THE DESIGN AND ANALYSIS OF ARMORED EXPANSION JOINTS FOR HIGHWAY BRIDGES

by

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Karl H. Frank

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Michael E. Kreger

To

My Parents.

Who never sought to stifle my inquisitiveness and curiosity

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Chapter 1 Introduction

To the design engineer, an expansion joint is a discontinuity in the bridge deck which allows for movement of the bridge due to environmental temperature changes. In the past this statement was completely true but advances in design and analysis have resulted in bridges with longer spans and complex geometries which require joints to more than expand and contract. These changes in joint requirements have resulted in the implementation of new joint types. With these new joint types there have been problems which have resulted in joint failures and subsequent replacement before the joint reaches its intended service life. Investigations of these failures have resulted in recommendations for changes in the design, detailing and construction of bridge deck joints. Unfortunately, the results of these recommendations have been slow to propagate through the bridge community. A recent memo from the Texas Department of Transportation (TexDOT) stated that between ten and fifteen percent of armored bridge deck joints installed in Texas had failed [23]. A survey of related joint literature indicates that this number is not out of line with the experiences encountered in other areas. In an effort to reduce the possibility of joint failure this thesis has compiled information for engineers, contractors and inspectors to design, detail and install armored joints to increase the service life of armored joints.

One of the problems of working with bridge deck joints is defining what functions they must provide. A more complete description of those factors which must be considered in the design of bridge deck joints would be:

> Bridge deck joints are designed to accommodate cyclic and long term structural movements, to support and provide smooth quiet passage of traffic, to prevent water runoff from damaging the supporting structural elements and to have a long service life.

This statement was made by John Van Lund who is a bridge design specialist in the area of bridge deck joints with the Washington State Department of Transportation [26]. Based on this definition, the design of bridge deck joints must consider several different objectives. The importance of each objective in the design of the joint relative to the total joint design varies from bridge to bridge. Some bridge factors which influence the importance of bridge joint design objectives are: bridge structure type, bridge geometry, seasonal variations in local weather, distance between joints and intended service life. Based on this design criteria several different joint types have evolved. Although each joint type has different methods of achieving design objectives, joints are generally grouped by the bridge movement they can accommodate into categories of small, medium and large-movement joints.

Small-movement joints are considered to be those joints with a capacity for movement of less than two inches. Typically compression seals are used for these joints. Compression seals are made of a webbed elastomeric material which is compressed as they are placed in the bridge deck joint gap. Once in the gap, these seals try to expand and exert a small pressure on the sides of the joint which holds them in place.

Joints with anticipated movements between two and five inches are considered to be medium-movement joints. These are the most common joints found in current bridge construction. Strip seal joints are widely used in this joint range. The strip seal is an elastomeric gland which is mechanically locked in place between the two sides of the joint by steel retainer members.

Both of these joints, compression and strip seal, can be installed as armored joints. Armored joints use an edge rail at the end of the concrete bridge deck to protect it from traffic loads and prevent spalling of the concrete. For compression seals the armoring consists of either a straight vertical plate at the end of the bridge deck or an angle section at the end of the bridge deck, Figure 1.1(a,b). Proprietary systems such as those in Figure 1.1(c) are typically used for strip seal expansion joints.

Figure 1.1 a) Vertical Plate Armoring b) Angle Section Armoring
c) Proprietary Armored Joint

Large movement joints are typically either finger or modular joints. Finger joints are plates cantilevering from the end of the bridge deck on each side of the joint. Each plate has fingers extending out from the end of the bridge deck which mesh with those on the opposite side of the joint and allow for movement. These joints require separate seal in order to create a watertight joint. The limits of movement for finger joints are approximately eighteen to twenty-four inches.

Modular joints can be used in place of finger joints and for bridges requiring even larger joint openings. First developed in Europe, modular joints are an extension of the compression and strip seal joints. A modular joint can be viewed as several connected strip or compression seal joints. Each modular joint has several separator beams across its width which are connected by seals. For every separator bar there is a support bar which spans the joint gap and supports the separator bar and the traffic load. As bridge movements occur the support bars move, keeping equal distance between the separator bars. This movement of the support bar allows for the medium size openings (1”to 4”) between the separator bars to be additive and produce a joint which can span large openings.

The choice of joint type is important. Once the joint type is chosen other details must be considered, such as how the concrete will be placed around the joint and what affect does the bridge’s overall geometry have on the joint’s ability to be properly installed. These two factors were relevant in the investigation of the failure of several armored joints that served to raise the questions about armored joints that this thesis investigates. In the course of the investigation questions such as “How were the joints installed?" and "What alternatives are there to this joint type and installation procedure?" led to the discussion of problems associated with joints and their installation. Another question raised in the investigation was, "Is the failure of armored joints a common occurrence." The answer to this question turned out to be "Yes" as two other sites of failed armored joints were investigated in the course of this thesis. Discussions of failed joints always raised the question, were the joints properly designed and what is the design load for joints? This led to the investigation of the design load for joints and how this load is to be applied. Any discussion involving the failure of joints inevitably leads to the question of "How can I prevent this (premature joint failure) from happening again?" This is discussed in Chapter 5 which presents recommended procedures to be followed to prevent failures, or detect and repair faulty joints before the bridge is opened to traffic. These recommendations are simple and are presented so that they can be used to extend the service life of armored bridge deck joints.

Chapter 2. Survey of Practice and Problems

2.1 Introduction

Much of the literature that has been written about armored joints develops performance criteria for the use of joints concerning the joint's ability to be watertight and what the design life of a joint should be. In developing these criteria many of these same articles do not fully explain the problems associated with armored joints and how they can be avoided. This section will present a survey of field problems with armored joints. Any discussion of factors leading to problems in the field with armored joints may be independent or dependent of joint type. Therefore problems will be explained in regard to the type or types of joints in which they occur. Typical joint failures occur due to traffic loading, incompatible joint materials, improper installation techniques, bad design or some combination of these.

The need for expansion joints in bridge structures is an environmental one. Daily and seasonal variations in temperature can induce large forces in bridge structures due to restrained movements. Failures of expansion joints due to cyclical temperature changes rarely occur [21]. The mode of joint failure due to this thermal cycling is typically loss of watertightness at the applicable joint seal. Loss of this seal results in the free flow of contaminated water from the bridge deck to the supporting substructure resulting in deterioration. Improper sizing of the joint or not adjusting the joint gap for climatic conditions at time of installation are the causes of such actions.

Improper installation can be attributed to almost every joint failure to some extent. Armored joints are installed in a variety of arrangements. These arrangements can be summarized into two categories; joints installed at the time of deck concrete casting and joints installed after the deck concrete has cured. Joints which are installed after the bridge deck concrete has cured require a blockout that is formed in the bridge deck during concrete casting. An illustrative example of each installation type is shown in Figure 2.1. The decision of which installation method to use is based on the type of joint anchorage used.

Figure 2.1 a) Cast-in-place joint **b)** Joint utilizing a blockout

2.2 Anchorages

For any deck joint to reach its expected design life substantial anchorage of the rail must be provided [6]. Insufficient anchorage of the joint rail leads to rail movements. These movements can create excessive noise levels as traffic traverses the joint. Such noises may be disturbing enough to those close to the bridge that premature replacement of the joint is required. Movements will also require replacement of the joint due to fatigue of the anchorages or failure of the supporting concrete or joint bedding materials. Anchorages utilizing both installation methods must be able to prevent such failures from occurring.

<u>2.2.1 Sinusoidal Anchorages</u>- For joints installed using a blockout, sinusoidal anchorages are typically used [20]. This anchorage shown in Figure 2.2, looks similar to a sawtooth pattern in the plane of the bridge deck. This anchorage provides positive anchorage of the rail section to the bedding material. A minimum diameter of 1/2 inch for the reinforcing steel is recommended [18]. Anchorages provided for this type of joint are typically adequate as the true strength of the joint is controlled by the bond between the bedding material and the concrete deck..

Figure 2.2 Sinusoidal Anchorage

2.2.2 Bolted Anchorage- Other methods of joint anchorage when using a blockout use a bolted connector. There are three typical bolt arrangements used: anchor bolts, cast-in-place bolts, and through bolts (Figure 2.3). Torquing the bolts after installation provides a constant state of compression (prestressing) at the joint concrete interface under traffic loading. This prestressing prevents movement between the rail and concrete deck. Each bolt installation procedure accomplishes this in a different manner.

Anchor bolts are the most common form of bolted anchorage. Anchor bolts are popular because they are easily installed and the holes are drilled after the deck is cast. By drilling the holes after deck casting, the rail can be used as a template resulting in precise alignment of the holes. Bolts are installed in the holes with or without grout depending on the specific brand and model of bolt used. When the deck concrete and the bolt hole grout, if used, has reached sufficient strength the bolts are torqued to specified values. Torquing of the bolts results in the desired uniform compressive force along the rail edge. This method of joint installation appears to solve most anchorage problems and is easily installed. Unfortunately it has not proven to be very effective [25]. Traffic loads on the joint have resulted in anchor bolts working loose in the adjoining concrete or grout hole. This looseness of the bolt causes the joint to work free and results in joint failure. Congestion of the deck reinforcing at the joint location may prevent drilling all of the required anchor bolt holes, resulting in fewer bolts and overstressing of the installed bolts which leads to joint failure. Some short term success has been reported in the use of epoxy instead of grout to fill the holes and secure the bolts. Long term results on the use of epoxies have been inconclusive [25]. Until a more durable method of anchoring the bolts in the holes is devised this method is not recommended.

A more durable method of using bolts as an anchorage is to cast the desired bolts directly into the concrete deck. This method which is shown in Figure 2.3(b),

Figure 2.3 Bolted Anchorages **a)** Anchor bolt **b)** Cast-in-place bolt **c)** Through bolt

significantly decreases the possibility of bolts working loose and requiring replacement. Bolts used in this manner should be of sufficient length to prevent them from pulling out of the concrete deck. The use of headed studs or washer plates on the bolts will reduce the required embedment length and increase the bolt's pullout capacity [15]. Additional reinforcing around the embedded end of the bolt is recommended to reduce the possibility of concrete failure. Careful placement of the bolts prior to deck casting is required so that the bolts will be properly aligned when the rail is installed. Sturdy formwork should be used in these areas to prevent any movement of the bolts during casting procedures. Use of the rail to hold the bolts in place during casting is not recommended. Such use of the rail should be avoided as voids may form on the underside of the rail during casting. Formwork should be removable so that complete inspection of the concrete in the rail area can be made and the presence of voids detected and corrected. These installation procedures require considerable coordination between the engineer, contractor and joint manufacturer. Such coordination is required so that the proper bolt pattern is used, the deck reinforcing allows for this pattern, and the field crews properly lay out the bolts. It is evident that this method requires more time and expense than the anchor bolt installation procedure but this method has proven to be very durable and requires little maintenance [25].

