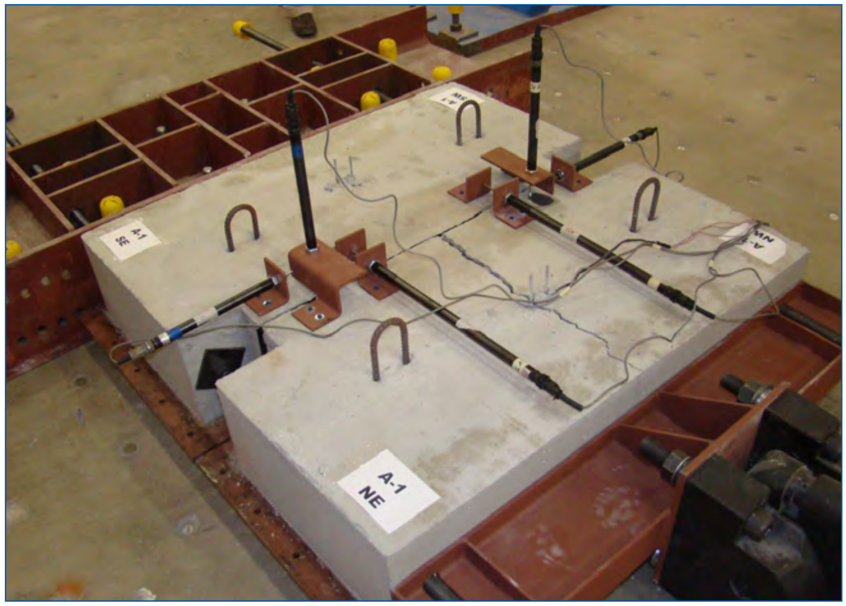
# MOUNTAIN-PLAINS CONSORTIUM

**MPC 20-417** | A.A. Ghadban, N.I. Wehbe, and W. Olson

Tolerances for Placement of Tie Bars in Portland Cement Concrete Pavement




MOUNTAIN-PLAINS CONSORTIUM REGION 8

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# Tolerances for Placement of Tie Bars in Portland Cement Concrete Pavement

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August 2020

## Acknowledgment

The authors would like to acknowledge the financial support of the Mountain-Plains Consortium (MPC) and the South Dakota Department of Transportation for funding this study through project MPC-440.

## Disclaimer

## ABSTRACT

Longitudinal joints in Portland Cement Concrete (PCC) pavement slabs require tie (dowel) bars to control joint opening resulting from thermal stresses. Inspections of PCC pavements by the South Dakota Department of Transportation (SD DOT) using ground penetrating radar (GPR) revealed that it is common for tie bars to be either misplaced or completely missing at some locations. It is unclear if tie bar misplacement results in increased maintenance costs or reduced pavement service life. Tie bar misalignment tolerances that have been established in existing specifications are not based on PCC pavement research or economic data. This research study investigates the effects of different tie bar misalignments on the tie bar performance to establish acceptable placement tolerances. A comprehensive literature review was carried out to assess existing specifications and past studies. The effect of different misalignment configurations and magnitudes on the longitudinal joint performance was examined by conducting laboratory experiments on PCC slabs incorporating four different misalignment configurations and four different misalignment magnitudes. The results showed that vertical and longitudinal translation misalignments had no significant effect on the performance of the longitudinal joint, while vertical skew misalignment had a mild effect only on joint faulting. Horizontal skew misalignment, however, caused a significant increase in both joint opening and joint faulting. Horizontal skew misalignment also caused a significant decrease in the tie bar load that initiates bar yielding. A simplified method was developed to conservatively estimate the allowable load in a tie bar for horizontal skew misalignments. Based on the results, recommendations are to reduce the horizontal skew tolerance limit from 18 in. to no more than 16 in.

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# EXECUTIVE SUMMARY

Concrete pavements are widely used for roadways across the United States. Jointed plain Portland Cement Concrete (PCC) pavement is a common type of concrete pavement that consists of unreinforced concrete slabs with longitudinal and transverse joints. The longitudinal joint runs parallel to the direction of traffic and is reinforced using tie bars. Tie bars are typically deformed, epoxy coated steel bars that control joint openings due to thermal stresses in the concrete slab by providing load transfer across the joint. The transverse joint runs perpendicular to the direction of traffic and is reinforced using dowel bars. Dowel bars are smooth, round bars that provide load transfer between slabs without restricting expansion and contraction of the pavement due to temperature and moisture changes.

Inspections of PCC pavements by the South Dakota Department of Transportation (SDDOT) using ground penetrating radar (GPR) revealed that it is common for tie bars to be misplaced or missing. A misplaced tie bar could inhibit the tie bar's ability to provide load transfer across the joint and to prevent excessive joint opening. However, the short- and long-term effects of misplaced or missing tie bars on the performance of the longitudinal joint are not well understood. Missing or misplaced tie bars can cause additional maintenance costs and reduce pavement life.

Placement tolerances for dowel bars have been implemented by most state departments of transportation (DOTs). However, very few states have set requirements on the placement of tie bars in PCC pavements. The tolerances that have been established for tie bar placement are not based on any previous PCC pavement research or economic data, making it impossible to know if these tolerances are too strict or too relaxed. Missing or misplaced tie bars could be costing SDDOT a substantial amount of money in the long term. Therefore, there was a need for a study to determine the effects of various tie bar misalignments on tie bar performance in order to establish acceptable placement tolerances. This report responds to that need.

This research involved two main tasks in order to identify current specifications regarding tie bar misalignment tolerances in PCC pavements and to provide recommendations to improve these specifications. These tasks were: 1) conducting a comprehensive literature review and 2) carrying out experiments involving several tie bar misalignment configurations and magnitudes. The literature review includes sources for existing design practices and specifications, in addition to the most recent studies about longitudinal joints in PCC pavements.

A total of 35 PCC slabs were tested at the Lohr Structures Laboratory at the Civil and Environmental Engineering Department at South Dakota State University (SDSU). All slabs had the same concrete mix design, with the only difference among them being the tie bar misalignment configurations and magnitudes. Three slabs acted as controls, having perfectly aligned tie bars. The other 32 slabs incorporated four different misalignment configurations and four different misalignment magnitudes for each misalignment configuration. The misalignment configurations were vertical and longitudinal translations, and vertical and horizontal skews. A direct mechanical tensile force was applied on each specimen. Allowable load, joint opening, and joint faulting were measured to assess the performance of the longitudinal joint.

The study presented in this report was conducted to 1) identify current specifications for tie bar placement tolerances in PCC pavements, 2) conduct experimental testing to examine the effect of various tie bar misalignment configurations and magnitudes on the performance of longitudinal joints, and 3) give recommendations to improve current specifications if needed.

The following findings and conclusions are based on the experimental tests that were conducted in this study.

- Vertical and longitudinal translation misalignments had no significant effect on the performance of the longitudinal joint.

- Vertical skew misalignments did not have any significant effect on maximum allowable load or joint opening.
- Vertical skew misalignments resulted in joint faulting that reached as high as 25 times that of aligned specimens (0.152 in. at an offset of 8 in.).
- Horizontal skew misalignments resulted in a decrease in maximum allowable force and an increase in both joint opening and joint faulting.
- The joint opening limit of 1/8 in. was exceeded at 20 in. horizontal skew offset.
- Joint faulting for horizontal skew misaligned specimens reached as much as 35 times that of aligned specimens at an offset of 20 in.

Based on the findings of this study, the research team offers the following recommendations.

- The current SDDOT tie bar tolerance limit for horizontal skew misalignment should be reduced from 18 in. to, at most, 16 in.
- Further reduction in the horizontal skew tolerance limit might be required if joint faulting is a significant issue.
- The vertical skew tolerance limit is sufficient, but contractors need to strictly abide by it in order to avoid excessive joint faulting.
- For future research, experiments can be conducted on slabs with multiple tie bars that have different horizontal skew magnitudes, examining more realistic scenarios.

# 1. INTRODUCTION

## 1.1 Project Description

Concrete deterioration is one of the major causes of poor performance and shortened life expectancy of concrete roadway infrastructure nationwide. Due to the low tensile strength of traditional concrete, reinforced concrete structures often experience cracking and spalling, leading to accelerated corrosion of imbedded reinforcement, failure under severe loading, and lack of durability. Fiber-reinforced concrete (FRC) has a solid reputation for superior resistance to crack development and abrasion, along with improvement on strength, ductility, resistance to dynamic loading, and resistance to freeze-thaw effects. Due to these properties, FRC has been used in many applications, such as bridge decks, repairs, and building beam-column connections.

A wide variety of FRC products are currently available for engineering applications, but the applicability and cost-effectiveness of different products has not been evaluated systematically for SDDOT in the past. There are many factors that play a role in the selection of FRC products. Depending on the application, different types and dosages of fibers will result in different performances. Guidelines are needed in order to facilitate selection of fiber type and dosage required to achieve optimal performance at a reasonable cost. Engineers find it challenging to interpret performance claims by manufacturers based on unstandardized testing procedures and what seem to be high fiber dosage recommendations.

It has been nearly 20 years since SDDOT delved into the topic. Many of the fiber materials used in SDDOT projects have been phased out or discontinued, and many more new products have been developed. What little available guidance on the proper specifications and use of FRC comes from the American Concrete Institute (ACI), and is generic in nature. Research is needed to investigate recent product development, evaluate fiber products currently on the market, and generate guidance for use, testing, and potential application of FRC. For lack of guidance, SDDOT may be sacrificing improved durability and performance as implementation lags technological developments in the area of fiber reinforced concrete structural components.

## 1.2 Objectives

The three main objectives of this study are listed below.

- Identify and describe best practices for design and construction of fiber reinforced concrete structural components. This objective was accomplished through an extensive literature search and interviews with various state DOTs and fiber manufacturers. The effort was focused on FRC products related to structural applications that are relevant to DOT projects. Moreover, the most commonly used products were identified and the most relevant SDDOT applications were looked at in more detail.

- Assess potential application, performance, costs, benefits, and drawbacks of fiber reinforced concrete structural components. After identifying the structural applications of FRC in common SDDOT projects through interviews, the FRC materials were evaluated experimentally at SDSU’s structures lab. The testing results, together with the literature review and interview findings, were combined to provide realistic assessment of performance, costs, benefits, drawbacks, and constructability of these structural applications.

- Develop guidance for design, material selection, construction, testing, and application of fiber reinforced concrete structures in South Dakota. A South Dakota specific guideline of using FRC in structural applications was developed with consideration to the availability, experience, and economic aspect of FRC applications in South Dakota. The guideline is very concise and incorporates the findings obtained from the literature review, DOT interviews, and experimental testing.

# 2. LITERATURE REVIEW

This chapter provides a summary of existing literature pertaining to longitudinal PCC joints with tie bars. The literature review focused on identifying common methods for installing tie bars, discussing current tie bar design procedures, and describing tie bar specifications used by various state DOTs. It also summarized findings from previous studies conducted on tie bars.

## 2.1 Introduction

Concrete pavements are widely used for roadways across the United States. Jointed plain Portland Cement Concrete (PCC) pavement is a common type of concrete pavement that consists of unreinforced concrete slabs with longitudinal and transverse joints. Two types of joints are depicted in Figure 2.1. The longitudinal joint runs parallel to the direction of traffic and is reinforced using tie bars. Tie bars are typically deformed, epoxy coated steel bars that control joint opening due to thermal stresses in the concrete slab. The transverse joint runs perpendicular to the direction of traffic and is reinforced using dowel bars. Dowel bars are smooth, round bars that provide load transfer between slabs without restricting expansion and contraction of the pavement due to temperature and moisture changes.

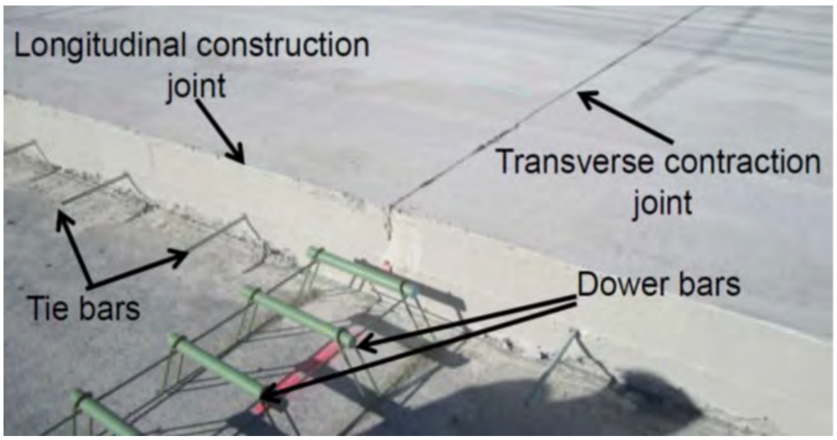



**Figure 2.1** Dowel and Tie Bars in PCC Pavement (Khazanovich, 2011)

Inspections of PCC pavements by South Dakota Department of Transportation (SDDOT) using ground penetrating radar (GPR) revealed that it is common for tie bars to be misplaced or missing. A misplaced tie bar could inhibit the tie bar's ability to provide load transfer across the joint and to prevent excessive joint opening. However, the short- and long-term effects of misplaced or missing tie bars on the performance of the longitudinal joint are not well understood. Missing or misplaced tie bars could be the reason for additional maintenance costs and reduced pavement life.

Placement tolerances for dowel bars have been researched and implemented by most state departments of transportation (DOTs). However, very few states have set requirements on the placement of tie bars in PCC pavements. The tolerances that have been established for tie bar placement are arbitrary and not based on any engineering or economic data, making it impossible to know if these tolerances are too strict or too relaxed. With millions of dollars spent each year on roads, the financial impacts of missing or misplaced tie bars could be costing SDDOT a substantial amount of money in the long term. Therefore, there is a need for a study to determine the effects of various tie bar misalignments on tie bar performance in order to establish acceptable placement tolerances.

## 2.2 Bar Installation Methods

The tie bar installation depends primarily on the longitudinal joint type used. There are two main types of longitudinal joints: sawed joints and construction joints. A sawed longitudinal joint is used when the two sides of the joint are poured monolithically, and the joint is saw cut after the concrete starts to set. A construction longitudinal joint is used if the two sides of the joint are poured at separate times, thus creating a cold joint. The type of longitudinal joint used depends on many factors, such as the width of the roadway, capabilities of the paver used, and site restrictions.

For sawed joints, tie bars can be installed either prior to paving using p-stakes or seats or mechanically during paving using automatic inserters. Using p-stakes or seats to place tie bars prior to paving is the most common way to install tie bars in South Dakota. This method involves setting the tie bars in p-stakes or on seats that have been attached to the roadway base prior to paving. Tie bars installed in p-stakes prior to placing concrete are illustrated in Figure 2.2.

**Figure 2.2** Tie Bars Prior to Placing Concrete (Perera, Kohn, & Tayabji, 2005)

Mechanically placing tie bars with automatic inserters involves inserting the tie bars in the plastic concrete as it is being placed. However, installing tie bars with automatic inserters is not allowed by many state DOTs, including South Dakota, due to the increased number of missing or misplaced tie bars commonly found with this installation method.

Along construction longitudinal joints, the tie bars can either be installed into the plastic concrete or drilled in after the concrete has hardened. Installation of tie bars into the plastic concrete can be done either mechanically or by hand. Mechanical placement of tie bars is done by the paving machine prior to final strike off of the paver. SDDOT does not allow hand placing tie bars into the plastic concrete. When drilled in, the tie bars are installed into holes that have been drilled into the face of the hardened concrete (Figure 2.3). An epoxy adhesive is used to form a bond between the tie bar and the hardened concrete.

**Figure 2.3** Installation of Drilled-in Tie Bars (South Dakota DOT, 2016)

## 2.3 Design Methods

There are two main design procedures for determining the required size and spacing for tie bars: 1) the AASHTO Guide for Design of Pavement Structures (or AASHTO 1993 design guide) (AASHTO, 1993) and 2) the AASHTO Mechanistic-Empirical Pavement Design Guide (or M-E design guide) (AASHTO, 2008).

### 2.3.1 AASHTO Guide for Design of Pavement Structures (AASHTO, 1993)

The AASHTO (1993) design guide procedure is commonly used for selecting tie bar size and spacing. That design procedure provides the following general recommendations for tie bars:

1. Made from grade 40 deformed steel bars or connectors
2. Have a corrosion resistant coating, such as epoxy, in regions where salts are applied to pavements
3. Minimum length for #4 and #5 tie bars of 25 in. and 30 in., respectively
4. Minimum center-to-center spacing of 48 in.

The AASHTO (1993) design procedure is based on the subgrade drag theory (SDT). The SDT determines the amount of steel required to "drag" the concrete slab across the base material without yielding or pulling out the tie bars. The tie bar spacing using SDT can be found using Equation 1 through Equation 3.

$$F_{drag} = L_{fe} D W_{conc.} F \qquad \text{Equation 1}$$

Where:

$F_{drag} = Required\ force\ to\ drag\ slab\ accross\ the\ base, lb/in$

$L_{fe} = Distance\ from\ longitudinal\ joint\ to\ closest\ free\ edge, ft$

$D = Pavement\ slab\ thickness, in$

$W_{conc.} = Unit\ weight\ of\ concrete\ \left(Typcally\ use\ 1.0\ \frac{lb}{in^2 ft}\right)$

$F = Friction\ factor\ (See$ Table 2.1)

$$F_{TB} = f_s A_s \qquad \text{Equation 2}$$

Where:

$$F_{TB} = Allowable\ tie\ bar\ force, lbs$$

$$f_s = Allowable\ steel\ working\ stress, psi\ \left(\frac{3}{4} f_y\right)$$

$$A_s = Tie\ bar\ cross\ sectional\ area, in^2$$

$$J_{TB} = \frac{F_{TB}}{F_{drag}} \leq 48\ in. \qquad \text{Equation 3}$$

Where:

$$J_{TB} = Tie\ bar\ spacing, in$$

$$F_{drag} = Force\ required\ to\ drag\ slab\ accross\ the\ base, lb/ft$$

The friction factor, *F*, value used in Equation 2 is given in Table 2.8 of the AASHTO (1993) design procedure for many common base materials. Table 2.1 is a replica of Table 2.8 from the AASHTO (1993) design guide.

**Table 2.1** Recommended Friction Factor Values (After AASHTO, 1993)

| Type of Material Beneath Slab | Subgrade Friction Factor (F) |
|---|---|
| Surface Treatment | 2.2 |
| Lime Stabilization | 1.8 |
| Asphalt Stabilization | 1.8 |
| Cement Stabilization | 1.8 |
| River Gravel | 1.5 |
| Crushed Stone | 1.5 |
| Sandstone | 1.2 |
| Natural Subgrade | 0.9 |

Despite being the most widely used design approach for determining tie bar size and spacing, the AASHTO (1993) design procedure has deficiencies. Some deficiencies discussed in the American Concrete Pavement Association (ACPA) article, "Mechanistic-Empirical Tie Bar Design Approach for Concrete Pavements" (Mallela, Gotlif, Darter, Ardani, & Littleton, 2009), are presented below:

- Does not consider the stresses induced from temperature changes or drying shrinkage of the concrete slab.
- Does not compute the actual stresses in the steel.
- The distance to a free longitudinal joint is hard to define when more than two lanes are tied together.
- Is based on a simple friction model and assumes a single parameter to define the behavior at the slab-based interface.
- Includes a large safety factor by reducing the steel yield stress.
- Does not account for displacement of the base layer since the base is assumed to be rigid.

