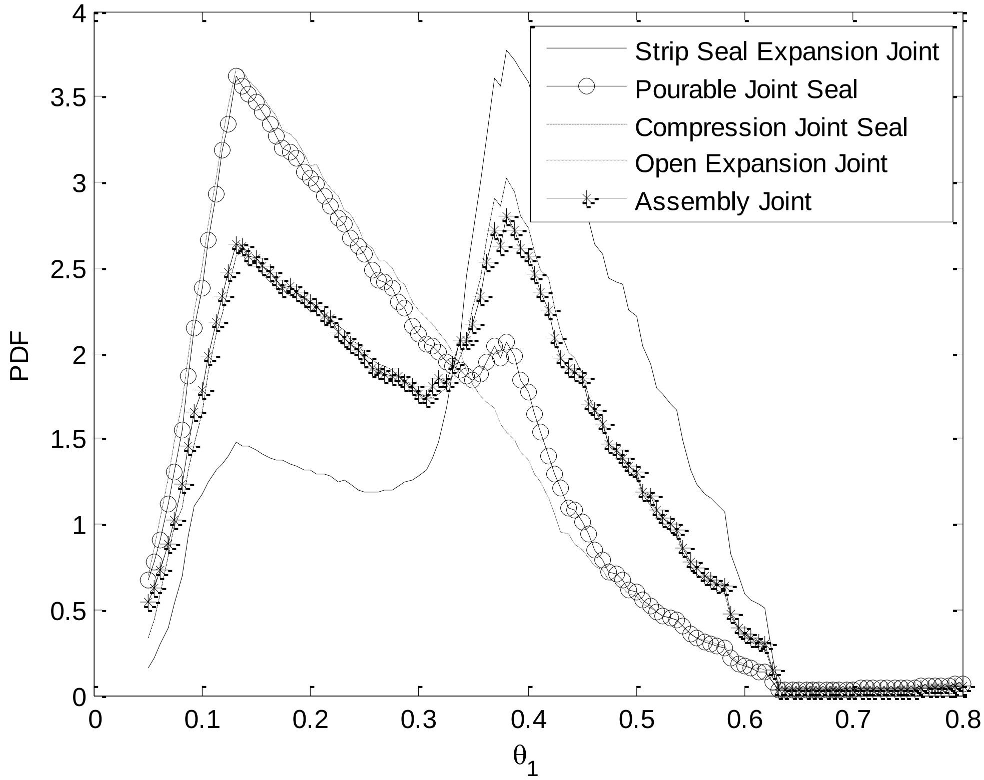


Figure 17. Marginal Probability Density Function θ_1

6 Conclusions and recommendations for SCDOT

6.1 Performance of current joints

This report presents a study focused on identifying the most durable expansion joints for the South Carolina Department of Transportation. This is performed by proposing a degradation model for the expansion joints and updating it based on bridge inspections. The degradation models are similar to those used in the literature for structural members. The models are updated based on a performance metric proposed in this research.

The proposed degradation model consisted of two parameters (θ_1 and θ_2). After the model was updated it was found that the most likely value of θ_2 is zero, having the degradation of the expansion joints be explained by a linear model. The marginal probability density function of this parameter was calculated and it is presented in Figure 17. A lower value of θ_1 corresponds to a better performance (less degradation). Based on this model the type of expansion joints with the best performance are open expansion joints and pourable joint seal. Assembly joints and compression joint seal have an intermediate performance and strip seal expansion joints have the worst performance of the type of expansion joints studied. However, it is important to indicate that based on the literature review assembly joints are found to be problematic because of the different moving parts composing the joint. It is possible that the low number of assembly expansion joints studied here had an effect on the results shown in Figure 17 and a larger study could find that assembly joints have a lower performance than strip seal expansion joints.

The literature review indicated different typical failure mechanisms on the different type of expansion joints. Overall, the literature indicates that the construction phase of the expansion joint is critical. A joint that has been correctly installed is going to have less maintenance problems. Modular expansion joint are some of the most studied expansion joints. This type of expansion joint has multiple mobile parts, which are prone to fatigue fracture, especially on critical components such as welds and support bars. Dynamic effects can amplify these fatigue problems. The second most common expansion joint studied was the seal expansion joint. The predominant failure mechanism in seal joints was the loss of elasticity of the material over time (loose of elongation capacity). In addition, the seal can be separated from the deck and the anchorage at the deck can produce some maintenance problems. Finger joints could have problems due to debris accumulating inside the joint or the connection with the deck. However, finger joints are easier to maintain than modular joints because it has less movable parts.