The third method of using bolts to anchor the rail is a variation of either the anchor bolt or cast-in-place methods. This method uses bolts which extend completely through the bridge deck, Figure 2.3(c). Through bolts provide a design which allows for replacement of all the joint parts including the anchorage with a minimum of traffic disruption. The typical installation procedure for through bolts is to use PVC pipe to form the bolt holes. The deck concrete is then cast, resulting in a hole through the deck into which the bolt can be placed. Constructabilty problems with this method are the same as those for the cast-in-place bolt method. An alternative to this forming problem may be drilling holes in the deck for the bolts after the concrete is cast. If drilled holes are to be used, special care should be taken so that the intended location of the holes does not coincide with the reinforcing steel location. Through bolts may provide a small degree of rail adjustment if the outside diameter of the bolt is less than the inside diameter of the hole or PVC pipe. This difference in diameter should be less than 1/8 of an inch so that the bolt cannot move in the hole once it is torqued.

Placement of the bolt head up or down is left to the discretion of the joint manufacturer and engineer. It is recommended that washers be used at both ends of the bolt to ensure uniform bearing of the nut and bolt head against the rail and concrete surfaces. Oversized washers should be used at the bottom of the slab to decrease the bearing stresses and increase the anchorage pullout strength. Reinforcing steel in addition to that provided for deck loads should be provided at the bottom of the slab around the bolt holes to reduce the effect of stress concentrations.

<u>2.2.3 Cast-in-place Anchorages</u>**-** For joints installed at the time of deck concrete casting there two basic types of anchorages, welded shear studs and plate anchorages (Figure 2.4). Studs are typically 1/2 or 5/8 of an inch in diameter and attached alternating along the top and side rail flanges at either 9 or 12 inch spacing. Plates come in various sizes depending on the joint manufacturer and applicable state standard. As a minimum, plates should be 3/8 of an inch thick and placed at one foot intervals with welds to both the top and side rail flanges. Most plate anchorages provide either a loop of reinforcing steel or a cutout in the plate. This loop or cutout provides a location where reinforcing steel can be placed through, once the joint is installed in the uncast bridge deck. This added reinforcing will provide a greater degree of anchorage for the joint rail. Attachment of the studs or plates to the rail is completed by the joint supplier at the fabrication shop.

Depending on the deck reinforcing, joints using stud or plate anchorages may be

Figure 2.4 Cast-in-place anchorages **a)** Shear studs **b)** Tab anchorages

difficult or easy to install in the deck. The difficulty of installation for joints with cast-in-place anchorages is related to the amount of reinforcing steel present in the joint area. For highly reinforced sections, joint installation can be difficult due to interference between the joint anchorages and the deck reinforcing steel. For heavily reinforced bridge decks it is advisable to install the joint before all the reinforcing steel is in place. When placing joints in reinforcing steel, strict adherence to minimum spacing requirements must be followed to allow for the proper placement of concrete during casting.

It is common practice to weld joint anchorages to the deck reinforcing steel for support of the joint prior to casting the deck concrete. This practice is discouraged as it creates an unfavorable condition in both the reinforcing steel and anchorage. Another method of supporting the joint prior to concrete casting uses bolts which protrude from the tops of the girders. These bolts often called leveling bolts allow for adjustment of the joint height during installation and provide the necessary joint support. Leveling bolts are not the perfect solution to the joint installation problem as they carry the load directly from the joint rail to the bridge girder; something which they are not designed to do. In carrying this load they reduce the transfer of load from the top flange of the rail to the deck concrete. This additional load on the leveling bolt may eventually cause movement of the bolt and lead to movement of the joint. Once a joint begins to move there is rapid deterioration in the concrete under the joint which ultimately leads to failure. In order to prevent this, when leveling bolts are being used they should be used sparingly and of the smallest possible size so they promote the transfer of load between the rail and deck concrete.

2.3 Installation Procedures

The installation of armored joints with cast-in-place anchorages (studs and tabs) requires precise positioning of the joint with respect to the profile grade line. Precise alignment of the joint is difficult to achieve at the time of installation due to the absence of suitable joint supports and a local reference point. Typical joint installation procedures involve placing both sides of the joint simultaneously. It is easier to set both sides together as this is the way in which they are shipped. The joint is set in position using a crane. While suspended from the crane the joint is surveyed into its final position. Exact placement of the joint is difficult because the joint is moving while suspended from the crane. To reduce this movement some contractors use bolts placed along the top flange of each girder to support some of the joint weight and reduce swaying of the suspended joint. These bolts also help to set the joint elevation at the bolt location by raising or lowering the bolt. Once the contractor locates the joint in its final position the joint anchorages are tack welded to the deck reinforcing steel to hold the joint in position. Deck reinforcing normally deflects under the weight of the joint. The degree of this movement must be estimated in the field before the joint is welded in place. Cutting of welds and realigning the joint is very time consuming and is not common practice unless the joint is grossly mispositioned. It is expected that should a joint be too high or low, the contractor would finish the deck to the top of the joint. This adjustment is not always completed as shown in Figure 2.5. By finishing the deck to the misaligned joint, the thickness of the deck concrete has been changed. This change in thickness is usually small enough that it does not have an affect structurally. This local change in the deck profile may induce unwanted forces on the joint especially if the two sides of the joint do not follow the same profile grade line.

Another consideration often overlooked when installing a joint with cast-in-place anchorages is the effect of construction loads on the joint rail. Adequate support must be

Figure 2.5 Joint not finished level to deck joint rail

provided for the rail during casting and curing of the concrete. Deflections or movements of the rail prior to complete concrete curing may cause gaps or openings between the rail or anchorage and the plastic deck concrete. These gaps may result in movement of the rail under traffic loading. Such rail movement can lead to fatigue failure of the joint anchorages due to unintended bending moments. It is necessary that all joints be installed to the correct location along the profile grade line and that the sturdiest method be used to hold the joint in place during concrete casting to prevent unintentional movements of the rail.

Joints using bolted anchorages are easily installed once the forming in the deck blockout has been removed. The blockout should be measured to ensure that it is the proper depth. If the blockout is too shallow the excess concrete should be removed. A shallow blockout will result in the top surface of the rail being left in the roadway and subject to unintended impact due to passing traffic. Removal of the excess deck concrete can be accomplished by several methods. The blockout area directly under the rail should be as smooth as possible to provide uniform bearing along the bottom of the rail. High or low spots, voids, and any area where the concrete may have been removed should be properly prepared and leveled with a high strength grout or other approved material. For blockouts which are too deep, the elevation of the joint may be raised using these same leveling materials. It should be noted that there are some benefits to having the joint slightly below the profile grade line which are discussed later in this chapter. Once the bottom surface of the blockout is properly prepared the rail, bolts and any required holes may be installed in the concrete deck. The bolts are then torqued to the specified values. Torquing of the bolts should not commence until the leveling material in the blockout has completely cured. Once these bolts are torqued the remainder of the blockout can be prepared for installation of the filler materials in accordance with the specifications. Blockout filler materials are typically portland cement, and asphaltic or elastomeric concretes. The durability of these materials as well as their compatibility with the concrete deck should be considered when they are specified or approved.

Joints with sinusoidal anchorages require a blockout which provides sufficient depth to allow for adequate flow of the bedding material under the rail and proper clearances around the anchorages. Once the blockout dimensions conform with these clearance requirements, installation of the joint can commence. The initial step in the installation process is to prepare the blockout surfaces in accordance with the project specifications. Proper preparation of the blockout typically involves sand blasting of the concrete surfaces. Sandblasting is used to roughen the concrete surface and increase the bond strength between the bedding material and the deck concrete. A strong bond between the deck concrete and the bedding material is desired because this bond is what holds the joint in place. Once this area has been properly prepared the joint rail and its anchorages can be installed. Installation of the joint rail with sinusoidal anchorages is similar to that of a joint with cast-in-place anchorages. As with cast-in-place joints these joints are shipped and are installed in pairs. The difference between installing the two joint types is for the joint with a sinusoidal anchorage the concrete deck is in place at the time of joint installation. The deck provides a method of supporting the joint during alignment. Support of the joint is usually made by small beams traversing the blockout and the supporting rail and anchorage (Figure 2.6). These beams allow for more precise setting of the joint height and grade. The deck aids in this alignment procedure as it provides ready reference to the final bridge profile. After the rail has been properly aligned the blockout is filled with the specified bedding material.

Bedding materials have included standard concrete, asphaltic concrete, epoxies, quick setting concretes or grouts and elastomeric concrete. Each of these bedding materials has been used with limited

Figure 2.6 Temporary joint rail support success. Asphaltic concrete has been found to be prone to premature spalling. Higher strength materials such as epoxies and quick setting concretes appear to have additional strength which may be beneficial for bridge joints. These high strength materials are brittle and tend to break down in colder climates. Elastomeric concretes have shown great promise due to their flexibility and bond strength to normal concrete. Some concerns over the use of elastomeric concrete have been raised about its susceptibility to creep and how this affects the anchorage of the joint.

Incomplete concrete or bedding consolidations is one of the leading causes of joint failures. Without support along the underside of the rail from the concrete or bedding materials, movement of the joint rail will occur. These movements ultimately lead to complete joint failure by either anchorage or bond failure. For joints with cast-in-place anchors the formation of voids or "rock pockets" (areas of aggregate only and no paste) under the top flange of the rail has lead to premature joint failure. These deficiencies may be formed by concrete or bedding materials that are too stiff or too thin, improperly vibrated, or cast in the incorrect sequence. When using bedding materials especially, elastomeric concrete, the joint should be pressed down into final position so as to allow excess material to flow out from underneath the rail [19].

2.4 Initial Joint Opening

Joint manufacturers provide temporary connectors between each rail for ease of shipping and installation. Connectors are usually a bolt and a spacer between the rails, spaced along the vertical flange of the rail as shown in Figure 2.7. This arrangement allows for the joint to be moved and installed as a unit. This connector should be removed at the time of joint installation but often is not. The reason for not removing the connector is it provides additional support to hold the joint in place prior to concrete casting. For joints with cast-in-place anchorages this connector can create voids around the anchorages. These voids occur because as the fresh deck concrete on one side of the deck begins to cure shrinkage occurs, movement of the joint in the direction of this shrinkage is restrained by the connection between the two sides of the joint and voids are formed. These voids generally do not form on the second side of the joint because once the deck concrete has been placed the connections are cut using an oxy-acetalene torch. In some cases these connectors have been left in place until the time of the joint seal installation. Joint seal installation may be several months after casting and most of the deck shrinkage and several temperature cycles have occurred resulting in significant voids around the joint anchorages. These voids will lead to premature failure of the anchorage and the joint. It is recommended that all connections between each side of the joint be released as soon as the joint is properly aligned to prevent unnecessary distress in the joint due to movements of the bridge.