These shortcomings led AASHTO to develop the Mechanistic-Empirical Pavement Design Guide (2008).

### 2.3.2 AASHTO Mechanistic-Empirical Pavement Design Guide (AASHTO, 2008)

The AASHTO (2008), sometimes called the M-E design guide, is based on engineering mechanics and has been validated by road test performance data. In order to use the M-E design method, the pavement must first be designed according to the AASHTO 1993 design procedure. The pavement design can then be incorporated into the M-E design guide software, along with such conditions as traffic, climate, and subgrade. The M-E design guide software assesses the incremental damage to pavement over time, resulting from the applied stresses. The incremental damage is used to predict the pavement distresses and smoothness at any given time throughout the pavement's lifespan. The user can then use the predicted pavement distresses and smoothness to determine if the pavement design needs to be improved.

This design method is used to "fine tune" or "double check" the pavement design developed by the AASHTO 1993 design procedure. Therefore, no changes are made to the allowable tie bar design force. Instead of the SDT, the load experienced by the tie bar, in this method, is a function of the traffic, climate, and subgrade conditions. If the M-E design guide software output shows that the input design performs adequately over the design life of the pavement, no changes need to be made to the tie bar design. However, if the M-E design guide software shows that the design is inadequate, the design procedure recommends iterative changes using engineering judgment until adequate results are obtained.

## 2.4 Available Standard Specifications

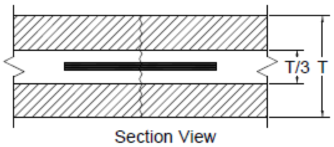
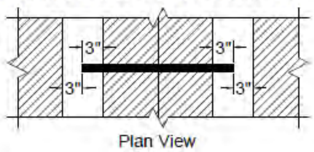
Rather than calculating tie bar spacing for each individual roadway design, most state DOTs have adopted one or several different standard tie bar designs. The standard tie bar designs are determined for various combinations of parameters, which may include pavement thicknesses, joint types, tie bar steel grades, tie bar diameters, installation methods, and free edge spacing. The number of parameters considered depends on what each state DOT deems necessary for their specific roads.

### 2.4.1 South Dakota DOT

SDDOT publishes standard tie bar design specifications in the annual version of the Concrete Paving Manual (South Dakota DOT, 2016). The Concrete Paving Manual specifies #5, grade 40 or grade 60, epoxy coated, deformed tie bars. The length and spacing of the tie bar depend on the type of longitudinal joint. For tie bars that are not drilled in, the tie bar length should be 30-in. long with 15 in. embedded on each side of the joint. For drilled-in tie bars, the tie bar should be 24-in. long, with 9 in. embedded in the in-place concrete. Center-to-center spacing of the tie bars is specified to be 48 in. for sawed joints and construction joints with a female keyway. For construction joints with a male keyway or no keyway, the center-to-center tie bar spacing is reduced to 30 in.

SDDOT provides vertical and transverse placement tolerances for tie bars in their concrete paving manual. Among all of the state DOT specifications, SDDOT was the only state DOT that provided a tie bar placement tolerance specification. However, no explanation is provided on how the placement tolerances were developed. Table 2.2 shows the current tie bar placement tolerances recommended by SDDOT.

**Table 2.2** Current SDDOT Tie Bar Placement Tolerance Limits

| Direction | Tolerance Limit |
|---|---|
| Vertical Placement:  | All parts of the tie bar must be within the middle 1/3 of the pavement depth. |
| Transverse Placement:  | ± 3.0 in. measured perpendicular to the longitudinal joint. |

### 2.4.2 Federal Highway Administration

The Federal Highway Administration (FHWA) recommended a tie bar design in a technical advisory for concrete pavement joints, last updated in November 2011 (Federal Highway Administration, 2011). The technical advisory states that longitudinal joints with tie bars should be used for slabs that have a width exceeding 15 ft. The FHWA recommends that tie bars should be either #4 or #5, epoxy coated, deformed bars. They should be made from either grade 40 or grade 60 steel. The length of the tie bar is a function of the tie bar size and steel grade. The tie bar center-to-center spacing is a function of the pavement thickness, joint type, tie bar size, tie bar material grade, and the distance to the free edge. Table 2.3 and Table 2.4 show the FHWA's recommended tie bar length and spacing requirements.

**Table 2.3** FHWA's Recommended Tie Bar Length

| Tie Bar Grade | Tie Bar Length for a #4 Tie Bar | Tie Bar Length for a #5 Tie Bar |
|---|---|---|
| Grade 40 | 24 in. | 30 in. |
| Grade 60 | 32 in. | 40 in. |

**Table 2.1** FHWA's Recommended Tie Bar Spacing

| Pavement Thickness, in. | Joint Type* | Tie Bar Spacing for a #4 Tie Bar, in. | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Grade 40 | | | | | Grade 60 | | | | |
| | | Distance to Free Edge, ft. | | | | | Distance to Free Edge, ft. | | | | |
| | | 10 | 12 | 16 | 22 | 24 | 10 | 12 | 16 | 22 | 24 |
| 9 | Wrap | 37 | 31 | 23 | 17 | 16 | 48 | 47 | 35 | 25 | 23 |
| | Butt | 26 | 22 | 16 | 12 | 11 | 40 | 34 | 25 | 18 | 16 |
| 10 | Wrap | 34 | 28 | 22 | 16 | 14 | 48 | 42 | 32 | 23 | 20 |
| | Butt | 24 | 20 | 16 | 11 | 10 | 36 | 30 | 23 | 16 | 14 |
| 11 | Wrap | 31 | 25 | 20 | 15 | 13 | 47 | 38 | 29 | 21 | 19 |
| | Butt | 22 | 18 | 14 | 11 | 9 | 34 | 27 | 21 | 15 | 14 |
| 12 | Wrap | 28 | 23 | 18 | 13 | 12 | 42 | 35 | 27 | 19 | 18 |
| | Butt | 20 | 16 | 13 | 9 | 9 | 30 | 25 | 19 | 14 | 13 |
| | | Tie Bar Spacing for a #5 Tie Bar, in. | | | | | | | | | |
| 9 | Wrap | 48 | 48 | 36 | 25 | 24 | 48 | 48 | 48 | 40 | 36 |
| | Butt | 42 | 35 | 26 | 19 | 17 | 48 | 48 | 39 | 29 | 26 |
| 10 | Wrap | 48 | 44 | 33 | 24 | 22 | 48 | 48 | 48 | 36 | 32 |
| | Butt | 38 | 31 | 24 | 17 | 16 | 48 | 47 | 35 | 26 | 23 |
| 11 | Wrap | 48 | 40 | 30 | 22 | 20 | 48 | 48 | 44 | 32 | 30 |
| | Butt | 34 | 29 | 21 | 15 | 14 | 48 | 43 | 31 | 23 | 21 |
| 12 | Wrap | 44 | 36 | 28 | 20 | 18 | 48 | 48 | 41 | 30 | 28 |
| | Butt | 31 | 26 | 20 | 14 | 13 | 47 | 39 | 29 | 21 | 20 |

* Warp Joint: A sawed or construction joint with a keyway
Butt Joint: A construction joint with no keyway

### 2.4.3 Minnesota DOT

The Pavement Design Manual of the Minnesota Department of Transportation (Minnesota DOT, 2014) specifies two tie bar designs based on the pavement thickness. For pavements less than, or equal to, 10-in. thick, the tie bars should be 30-in. long, #4, deformed bars spaced at 30 in. center-to-center. If the pavement thickness is greater than 10 in., the tie bars should be 36-in. long, #5, deformed bars spaced at 30 in. center-to-center. Regardless of the pavement thickness, the pavement design manual recommends grade 60 and epoxy coated tie bars.

### 2.4.4 Colorado DOT

The Colorado DOT standard tie bar designs are listed in the annual edition of the Colorado DOT's Pavement Design Manual (Colorado DOT, 2014). The pavement design manual specifies grade 60, epoxy coated, deformed tie bars that are 30-in. long and placed at 36 in. center-to-center spacing. When the pavement is placed on an unbound base, the tie bar should be a #5 bar. When the base material is lime treated soil, asphalt treated, cement treated, milled asphalt, or recycled asphalt pavement, the tie bar should be a #6 bar.

### 2.4.5 Nebraska Department of Roads

The standard concrete pavement details (Nebraska DOR, 2011) for the Nebraska Department of Roads (DOR) specifies different tie bar designs based on the type of longitudinal joint. For a sawed longitudinal joint, the tie bars should be 30-in. long and spaced at 33 in. center-to-center. The tie bar needs to be a #5 bar if the pavement thickness is between 6 in. and 10 in. or a #6 bar if the pavement thickness is greater than 10 in. For a construction longitudinal joint, the pavement thickness must be at least 8 in., so that a keyway can be installed. The tie bars are required to be a #5 bar that is 30-in. long and spaced at 33 in. center-to-center.

### 2.4.6 Iowa DOT

The Iowa DOT provides its standard tie bar designs in standard concrete pavement drawings (Iowa DOT, 2014). The standard concrete pavement drawings specify the tie bar size, length, and spacing, based on the joint type and pavement thickness. The standard tie bar designs as specified by the Iowa DOT are provided in Table 2.5.

**Table 2.5** Iowa DOT Standard Tie Bar Design

| Joint Type | Pavement Thickness, in. | Bars Number | Bar Length, in. | Tie Bar C-C Spacing, in. |
|---|---|---|---|---|
| Sawed Joint | < 8 | # 4 | 36 | 30 |
| | ≥ 8 | # 5 | 36 | 30 |
| Construction without a Keyway | < 8 | # 4 | 36 | 30 |
| | ≥ 8 | # 5 | 36 | 30 |
| Construction with a Keyway | < 8 | # 4 | 30 | 30 |
| | ≥ 8 | # 5 | 30 | 30 |

### 2.4.7 Indiana DOT

A set of standard concrete pavement design drawings (Indiana DOT, 2011) are used by the Indiana DOT to specify its standard tie bar designs. The standard tie bar designs vary primarily based on the joint type and the pavement thickness. The Indiana DOT standard tie bar designs are provided in Table 2.6.

**Table 2.6** Indiana DOT Standard Tie Bar Design

<table>
<tr><th>Longitudinal Joint Type</th><th>Pavement Thickness, in.</th><th>Tie Bar Length, in.</th><th>Tie Bar Size</th><th>Tie Bar Center-to-Center Spacing, in.</th></tr>
<tr><td rowspan="2">Sawed</td><td>≤ 9</td><td>30</td><td># 5</td><td>36</td></tr>
<tr><td>> 9</td><td>30</td><td># 6</td><td>36</td></tr>
<tr><td rowspan="4">Construction</td><td>< 9</td><td>30</td><td># 5</td><td>36</td></tr>
<tr><td>9 to 12</td><td>30</td><td># 6</td><td>36</td></tr>
<tr><td rowspan="2">> 12</td><td>30</td><td># 6</td><td>24</td></tr>
<tr><td>30</td><td># 7</td><td>36</td></tr>
</table>

### 2.4.8 Summary

Based on tie bar specifications adopted by few state DOTs, the upper and lower limits used for each tie bar design parameter were identified. The upper and lower limits for each tie bar design parameter are provided in Table 2.7.

**Table 2.7** Upper and Lower Limits for Tie Bar Design Parameters

| Tie Bar Design Parameter | Upper Limit | Lower Limit |
|---|---|---|
| Grade | 60 | 40 |
| Size | #7 | #4 |
| Length | 40 in. | 24 in. |
| Center-to-center Spacing | 48 in. | 9 in. |

## 2.5 Previous Studies

A rigorous literature review revealed only two previous research studies on tie bars placement or PCC pavement joint performance relevant to testing and development of tie bar placement tolerances. The following summarizes the work done in each study.

### 2.5.1 Mallela et al.

Mallela et al. (2011) conducted research to evaluate a longitudinal tie bar joint system. The research started with a preliminary field inspection of longitudinal joints. The inspection showed that the condition of the longitudinal joints was highly variable along sections of the roadway for no apparent reason. During the preliminary inspections, three forms of joint distresses were observed at the longitudinal joint: 1) excessive joint opening (most common), 2) joint faulting, and 3) joint slippage (Figure 2.4).

**Joint Opening** **Joint Faulting** **Joint Slippage**

**Figure 2.1** Longitudinal Joint Distresses (Mellala et al., 2011)

The researchers initially suspected the longitudinal joint distresses were caused by one or a combination of the following factors:

- Lane configuration (number/width of lanes, lane to shoulder connection)
- Pavement structure (pavement thickness, base friction, and stiffness properties)
- Portland cement concrete properties (compressive strength, modulus of elasticity, thermal expansion, shrinkage, unit weight)
- Weather conditions (changes in temperature/moisture)
- Construction factors (longitudinal joint type, tie bar installation method)
- Other factors (pavement support conditions, slope stability, and road geometry)

To investigate the effects of those six factors, two rounds of field tests were conducted. All of the field tests were performed on sections of roads that had both well and poorly performing longitudinal joints in close proximity to one another with similar tie bar designs, traffic conditions, and base conditions.

The first round of field testing was completed in the fall of 2008 on three roadway sections. At each of the three test sections, one lane of the road was closed from morning until early afternoon to collect data on longitudinal joint opening, pavement temperature, falling weight deflectometer (FWD), and magnetic induction tomography (MIT). The longitudinal joint opening and the pavement temperature were measured at regular intervals throughout the morning and early afternoon to determine how the joint opening changed as the temperature of the concrete changed. The FWD test was used to measure the load transfer across the longitudinal joint. This test was performed simultaneously with the longitudinal joint opening and concrete temperature measurements to see how the load transfer was affected by the width of the joint opening. The MIT scan testing was performed once during the morning to determine the position of the tie bars in the pavement along the longitudinal joint. The most significant conclusion found during this first round of field tests was that many of the tie bars at the poorly performing joints were either missing or severely misaligned. At the well performing longitudinal joint, the tie bars were found to be very close to their intended position. The MIT scan images are provided (Figure 2.5) for a section of the well and the poorly performing longitudinal joints at one of the sites. The black line on the image represents the longitudinal joint; the orange to red shading indicates the positions of the tie bars.

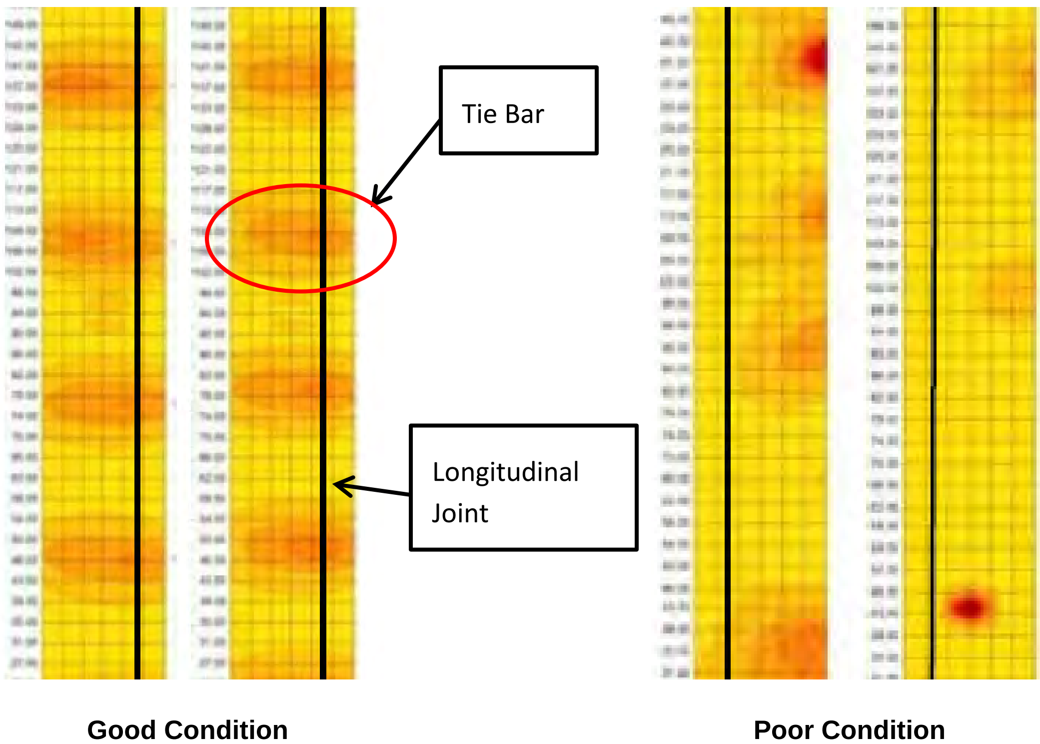



**Figure 2.5** MIT Scan Images for a Longitudinal Joint (Mellala et al., 2011)

Based on the MIT scan images for the five sites, it was concluded that the largest joint openings were seen when the tie bars were either missing or misaligned in a way that resulted in a reduced embedment length. The vertical placement of the tie bars and misalignments with adequate embedment depths appeared to perform adequately without allowing excessive joint openings.

The results of field tests by Mallela et al. (2011) indicate that tie bar placement has a significant impact on the future condition of the longitudinal joints. The investigators conclude that tie bars with "proper embedment" on both sides of the joint appear to be performing adequately. However, the investigators stated that more tests were needed to determine the minimum length for proper embedment.

### 2.5.2 Buch et al.

Buch et al. (2007) conducted a research study for the Michigan DOT. The objective of this research was to perform experimental and analytical studies to develop placement tolerances for dowel bars.

Experimental testing was performed on 67 specimens, which consisted of 54 different dowel bar configurations. The 54 different configurations were made by varying the number of dowel bars, the alignment configuration, and the dowel bar orientation with respect to the adjacent dowel bar. Specimens were tested with one, two, three, and five dowel bars. The specimens with one and two dowel bars were tested while embedded in 48-in. x 48-in. x 10-in. concrete slabs. The three and five dowel bar specimens were tested while embedded in 96-in. x 72-in. x 10-in. concrete slabs. Three alignment configurations were examined with the experimental tests: 1) horizontal skew, 2) vertical skew, and 3) a combination of those two. For each alignment configuration, four misalignment magnitudes were examined. For specimens with more than one dowel bar, the dowel bars were placed in one of three patterns: 1) Non-Uniform, 2) Uniform, and 3) Alternate (Figure 2.6)

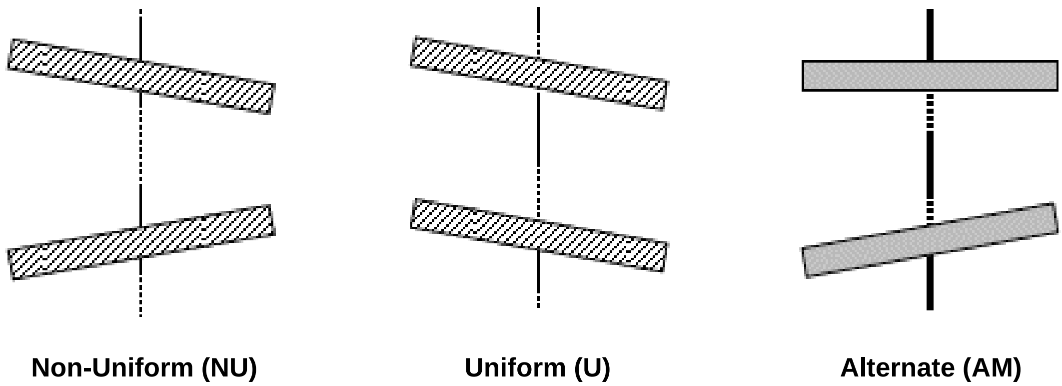



**Figure 2.2** Dowel Bar Relative Orientation (Buch et al., 2007)

The experimental test matrix with the dowel bar configurations tested is provided (Table 2.8).