6.2 Recommendations for SCDOT

6.2.1 Installation

After reviewing the SCDOT design manual and studying the different failure mechanisms of bridge joints it was concluded that the SCDOT standards are up to date and comparable to other DOT standards in terms of the design aspects of bridge joints. However, a significant number of bridge joint failures are caused because of incorrect installation. In particular, joints with complex anchor systems between the bridge deck and expansion joint. It is recommended that SCDOT continues with the practice of requiring contractors to seek the advice of the manufacturer at the time of installation. As specified in the current specifications, it is preferred that a technical representative of the manufacturer will be present for the joint installation. For projects with a significant number of joints, it is recommended that the technical representative should be present for the installation of a number of the joints to verify that the procedure of installation is correctly followed. In addition, it is recommended that SCDOT recommends a warranty when appropriate for the installation of the expansion joints. It is expected that a large number of problems related to joint maintenance can be addressed with these two measures. Other recommendations related to joint installations are:

- When possible, install joints when the ambient temperature is the average of the range of temperatures in the area. This allows the joint to be installed close to the "undeformed" position of the bridge.
- The support of the joint should be installed in good quality, cured concrete.
- Avoid spliced of any pre-manufactured material. If splices cannot be avoided, place the splice outside the wheel path.

6.2.2 Finger joints and modular joints

The use of finger joints is recommended over the use of modular joints due to maintenance issues. Modular joints have several movable parts that can fail and impede the correct functioning of the joint as reported in the literature by numerous reports and papers. In addition, the effect of the applied loads could be amplified due to the dynamics of the joint. The use of a drainage trough to collect the water runoff is recommended to protect other bridge components under the deck such as bearings.

6.2.3 Plugs

The recommendations performed by Johnson and McAndrew (1993) after studying 125 plug joints are suggested to the SCDOT. The recommendations during design are (Yuen, 2005): “i) Joints for roads with significant cross-sectional or profile gradients should be designed using relatively stiff binders to reduce debonding and binder flow; ii) Joints should be linear with uniform widths of at least 20 in; iii) Localized widening should be avoided especially on heavily trafficked roads; iv) If widening is unavoidable, stiffer binders should be used to minimize deformation and binder flow.” For installation the recommendations include (Yuen, 2005): “i) Before the joint is installed, all loose material should be removed from the deck, and the deck substrate should be thoroughly dry; ii) Bridging plates should be installed across the expansion gap to prevent extrusion of the joint material in the gap under traffic loads; iii) Joints should be continued straight through the curb. The depth of the curb over the joint should be reduced to ensure that the full joint depth can be maintained under the curb; iv) The joint and transition strips should be approximately level with the adjacent deck surfaces to provide good ride quality.” The maintenance recommendation is to replace joint and the surface adjacent to the joint if the adjacent area shows deterioration (Yuen, 2005).

6.2.4 Additional joint systems

Manufacturers are constantly producing and testing new type of materials and systems for bridge joints. The performance of these joints could depend on particular characteristics to South Carolina. For example, while damage due to plowing could be a dominant factor in Northern states, it is not a central issue for the low country of South Carolina. For this reason, it is important to test new bridge joint systems under the specific characteristics of South Carolina. It is recommended that pilot field tests of new joint systems and materials are performed under controlled conditions (i.e. next to other joints with known performance) before state wide installation or its inclusion in the SCDOT bridge design manual. Three joint types that can be considered are:

- Pourable silicone. Silicone sealants are also available in the market and can be used for new construction and rehabilitation projects. Some of the advantages of this type of joints are their self-bonding and self-leveling characteristics. Rapid cure silicone sealants are a viable solution for rehabilitation where significant traffic disruption is not possible. This type of joint has been adopted by other DOTs such as Washington state and are being used to replace other type of joints such as bolt-down panel joints (WSDOT bridge design manual, 2010). Most of the silicone sealant materials are relatively new and have been studied in a laboratory environment. No studies were found reporting the durability of these joints on the field for an extensive period of time.
- Silicone foam sealant. The work by Malla et al (2007) studied the characteristics of a foam silicone sealant in laboratory conditions. This foam was produced by modifying commercially available Wabo silicone. The produced foam had better mechanical characteristics than the traditional Wabo silicone such as lower stiffness, higher extensibility and higher resistance to cyclic loads under cold conditions. In addition, the foam experienced significant increase in volume during the curing process, leading to cost saving for the joint seal material needed.
- Polymer modified asphalt (PMA) joints. This type of joints is usually installed for displacement of up to 1.5 inches and is reported to provide a smooth riding

surface. Even though literature report some problems with this type of joints in Indiana (Chang and Lee, 2002) and Washington state (WSDOT Bridge design manual, 2010), this type of join could potentially work in South Carolina, where weather conditions are different. Some of the problems reported in Indiana include rusted plates which could be caused by the use of deicing salt. Other problems reported include missing PMA material from the joint and cracks in the material close to the shoulder. Washington state reports that the material tends to creep when the joint is subjected to significant acceleration, high traffic count and heavy trucks. However, it has been reported that these issues could be minimized by using good quality material, testing the material before installation, checking that the joint displacement is within acceptable ranges (Chang and Lee, 2002) and installing them in low truck traffic.

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