The opening between each rail of a joint is usually set by the manufacturer prior to shipping in accordance with the project plans. This is done by using the spacer provided at the shipping connector (Figure 2.7). Initial joint opening size at the time of joint installation is specified in the plans for a specific temperature (usually 70^0 F). Corrections for setting the joint opening for other conditions at the time of casting are

Figure 2.7 Shipping connector and spacer between rails also listed in the bridge plans. Field corrections of the initial joint opening based on these listed factors are difficult to perform. In order to make such a change the contractor needs to remove all the installed spacers, fabricate new spacers for the intended joint installation temperature and install the new spacers. Should the contractors construction be delayed and/or the weather change a another new set of spacers would be required. Quite often this additional work is neglected and the joints are installed as shipped. This may result in the improper operation of the joint and lead to loss of the joint seal.

As an alternative bridge designers could account for a temperature range in which the joint can be installed when specifying the initial joint opening. This may require a joint with a larger capacity but with strip seal type joints the movement capacity of a joint between two and five inches is dependent on the seal used and not the hardware, which remains the same. By allowing this leeway in specifying the initial joint opening the contractor has a greater opportunity to meet the specified initial temperature/initial gap opening coordination. Such coordination can be accomplished even when daily temperatures are out of the specified range. For example in the summer when daily temperatures may reach the high nineties the contractor may be willing to set the joint early in the morning when temperatures are in the low eighties and within the allowable range. With a range of setting temperature the contractor has the ability to set the joints while the weather is favorable or wait several days as opposed to several months that may be required to reach the temperature specified in the bridge plans. By providing this type of flexibility in the design phase failures due to construction problems can be prevented.

2.5 Self-cleaning Joints

Manufacturers of bridge deck joints claim to produce joints which are self cleaning. Self cleaning means that the passing of traffic over the joint will remove all debris in the joint including sand. Inspection of in-service joints has found that most joints are only self cleaning in areas along the wheel lines of the traffic. Other areas of the joint across the travel lane were often found to be full of sand and gravel, especially along the shoulder. To the accumulation of debris it is recommended that larger openings be specified so that runoff will be sufficient to wash the accumulated debris out of these areas between routine joint maintenance. The increased opening size will also prevent joint damage that may occur if the joint were to attempt to fully close with debris in place. Closure of a debris-filled joint usually results in the loss of the joint seal as the accumulated debris heaves into the roadway and is forced back down by the passing traffic and pushes the seal out of its retainer. The transfer forces from one side of the joint to the other may occur due to presence of debris in the joint resulting in structural damage of the joint. As a minimum the joint opening should always be greater than 3/4 inch to prevent such damage [19].

2.6 Arrangement of Joints on Bridges with Wearing Surface or Overlays

The use of armored joints with wearing surfaces involves consideration of several factors. Most important is the arrangement of the wearing surface and the joint

(a) **(b)**

Figure 2.8 a) Undesirable and **b)** Desirable wearing surface arrangements

rail. Direct contact of the wearing surface with the joint rail is undesirable as the wearing surface will tend to deform at the rail edge (Figure 2.8(a)). This will result in the joint rail being exposed to increased forces due to the step in the roadway surface. A more suitable arrangement includes the use of a one foot wide concrete surface between the joint rail and wearing surface. This will allow for better distribution of the joint forces into the deck and provide for a smoother transition from the wearing surface to the joint while preventing the deformation of the wearing surface at the joint rail edge [8].

2.7 Snow Plow Damage

Snow plow impact with bridge deck joints is the leading cause of shortened joint life. Bridge deck joints can be detailed to significantly reduce the probability of such damage. Factors affecting the joints susceptibility to snow plow damage include: joint height, angle of skew and joint material. When detailing a joint for snow plow impact the most critical factor is overall joint height with reference to the bridge deck. It has been shown that a significant decrease in joint damage will result by placing the top of the joint rail one-eighth to one-quarter of an inch below the roadway surface. Flush mounting of joints is not effective; the joint must be below the roadway surface [6]. An added benefit of such placement is a reduction in impact forces on the joint. Low joint placement reduces the probability of a joint being hit by a snow plow. If impact should occur substantial anchorages like those described herein are sufficient to survive such impact. For bridge skew angles of 30^o-35^o the probability of damage is significantly increased. This joint angle matches the angle of the plow blade allowing the blade to drop into the joint resulting in serious damage to the joint.

2.8 Aluminum Joints

The use of joints made of aluminum or aluminum components is not recommended. Joints made of aluminum have proven to be very susceptible to snow plow damage. The lower strength of aluminum causes it to be damaged by even the smallest impact or scrape. Based on the performance of aluminum bridge deck joints in New York, it has been recommended that aluminum not be used for any joint type in regions where snow plows are likely to be used [16].

In addition to aluminum’s inability to resist snow plow damage, several tests have shown an incompatibility in the coefficients of thermal expansion of aluminum and concrete leading to early joint failure. One test indicated that after ten freeze thaw cycles an aluminum joint leaked around the joint anchorages. Further testing of the joint subjected it to simulated traffic loads which resulted in spalling of the concrete along seventy-five percent of the joint length [21].

In addition to the lack of thermal compatibility between concrete and aluminum there is the possibility of a chemical reaction between fresh concrete and the aluminum. This reaction reduces the strength of the concrete around the joint anchorages [15].

Use of aluminum in bridge deck joints is not recommended due to aluminum's incompatible physical properties (chemical reactivity and coefficient of thermal expansion) and low strength.

Chapter 3. Analysis of Failures

3.1 Introduction

The impetus for this thesis was the investigation of the failure of several strip seal expansion joints in Texas. In this chapter the results of these investigations, observations from a another field examination of failed strip seal expansion joints and general observations on the causes of joint failures will be discussed.

3.2 US 83 and US 281 at McAllen, Texas

The first set of failed joints inspected were at the US 83 and US 281 interchange at McAllen, Texas. The interchange is composed of 4 unidirectional ramps which had been opened to traffic for less than six months at the time of inspection. Each of the ramps uses a mix of simple span AASHTO prestressed concrete girders and continuous steel girders. The supporting substructure includes both concrete and steel bent cap systems on concrete piers. The concrete systems are inverted tee bents, hammerhead or tee piers and concrete beam straddle bents. Decking for all of these structures is 8" reinforced concrete with a design strength of 3600 psi. All ramps have vertical grade and most have horizontally curved plans with super elevation.

Expansion joints for the interchange are elastomeric strip seals. The rails are held in place with 6" by 1/2" diameter studs cast in the concrete. Studs are alternately attached to the top and side of the rail at 12" on center. Spacing of the joints was typically every three spans

Figure 3.1 Arrangement of expansion joints at steel bent caps

in the concrete section (≈300' between joints) and at the ends of each continuous steel girder unit. The arrangement of expansion joints varied depending on the substructure type. For inverted tee bents only one expansion joint unit was used. At steel box girder bents two units were used. This arrangement is shown in Figure 3.1. The supplier of the joints on this interchange was Watson-Bowman.

At the time of the inspection the contractor was proceeding with the removal and reinstallation of the failed joints. Several failed joints were inspected. The inspected joints had either been removed from the bridge slab or were viewed in place, either with the concrete in place or removed.

The first failed joint inspected was completely removed from the slab at the time of inspection. Inspection of the rail revealed fractures of the studs along the rail. Stud failures were only present on the one side of the joint unit. The side of the joint with the stud failures was located on the downstream side of the traffic flow and was on the uphill end of the relevant slab. Stud failures had only occurred in the horizontal studs. Examination of the failed studs revealed that all the stud failures had occurred in the weld region where the stud was welded to the rail (Figure 3.2). The remaining studs along the rail were bent over using a

Figure 3.2 Fracture area along rail

hammer to test soundness of their welds (Figure 3.3). Bending over of the remaining studs along the rail resulted in no additional stud failures. Failed stud locations along the rail exhibited peening at the fracture faces indicating movement between the rail section and the stud after fracture. Inspection of the rail with stud failures showed visual evidence of a lack of concrete consolidation in areas with stud failures.

Figure 3.3 Left side of rail shows several bent over studs without failure

Lack of consolidation was determined by the brownish color of the rail on the surface adjacent to the slab when compared to whitish color of the rail (Figure 3.4) on the opposite side of the joint and on the downstream rail in areas without stud failures. Both sides of the joint showed no evidence of anchor stud failures where the whitish color existed. Weep holes or vent holes drilled in the top surface of the rail to vent any trapped air or water under the rail at the time of concrete placement, required by the recently instituted state standard, were not present. Several sections of the rail at this pier were removed for further analysis. Failed studs were also removed from the joint rubble for further study.

The second joint inspected was also removed from the deck. This rail exhibited the same patterns of incomplete concrete consolidation and associated stud failures as the

Figure 3.4 Lack of complete concrete consolidation is seen on the right rail when compared to the left rail. Also note missing studs on right.

first joint inspected. Stud failures and areas of incomplete concrete consolidation were located along the downstream side of the joint. Welds along these rails were bent over to check their soundness and no fractures occurred. Pounding or scarring of the horizontal flange of the rail was present, indicating excessive movement of the rail after stud fracture. Peening at the failed stud locations was also evident. No failed studs were found along the upstream rail.

Inspection of the third expansion joint was conducted while the contractor was removing the concrete around the joint. Stud failures were located along the downstream side of the joint which was the uphill side of the slab. Complete inspection of the rail was not possible as the contractor had not completed removal of the deck concrete completely along the rail length. From the areas that were inspected, the stud failures occurred in areas with a lack of concrete consolidation.

Since the inspection of the third joint occurred while the contractor was in the process of removing the concrete it was possible to view the joint with the reinforcing steel in

Figure 3.5 Close placement of the reinforcing steel to the rail

place. The presence of the reinforcing steel at this joint illustrated correct placement of the reinforcing steel was necessary for proper behaviour of the joint to occur. Figure 3.5, shows placement of reinforcing steel too close to the rail. Steel placement in this manner prevents the flow of medium aggregate (d>3/4") under the rail. Complete placement of the concrete cannot occur when this area is blocked and detrimental voids will form.

The fourth joint inspected was located at a box steel bent with two expansion joints. The downstream joint of the two present at this bent was examined in place. The

Figure 3.6 Depth of hole along top rail of joint

examination showed small cracks in the concrete deck along the lip of the downstream side rail. Using a screwdriver and a pry bar to lift the rail, small pieces of loose concrete were removed. Surface concrete near the rail edge was not very strong and could be described as "Punky". After removal of this concrete a tape measure was inserted into the resulting hole

Figure 3.8 Void is to a depth of 7/16”

Figure 3.7 Voids along the bottom of the rail

as shown in Figure 3.6 to a depth of 1 5/8". The rail was easily moved using a pry bar along the downstream rail, indicating that it was very loose.