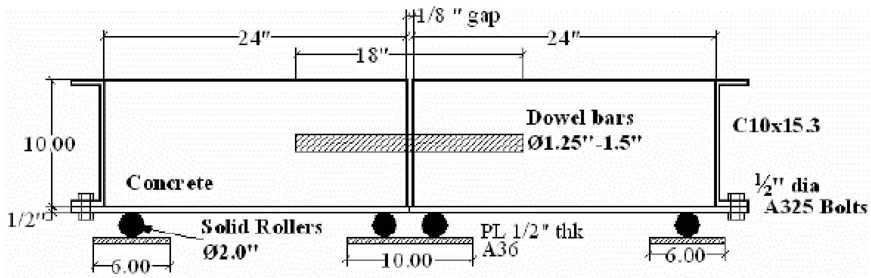
**Table 2.8** Experimental Test Matrix for the Dowel Bar Tolerances Study by Buch et al. (2007)

| Orientation | Misalignment | | 1 Bar | 2 Bars | | | 3 Bars | 5 Bars | |
|---|---|---|---|---|---|---|---|---|---|
| | End Offset (in.) | Bar Rotation (radians) | | U[1] | NU | AM | NU | AM | NU |
| Aligned | 0 | 0 | X | X | | | X | X | |
| Vertical | 1 | 9 | X | X | X | | | | |
| | 3/4 | 12 | X | | X | X | | | |
| | 1/2 | 18 | X | X | X | X | X | X | X |
| | 1/4 | 36 | X | X | X | | | X | |
| Horizontal | 1 | 9 | X | X | X | | | | |
| | 3/4 | 12 | X | | X | X | | | |
| | 1/2 | 18 | X | X | X | X | X | X | X |
| | 1/4 | 36 | X | X | X | | | X | |
| Combined | 1 | 9 | X | X | X | | | | |
| | 3/4 | 12 | X | | X | X | | | |
| | 1/2 | 18 | X | X | X | X | X | X | X |
| | 1/4 | 36 | X | | X | | | X | |
| | Total | | 13 | 9 | 13 | 6 | 3 | 6 | 4 |

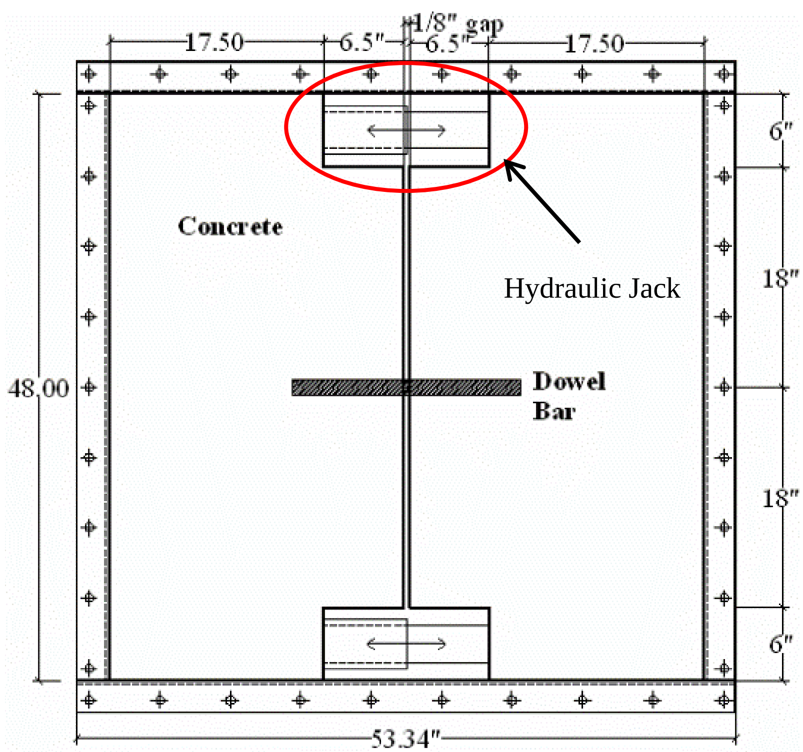
All specimens were cast using custom made steel forms that consisted of a ½-in. thick steel plate and C10 x 15 structural steel channels for the bottom and sides, respectively. The dowel bars were held in place with threaded bars that were bent into a "U" shape and attached to a C3 x 5 structural steel channel that spanned across the top of the forms. The joint between the two slabs was created with a 1/8-in. thick sheet of aluminum. The aluminum sheet was left in place throughout testing to simulate a completely cracked section of concrete. The 1/8-in. thick aluminum joint was held vertical on either end by steel forms that attached to the C10 x 15 structural steel side channels to create box cutouts for the hydraulic cylinders.

After the specimen was poured and cured, it was prepared for testing by first removing the two side channels that were perpendicular to the joint. The hydraulic cylinders were installed into the box cutouts on either end of the joint to apply the tensile force to the specimen during testing. The tensile force was used to simulate the forces imposed on the dowel bars when the concrete contracts. Displacement sensors were attached to the specimen across the crack to measure the relative displacement between the two concrete slabs. The specimen was supported on 2-in. diameter rollers throughout testing to eliminate any friction that would occur between the ground and the steel base.

A dimensioned plan and section views of a 48-in. x 48-in. x 10-in. specimen is shown in Figure 2.7. A 5-dowel bar specimen test setup is shown in Figure 2.8.




**Section View**




**Plan View**

**Figure 2.7** Details of a Test Specimen with One Dowel Bar (Buch et al., 2007)

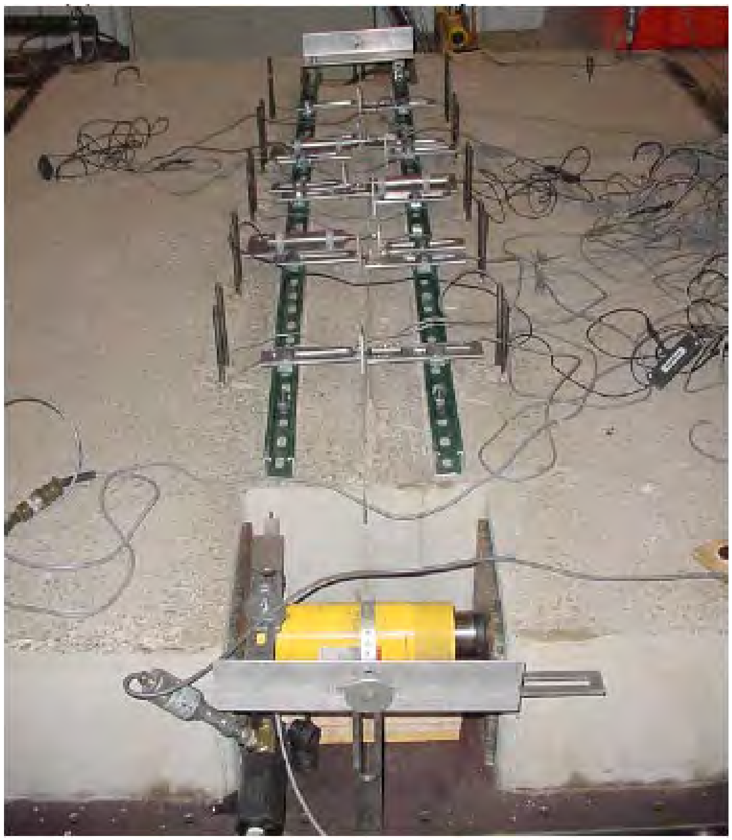


**Figure 2.8** A 5-Dowel Bar Specimen (Buch et al., 2007)

To better understand the effect of misalignments on the dowel bar-concrete bond and the concrete stresses, a finite element model was created for each of the 54 specimens tested in the laboratory.

Based on experimental and analytical results, a misalignment tolerance range was determined. The researchers recommended that for a skewed dowel bar, the offset of the dowel bar ends relative to one another should not exceed 1/8 in. to 1/4 in.

# 3. EXPERIMENTAL WORK

The experimental work conducted in this study consisted of laboratory testing of 35 slab specimens incorporating full depth joints with tie bars. The testing was performed at the Lohr Structures Laboratory at South Dakota State University. This chapter provides an overview of the measurement of material properties, testing matrix, specimen construction, instrumentation, testing procedures, and issues encountered during testing.

## 3.1 Material Properties

Material properties for the concrete batches and tie bars used to construct the specimens were determined in accordance with American Society for Testing and Materials (ASTM) standards (2014). Fresh and hardened concrete properties were measured for each concrete batch. Tension tests were performed to determine the mechanical properties of the tie bar.

The fresh concrete properties were measured prior to casting the specimens. The measured fresh concrete properties were temperature, air content, and slump. Air content and slump measurements are required by SDDOT to ensure that the concrete meets specifications (see Appendix A for the concrete mix design). For the well graded PCC pavement mix used for casting the specimens, SDDOT specifies an air content of 5.0% to 7.5% and a slump of 2 in. to 3 in.

The hardened concrete properties were measured by testing 6-in. x 12-in. standard concrete cylinders and 6-in. x 6-in. x 22-in. beams that were prepared according to ASTM 192 (2014). The samples were covered in wet burlap for three days to replicate the curing of the specimens. Compressive strength, modulus of elasticity, split tensile strength, and flexural strength were determined for each batch of concrete. The compressive strength was measured at three, seven, and 28 days according to ASTM C-39 (2014). The cylinders used for the compressive tests were caped with Tech-Lab Industries HYTECH #9 high strength capping compound according to ASTM C-617 (2012). The modulus of elasticity was during the compression tests at seven and 28 days. The split tensile strength was found by performing a split tensile test on a cylinder according to ASTM C-496 (2004) at seven and 28 days. The concrete flexural strength (modulus of rupture) was determined at seven and 28 days according to ASTM C-78 (2010). For every test, a minimum of three samples were tested and the average values were reported.

A tensile test was performed according to ASTM E-8 (2013) on a dog-boned sample made from a tie bar. Due to loading capacity limitations of the tensile testing machine used to test the tie bar samples, the middle segment of each bar sample was machined to a diameter of 0.35 in. A 25-mm. gauge length MTS extensometer was attached to the dog-boned tie bar sample to measure the tie bars' extension along the gauge length (Figure 3.1).

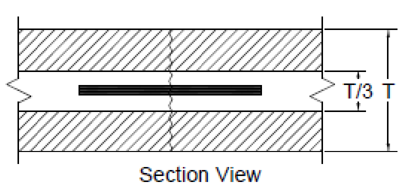
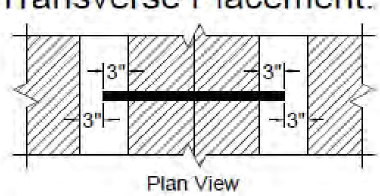

**Figure 3.1** Tensile Test of a Dog-Boned Tie Bar Sample

## 3.2 Testing Matrix

Each specimen consisted of two 48-in. x 24-in. x 10-in. concrete slabs that were connected with a tie bar. The tie bar was 30-in. long, grade 60, epoxy coated, #5 deformed bar as specified by SDDOT. The purpose of the testing was to investigate the effect of each tie bar alignment configuration on the joint behavior and anchorage strength under an increasing splitting force normal to the joint surface.

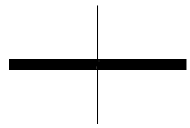
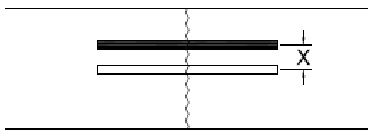
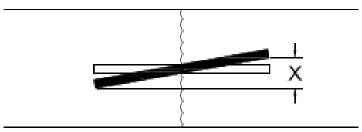
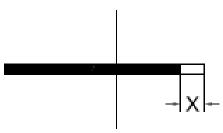
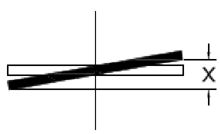
The testing matrix was developed to investigate the behavior of the tie bars and joints under various tie bar alignment configurations. Based on the literature review and discussions with the technical panel, four alignment configurations were selected to be tested: 1) vertical translation, 2) vertical skew, 3) longitudinal translation, and 4) horizontal skew. For each of these alignment configurations, four different misalignment magnitudes were selected. The misalignment magnitudes were based on both current SDDOT tie bar placement tolerances (Table 3.1) and typical as-built conditions as determined during ground penetrating radar (GPR) tests.

**Table 3.1** Tie Bar Placement Tolerances for 10-in. Thick Pavements (SD DOT, 2016)

| | | |
|---|---|---|
| Vertical Placement:   | Tolerance Limit | All parts of the tie bar must be within the middle 1/3 of the pavement depth. |
| | Vertical Translation | ± 1.25 in. |
| | Vertical Skew | 2.50 in. |
| Transverse Placement:   | Tolerance Limit | ± 3.0 in. measured perpendicular to the longitudinal joint. |
| | Longitudinal Translation | 3.0 in. |
| | Horizontal Skew | 18.0 in. |

Two specimens of each misalignment magnitude were constructed, totaling eight samples for each alignment configuration. In addition, three control specimens with aligned tie bars were built. A total of 35 specimens were built and tested in this study (Table 3.2).

**Table 3.2** Testing Matrix

| Alignment Configuration | Misalignment Magnitude, in.. | Number of Samples |
|---|---|---|
| Aligned:  Plan View | 0 | 3 |
| Vertical Translation:  Section View | X = 1 | 2 |
| | X = 2 | 2 |
| | X = 3 | 2 |
| | X = 4 | 2 |
| Vertical Skew:  Section View | X = 2 | 2 |
| | X = 4 | 2 |
| | X = 6 | 2 |
| | X = 8 | 2 |
| Longitudinal Translation:  Plan View | X = 3 | 2 |
| | X = 5 | 2 |
| | X = 7 | 2 |
| | X = 9 | 2 |
| Horizontal Skew:  Plan View | X = 16 | 2 |
| | X = 20 | 2 |
| | X = 24 | 2 |
| | X = 28 | 2 |
| | TOTAL SAMPLES: | 35 |

The aligned specimens were labeled A-X where X is either 1, 2, or 3 as the specimen number. All other samples were labeled using a series of letters and numerals separated by hyphens (e.g., XX-X-X). The first part represented the misalignment configuration (e.g., VT, VS, LT, and HS for vertical translation, vertical skew, longitudinal translation, and horizontal skew, respectively). The second part represented the misalignment magnitude in inches. The third part represented the sample's serial number when multiple samples were made from the same mix.

## 3.2.1 Aligned Specimens

Three aligned specimens were constructed. Each of the specimens contained an ideally placed tie bar. According to SDDOT's Concrete Paving Manual (South Dakota DOT, 2016), an ideally placed tie bar is located at the mid-depth of the slab and aligned perpendicular to the longitudinal joint (Figure 3.2).

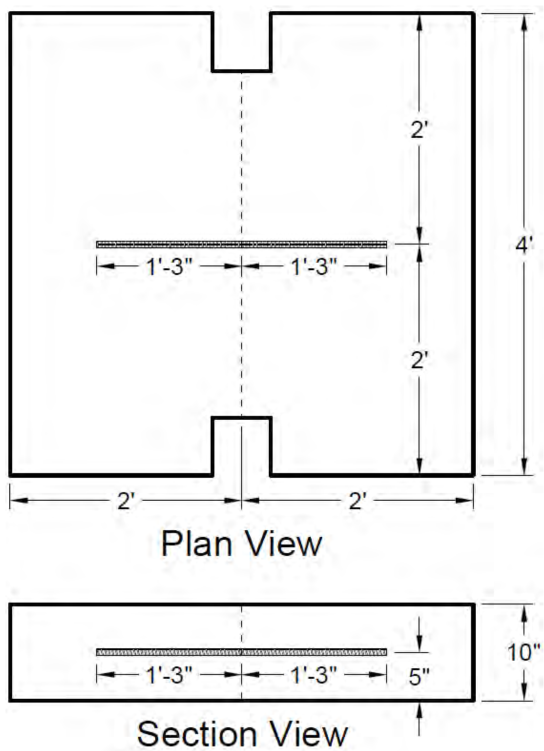



**Figure 3.2** Aligned Tie Bar Specimen Details

## 3.2.2 Vertical Translation

Eight vertical translation specimens were constructed. The specimens consisted of four different misalignment magnitudes with two samples of each. The four misalignment magnitudes selected for the vertical translation specimens had vertical offsets of 1 in. (VT-1), 2 in. (VT-2), 3 in. (VT-3), and 4 in. (VT-4) upwards from the position of an ideally placed tie bar (Figure 3.3).

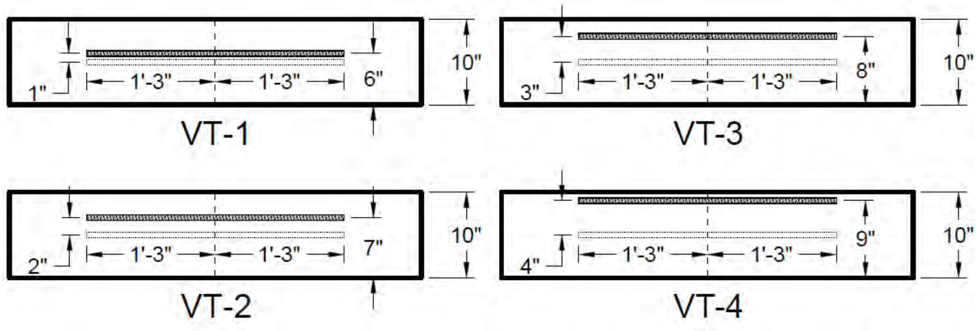



**Figure 3.3** Section Views of the Vertical Translation Misalignment Specimens

### 3.2.3 Vertical Skew

Eight vertical skew specimens were constructed. The specimens consisted of four different misalignment magnitudes with two samples of each. The four misalignment magnitudes, measured as the vertical offset between the two ends of the tie bar, were 2 in. (VS-2), 4 in. (VS-4), 6 in. (VS-6), and 8 in. (VS-8). Section views of the four vertical skew misalignment magnitudes and how they compare to an ideally placed tie bar are illustrated in Figure 3.4.

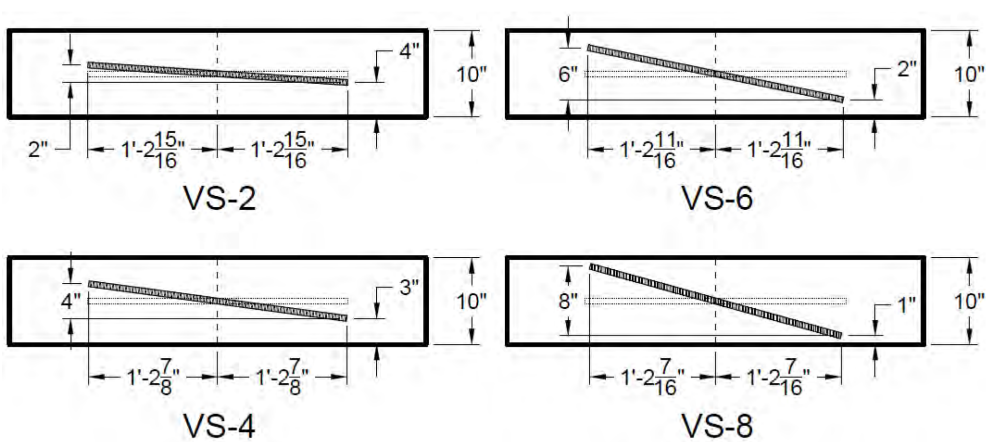



**Figure 3.4** Section Views of the Vertical Skew Misalignment Specimens

### 3.2.4 Longitudinal Translation

Eight longitudinal translation specimens were constructed. The specimens consisted of four different misalignment magnitudes with two samples of each. The four misalignment magnitudes selected for the longitudinal translation specimens had the tie bar embedment length on the stationary side of the longitudinal joint reduced by 3 in. (LT-3), 5 in. (LT-5), 7 in. (LT-7), and 9 in. (LT-9). Section views of the four longitudinal translation misalignment magnitudes and how they compare to an ideally placed tie bar are illustrated in Figure 3.5.

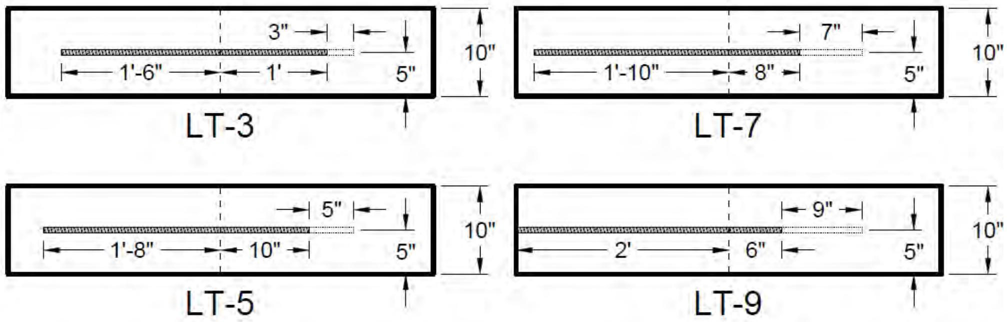



**Figure 3.5** Section Views of the Longitudinal Translation Misalignment Specimens

### 3.2.5 Horizontal Skew

Eight horizontal skew specimens were constructed. The specimens consisted of four different misalignment magnitudes with two samples of each. The four misalignment magnitudes, measured as the horizontal offset between the tie bar ends, were 16 in. (HS-16), 20 in. (HS-20), 24 in. (HS-24), and 28 in. (HS-28). Plan views of the four horizontal skew misalignment magnitudes and how they relate to an ideally placed tie bar are depicted in Figure 3.6.

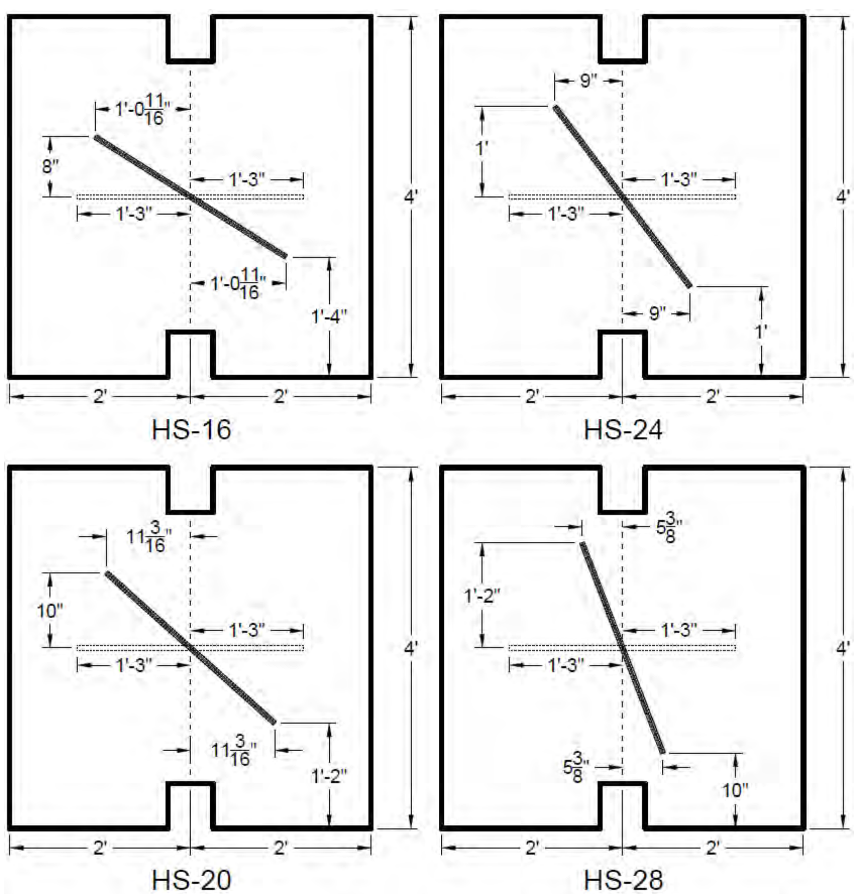



**Figure 3.6** Plan Views of the Horizontal Skew Misalignment Specimens

## 3.3 Construction of Specimens

The specimens were cast inside steel forms. Eight forms were fabricated to allow using the same concrete batch for the casting specimens of the same alignment configuration. Each steel form consisted of ½-in. thick A36 steel plate bottom and C10 x 15 structural steel channel sides. The joint between the two concrete slabs was created by a 1/8-in. thick acrylic sheet. The acrylic sheet had a hole to allow for the passage of the tie bar. The acrylic sheet, which was left in place throughout the testing, provided a completely cracked section condition. The acrylic sheet was held in place by means of two 6-in. x 6-in. x 10-in. wooden boxes placed on either end of the sheet. The two wooden boxes were also used to hold polyvinyl chloride (PVC) sleeves in place and to create block outs. The PVC sleeves and block outs were needed to facilitate the testing setup. The tie bar was held in place using a tie bar support assembly. The tie bar support assembly consisted of a C3 x 5 structural steel channel and a ¼-in. diameter threaded steel rod that was bent into a "U" shape. The threaded steel rod was fastened to the C3 x 5 structural steel channel, which spanned across the top of the forms. The tie bar was then secured to the "U" of the threaded steel rod in its required position using zip ties. Figure 3.7 and Figure 3.8 show a dimensioned top view and a picture of the steel forms prior to placing concrete, respectively.

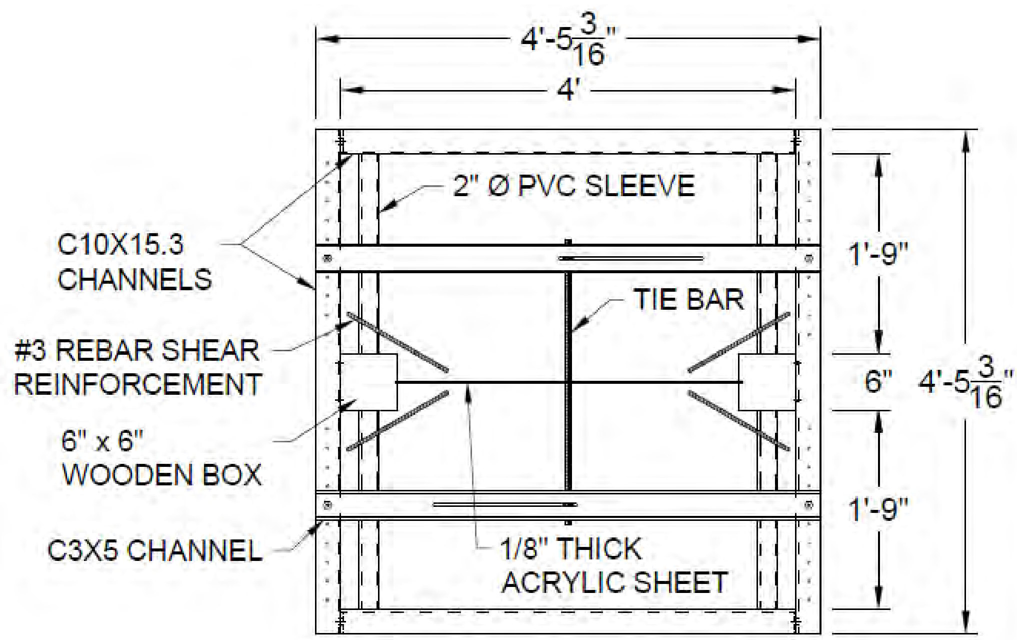



**Figure 3.7** Plan View of the Casting Form

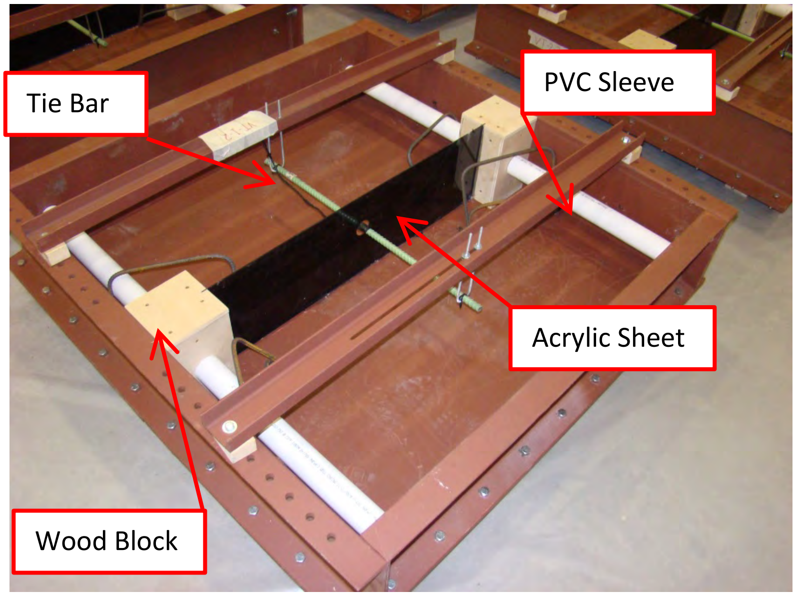



**Figure 3.8** Steel Casting Forms

The specimens for each alignment configuration were constructed and tested in two-week cycles. In the week prior to casting, the eight steel forms were assembled and the tie bars were secured in the positions defined by the testing matrix. On the day of the concrete pour, the fresh concrete properties (temperature, air content, and slump) were checked to ensure that the concrete was within the specified ranges for the well-graded PCC pavement mix. Following the measurement of fresh properties, all eight specimens along with eighteen 6-in. x 12-in. concrete cylinders and eight 6-in. x 6-in. x 22-in. concrete beams were cast (Figure 3.9).


**Figure 3.9** Concrete Casting

The specimens, cylinders, and beams were all cured for three days while covered with wet burlap and plastic sheets. The forms were stripped one day after concrete casting. On the fifth day after casting, the custom steel LVDT brackets were installed on the specimens using a hammer drill and masonry screws.

The two C10 x 15 structural steel side channels were reinstalled, on the sixth day after casting, to allow the samples to be moved over to the testing position without being damaged.

## 3.4 Instrumentation

Each of the 35 specimens was instrumented with strain gauges and linear variable displacement transducers (LVDT) to measure strain in the tie bar and the relative displacement between the two sides of the concrete slab across the joint.

Three Vishay CEA-06-250UN-350 strain gauges were installed on the tie bar at the location where the tie bar crosses the joint. The three strain gauges were attached to the surface of the tie bar, 120 degrees apart around the circumference of the tie bar. The strain gauges arrangement allowed for identification of the location on the circumference where yielding initiates. The orientation of the strain gauges on the tie bar circumference is shown in Figure 3.10.

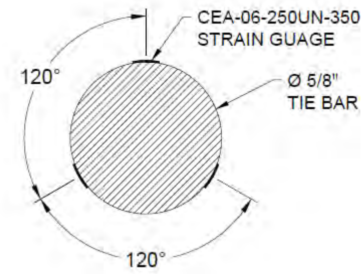



**Figure 3.10** Placement of the Strain Gauges

Six LVDTs were mounted to the top of each specimen using custom steel brackets (Figure 3.11). Three LVDTs were mounted on each end of the joint to allow for measuring the relative displacement of the two slab segments across the joint in three orthogonal directions and to calculate rotations and twisting about the joint.

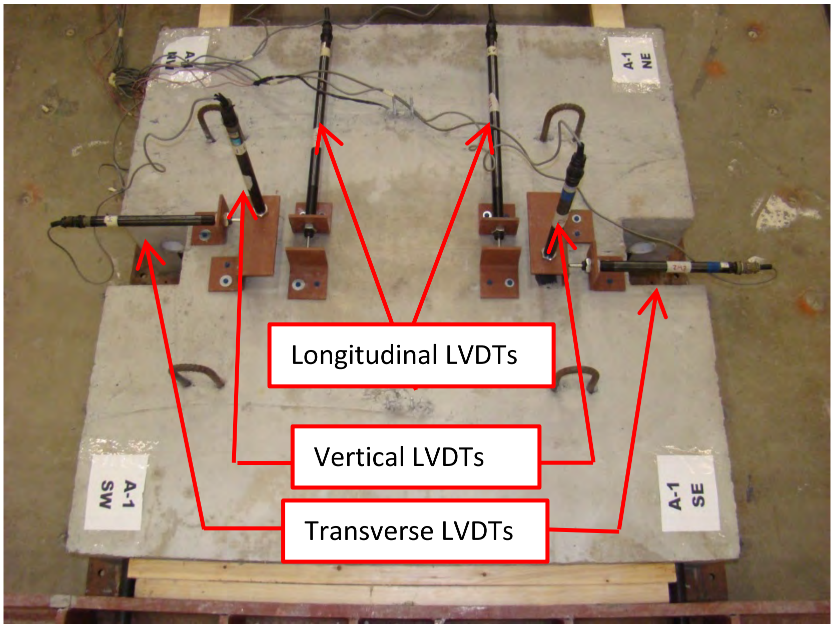


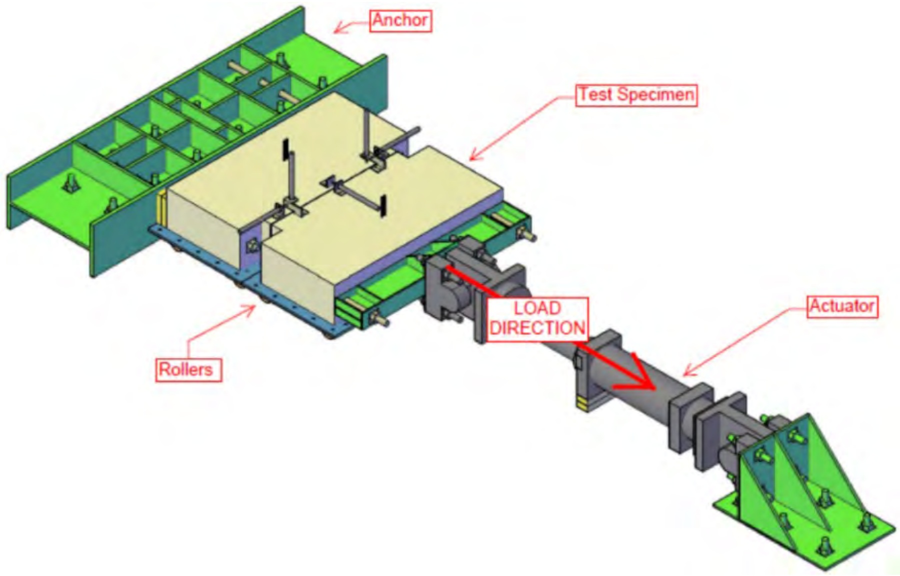

**Figure 3.11** LVDT Arrangement

The longitudinal LVDTs measure the relative joint opening between the two concrete slab segments parallel to the direction of the applied tensile force. The transverse LVDT's measure the relative joint slippage between the two concrete slabs perpendicular to the direction of the applied force. The vertical LVDTs measure the relative joint faulting that occurs between the two concrete slabs in the vertical direction. In the following chapters, the relative displacement readings from the longitudinal, transverse, and vertical LVDTs will be referred to as the joint opening, joint slippage, and joint faulting, respectively.

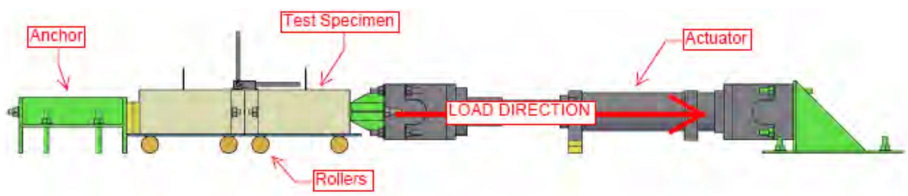
The data measured by the strain gauges and LVDTs were collected using the Vishay Micro-Measurements System 7000. It allowed for measurements to be recorded at a rate of 10 hertz throughout testing.

## 3.5 Testing Procedure

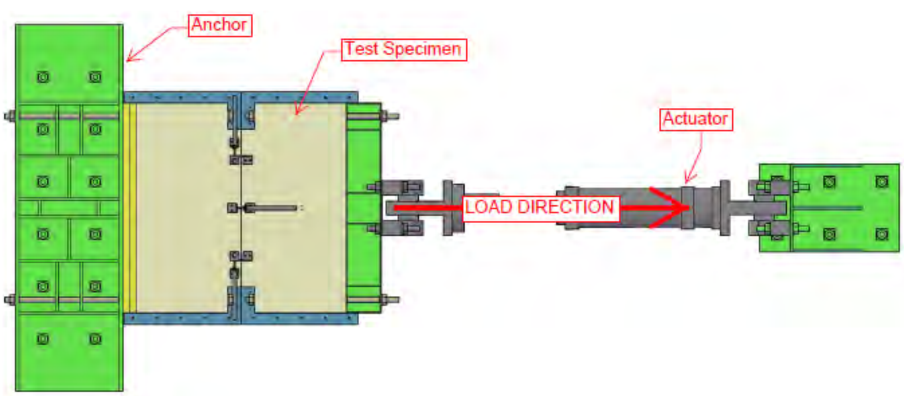
All specimens were tested seven days after pouring. The specimens were tested by securing one end to an anchor and the other end to a hydraulic actuator. The hydraulic actuator then applied a splitting force normal to the face of the joint until failure occurred. Figure 3.12 presents a rendering of the testing setup.




**Isometric View**




**Side View**




**Top View**

**Figure 3.12** Schematic Views of the Testing Setup

On testing day, a specimen was moved into the testing position and placed on a set of rollers. Once the specimen was in position, the stationary side of the specimen was secured to the anchor beam using two threaded rods. The two C10 x 15 structural steel side channels were then removed, and the six LVDTs were installed at their respective brackets. The connection to the anchor beam is shown in Figure 3.13.