Also while at this joint, the gland between the two rails was removed. By removing the gland, voids along the bottom of the rail were revealed. Figures 3.7 and 3.8, show one of these voids with a depth of 7/16". It was also noted that the finish of the concrete along this face of the slab did not appear to be flush with the edge of the rail. The surface appeared to have numerous voids along the edge the form (Figure 3.9). The inconsistent finish of the slab along this edge appears to be an indicator of consolidation problems at the joint.

The opportunity to inspect two severely distressed joints was also provided. Unfortunately the roadway was open to traffic and the inspection could only be completed from the shoulder. The first of these joints inspected showed small movement and produced an audible clanking noise under traffic. No cracking of the concrete at this joint was apparent when viewed from the shoulder. The second distressed joint showed severe signs of distress with large visible movements under traffic. The joint also had three cracks in the concrete running transversely across the roadway on the downstream side of the joint (Figure 3.10). Each of these cracks was 4' to 6' long. A section of the roadway under the right-hand wheel line at the first severely distressed joint inspected was marked for removal. The purpose of removing this section intact was to provide a specimen which could be dissected and the placement of concrete under the rail section observed. This marked section when removed from the roadway would become the "Roadway Section" discussed later in this chapter.

Figure 3.9 Voids along the end of slab surface

Figure 3.10 Cracking of the concrete along the rail

After the inspections were complete, discussions were made with the construction inspectors who were on the bridge at the time the deck concrete was cast. They confirmed that all the spans had been cast downhill, that is from the highest to the lowest point. Casting in this manner probably aggravated the incomplete consolidation under the rail with the tendency of the plastic concrete to flow away from the rail.

3.3 Route 79 and MacMill Road Bridge Rails

A field trip to the Georgetown field office of TexDOT was made to examine another set of rails which had failed in service. The office had several rails from two bridges in their district which had failed shortly after placement (6 months) and were replaced. These bridges were the Route 79 and MacMill Road Bridges on I-35 near Round Rock, Texas. Unfortunately the rails had been removed almost two years before the time of inspection and oxidation of the surfaces had occurred.

Even with the oxidation of the rail surfaces several observations were made. Joints

Figure 3.11 Rail cross section

in the Round Rock bridges were used on bridges with asphaltic overlays. The rails used were made of studs attached to an angle section with a grooved channel section for the seal welded to the angle section as shown in Figure 3.11. Even with oxidation due to the extended exposure, the rails showed the patches of white along them as seen on the McAllen rails. These rails also displayed extensive scarring due to movement of the joint. Over 100' of rail was inspected with only seven studs found. Of these seven studs, five were located completely intact on one rail. These 5 studs on one rail were bent over without failure. The other two studs were only portions of studs and failed at the weld when bent over. Inspection of the failure surface of the two welds that failed the bend over test indicated that fatigue cracks had formed in the studs. Weep holes were located along the vertical face of the angle making up the joint.

While inspection of these joints revealed fractures of almost all studs and the presence of white areas along the joint, the fact that an overlay was present must be considered. Compaction of the overlay at the top edge of the rail results in an increase in the horizontal load on the rail. In addition the overlay would mask movement of the joint until complete failure of the joint occurred. This failure is probably due to the combination of increased horizontal force due to compaction of the overlay, fatigue failure of studs in unconsolidated areas and subsequent overloading of the remaining studs. Several sections of rail were removed for further investigation.

3.4 Rail and Stud Sample Testing

Test were performed on the recovered sections of rail at Bent 25 Ramp 4 in McAllen and the sections of rail recovered from the Georgetown district to determine the properties of joint materials and evaluate the fabrication techniques used. To further widen the study, a section of rail from another research project was also evaluated. This rail section had been subjected to a static load test in the laboratory and performed acceptably [14].

3.4.1 Chemical Analysis - The first test conducted was a spectrographic chemical analysis of the rail and stud metals from each of the rails. The purpose of this test was to determine if the chemical composition of the component materials predisposed the rails to either fatigue or weldability problems. A complete listing of the results is located in the report to TexDOT [9]. The rail steel met the requirements of ASTM A36 with copper specified for enhanced corrosion protection. It should be noted that the rail steel and studs are subject to the general ASTM requirements but are not subject to the same requirements as other bridge steels such as toughness.

3.4.2 Weld Structure - Sections were then cut through four of the rail welds for etching. Each specimen was cut, ground, polished and then etched with a Ferric Chloride solution. This solution made the macrostructure of the weld clearly discernible. Etching showed the welds to be of consistent quality and to be free of any harmful flaws. The welds indicated that stud guns used for placing these studs were properly calibrated and operated consistently [9].

3.4.3 Hardness Testing - After etching, each of the sections was subjected to hardness testing. Hardness testing was used to see if the welding process caused a significant change in the rail or stud strength. Tests were performed using the Rockwell C scale. These results show some hardening of the base metal in the heat affected zone but not enough to cause a brittle weld. The ductility of the welds was confirmed by the bend over tests performed in the field [9].

3.5 Roadway Section

During the observation of the joint removal and reinstallation process in McAllen a section of the roadway was marked for careful removal. Careful removal meant that the rail and surrounding concrete were to be removed intact. The section was saw cut from the end of the slab. The removed block was then shipped to the Ferguson Structural Engineering Lab at the University of Texas, Austin. The purpose of removing the section intact was to provide an opportunity to inspect the state of concrete under the rail. From the observations made at McAllen there appeared to be a direct correlation between the lack of consolidated concrete under the rail and stud failure.

The specimen that arrived at the Ferguson Lab is shown in Figures 3.12 and 3.13 (Front and End view). Inspection of the block immediately showed a lack of complete concrete consolidation. This lack of concrete consolidation was illustrated by the presence of voids at both ends of the specimen as well as voids along the lower lip of the rail. Figures 3.14 and 3.15 identify these voids and quantify their depth. Another feature of the block, which was symptomatic of the failed joints in McAllen, was the close proximity of the reinforcing steel to the rail. Presence of these bars close to the rail causes the rail and reinforcing bar to act as a filter. Filtering action by the rail and reinforcing bar catches large aggregate and prevents it from flowing underneath the rail. The "caught" aggregate increases the filtering and impedes the flow of smaller aggregate and cement paste under the rail which ultimately leads to voids. Once these preliminary observations were complete a portion of the rail was removed from the section.

Figure 3.12 Front View of Roadway Section

Complete removal of the rail was not necessary to check for proper concrete consolidation. Instead the corner section of the rail was removed. The corner section of the rail was of prime interest. Field observations had shown a correlation between lack of concrete in this area and stud failure. In order to remove the corner section of the rail two cuts were made in the rail section. The locations of these cuts are shown in Figure 3.16. By choosing these two lines for cutting, the most rail material could be removed without cutting at a stud weld. To perform these cuts the block was moved to a horizontal milling machine and anchored in place. Upon completion of the second cut it was realized that a stud connected to the top lip of the rail had failed.

Figure 3.13 End View of Roadway Section

Figure 3.14 Approximately 2” void along end and note reinforcing bar location (round dot)

Figure 3.15 Void at end also note vertical reinforcing bar

Figure 3.16 Location of cuts to be made in rail

With removal of the rail complete, it was readily apparent that complete consolidation of the concrete along the corner of the rail had not occurred. Figures 3.17 and

Figure 3.17 Edge view of roadway section after rail is removed

3.18 show views of the section looking down the corner of the block section. It is apparent in the figures (3.17 & 3.18) that concrete did flow into the corner of the rail as indicated by the highest or hump areas. The now exposed concrete section also exhibited a pasty or crusty white coating on the exposed surface. This pasty coating indicated the presence of bleed water in this area during curing. By allowing bleed water to collect the strength of the concrete was further reduced in this critical area. The section also showed the presence of voids along the underside of the rail.

Inspection of the removed rail exhibited the same pattern of brown coloration seen in the field observations . The coloration along the rail was compared to the humps and voids along the block edge and a direct correlation was found. This correlation is that brown patches along the rail indicate a lack of concrete in these areas. This is in agreement with the observations of brown areas and stud failures made during the trip to McAllen.

Before excavation of the stud, the excess rail on both sides of the stud was removed

Figure 3.18 Edge view of roadway section after rail is removed

Figure 3.19 Small void at base of stud

from the block. It was also noted that the area near the base of the stud had a void. This void is shown in Figure 3.19. Excavation revealed that there was a small open area around the base of the stud. After completely exposing the stud it was removed from the block. The removed stud was then placed in a vise and the stud was bent over using a hammer. This bending over of the stud was done to test the soundness of the stud weld. The stud failed in this test and revealed an interesting fracture surface. By examining the fracture surface (Figures 3.20, 3.21 and, 3.22) it is apparent that the smooth semicircular region is a fatigue crack propagating through the stud. This type of crack would ultimately lead to failure of the stud. Figure 3.23 shows the orientation of the stud, fatigue crack, rail and roadway.

Figure 3.20 Failure surface of Excavated Stud

Figure 3.22 Failure surface of excavated stud

Figure 3.21 Failure surface of excavated stud

Figure 3.23 Orientation of crack in stud, rail, and loading

3.6 Trinity River Bridge Dallas, Texas

A trip was made to Dallas, Texas to inspect two failed strip seal expansion joints.

Figure 3.24 Construction sequence a) Existing, b) Stage I, c) Stage II, d) Stage III

The joints were on Interstate 30 across Lake Ray Hubbard. At the time of inspection the bridge was still under construction but traffic had been on the joints for less than one year. The bridge is being built in three stages with traffic being shifted off of two existing parallel structures (Figure 3.24). The superstructure consisted of prestressed concrete AASHTO Girders with an eight inch concrete deck. There is no riding surface or overlay. The joints are strip seal expansion joints with typical one-half inch studs spaced at twelve inches. Weep or vent holes were not present in the top flange of the rail. Joints on this project were supplied by D.S. Brown.

The first joint inspected was located at the west approach slab on the downstream side of the traffic flow. At the time of inspection the joint had been in service for less than four months and had failed completely due to loss of the anchoring studs. Failure of all the studs along a twelve foot portion of the joint rail had occurred. To prevent damage to passing traffic the loose rail was pulled into the gap and held in place by a wire rope. In order to inspect the joint one lane of traffic had been closed.

The inspection from the deck showed a lack of complete concrete consolidation

Figure 3.25 Failed rail

under the rail. Lack of concrete consolidation was evident due to the absence of a continuous ridge along the slab edge where the rail had been removed (Figure 3.25). Large voids were also noted at other areas along the slab edge particularly at the location of the leveling bolts. Failure of both horizontal and vertical studs could be observed. An inspection was then made from underneath the deck.

The inspection from underneath the bridge deck showed that the abutment backwall concrete had been cast before the approach slab concrete. From underneath the slabs, it was evident that incomplete consolidation of the concrete had occurred. The lack of consolidation was evident due to the lack of cement paste with the concrete aggregate (Figure 3.26). A complete inspection for incomplete consolidation along the bridge rail was made from underneath the bridge deck. This inspection showed no other signs of incomplete consolidation in areas other than those with failed studs.