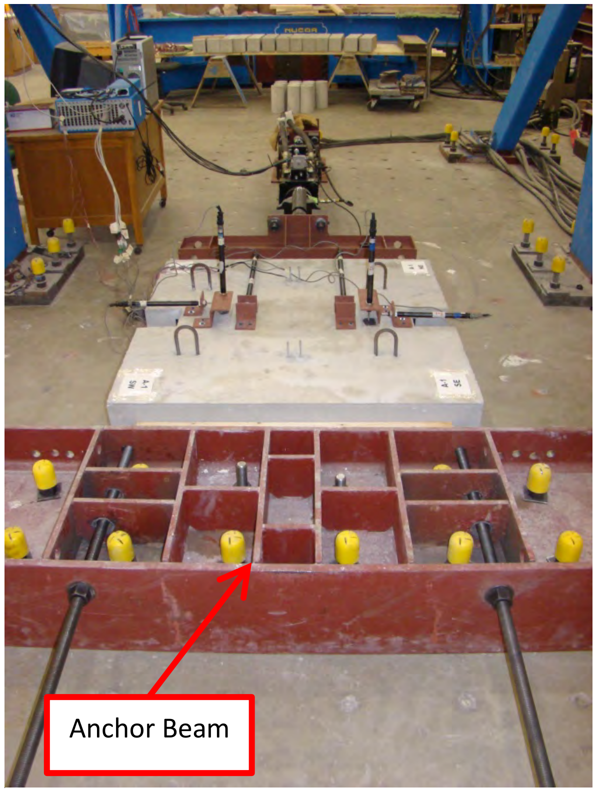



**Figure 3.13** Attachment of a Test Specimen to the Anchor Beam

The LVDT and strain gauge wires were then connected to the Micro-Measurement Data Acquisition System and an initial reading was taken for all of the strain gauges and LVDTs. With the initial reading taken, the hydraulic actuator could then be seated and connected to the specimen with two threaded rods (Figure 3.14).

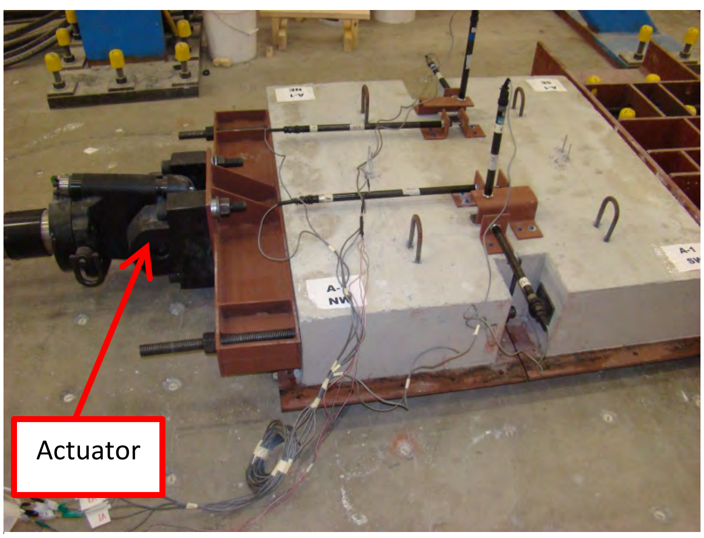



**Figure 3.14** Attachment of a Test Specimen to the Actuator

The specimen was tested by applying a splitting force normal to the face of the joint using the hydraulic actuator. The hydraulic actuator was operated in displacement control mode with intervals of 0.005 in. until the yielding of the tie bar. When the incremental displacement was being applied to the specimen, the data acquisition system was activated to record the strain and relative displacement data at a frequency of 10 Hz. All of the specimens were tested until bond failure occurred between the concrete and the tie bar. The typical mode of failure for the specimens was splitting of the concrete along the length of the tie bar.

The longitudinal joint width was evaluated when the measured strain in the tie bar reached $0.75\varepsilon_y$, where $\varepsilon_y$ is the yield strain corresponding to the tie bar yield stress, $f_y$. The $0.75\varepsilon_y$ threshold was established based on the allowable tie bar design force for a single tie bar given in AASHTO (AASHTO, 1993). According to AASHTO (1993), the allowable tie bar design force, $F_{TB}$, is calculated using Equation 4.

$$F_{TB} = f_s A_s = 0.75 f_y A_s \qquad \text{Equation 2}$$

The tie bars used in the experimental testing were grade 60 ($f_y$ = 60 ksi), #5 ($A_s$ =0.31 in$^2$) bars. Therefore, the allowable tie bar design force, $F_{TB}$, for a single tie bar is 13.95 kip.

The joint opening performance limit was based on the typical SDDOT sawed joint detail for longitudinal joints; the detail calls for a ¼-in. wide sawed joint, filled with a hot poured elastic joint sealer. This elastic joint sealer allows the joint to expand while remaining watertight. Based on the manufacturer’s specifications, the hot poured elastic joint sealers should be able to elongate at least 50% of their original lengths before bond failure occurs. This specification is approved by SDDOT (South Dakota DOT, 2016). The hot poured elastic joint sealers approved by SDDOT include 3405 Sealtight – Type II, Beram 195 – Type II, Roadsaver 221 – Type II, and Hi-Spec – Type II. The 50% elongations mean that 1/8 in. would be an acceptable performance limit for the joint opening to ensure that the hot poured elastic joint sealer is still able to keep the joint watertight.

Joint faulting and joint slippage were outside the scope of this study; therefore, no allowable limits were established for joint faulting and joint slippage. However, the measured data for the joint faulting and joint slippage were compared between the specimens, with aligned and misaligned tie bars.

Some issues were encountered during the testing of specimens with vertical translation misalignment. During testing of these specimens, the actuator load at the slab mid-height and the eccentric resisting force in the tie bar created a force couple about the horizontal axis of the joint plane. Figure 3.15 depicts a free body diagram of the initial forces acting on the joint plane.

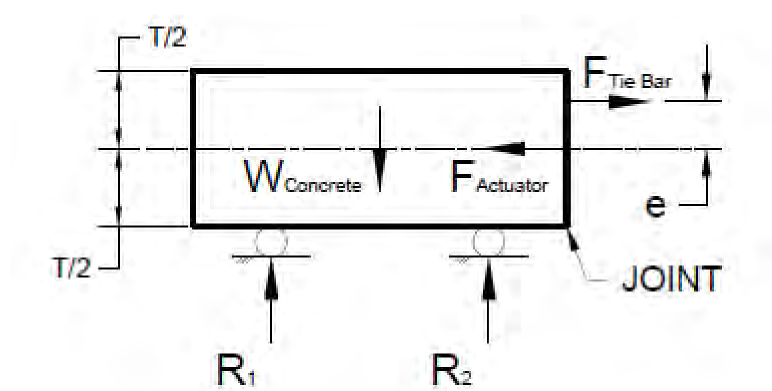



**Figure 3.15** Free Body Diagram of One Side of the Test Specimen

The couple created by the eccentric loading caused the joint plane to rotate, thus inducing compressive stresses in the concrete at the top of the joint. Figure 3.16 shows the rotation resulting from the eccentric loading. The couple tends to increase the tension force in the tie bar for a given applied actuator load.

**Figure 3.16** Slab Rotation during Testing of Specimen VT-4-1

Increasing the actuator load caused the joint rotation and compressive concrete stresses to increase to a level that induced concrete crushing and spalling at the top of the joint (Figure 3.17).

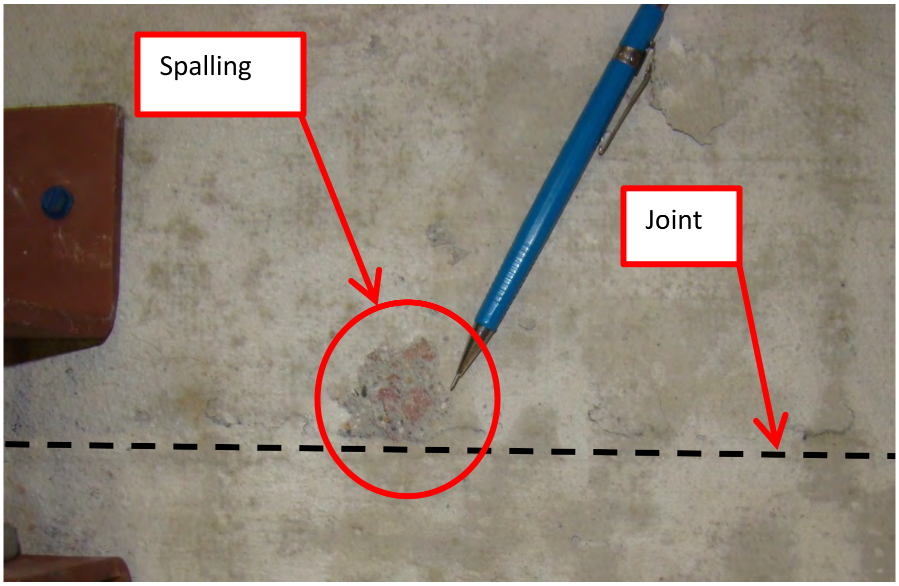



**Figure 3.17** Spalling of Compression Concrete at the Joint

As a result, only one specimen of each misalignment magnitude of the vertical translation specimens was tested under eccentric loading with unrestricted rotation. The specimens tested under unrestricted rotation were VT-1-1, VT-2-1, VT-3-1, and VT-4-1. Since in actual pavements, the weight of the slab will restrain rotation about the joint, it was decided to test the remaining four specimens (VT-1-2, VT-2-2, VT-3-2, and VT-4-2) under restrained rotation conditions.

Rotational restraint was achieved by installing two C10 x 15 side forms and two C3 x 5 top braces (Figure 3.18). The C10 x 15 provided resistance to the horizontal sliding of one side of the joint relative to the other. The C3 x 5 top braces provided resistance to slab uplift and thus to joint rotation. The sides of the steel channels in contact with the concrete were greased prior to installation in order to reduce the frictional stresses. The LVDTs measuring relative transverse displacement were removed from this test setup to allow for the installation of the C10 x 15 side forms.

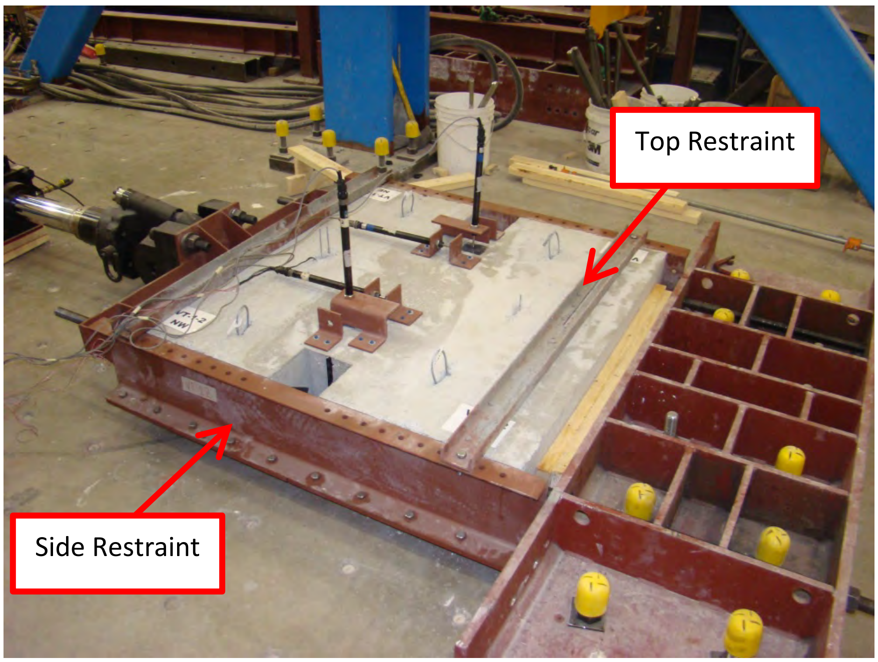



**Figure 3.18** Testing Setup of a Specimen Restrained against Joint Sliding and Rotation

The restrained joint against sliding and rotation test setup was adopted for the remainder of the specimens with the vertical skew, longitudinal translation, and horizontal skew alignment configurations.

# 4. EXPERIMENTAL RESULTS AND ANALYSIS

This chapter provides a summary of the measured data and analysis of the experimental results. A simple and conservative analytical model for determining the force corresponding to $0.75\varepsilon_y$ in a horizontally skewed tie bar is also presented.

## 4.1 Material Properties

Table 4.1 shows the measured fresh concrete properties for all five batches of concrete used to make the test specimens. The measured air content and slump for all five batches were within the SDDOT specified limits.

**Table 4.1** Measured Fresh Concrete Properties

| Alignment Configuration | Temperature, °F | Air Content, % | Slump, in. |
|---|---|---|---|
| Aligned | 85 | 7.5 | 2.50 |
| Vertical Translation | 62 | 6.2 | 2.50 |
| Vertical Skew | 63 | 5.0 | 2.75 |
| Longitudinal Translation | 81 | 5.7 | 2.75 |
| Horizontal Skew | 80 | 6.7 | 2.75 |

Table 4.2 shows a summary of the hardened concrete properties (see Appendix B for the complete fresh and hardened concrete properties for each concrete batch).

**Table 4.2** Measured Hardened Concrete Properties

| Alignment Configuration | Concrete Cure Time | Compressive Strength, psi | Modulus of Elasticity, ksi | Flexural Strength, psi | Split Tensile Strength, psi |
|---|---|---|---|---|---|
| Aligned | 3 Day | 2740 | - | - | - |
| | 7 Day * | 3973 | 3780 | 435 | 482 |
| | 28 Day | 4785 | 4010 | 647 | 434 |
| Vertical Translation | 3 Day | 4562 | - | - | - |
| | 7 Day * | 5357 | 4600 | 460 | 565 |
| | 28 Day | 6635 | 4810 | 699 | 629 |
| Vertical Skew | 3 Day | 4383 | - | - | - |
| | 7 Day * | 5261 | 4590 | 644 | 478 |
| | 28 Day | 6216 | 4890 | 738 | 567 |
| Longitudinal Translation | 3 Day | 4297 | - | - | - |
| | 7 Day * | 5241 | 4790 | 502 | 490 |
| | 28 Day | 6320 | 4320 | 608 | 553 |
| Horizontal Skew | 3 Day | 4103 | - | - | - |
| | 7 Day * | 5297 | 4440 | 486 | 488 |
| | 28 Day | 6384 | 4510 | 776 | 563 |

*All specimens were tested after seven days of curing

Figure 4.1 shows the measured stress-strain of a tie bar sample. The measured yield strength, ultimate strength, and the modulus of elasticity were 74 ksi, 124 ksi, and 29000 ksi, respectively.

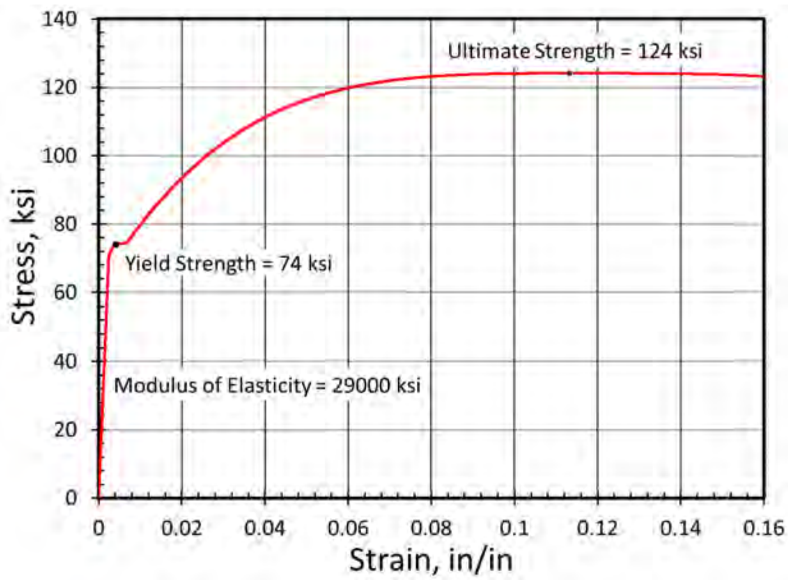



**Figure 4.1** Measured Stress-Strain Relationship for Tie Bar Material

## 4.2 Experimental Results of the Test Specimens

### 4.2.1 Aligned Specimens

In all three aligned specimens, the tie bar yielded prior to bond failure. Bond failure occurred by splitting of the concrete along the length of the tie bar. The longitudinal crack and bond failure of specimen A-1 are shown in Figure 4.2 and Figure 4.3, respectively.

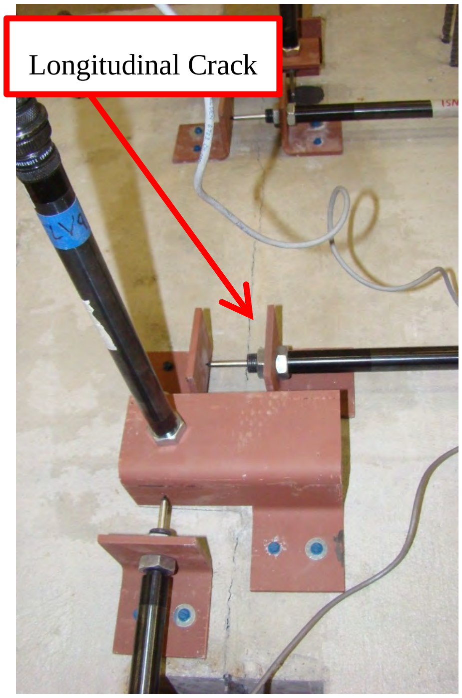



**Figure 4.2** Longitudinal Crack on Specimen A-1 at a Tie Bar Strain of $0.75\varepsilon_y$

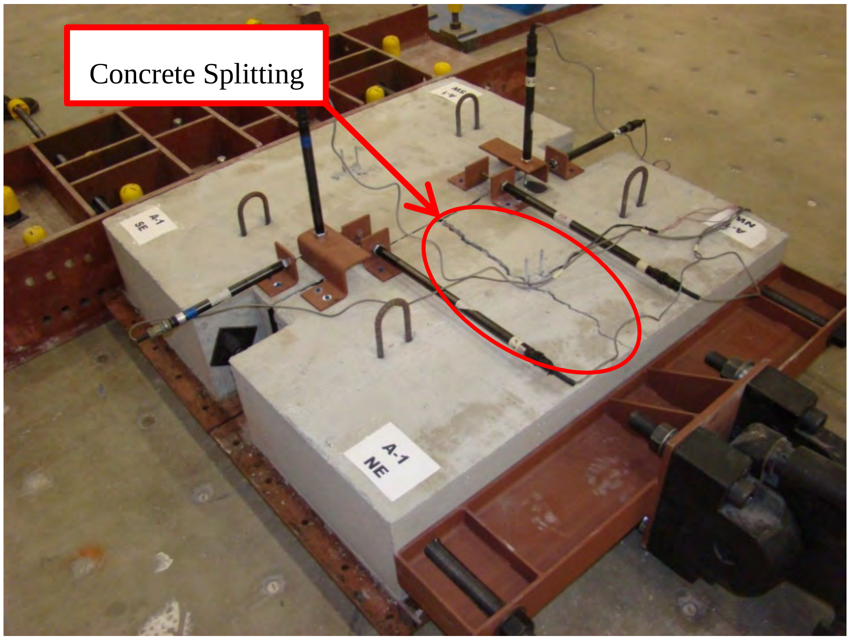



**Figure 4.3** Bond Failure of Specimen A-1

Plots of the measured tie bar strain, joint opening, joint slippage, and joint faulting, versus applied actuator load, are shown in Figure 4.4. The actuator load-tie bar strain relationships for the three aligned specimens were almost identical. The joint opening increased as the actuator load increased, with the exception of specimen A-2. The joint opening values for specimen A-2 increased initially with the actuator load until a load of 12 kip, after which the joint opening values began decreasing. The probable explanation is that some rotation about the joint must have occurred, causing the joint at the top of the slab to begin closing. There was no significant joint slippage and faulting for any of the three aligned specimens.