The second joint inspected was at the east approach slab of the bridge. This joint

Figure 3.26 Incompletely consolidated concrete under rail

had not failed to the same degree as the first joint inspected but did show signs of impending failure. Indications of impending failure of this joint were; visible movement of the joint due to passing traffic and a loud banging sound associated with this movement. One lane of traffic had been closed on this joint which allowed full inspection of the joint from the deck. Inspection from the deck showed signs of "Punky" concrete at the rail edge. The concrete had broken out and left a void along the rail edge

(Figure 3.27) in the area of movement. An inspection of the joint and associated concrete was made from underneath the bridge deck.

The inspection of the rail from the underside showed small areas of incomplete consolidation which were similar to those found during the inspection of the westbound approach slab. Unlike the westbound approach slab where the incomplete consolidation extended over a twelve foot section, the eastbound approach slab had localized areas of incomplete concrete consolidation. These areas of incomplete concrete consolidation coincided with the locations of impending joint failure.

Figure 3.27 Void at rail edge due to “Punky” concrete

Both inspected joints on the Trinity River Bridge exhibited incomplete concrete consolidation of the deck concrete in the area of the joint rail. Complete inspection of the joint rail along the west approach slab indicated that joint failure occurred only in those areas where the lack of concrete consolidation was visibly apparent. Further examination of the backwall showed areas where cement paste had flowed down the backwall at the time of approach slab concrete placement. From this concrete paste flow it is felt that the contractor attached formwork to the existing backwall at the time of approach slab concrete placement. In the areas where the aggregate was observed, without concrete paste, the form deflected away from the back wall and left a small gap. This small gap allowed for the free flow of cement paste down the backwall and away from the aggregate. Free flow of cement paste resulted in the incompletely consolidated concrete even though proper concrete vibration methods may have been followed. Proper vibration of the concrete may have precipitated the flow of the cement paste away from the aggregate and aggravated this condition. As was explained in the sections on the McAllen and Georgetown joint failures; the lack of complete concrete consolidation is often indicated by the condition of the concrete at the end of the slab when the formwork is removed. This area at the end of the slab should be uniform in appearance and free of voids and rock pockets or joint failure may occur [9]. Inspection of the joints on the Trinity River Bridge followed this same trend. Inspected joints with properly consolidated concrete exhibited no signs of failure, which contrast the joints discussed earlier which showed failure of the joints occurring in those areas with poor quality concrete.

3.7 General Observations

Based on the information presented in Chapter 2 and in this chapter it is apparent that the two leading causes of joint failure are snow plow damage and incomplete consolidation of the deck concrete in the rail area. Failure of joints by both of these modes can be prevented by proper installation at the time of construction. Reports on bridge deck joints written by the states of Colorado, Indiana, Kansas, and Michigan indicated that proper installation techniques were vital to the success of a joint [6]. Swanson of the Colorado Department of Transportation specifically says: "The most important factor which will ultimately determine the success of an expansion joint is proper installation during construction." [22].

The observations in both field investigations showed that visual inspection of the bridge deck end after concrete placement is an excellent indicator of the probable success or failure of a joint. Inspection of the joint after the concrete has cured and the forms have been removed would provide an opportunity to check the level of the joint in regards to the roadway to ensure that the joint is placed following the recommendations in Chapter 2 preventing snow plow damage. In addition, if post-casting inspection of the end of the bridge deck revealed a surface that was non-uniform or voided the quality of the concrete under the joint rail should be questioned. Non-uniform and voided concrete was identified as an indicator of poor quality concrete leading to failure in both the McAllen and Dallas investigations. If this simple procedure is followed the probability of a joint reaching its design life can be immediately assessed and should any repair action be necessary it can be made before the bridge is opened to traffic.

Chapter 4. Loading on Joints

4.1 Introduction

One of the greatest areas of uncertainty in the design of deck joints is what the design load is and how this load is applied. Current AASHTO Specifications outline no specific procedure for the loading of bridge joints. Due to the lack of guidance in the AASHTO Specifications for the design of deck joints, several states have adopted their own deck joint design procedures or left the design of the joints to the joint manufacturers. In order to clarify this area, a survey of several published loading procedures and related experiments are presented in this chapter. A summary of the salient points in each of these references is given below.

4.2 FHWA Technical Advisory [13]

The Federal Highway Administration has issued a design directive on expansion devices for bridges. In this directive, 300 pounds per foot is stated as the maximum capacity for the design of deck joint anchorage devices. In specifying the joint anchorage capacity no information is provided to determine what or how loads are to be applied to the joint.

4.3 Anchorage of Edge Sections for Expansion Joints [18]

It is of interest however to compare the FHWA recommendation to a similar value presented by Koster in which he recommends that anchorages for joints be capable of carrying 12 tons per foot of run, eighty times that presented by the FHWA. Koster states that to achieve the level of anchorage force he recommended, prestressed bolts would be needed. The article continues that this form of anchorage has a high installation cost but the behaviour of such joints has been “very satisfactory.”

4.4 Road Research Laboratory Test [5]

An experiment was carried out at the Road Research Laboratory (RRL) in England to determine the effect of expansion joint gap width and expansion joint height above the roadway grade on horizontal joint forces. These tests were performed to check the validity of a Transportation Ministry Directive which recommended a horizontal design force of 5.5 kips per foot (80 kN/m) for deck joints. To accomplish this test a special joint was installed in the roadway at the Road Research Laboratory test track. The test joint was designed and built so the joint could be raised by placing shim plates underneath the joint. By adding shim plates the effect of joint height on horizontal joint forces could be measured. The joint was installed so that the gap width could be varied from 1/4 of an inch to 3-1/4 inch. Packing was placed between the vertical face of blockout and the joint to vary gap width. Testing was conducted using four vehicles: a car (1.0 K wheel load), a Land Rover (1.5 K wheel load), a two axle truck (5.4 K wheel load), and a three axle truck (5.05 K wheel load),.

The first test carried out measured the effect of gap width on horizontal forces applied to the joint. Two different gap widths were investigated 1/4 inch and 3-1/4 inch. The results of this test showed that at lower speeds (5-10 MPH) the horizontal force was lower for the 3-1/4 inch gap than for the 1/4 inch gap. This trend was reversed at higher speeds (30-40 MPH) where higher forces occurred for the larger 3-1/4 inch gap than the 1/4 inch gap. Overall the researchers felt that the difference in horizontal joint forces between the two cases at each speed were not significant enough to merit further study and the remaining tests were conducted using a 1/4 inch gap.

A second test series was carried out varying the height of the joint above the roadway. These tests were performed using joint heights from zero to 7/16 inch above the roadway grade. Testing showed that there was no significant increase in horizontal forces on the joint when it was raised above the roadway.

Both of the tests described here tested each vehicle under rolling and braking conditions. It was found that the horizontal load under braking was higher than that due to rolling. Horizontal joint forces were also found to be directly related to the vehicle braking efficiency. Horizontal joint forces were found to be equal to the product of the vehicle braking efficiency and the vertical wheel load. This was also compared to the effect of dynamic magnification of the wheel load. Based on the test measurements it was recommended that static loads should be multiplied by 1.67 to account for the dynamic effects of the vertical wheel load.

From this test information the authors felt that the recommended design value of 5.5 k/ft was a realistic value when it is considered that certain cases may exceed this value on a per foot basis but that the load would be shared over the length of the rail and therefore the design value would not be exceeded. It was also noted that the recommended force is for the design of joints only and is not intended to be applied to the bridge deck or substructure.

4.5 Theoretical and Experimental Study of Dynamic Highway Loading [1]

Several tests were conducted at the University of Texas in the late 1960’s to develop a computer model of actual roadway loading due to interaction between vehicle suspension and the roadway profile. As part of this test a series of sensors were placed along a stretch of interstate highway to monitor the weight of trucks as they passed over them. Readings were recorded and compared to values found using the computer model which accounted for this interaction. In one series of tests a 3/8 inch piece of plywood was placed over a weight sensor and traversed by known truck traffic. The results of this test showed a 75 to 100 percent increase in the sensors readings due to the presence of the plywood. This data was then verified using the computer model showing that large increase in the vertical loading of roads or bridges can occur due to small, but abrupt, changes in the roadway profile.

4.6 Behaviour of Expansion Joints [7]

In this test, the researchers wanted to determine the effect of joint height in the roadway and vehicle speed on the vertical joint forces. To do this a dynamic weighing scale was placed in a specially fitted pit and several vehicles driven over the scale at various speeds with the scale at different heights above the roadway. These tests took place at an airport aircraft parking area where the roadway was extremely level, reducing the effect of roadway roughness on the results. A total of 420 tests were performed using three different types of vehicles, joint heights between -10 and 10 mm and vehicle speeds of 0 (static) to 60 KPH.

The results of these tests showed little increase (<5%) in the load applied to the scale when it was level with the road, regardless of speed. When the joint was lowered below the roadway surface there was a decrease in the measured vertical forces with an increase in speed. An opposite trend was found when the joint was raised above the roadway. In elevated positions the joint forces increased with speed and joint height. A graph of one set of data is shown in Figure 4.1.

Figure 4.1 Data from Lorry Test [7]

From this data the researchers recommended that the design load for joints be based on the heaviest allowable axle load, modified by the applicable service load factor and increased by a factor of two. The factor of two was intended to consider the case where the joint is installed 10mm (3/8”) above the roadway surface. Installation in this manner was considered to be within reason. For the case of a 10 mm step the maximum increase in recorded load was 1.69. Several tests resulted in measured values of 1.50. Using this information the factor of two is reasonable if such an installation is considered to be likely.

4.7 The Principle of Elasticity for Expansion Joints [17]

This paper by Koster presents a procedure for developing the design loads on modular expansion joints. This paper treats the loading of an expansion joint as a problem in structural dynamics and not statics. The moving wheel is modeled by the sudden application of a load as a function of time. The analysis considered several factors.

The first factor is the rate of the loading on the joint. Loading rate is determined by dividing the length of the vehicles contact area with the road, by the speed of the vehicle. This loading is then considered to be one-half cycle of a sinusoid. The natural period of the joint is compared to impact time. The closer the impact time is to the natural period the greater the force on the joint will be due to resonance. The maximum increase in the load, 1.7, occurs when this condition exists.

Another factor which may add to the resonant condition is the loading of the joint by multi-axle vehicles or truck trains. Such vehicles create a resonant condition if additional loading of the joint occurs during a peak in the decay of the original loading. To account for this, several different axle arrangements were considered and the conditions of resonance which they may create calculated. From this analysis it was found that, for several axle arrangements at different impact and joint vibration times, a resonant condition may occur. Resonant conditions due to multiple loadings are plotted as circular points on the graph in Figure 4.2. Several of these points are above the normal loading curve. To account for this loading, the bold line is introduced as an envelope of probable loading. This graph can then be used to find the ratio of the static to dynamic force on the joint based on the velocity of the traffic and the vibration time of the joint.