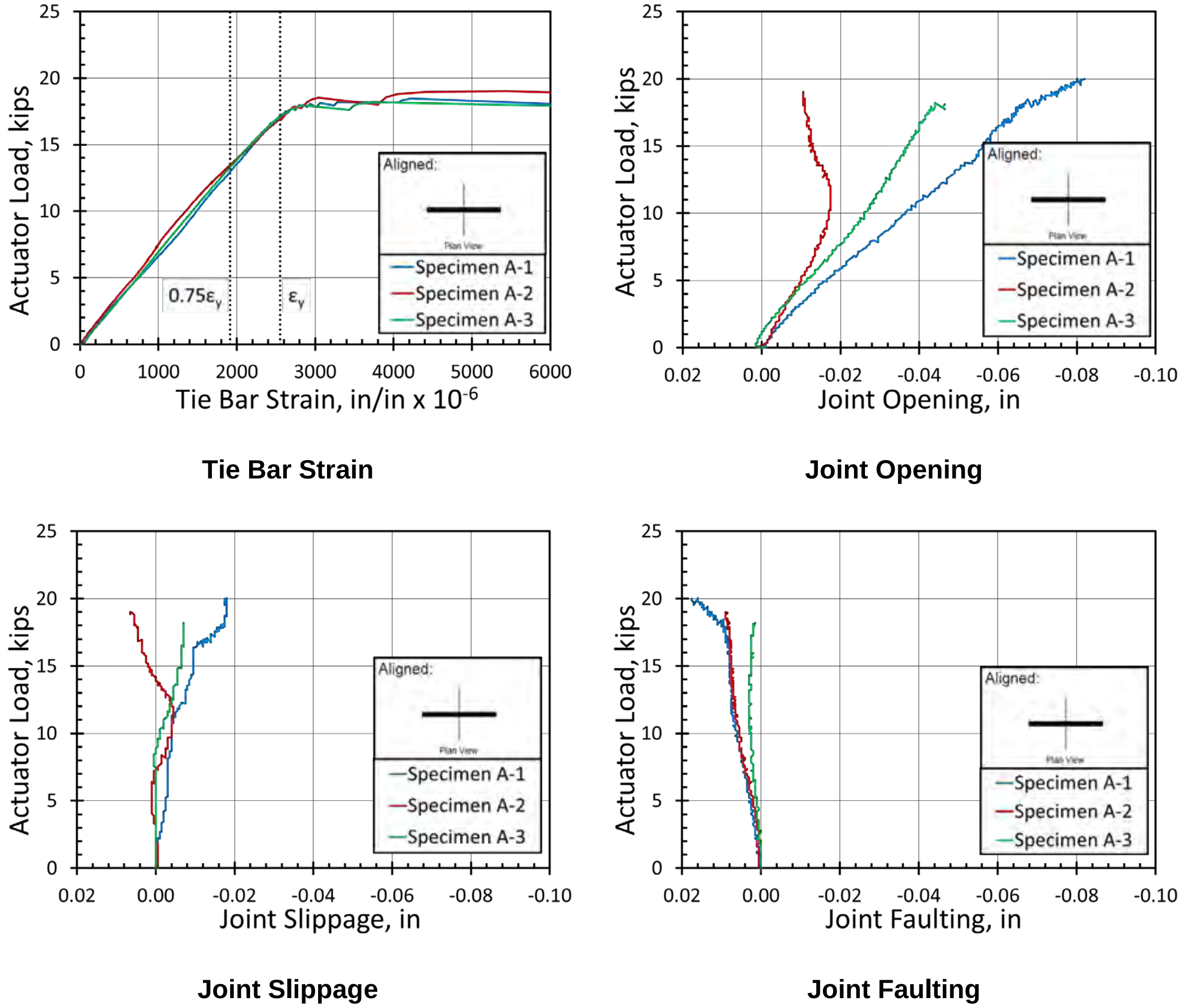



**Figure 4.4** Testing Results for the Aligned Specimens

Table 4.3 presents a summary of the actuator load and the corresponding relative displacement results at the point when the first measured tie bar strain reached $0.75\varepsilon_y$.

**Table 4.3** Measured Hardened Concrete Properties

| Specimen | At First Measured Tie Bar Strain Equal to $0.75\varepsilon_y$ | | | |
|---|---|---|---|---|
| | Actuator Load, kip | Joint Opening, in. | Joint Slippage, in. | Joint Faulting, in. |
| A – 1 | 12.8 | -0.048 | -0.008 | 0.008 |
| A – 2 | 13.5 | -0.016 | -0.002 | 0.007 |
| A – 3 | 13.2 | -0.033 | -0.005 | 0.003 |
| AVERAGE | 13.2 | -0.032 | -0.005 | 0.006 |

The average actuator load required to cause a tie bar strain of $0.75\varepsilon_y$ for the aligned specimens is 13.2 kip. In addition, the 13.2 kip is slightly less than the allowable 13.95 kip tie bar design force for a single tie bar. The slight difference between the values is likely due to the fact that the actuator load is recorded when the first strain gauge reading reaches $0.75\varepsilon_y$. The actuator load of 13.2 kip will be used as the baseline for comparison with other alignment configurations.

The joint movements in all three dimensions at $0.75\varepsilon_y$ were extremely small. The average joint opening for the three aligned specimens was well below the 1/8-in. performance limit. The joint slippage and joint faulting values were nearly negligible.

### 4.2.2 Vertical Translation

Specimens VT-1-1, VT-2-1, VT-3-1, and VT-4-1 were not restrained against joint rotation, resulting in an eccentric loading condition. Joint rotation of specimen VT-4-1 and bond failure of specimen VT-3-1 are shown in Figure 4.5 and Figure 4.6.

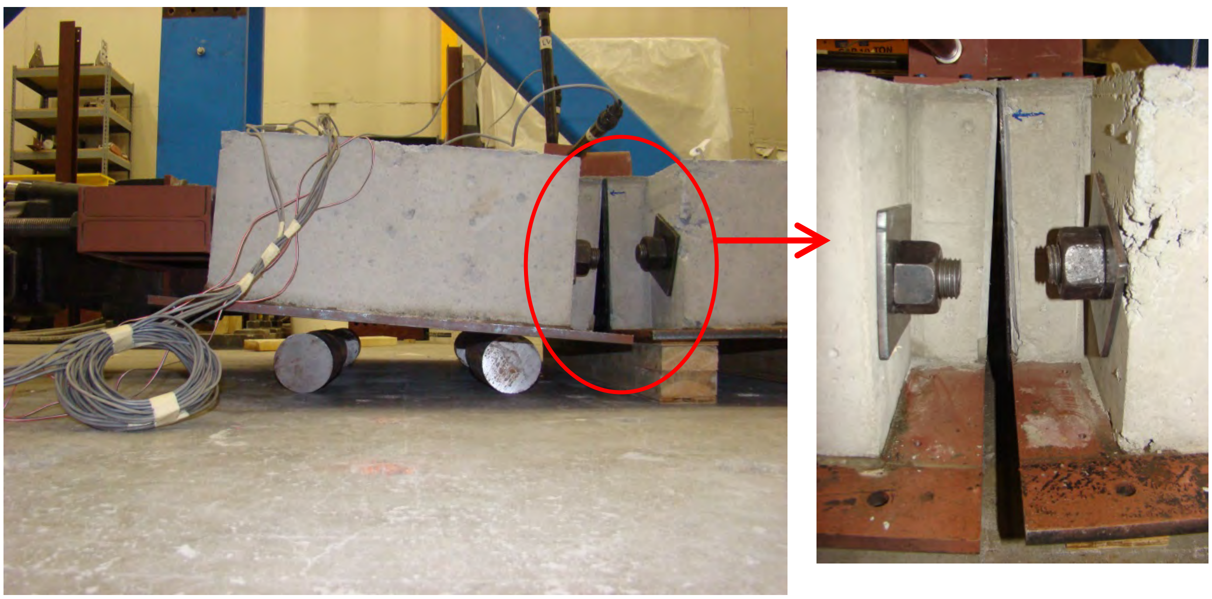


**Figure 4.5** Joint Rotation of Specimen VT-4-1

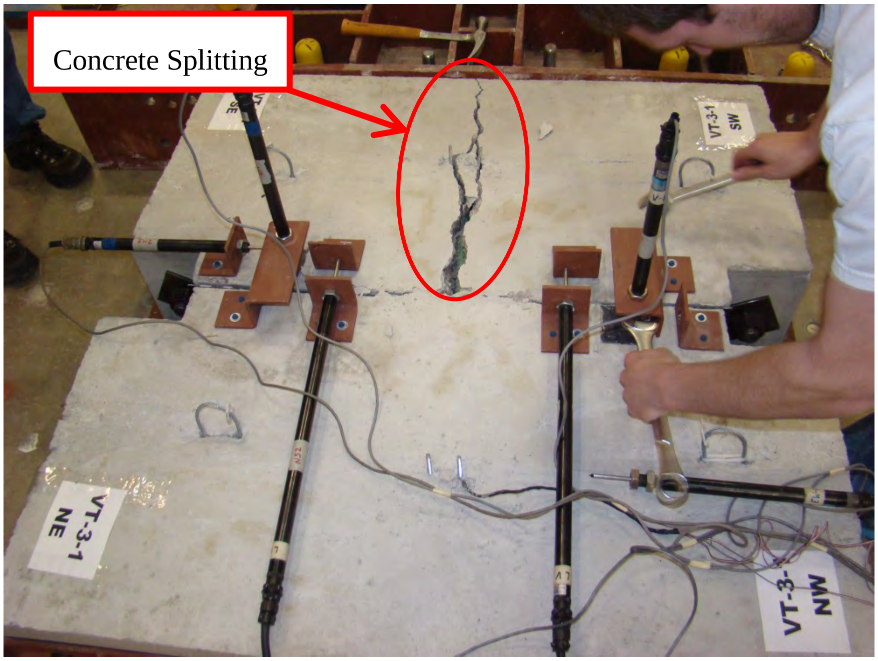



**Figure 4.6** Bond Failure of Specimen VT-3-1

The remaining four specimens, VT-1-2, VT-2-2, VT-3-2, and VT-4-2, were restrained to prevent joint rotation. Restrained specimens during testing were documented through photography (Figure 4.7 and Figure 4.8).

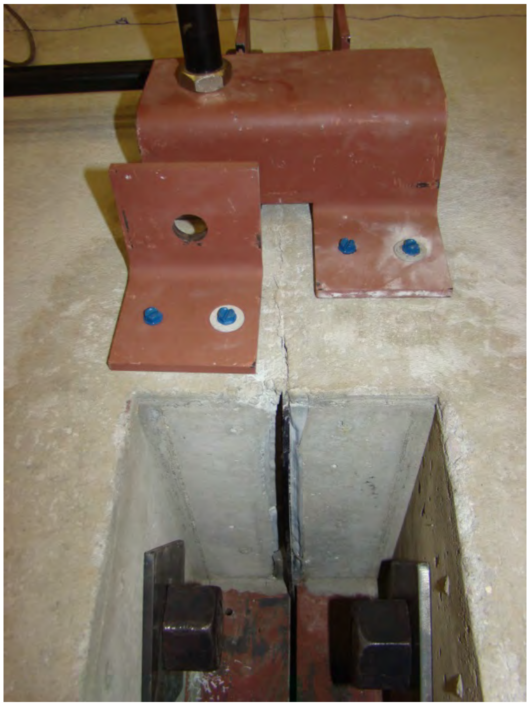


**Figure 4.7** Reduced Joint Rotation of Specimen VT-4-2

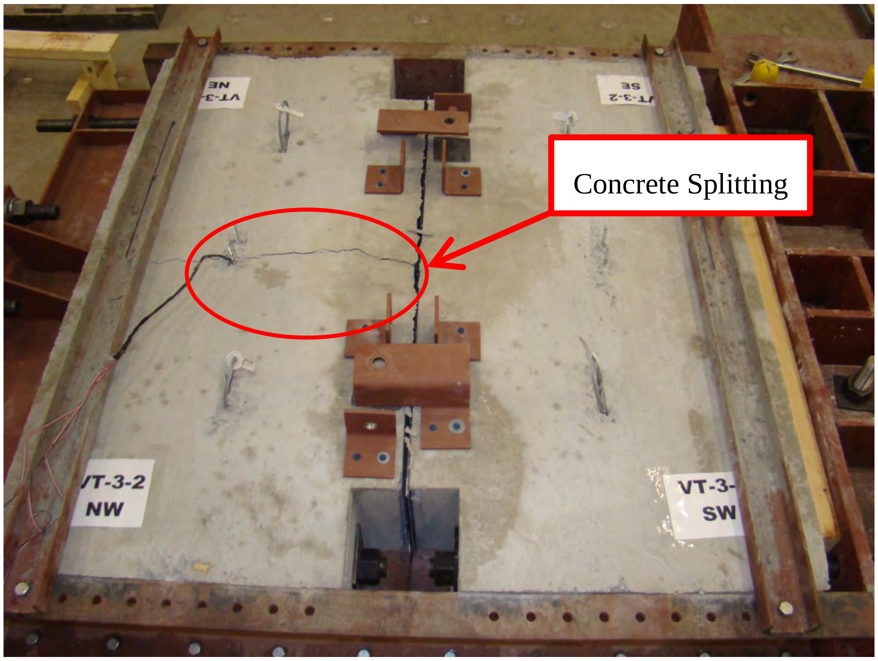


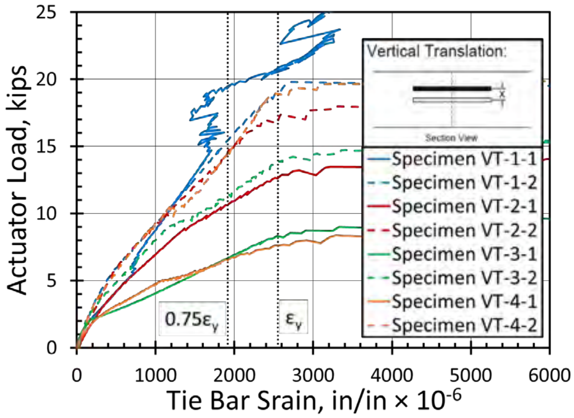

**Figure 4.8** Bond Failure of Specimen VT-3-2

The LVDTs measuring the joint slippage were removed from the restrained specimen setups since slippage was restricted. A comparison between Figure 4.5 and Figure 4.7 indicates a significant reduction in joint rotation in the restrained specimens.

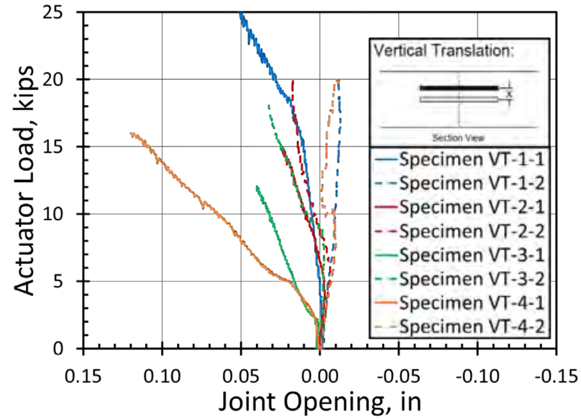
Measured tie bar strain, joint opening, and joint faulting, versus applied actuator load, were plotted (Figure 4.9). With the exception of the 1-in. offset case (VT-1-1 and VT-1-2), the strain in the tie bars of the unrestrained specimens (VT-2-1, VT-3-1, and VT-4-1) were much higher than the tie bar strains in the respective restrained specimen (VT-2-2, VT-3-2, and VT-4-2) for a given actuator load. Those results indicate that the eccentric loading condition and the resulting moment increase the tensile stress in the tie bar for a given actuator load. Since the unrestrained condition is not representative of real pavement conditions, the unrestrained test should not be used to draw conclusions regarding the effect of the vertical translation alignment configuration. The presence of a moment resulting from eccentric loading conditions is also apparent. The unrestrained specimens with 3-in. and 4-in. offsets show high positive joint opening values. Positive joint opening values indicate that the longitudinal joint is closing.

With the exception of specimen VT-3-2, the restrained specimens showed almost identical tie bar strain development up to $0.75\varepsilon_y$ (Figure 4.9). Specimen VT-3-2 exhibited higher tie bar strain values than the other three specimens. Moreover, the tie bar strain in specimen VT-2-2 past $0.75\varepsilon_y$ increased at a higher rate than the strain in specimens VT-1-2 and VT-4-2. The inconsistency in the tie bar strain results in specimen VT-3-2 and VT-2-2 can be explained by examining Figure 4.9. The figure shows that specimens VT-1-2 and VT-4-2 exhibited negligible joint opening throughout the test, while specimens VT3-2 and VT-2-2 exhibited joint closing at the top of the joint past an actuator load of approximately 10 kip. This indicates that specimens VT-3-2 and VT-2-2 were not properly restrained. Excluding the results from specimen VT-1-2, which had the lowest bar offset, it seems probable that the vertical translation has little to no effect on the actuator load required to induce the given tie bar strain. The joint openings for the restrained specimens appear to be small and unaffected by vertical translation.

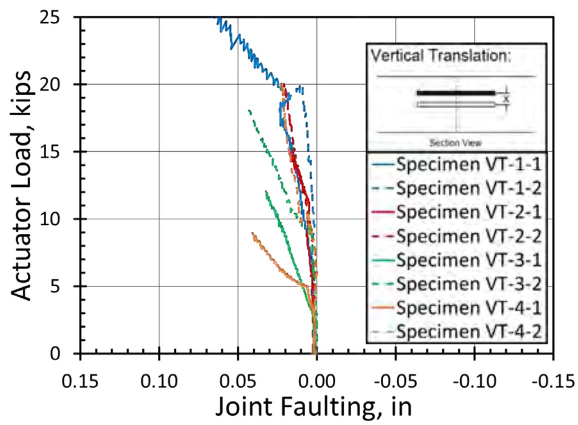
The joint faulting values for the vertical translation specimens (Figure 4.9) indicate that the joint faulting was minimal until after the tie bar strain had exceeded $0.75\varepsilon_y$, regardless of the restraining conditions.