Road surface conditions can increase the dynamic portion of wheel loads. For heavily loaded vehicles increases in the dynamic load can be between 1.15 and 1.35 and as high as 1.70 times the axle load. A value of 1.25 which is an average of the two more common values is recommended for joint design.

The width of the gap between the two rails affects the load applied to the joint. The larger the gap between the two rails the higher the rail load will be. A narrow gap allows load sharing between the rails. To determine the amount of load on a particular rail, the top flange width of one rail should be added to the maximum gap width and the

Figure 4.2 Impact Isolation and Dimensioning Limit [17]

sum divided by the length of the contact area. This value is to be set equal to unity if the sum of the top flange length and the gap width are greater than the length of the contact area. Koster indicates that this value should be evaluated for different times of the year as the gap width will vary and more importantly joint vibration time will vary with temperature. This is not the case with armored joints as elastomeric materials are not used in their construction.

Overall this article provides in a rational way a method for analyzing expansion joints. The only drawback is that a means of calculating the joint vibration time is not presented. Instead the article makes assumptions of joint vibration times with no background on how such assumptions were made. Also the article does not explicitly consider a horizontal joint load but indicates that the procedure can be applied to such problems. A design method following the theory presented in this article and expanding the analysis is covered in the next section.

4.8 Hawkins and Babaei Report [15]

This report recommends a design procedure for expansion joints and their anchorages. The procedure follows the work of Koster but applies it directly to armored joints. In addition to calculating design loads for joints the Hawkins procedure presents a method for the design of joint anchorages which considers fatigue. One specific difference between the Hawkins and Koster procedures is that Hawkins takes the view that the critical load on the anchorage is the rebound of the joint after the load is removed and not the direct application of the load. The report specifies:

> When a vertical impact (caused by vehicle weight) acts on the edge beam, the load transmitted to the edge beam is mainly taken by the concrete directly under it. Only an insignificant portion of the impact load is carried by the anchor device.

As this load moves off of the rail, the rail will rebound and this rebound force must be carried by the anchors either in tension, shear or a combination of the two. The magnitude of the rebound force is a function of the joint system's structural damping. With this hypothesis, the method presented develops a procedure for the design of joint anchorages.

The first step in this procedure is to determine the vertical component of the design load. Four factors are recommended which modify the design truck. The first of these factors K_I amplifies the vertical load for the effects of road roughness, expansion joint roughness and the suspension system of the design vehicle. A value of 1.5 is recommend for K_I which is higher than the 1.25 recommended by Koster. The second factor K_L accounts for the degree of loading on the rail when considering gap width. This factor is of the same form presented by most authors, where the width of the gap is added to the width of the top flange of the joint rail. This sum is then divided by the length of the tire contact area and is always less than or equal to unity. In addition K_L shall be taken as unity if the joint protrudes vertically into the roadway.

One of the shortcomings of Koster's method was that it did not provide a value for the natural period of armored joints. This procedure recommends the use of 100 Hz as the natural frequency for armored joints. This value can be used in conjunction with the graph presented in Figure 4.2 to find factor K_D. K_D represents an increase or decrease in the load transmitted to the edge beam by the traffic. In lieu of using the graph in Figure 4.2 a value of 1.35 is recommended.

The last factor used in this procedure is K_R. K_R modifies the loading for the maximum rebound force which occurs after the load moves off of the joint. Determination of the rebound force is based on the amount of structural damping present in the system. Two formulas are used to define this factor:

$$K_R = \frac{(1+D)}{2} \qquad\qquad \ln(D) = \frac{-2\pi n}{\sqrt{1-n^2}}$$

In these equations *n* is the structural damping ratio of the joint system. Two percent is a commonly used value for damping in this type of structure. This results in a value of K_R equal to 0.94 which means that the maximum upward load on the joint is 94 percent of the total load applied to the joint.

Determination of the value of the horizontal edge load follows the same procedure described for vertical loads. One difference between the two procedures is the rebound factor, K_R is replaced by K_H. K_H is the ratio of horizontal to vertical impact load, and values between 1/3 and 1/2 are recommended. This procedure states that the vertical and horizontal loads occur at different times and the joint be analyzed as such. The procedure mentions that greater spacing between anchor devices would provide more flexibility of the joint and this would reduce the intensity of the loads transmitted to the anchorages using the principles described by Koster. Unfortunately, guidelines are not presented on how to calculate such a reduction.

4.9 Austria Tests [24]

This report outlined the testing and design of modular expansion joints. Again the topics of this report are not directly related to this thesis, but background that is provided by this report is applicable to the design of armored joints.

In-service testing of several modular joints was conducted by Professor Tschemmernegg in Europe. The results of these tests showed that for a given wheel load the horizontal force on a joint due to braking is one-half the vertical load. Laboratory tests were then performed to duplicate these results, without success. Further analysis of the field tests showed that the stress in the joint due to horizontal forces was the sum of two actions. The first component was the horizontal braking force, the second component was caused by the eccentricity of the vertical force on the joint when the load was initially applied. The effect of each component was about equal. From these tests a vertical wheel load of 14.6 kips with a concurrently applied horizontal force of 2.9 kips is recommended. Each of these loads should be considered to act in both the positive and negative direction for design. Impact factors, for both forces, of 40 and 30 percent are presented for the positive (down) and negative (up) directions.

Another area in which test data was collected was the effects of a joint being rigid or semi-rigid on the applied joint force. Measurements were made on a joint that was semi-rigid both vertically and horizontally. The horizontal direction was then blocked resulting in rigid behaviour. Measurements of the horizontally rigid joint were made. The results were the same as those for the semi-rigid joint. The lack of change in the measured forces indicated that there was no reduction of the forces on the joint if it is semi-rigid or rigid. This test data conflicts with the ideas developed by Koster and presented earlier in Section 4.7 which indicated that analytically a semi-rigid system will reduce the loads applied to the joint. As an explanation Tschemmernegg indicates the wheel load is in contact with joint for such a short period of time that it is actually an impact force. He continues that the system is not free to move and therefore must absorb the impact load and this is why there is no reduction in force for the semi-rigid joint.

Some other factors that may contribute to this behaviour are: the tested joint and vehicle speed may not have been in the resonance area described by Koster or that after being in service the joint may not behave as Koster assumed. In either case this data reaffirms both authors contention that expansion joints are highly complex systems and cannot be analyzed with simple models.

4.10 ATLSS Reports [10,11,12]

Full scale testing of modular joints was conducted at the Advanced Technology for Large Structural Systems Laboratory (ATLSS) at Lehigh University. From these tests several reports were written detailing the results of the tests and a review of the current provisions for the design and/or testing of expansion joints presented in the current AASHTO Specifications, the new LRFD format for these specifications and the recent State of Washington Department of Transportation provisions for the fatigue testing of modular expansion joints. While testing performed at ATLSS was concerned with the fatigue life of modular joints, static tests were included. Static tests were completed to gain a greater understanding of the behaviour of expansion joints and how each member contributes to the load carrying capacity of the joint. One static test provided extensive information on the relationship between gap width, rail width and applied load.

The current AASHTO specification provides no general or fatigue design provisions for the design of expansion joints. Based on a general interpretation of this specification joints should be designed for a maximum wheel load of 16 kips (Section 3.7) with an impact factor of 30% (Section 3.8.2). No specific horizontal force is specified with these loads but the longitudinal braking force equal to 5 percent of the live load (Section 3.9), for the design of the substructure could be extended to this case. This would result in a design wheel load of 21 kips vertically and 1 kip horizontally.

An impact factor of 75% for deck joints has been implemented in the new LRFD AASHTO specification (Section 3.6.2). This impact factor is to be used in conjunction with a 25 kip tandem axle (12.5 kip wheel load) for the design of deck joints (Section 3.6.1.2.3). This specification also recommends a load factor of 0.75 be applied to the axle load when checking fatigue. The LRFD AASHTO specification like the previous version does not make any recommendations for a horizontal force on the deck joint.

As part of the static testing the effect of gap width on joint loading was investigated. This testing provided an opportunity to check the validity of the equation presented by Koster and used by others regarding the effect of gap width on joint load. To perform this test the width of the gap was set at one, two and three inches. At each gap width the joint was loaded in a testing machine through a truck axle and the reactions on each of the beams was recorded. It was found that the load on the rail was independent of gap width. The formula presented by Koster was conservative at load levels induced by truck traffic and therefore is a reasonable assumption.

4.11 Washington Modular Bridge Specification [10]

The ATLSS reports state that the State of Washington has adopted a specification for modular expansion joints. A wheel load of 13 kips vertical and 4 kips horizontal is specified for the fatigue testing of modular joints. No increase in these values for impact is specified.

4.12 Texas Tests [14]

Several static and dynamic tests of strip seal expansion joints were performed by the Ferguson Structural Engineering Laboratory (FSEL) at the University of Texas at Austin. These tests unlike any other reported in this chapter evaluated both the load applied to the joint rail and the corresponding stress in the joint anchorage. Tested joints were subjected to dynamic and ultimate static loads. A total of four joints were tested. Three were of the standard flanged design and one was a new flangeless shape. Anchorage for each specimen was different. One specimen followed the recently mandated Texas standard of 5/8” ϕ studs spaced at 6 inches alternating along the top and side flange of the rail. The second specimen used a tab or loop style anchorage made of 3/8 inch plate. This tab anchorage was 3 inch by 6 inch with an elongated hole in the center and was welded to the rail at one foot intervals. The third specimen followed the Illinois standard 1/2” ϕ studs spaced at 9 inches alternating along the top and side flange. The flangeless design used 5/8” ϕ studs at 6 inches alternating between the side flange and the back of the seal pocket.

Dynamic testing was performed on all the rails except the flangeless design. Testing consisted of traversing the joint with a 14,000 lbs. capacity forklift. Wheel loads for the forklift were 7,600 lbs. when loaded and 3,500 lbs. unloaded (Axle loads of 15.2 and 7.0 kips). Tests were performed using both loaded and unloaded configurations. Due to the weight of the forklift and the small distance available for accelerating and braking, a maximum vehicle speed of 4.5 MPH was achieved for testing.

Measurement of the horizontal and vertical forces applied to the rail were computed by placing a 3 foot by 15 foot slab section on rollers. Load cells were then placed at each roller location to measure the vertical load applied to the joint. Additional load cells were attached to the sides of the slabs to measure horizontal loads. Strain gauges were placed on joint anchorages to record any transfer of load to the studs or tabs. Data was recorded during testing using the labs data acquisition system.

A total of 54 dynamic tests were conducted each varying speed, axle load and joint crossing direction. The results showed that at the low test speeds used loading of the joint was approximately equal to the static case. Measured horizontal forces applied to the joint were found to be less than 1/10 of the corresponding vertical load. Strains in the joint anchorages indicated that for an applied wheel load of 7,600 lbs. less than 1 ksi. of stress was developed. From these tests it was concluded that the ratio of horizontal to vertical load applied to joints by truck traffic was very small and at service level loads the transfer of the wheel load is from the top flange of the rail directly into the concrete deck through bearing and the anchorages are not affected. This last point strengthens the hypothesis that fatigue failure of anchor studs is caused by voids in the concrete which induce moment in the studs under traffic loading.

Static tests were performed on all four rail-anchorage types. Testing was performed in the FSEL 600 kip universal testing machine. The purpose of these tests was to determine the static ultimate capacity of the rail-anchorage configuration. The slabs were placed on the testing machine platen. Arrangement of the specimen in this manner prevented the probable failure of the slab which would occur under the high test loads. Based on the results of the dynamic tests only vertical load was applied to the joint rails. Loads were applied through two rollers at the top flange of the rail. Each roller was 10 inches long with a 3 inch space between them. This arrangement matched the load pattern presented by the forklift.

Testing of the three flanged rails to a load of 300 kips was completed without complete failure of the joints. The Texas anchorage and Illinois anchorage supported loads of 160 kips before the first signs of cracking were noticed. No substantial cracking of the tab anchorage was noticed up to the capacity of the testing machine. Testing of the flangeless rail showed signs of cracking at a load of 65 kips and ultimately failed at 160 kips. One unexpected outcome from the testing of the flangeless rail was that up to the cracking load of 65 kips this system provided the least deflection for applied load (stiffest) of all the rails tested.

Based on these tests, it was concluded that for properly installed joints under an AASHTO wheel load of 11 kips a factor of safety against cracking is approximately 6 for the flangeless design and almost 15 for the flanged rails.

4.13 Summary of Suggested Loads

When examining the recommendations reviewed in this chapter in appears that the design loading for joints should be higher than the standard design load in the current AASHTO Specification. Each recommendation is listed in Table 4.1.

Comparison of each of the loads listed in the chart shows a wide range of recommended design values. When the most conservative of these values is chosen (34.0 kips) and compared to the data in the Texas tests (Section 4.12) it is clear that the joint has sufficient strength to support the applied load without cracking of the concrete or failure of the embedded connectors.

Table 4.1

	Impact Factor	Other Factors	Recommended Wheel Load (Kips)	Design* Wheel Load (Kips)	Horizontal Force
RRL Tests	1.67	--	--	26.7	5.5 k/ft
Al-Rashid	2.00	--	--	32.0	--
French	2.00	--	--	32.0	--
Koster (1986)	1.70	1.25	--	34.0	1/3-1/2 Vert.
Hawkins	1.35	1.41	16.0	30.5	1/3-1/2 Vert.
Austria	1.40	--	14.6	20.4	2.9 K
AASHTO	1.30	--	16.0	20.8	1.0 K
AASHTO LRFD	1.75	--	12.5	21.9	--
Washington Modular**	--	--	17.3	17.3	5.3 K

* If a Wheel Load was not recommended the AASHTO Value of 16 kips was used.
** Washington Modular values for fatigue testing were divided by 0.75 for comparison.

The tests performed by Al-Rashid and the French recommended equal values for the impact factor. This is not surprising as in both tests the recommended design value was based on a joint protruding 3/8 of an inch above the roadway surface. The French researchers felt that this was a probable condition and should be designed for. The probability that a state Department of Transportation would accept a joint installed in this manner is unlikely as there are specifications regarding required highway smoothness. A joint with this large protrusion would not meet the specification.

The recommendations of Koster and Hawkins are based on analytical procedures of modeling the joint which assume the joint has significant elastic properties. Such an assumption is valid for modular joints which utilize preloaded springs and elastomeric bearings. Extension of these theories to armored joints would require that the bridge deck and joint rail behave as a flexible elastic member in the joint region. Behaviour of the bridge deck and joint rail is more rigid than the flexible condition assumed in their analysis.

In addition the relevance of the rebound theory presented by Hawkins and Babaei must be questioned based on the results of the McAllen failure investigation. Disassembly of the roadway section and the subsequent discovery of the fracture surface on the excavated stud brings the validity of this theory into question. The fracture surface on the excavated stud show's the impending failure of the stud to be due to the vertical traffic loading acting in a downward not upward direction. The investigations presented by Hawkins only mention the failure of joints in the context of failure of the deck concrete behind the rail and not the anchorages.

This leaves five recommendations which are applicable to normally installed and detailed bridge deck joints. Of these five recommendations all but one make specific recommendations (wheel load and impact factor) for the design of bridge deck joints. The exception is the Road Research Laboratory Report which presents only an impact factor of 1.67 and no wheel load. The design load presented in Table 4.1 with this recommendation is 26.7 kips. This value uses the AASHTO wheel load of 16 kips. If the wheel load value of 12.5 kips from the AASHTO LRFD Specifications were used with the load factor of 1.67, the load would be 20.9 kips. The relevance of using the standard AASHTO wheel load versus the LRFD wheel load are discussed in Section 4.14.

Table 4.1A

	Impact Factor	Wheel Load (Kips)	Design* Wheel Load (Kips)	Horizontal Force
RRL Tests	1.67	12.5	20.9	5.5 k/ft
Austria	1.40	14.6	20.4	2.9 K
AASHTO	1.30	16.0	20.8	1.0 K
AASHTO LRFD	1.75	12.5	21.9	--
Washington Modular**	--	17.3	17.3	5.3 K

Making the change in the wheel load used with the RRL recommended impact value of 1.67 results in four out of five of the recommendations presenting design values within 1.5 kips of each other. Based on this information, presented in Table 4.1A, the design load for bridge deck joints in the AASHTO LRFD Specification value of 21.9 kips is recommended. This value uses a wheel load that is based on a tandem axle loading the joint and a conservative impact value.

4.14 Distribution of Loads to Studs

Once the actual load to be applied to the rail has been decided, the design of the joint is not complete. The next consideration is the lateral distribution of the load along the rail to the studs. This calculation is necessary to determine if the stresses in the studs induced by the applied wheel load will lead to fatigue failure of the studs. Several different methods of load distribution will be evaluated. The fatigue criteria for the stresses in the studs will be those outlined in the AASHTO Specification for the design of shear studs in composite beams (Section 10.38.5.1 and Section 6.10.7.4.2 in the LRFD Specification). This criteria is used as it is the only one present in the AASHTO Specification for fatigue design of studs in shear.

Fatigue design of a bridge normally considers the number of trucks which cross the bridge in its intended life. For the design of a bridge deck joint the number of axles which cross the joint is the important consideration. Each axle crossing the joint, and not each truck crossing the joint, composes one cycle of the joint loading. In addition to the number of axles crossing the joint, the load each axle exerts on the joint is important also. The LRFD Specification specifies that the design of deck elements shall use a tandem pair of twenty-five kip axles. Initially this may seem less conservative than the thirty-two kip axle load specified in the AASHTO Specifications, but this is not true. The pair of tandem axles are to replace one axle of a standard AASHTO HS20-44 truck. This replacement results in a new axle load of fifty kips instead of thirty-two and is more realistic as the normal truck on the highway has two axles at the location of the HS20-44 single axle.

Both the AASHTO Guide Specification for Fatigue Design of Steel Bridges [2] and the LRFD Specifications specify a reduction in loading when checking fatigue for finite life. This reduced load is to account for the majority of the trucks on the highways not being as heavy as the design truck or axle loads, and the designs are reduced to reflect the distribution of truck loads. The LRFD Specification states that, the design load is to be multiplied by a load factor of 0.75 when accounting for the affects of fatigue. In the AASHTO Guide Specifications for Fatigue Design of Steel Bridges the standard design truck is replaced with the HS15 truck.[10] The HS15 truck is 25 percent lighter than the HS20 truck resulting in the same 25 percent reduction to the design load as seen in the LRFD Specification.

To perform this evaluation the recommended wheel load of 21.9 kips presented in the last section was used. The 21.9 kip load was multiplied by the LRFD specified 0.75 load factor which resulted in a fatigue evaluation load of 16.4 kips for finite life design. As part of this evaluation two different stud arrangements were considered: the Texas rail standard of 5/8" ϕ studs at 6 inches welded alternatingly along the top and side flange of the joint rail and the Illinois rail standard of 1/2" ϕ studs at 9 inches welded in the same manner as the Texas standard. For this evaluation the following was assumed: horizontal studs carry all of the vertical load, stud design is for fatigue loading with infinite life, the design fatigue shear stress for studs is 5.5 ksi from AASHTO (Section 10.38.5.1) and the tire contact length along the rail is 20 inches based on AASHTO (Section 3.30). The assumption that only horizontal studs carry vertical load is very conservative based on the results of the Texas test. For joints with other than horizontal studs attached to the vertical flange (as in flangeless rails) it may be assumed that these additional studs carry some portion of the load based on the additional studs, geometry and size.

The first calculation made was what would be the stress applied to the stud if only studs directly under the contact area carried load. With this distribution of load only two horizontal studs would be loaded for the Illinois rail standard resulting in a stress of 33 ksi per stud. Stress on the studs in the Texas rail standard arrangement would be 14 ksi. The large reduction in stress between the two standards is due to three studs being loaded in the Texas versus two in the Illinois coupled with the larger diameter Texas studs. These calculations indicate that if the assumption that only studs directly under the wheel contact area carry load the rail would fail for both cases as the stress is greater than the 13 ksi limit for a finite life of 100,000 cycles in the AASHTO Specification.

A second calculation was made to find the number of studs required to carry the wheel load at the endurance limit fatigue stress of 5.5 ksi associated with infinite life. In these calculations the full design load of 21.9 kips was used due to the calculation of the stud's capacity for infinite life For the Illinois rail standard sixteen studs were required to carry the wheel load at this stress. Sixteen studs would require over eleven feet of rail for each wheel using this standard. The Texas rail standard required ten studs per wheel load, due to the larger stud size and tighter spacing. Ten studs correspond to a rail length of four and one-half feet or fifty-four inches per wheel.

A third method of distributing the load to the rail is to consider the axle load "smeared" across the lane width. Again this would be a calculation considering the infinite life of the stud and the rail is to be designed using a load of 4.38 kips/ft. Given the spacing of the studs in each rail standard there would be 1.33 studs/ft for the Illinois rail standard and 2 studs/ft for the Texas rail standard. Stud stresses for these distributions would be 13.2 ksi for the Illinois standard and 5.6 ksi for the Texas standard.

Based on these calculations the Texas standard of 5/8” ϕ studs at 6 inches is recommended over the Illinois standard. Again it is reiterated that these calculations are based on the very conservative assumption that the vertical load is carried by the horizontal studs only and that the horizontal flange transmits no force directly into the slab as shown in the Texas tests.