**Tie Bar Strain**




**Joint Opening**




**Joint Faulting**

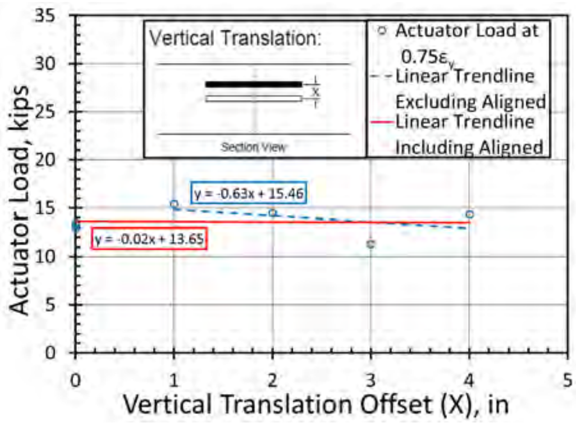
**Figure 4.9** Testing Results for the Vertical Translation Specimens

The actuator load and relative displacement testing results at the point when the first measured tie bar strain reaches $0.75\varepsilon y$ are summarized in Table 4.4.

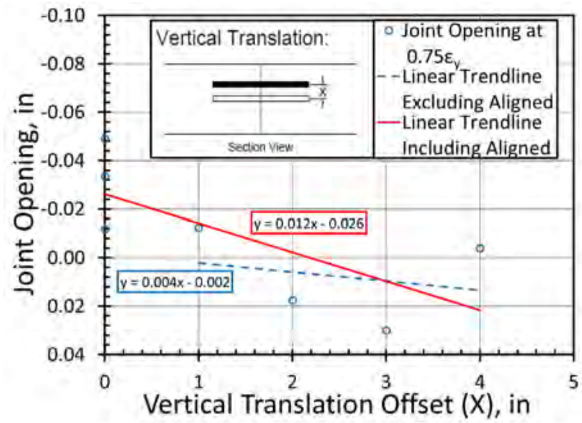
**Table 4.4** Actuator Load and Joint Opening, Slippage and Faulting for the Vertical Translation Specimens

| Specimen | At First Measured Tie Bar Strain Equal to $0.75\varepsilon_y$ | | | |
|---|---|---|---|---|
| | Actuator Load, kip | Joint Opening, in. | Joint Slippage, in. | Joint Faulting, in. |
| VT – 1 – 1 | 19.3 | 0.027 | -0.002 | 0.021 |
| VT – 1 – 2 | 15.4 | -0.013 | - | 0.006 |
| VT – 2 – 1 | 10.6 | 0.011 | -0.001 | 0.005 |
| VT – 2 – 2 | 14.5 | 0.018 | - | 0.014 |
| VT – 3 – 1 | 6.6 | 0.022 | 0.001 | 0.014 |
| VT – 3 – 2 | 11.3 | 0.030 | - | 0.018 |
| VT – 4 – 1 | 6.5 | 0.040 | 0.040 | 0.026 |
| VT – 4 – 2 | 14.4 | -0.004 | - | 0.017 |

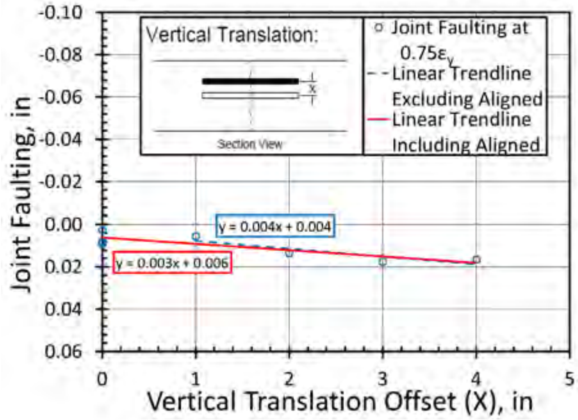
The effect of vertical translation on the actuator load, joint opening, and joint faulting at $0.75\varepsilon_y$ is documented in Figure 4.10. The plots in Figure 4.10 exclude the data from the unrestrained specimens. It is clear that the vertical translation does not induce significant performance variation from the ideally placed tie bar (aligned). The average actuator load at $0.75\varepsilon_y$ for the vertical translation specimens is 13.9 kip, which is 1.05 times that for the aligned specimens. The change in joint opening/closing and faulting is negligible. The joint opening remains well below the 1/8-in. performance limit.




**Actuator Load**




**Joint Opening**




**Joint Faulting**

**Figure 4.10** Effect of Vertical Translation Offset on Joint Parameters

### 4.2.3 Vertical Skew

The eight vertical skew specimens were tested while restrained against rotation. In all specimens, the tie bar yielded prior to bond failure. Bond failure occurred by splitting of the concrete along the length of the tie bar. Figure 4.11 and Figure 4.12 show the test specimen and the dowel bar at bond failure.

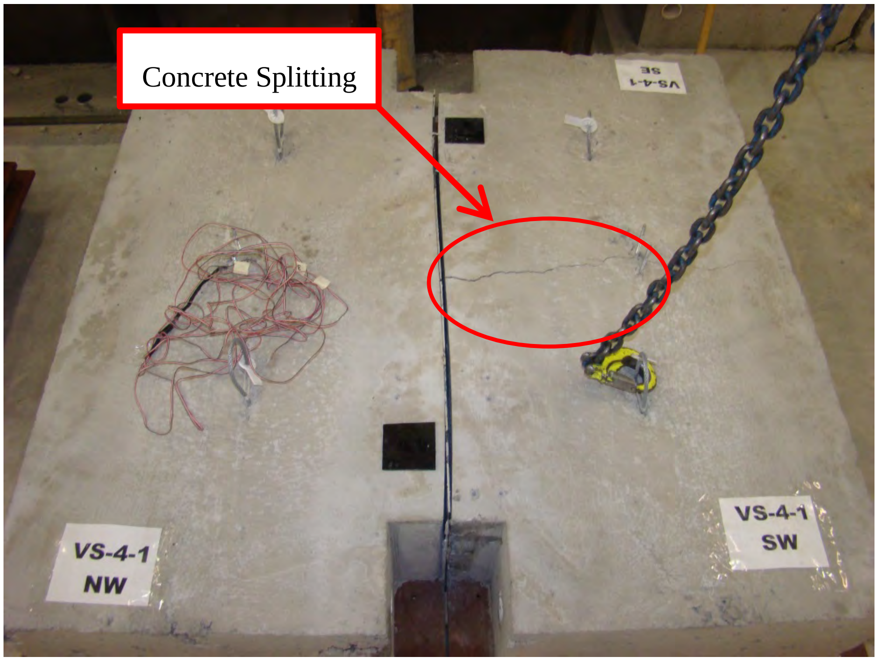
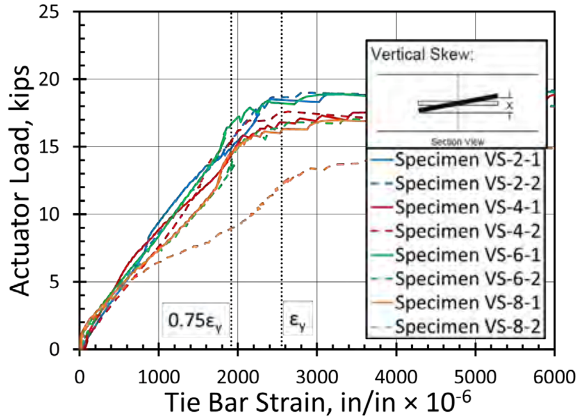


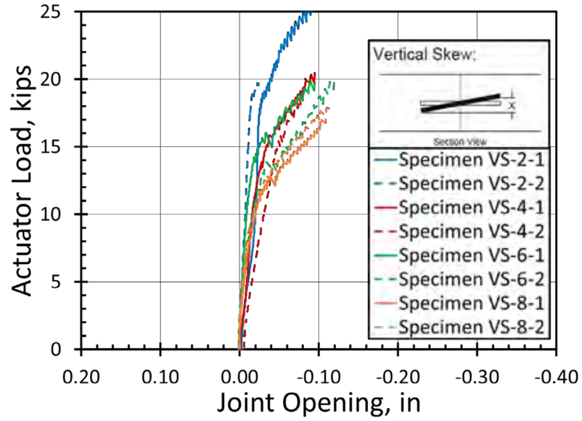

**Figure 4.11** Bond Failure of Specimen VS-4-1

**Figure 4.12** Specimen VS-4-1 Dowel Bar at the Failure Plane

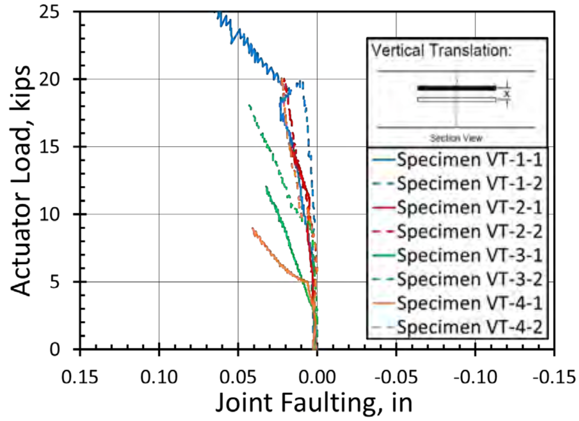
Figure 4.13 shows the measured tie bar strain, joint opening, and joint faulting versus applied actuator load. Excluding specimen VS-4-2, the development of tensile tie bar strain in the tie bar was not significantly affected by the magnitude of the vertical skew. The results indicate, however, that joint slippage and faulting increase with an increase in misalignment magnitude.




**Tie Bar Strain**




**Joint Opening**




**Joint Faulting**

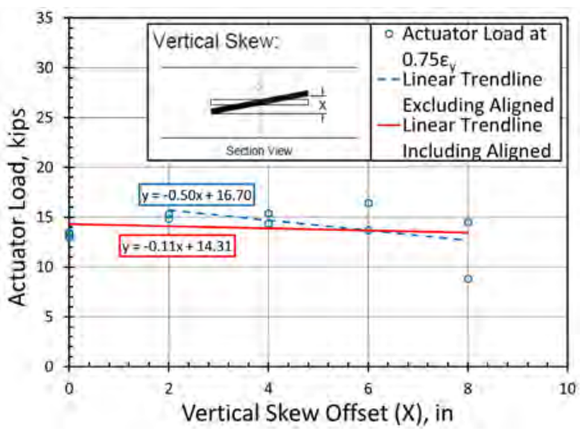
**Figure 4.13** Testing Results for the Vertical Skew Specimens

The actuator load and relative displacement testing results at the point when the first measured tie bar strain reached 0.75εy are summarized in Table 4.5.

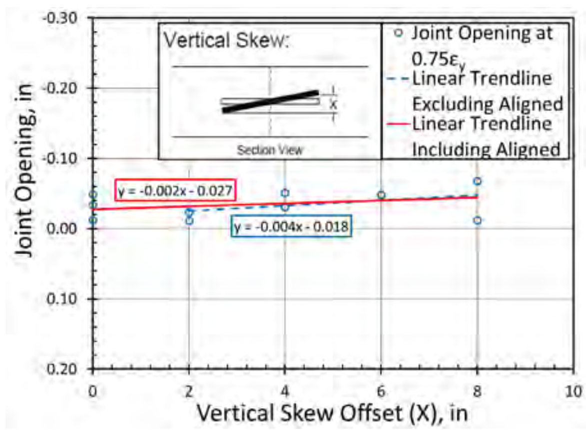
**Table 4.5** Actuator Load and Joint Opening and Faulting for the Vertical Skew Specimens

| Specimen | At First Measured Tie Bar Strain Equal to $0.75\varepsilon_y$ | | |
|---|---|---|---|
| | Actuator Load, kip | Joint Opening, in. | Joint Faulting, in. |
| VS – 2 – 1 | 14.9 | -0.024 | 0.058 |
| VS – 2 – 2 | 15.3 | -0.012 | 0.017 |
| VS – 4 – 1 | 14.4 | -0.031 | 0.138 |
| VS – 4 – 2 | 15.4 | -0.051 | 0.065 |
| VS – 6 – 1 | 16.4 | -0.048 | 0.127 |
| VS – 6 – 2 | 13.7 | -0.048 | 0.115 |
| VS – 8 – 1 | 14.5 | -0.068 | 0.132 |
| VS – 8 – 2 | 8.8 | -0.012 | 0.171 |

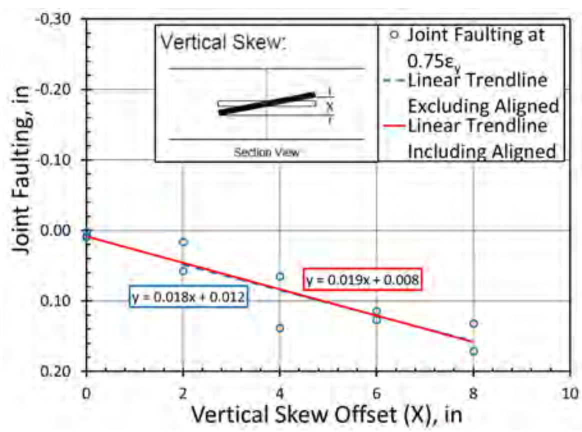
The effect of vertical skew on the actuator load, joint opening, and joint faulting at $0.75\varepsilon_y$ is shown in Figure 4.14. With the exception of one of the VS-8 specimens, the magnitude of the vertical skew had no significant effect on the actuator load at a tie bar strain of $0.75\varepsilon_y$. The average actuator load at $0.75\varepsilon_y$ for the misaligned specimens is 14.2 kip, which is 1.08 times that for the aligned specimens. The joint opening increased slightly as the misalignment magnitude increased, but the maximum measured joint opening was 0.068 in. (VS-8-1), which is still well below the allowable joint opening limit of 1/8 in. As the vertical skew increased, the joint faulting also increased. At a vertical skew magnitude of 8 in., the average measured joint faulting was 0.151 in., which is 25 times the average joint faulting experienced by the aligned specimens.




**Actuator Load**




**Joint Opening**




**Joint Faulting**

**Figure 4.14** Effect of Vertical Skew Offset on Joint Parameters

## 4.2.4 Longitudinal Translation

The eight longitudinal translation specimens were tested while restrained against rotation, as shown in Figure 4.14. All specimens failed due to bond failure after the tie bar had yielded. Bond failure occurred by splitting of the concrete along the length of the tie bar, as shown in shown in Figure 4.15 and Figure 4.16.

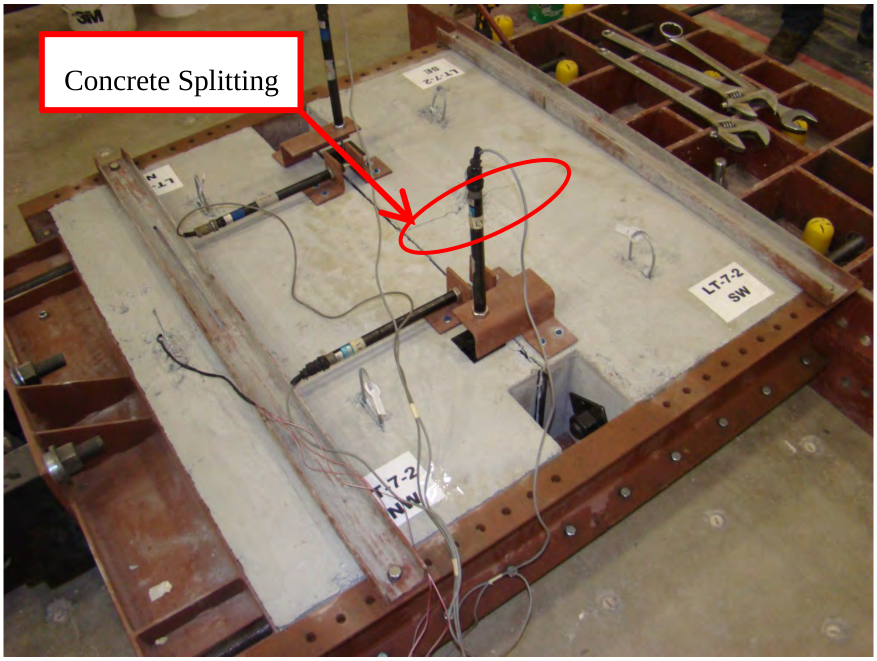
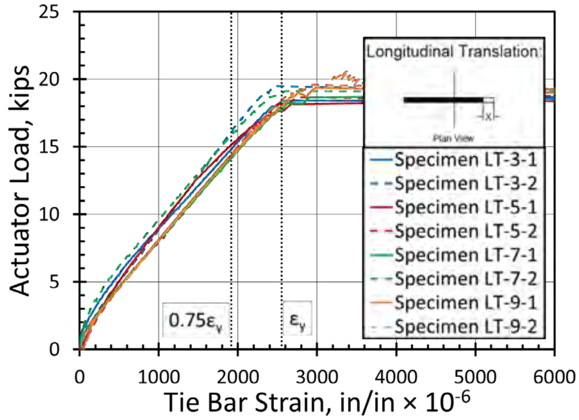


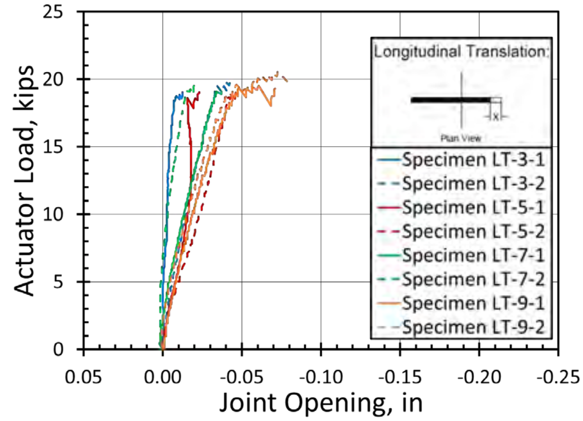

**Figure 4.15** Bond Failure Specimen LT-7-2

**Figure 4.16** Specimen LT-7-2 Failure Plane

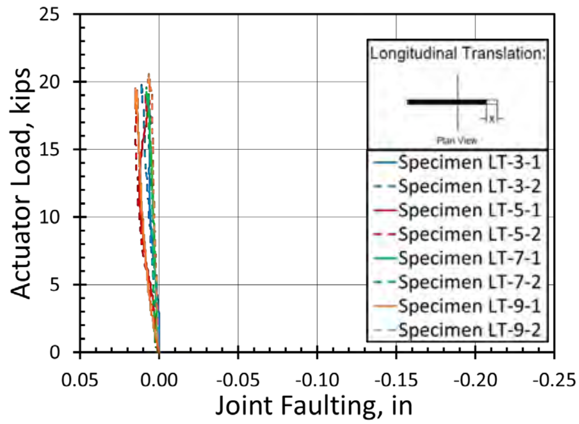
The measured tie bar strain, joint opening, and joint faulting versus applied actuator load are shown in Figure 4.17. The development of tensile strain in the tie bar and the joint faulting were not significantly affected by the magnitude of the longitudinal translation. The joint opening was slightly affected as the magnitude of the longitudinal translation misalignment increased.