Chapter 5. Conclusions & Recommendations

5.1 Conclusions

Armored bridge deck joints are used in the construction of short to medium span bridges. These joints provide an economical solution to the problem of concrete spalling at the edge of the bridge deck and the flow of water off the bridge deck onto areas below the deck, including the supporting substructure. Recent failures of several strip seal expansion joints brought into question the cause of the failures. Ancillary issues are: the design loads for bridge deck joints, what type of anchors should be used, and how the joints should be installed.

Design load values for bridge deck joints have been derived experimentally, analytically, and sometimes it appears randomly. With each presented value there are several assumptions that are made or special test cases examined. A comparison of the results from each of these highly variable methods shows design loads which are relatively equal. When these values are then compared to the results of ultimate strength tests, the reserve strength capacity of the joint beyond these “recommended” design loads is sufficient to prevent cracking of the concrete bridge deck let alone joint failure.

The initial reaction of those inspecting failed joints is that a failure of the weld between the rail and stud has occurred. Failures of this type would be unusual for several reasons. The first reason is that fabrication and welding of the joint rails generally occurs in a fabrication shop where a high degree of quality control can be exercised. Second, stud welds are typically performed with a stud welding machine, which is highly automated and provides consistent quality welds once calibrated. Proper calibration of the stud welding machine raises the most important point in regards to the quality of welds. Testing of the welds can be completed at any time by bending over any welded stud. Any signs of poor welding or improper calibration of the stud welding machine would occur during this process indicating the presence of poor quality welds and require the recalibration of the stud welding machine.

Improper installation techniques appear to be the cause of most armored bridge deck joint failures. The two most commonly occurring problems are incomplete concrete consolidation under the rail and incorrect setting of the joint height relative to the roadway. Incomplete concrete consolidation results in poor quality concrete under the rail. Poor quality concrete may result in large voids under the rail caused by trapped air or bleed water which forms during the vibration of the freshly placed concrete. These voids lead to the introduction of unintended bending stresses in the horizontal studs due to movement of the unsupported areas of the joint. The formation of these voids can occur even when following proper concrete placement practices. The presence of weep or vent holes along the top edge of the rail reduces the formation of such voids. Poor quality concrete also occurs with the formation of "rock pockets" or areas where the concrete paste has flowed from between the aggregate leaving only the aggregate behind to support the joint rail. This condition is caused by loose formwork which allows flow of the cement paste away from the concrete mix during concrete placement and vibration. The cost of preventing these two problems is quite low when compared to the replacement cost of the joint rail.

A current estimate of the cost to replace an armored bridge deck joint is \$250.00/ft of joint. Joint manufacturers estimate the cost of placing weep holes along the top flange of the rail to be less than \$2.00/ft. It is more difficult to estimate the cost of providing formwork which does not leak. A simple estimate would be \$240.00 based on a crew of eight working two additional hours at fifteen dollars an hour.

Typical installation of an armored bridge deck joint places the top flange of the rail flush with the final grade of the roadway. In areas where snow plows are used, positioning a joint in this manner makes it susceptible to snow plow impact. Leaving the joint 1/8 to 1/4 of an inch below the roadway profile prevents such damage from occurring. The cost of placing the joint in this manner versus flush with the roadway is immeasurable if it even exists, yet the returns are quantifiable.

All of this indicates that it is economically feasible to install an armored joint as currently designed so that it will reach its intended design life.

5.2 Recommendations

It appears that designers, suppliers, contractors, and inspectors do not have adequate guidance in the matters of joint design, fabrication, installation and inspection. In order to remedy this situation and prevent the continuing payment of unnecessary costs and disruptions to traffic the following recommendations are made for the use of armored bridge deck joints:

5.2.1 Recommendations to Bridge Designers:

1) For the design of bridge deck joint rails the design load shall be the two-axle tandem recommended by the LRFD AASHTO Specifications multiplied by 1.75 to account for the effects of impact on the joint. No reduction of this load for additional lanes is to be used. This same value multiplied by a load factor of 0.75 is to be used for the fatigue of the joint and its anchorages with no reduction for additional lanes.

2) Fatigue design of anchorages shall follow those procedures in either the Standard or LRFD AASHTO Specifications. Specifically the design fatigue strength of the studded anchorages shall follow the procedures detailed in either specification for the design of shear connectors for composite plate girders.

3) The design of studded anchorages shall assume that horizontal studs resist all vertical load. *As stated in section 4.14 this is a very conservative statement for flanged rails but allows for ease of design and additional studs will be of low cost and prevent joint failure*.

4) The number of studs required to carry the vertical wheel load at the allowable fatigue stress shall be calculated. From this number of studs, the length of rail needed to carry the load is to be found using the intended horizontal stud spacing. This distance shall not exceed six feet. *The basic theory is to look at how much of the rail is needed to participate in carrying the load as the rail probably will not transfer a wheel load more than a few feet, and the six feet is the AASHTO*

width of a truck. In addition, the loads from one wheel should not overlap another.

5) A minimum deck thickness of 12 inches shall be used in the joint area (deck width by four feet in the longitudinal direction). A minimum of three layers of transverse and longitudinal reinforcing shall be present in this area placed at the same spacing as the remaining sections of the deck. Bottom layer longitudinal bars in the main deck area shall be straight at their extension into the joint area. The joint area bottom layer bars shall extend into the main deck area and be fully

Figure 5.1 Additional Reinforcing Detail

developed in the main deck area. Epoxy coated reinforcing need only be used in the top steel layer and then only if the remaining sections of the deck are epoxy coated. (Figure 5.1) *This recommendation is based on observations made at McAllen where the deck reinforcing was very light when compared to that used in the Texas test (Which came from an Illinois DOT detail). The longitudinal reinforcing steel extended to the end of the deck and stopped. It was not developed at the end of the deck to where any forces at the end of the deck would have engaged. Also the drop panel used at the end of the deck was not uniform in depth. These recommendations are intended to increase the stiffness at the end of the bridge deck.*

6) Longitudinal steel shall be fully developed at the end of the slab. In lieu of this hairpin bars extending into the top and bottom shall be placed along the end

face of the bridge deck. Spacing and size shall be the same as the longitudinal reinforcing steel. *This addresses the last comment made for recommendation 5 where the reinforcing steel was not developed at the end of the deck. It is intended that the hairpins will carry forces back into the deck to the point where the longitudinal steel is developed.*

7) When sizing the initial joint opening for joint installation the specified installation temperature (Typically 70^0 F) shall account for installation of the
joint at temperatures higher and lower than this temperature with no need for resizing
of the initial joint opening. *A high to low range of 25^0 F is recommended but it is realized that it may not always be possible. When performing this calculation the minimum gap opening under temperature rise may be reduced to 3/8" when considering concrete structures because long term effects will increase this with time.*

5.2.2 <u>Recommendations for Contract Specifications:</u>

1) Prior to installation (shipping and storage) joints shall be covered and kept out of direct contact with the ground unless joints are hot dipped galvanized. *Inspection of the uninstalled rails at Dallas showed surface rusting on what was to become the inner faces of the rail when installed. Corrosion protection of these surfaces would be expensive. By covering and keeping the rail out of mud the amount of corrosion will be reduced and prevent any possibility of long-term corrosion problems*.

2) Any joint rail configuration in which the free vertical flow of air bubbles during concrete placement is impeded shall have 1/2" ϕ vent holes along the impeding surface spaced at one foot intervals.

3) For rails with vertical flanges, formwork shall be placed so that the finished concrete at the bottom of the joint rail is flush with the outer face of the vertical flange.

4) All formwork is to be flush, tight and give the appearance to the inspector and construction engineer that the formwork is sufficient to prevent the leakage of any concrete during concrete placement. If formwork should begin to leak during concrete casting, placement and vibration of concrete is to cease until the leak is sealed.

5) The contractor shall provide the construction and design engineer with the intended sequence of concrete placement including casting directions. Consideration shall be given to the effect of bridge geometry (specifically slope) n casting sequence and the possibility of the flow of plastic concrete away from bridge deck joint rails.

6) The highest point on the bridge deck rail shall be placed at least 1/8” but no more than 1/4” below the finished roadway profile.

7) Once the joint has been installed the erection bolts shall be removed to prevent any secondary forces in the joint or bridge deck due to temperature variations.

5.2.3 Recommendations for Inspectors:

1) At the time of their arrival inspect the bridge deck joint rails. Specifically verify that the following are in accordance with the contract and approved shop drawings: stud size, spacing and location, size and spacing of weep holes, initial gap opening, rails are covered and rails follow the above guidelines. If joints do not meet these the supervising construction and design engineer should be notified.

2) Inspect reinforcing steel placement and verify that proper clearance between reinforcing steel has been provided especially between the joint rail and transverse reinforcing steel.

3) Verify the placement elevation of each joint at the time of installation. Once the joint has been installed keep all loads, including finishers, off of the rail to prevent deflection of the rail from the desired position.

4) Use personal experience when approving contractors formwork in the joint area that it will not leak and will perform as outlined above.

5) Review the contractors approved concrete placement procedure and ensure that it is followed.

6) Aggressively inspect placement of the deck concrete by the contractor especially in the joint area. Allow only fresh concrete to be placed in the joint area and ensure that proper vibration procedures are followed especially in the joint area. Keep all loads including finishers off of the rail until the concrete has been allowed to cure sufficiently so that deflections of the rail will not deform the concrete away from the underside of the rail.

7) During concrete placement procedures watch for leakage of formwork especially in the joint area. If the formwork begins to leak stop the placement and vibration of concrete until the leak is sealed.

8) If the connector between the rail could not be removed at the time of installation it should be removed as soon as possible after concrete has been placed.

9) At the time of form removal perform a post-casting inspection of the end surface to ensure that it is uniform and consistent without voids especially at the rail edge. Report any inconsistencies in the surface finish to the supervising construction engineer so that a further determination of whether voids exist can be made, and if necessary any repairs can be made before opening the bridge to traffic.

5.2.4 Recommendations for Future Research:

1) Testing similar to the tests performed at The University of Texas (Frank, 1994) should be carried out on a real bridge structure where actual speeds can be achieved and any changes in behaviour over time can be monitored. At the time of testing the rail should be strain gauged to measure the lateral distribution of forces in the joint rail.

2) For failed or failing joints ultrasonic testing should be tried to see if accurate determination of the presence of voids can be made. Epoxy injection of any voids detected via ultrasonics or other methods should be attempted. It is recommended that these procedures be followed and then the rail removed so that the accuracy of the testing and the effectiveness of the repair can be accessed.

3) An improved detail for holding the joint in place should be sought out or developed so that joints can be placed soundly without requiring welding to the deck reinforcing or erection bolts being left in place until concrete is placed.

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