**Tie Bar Strain**




**Joint Opening**




**Joint Faulting**

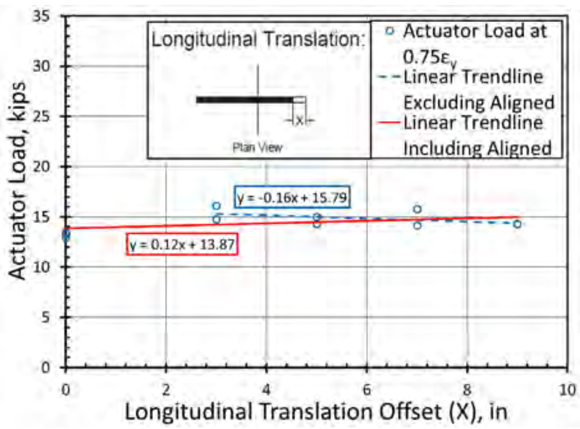
**Figure 4.17** Testing Results for the Longitudinal Translation Specimens

Table 4.6 presents the actuator load and joint relative displacement test results at the point when the first measured strain in the tie bar reached $0.75\varepsilon_y$.

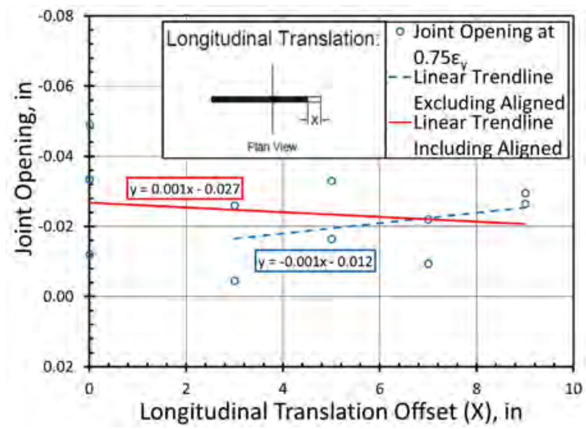
**Table 4.6** Actuator Load, Joint Opening and Joint Faulting for the Longitudinal Translation Specimens

| Specimen | At First Measured Tie Bar Strain Equal to $0.75\varepsilon_y$ | | |
|---|---|---|---|
| | Actuator Load, kip | Joint Opening, inches | Joint Faulting, inches |
| LT – 3 – 1 | 14.8 | -0.005 | 0.008 |
| LT – 3 – 2 | 16.1 | -0.026 | 0.010 |
| LT – 5 – 1 | 15.0 | -0.017 | 0.011 |
| LT – 5 – 2 | 14.3 | -0.033 | 0.015 |
| LT – 7 – 1 | 14.1 | -0.022 | 0.006 |
| LT – 7 – 2 | 15.8 | -0.010 | 0.008 |
| LT – 9 – 1 | 14.3 | -0.030 | 0.013 |
| LT – 9 – 2 | 14.3 | -0.027 | 0.004 |

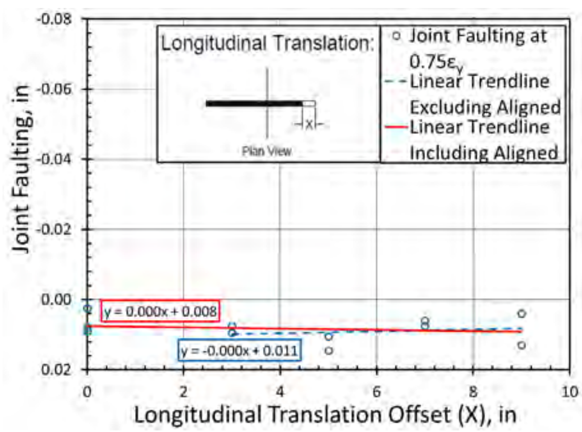
The effect of the longitudinal translation on the actuator load, joint opening, and joint faulting at $0.75\varepsilon_y$ is presented in Figure 4.18. The longitudinal translation does not seem to induce a performance variation from the ideally placed tie bar (aligned). The average actuator load at $0.75\varepsilon_y$ for the longitudinal translation specimens is 14.73 kip, which is 1.12 times that for the aligned specimens. The changes in the joint opening and joint faulting are negligible, and the joint opening is well below the 1/8-inch performance limit.




**Actuator Load**




**Joint Opening**




**Joint Faulting**

**Figure 4.18** Effect of Longitudinal Translation on Joint Parameters

### 4.2.5 Horizontal Skew

The eight vertical skew specimens were tested while restrained against rotation. In all eight of the horizontal skew specimens, the tie bar yielded prior to bond failure. Bond failure occurred by the splitting of the concrete along the length of the tie bar, as shown in Figures 4.19 and 4.20.

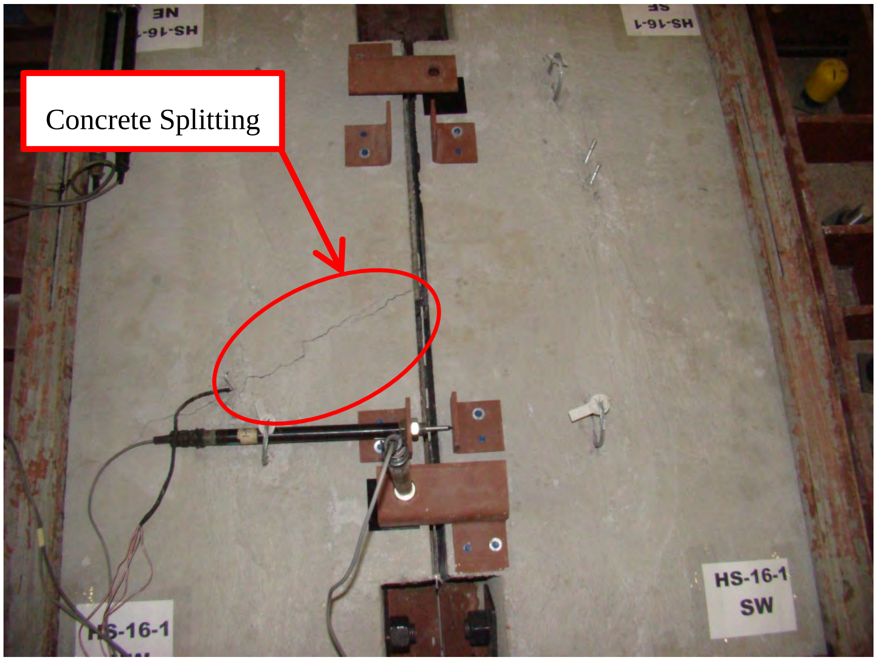



**Figure 4.19** Bond Failure of Specimen HS-16-1

**Figure 4.20** Specimen HS-16-1 Failure Plane

The measured tie bar strain, joint opening, and joint faulting versus applied actuator load are shown in Figure 4.21. The actuator load, joint opening, and joint faulting were all significantly affected by the magnitude of the horizontal skew. For a given actuator load, an increase in the horizontal skew misalignment resulted in an increase in tie bar tensile strain, joint slippage, and joint faulting.

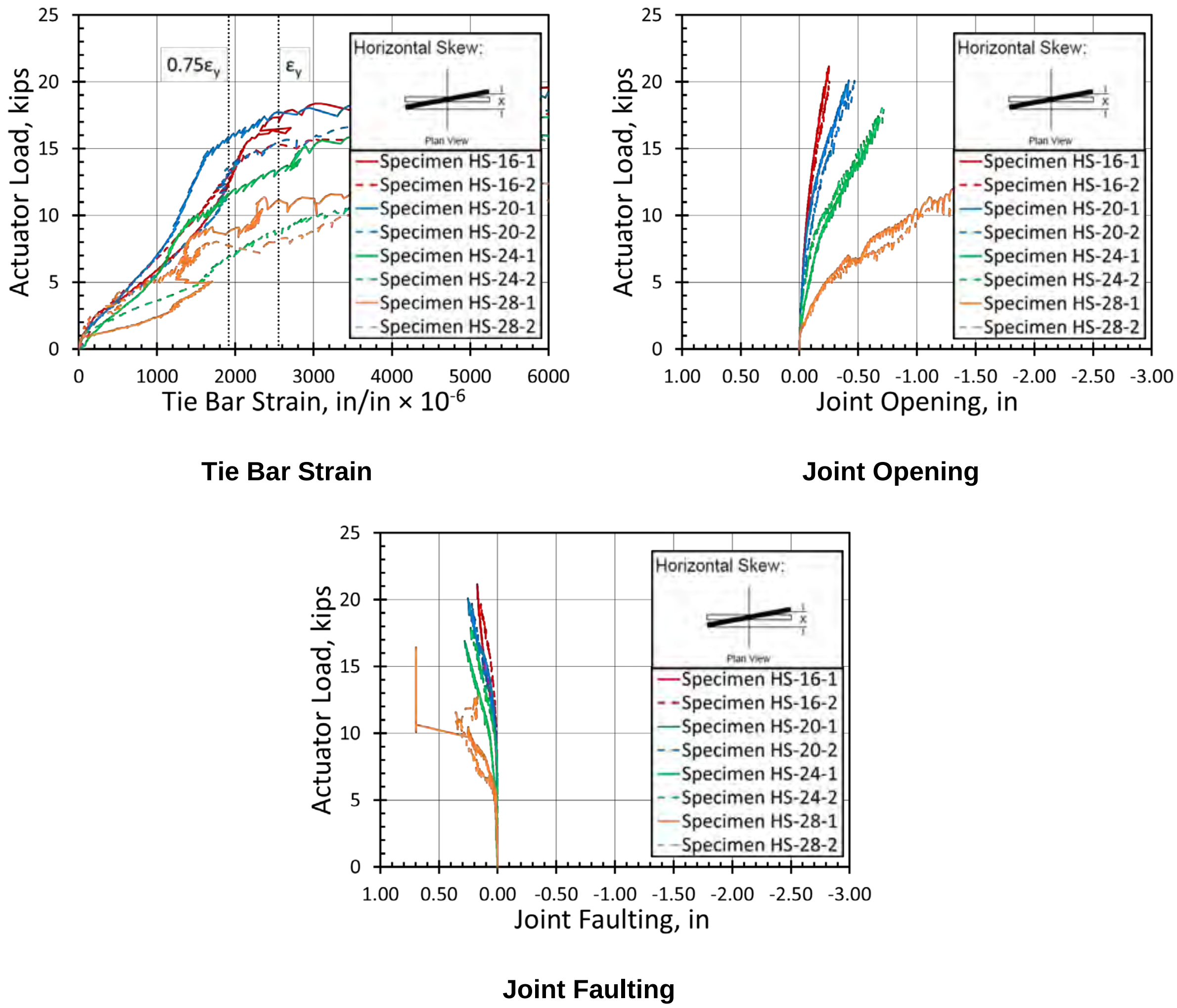


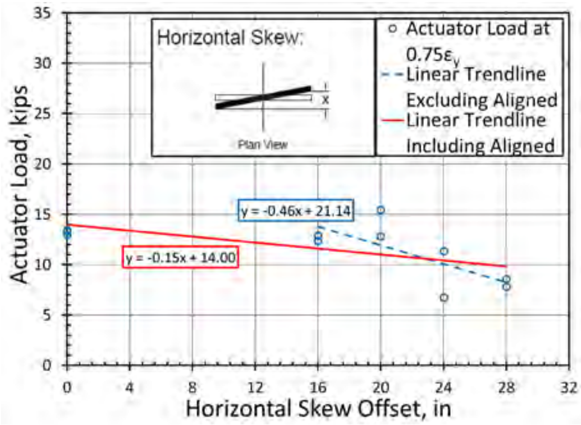

**Figure 4.21** Testing Results for the Horizontal Skew Specimens

Table 4.7 presents the actuator load and joint relative displacement test results at the point when the first measured strain in the tie bar reached $0.75\varepsilon_y$.

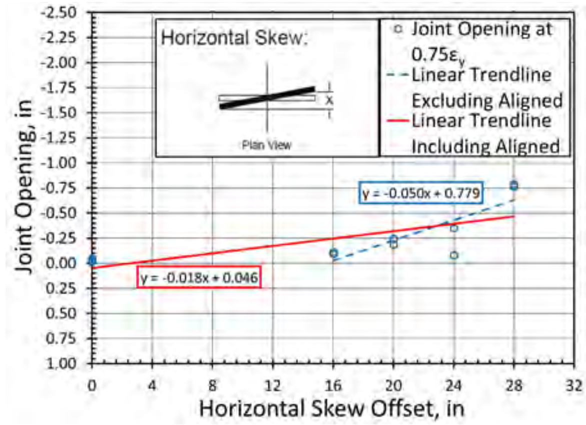
**Table 4.7** Actuator Load, Joint Opening, and Joint Faulting for the Horizontal Skew Specimens

| Specimen | At First Measured Tie Bar Strain Equal to $0.75\varepsilon_y$ | | |
|---|---|---|---|
| | Actuator Load, kip | Joint Opening, inches | Joint Faulting, inches |
| HS – 16 – 1 | 12.4 | -0.095 | 0.170 |
| HS – 16 – 2 | 12.9 | -0.115 | 0.134 |
| HS – 20 – 1 | 15.5 | -0.246 | 0.230 |
| HS – 20 – 2 | 12.8 | -0.188 | 0.194 |
| HS – 24 – 1 | 11.4 | -0.350 | 0.216 |
| HS – 24 – 2 | 6.8 | -0.082 | 0.215 |
| HS – 28 – 1 | 8.6 | -0.768 | 0.697 |
| HS – 28 – 2 | 7.8 | -0.786 | 0.343 |

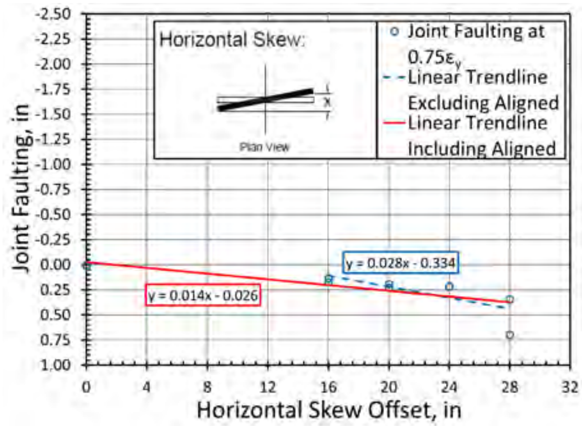
The effect of horizontal skew on the actuator load, joint opening, and joint faulting at $0.75\varepsilon_y$ is presented in Figure 4.22. An increase in the magnitude of the horizontal skew resulted in a decrease in the actuator load at a tie bar strain of $0.75\varepsilon_y$. The average actuator load at $0.75\varepsilon_y$ for the 16-in. and 20-in. misaligned specimens is 12.65 kip and 14.15 kip, respectively; both of which are close to the 13.2 kip experienced by the aligned specimens. The 24-in. misaligned specimen, however, resulted in an average actuator load at $0.75\varepsilon_y$ of 9.1 kip, which is 0.65 times that of the aligned specimen. With the exception of one of the HS-24 specimens, the horizontal skew caused the magnitude of the joint opening to increase at a tie bar strain of $0.75\varepsilon_y$. The average joint opening at $0.75\varepsilon_y$ for the 16-in. misaligned specimen was 0.105 in., which is more than three times that of the aligned specimen, but less than the 1/8 in. performance limit. However, the 1/8 in. performance limit was exceeded in the 20-in. misaligned specimen, where the average joint opening at $0.75\varepsilon_y$ was 0.217 in. The horizontal skew increased the magnitude of the joint faulting at a tie bar strain of $0.75\varepsilon_y$. At a horizontal skew magnitude of 20 in., the average measured joint faulting was 0.212 in., which is 35 times the average joint faulting experienced by the aligned specimens.




**Actuator Load**




**Joint Opening**




**Joint Faulting**

**Figure 4.22** Effect of Horizontal Skew on Joint Parameters

# 5. SUMMARY AND CONCLUSION

The study presented in this report was conducted to 1) identify current specifications for tie bar placement tolerances in PCC pavements, 2) conduct experimental testing to examine the effect of different tie bar misalignment configurations and magnitudes on the performance of longitudinal joints, and 3) provide recommendations to improve current specifications if needed.

## 5.1 Findings and Conclusions

The following findings and conclusions are based on tie bar specifications adopted by some state DOTs and the experimental tests carried out in this study.

1. The upper and lower limits on tie bar specifications adopted by some state DOTs are:
    - Steel grade: 40 ksi to 60 ksi
    - Tie bar size: #4 to #7
    - Tie bar length: 24 in. to 40 in.
    - Tie bar center-to-center spacing: 9 in. to 48 in.
2. Vertical and longitudinal translation misalignments had no significant effect on the performance of the longitudinal joint.
3. Vertical skew misalignment had no significant effect on the load resisted by the tie bar or joint opening at a tie bar strain of $0.75\varepsilon_y$.
4. Vertical skew misalignment resulted in joint faulting at $0.75\varepsilon_y$ that reached as high as 25 times that of the aligned specimens (0.152 in. at an offset of 8 in.).
5. Horizontal skew misalignment resulted in a decrease in the load resisted by the tie bar and an increase in both joint opening and joint faulting when the strain in the tie bar reached $0.75\varepsilon_y$.
6. The joint opening limit of 1/8 in. was exceeded at the 20-in. horizontal skew offset.
7. Through interpolation, the experimental results show that the joint opening limit of 1/8 in. would be exceeded at a horizontal skew offset of 18 in.
8. Joint faulting at $0.75\varepsilon_y$ for the 20-in. horizontal skew misaligned specimens reached as much as 35 times that of aligned specimens.

## 5.2 Recommendations

Based on the findings of this study, the research team offers the following recommendations.

1. The current SDDOT tie bar tolerance limit for horizontal skew misalignment should be reduced from 18 in. to, at most, 16 in. This would correspond to a reduction in the transverse placement tolerance limit from ±3.0 in. to, at most, ±2.25 in. measured perpendicular to the longitudinal joint.
2. Further reduction in the horizontal skew tolerance limit might be required if joint faulting is a significant issue.
3. The vertical skew tolerance limit is sufficient, but contractors need to strictly abide by that limit in order to avoid excessive joint faulting.
4. For future research, experiments on slabs with multiple tie bars that have various horizontal skew magnitudes can help examine more realistic scenarios.

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# APPENDIX A: CONCRETE MIX DESIGNS

**Contractor Concrete Mix Design** DOT-24 (10-10

| | | | |
|---|---|---|---|
| Project: | Tie Bar Tolerance | County: | PCN: |
| Concrete Supplier | GCC Brookings | Class of Concrete: | Well Graded PCCP |
| Supplier Signature | | Mix # (DOT use): | |
| Prepared by/ Title: | | Approved by (DOT): | |
| Date Prepared: | | Approval Date (DOT): | |

| MATERIALS: | | | Sp. Gr. | Absorption | F.M. |
|---|---|---|---|---|---|
| Fine Aggregate | (source, type): | LG Everist Washed Sand * | 2.66 | 1.1 | 2.99 |
| | (pit name, county): | Brookings | | | |
| | (Section-Township-Range): | | | | |
| Coarse Aggr. | (source, type): | LG Everist 1" Quartzite * | 2.63 | 0.3 | |
| | (pit name, county): | Dell Rapids | | | |
| | (Section-Township-Range): | | | | |
| Additional Aggr. | (source, type): | LG Everist 3/8" Chips * | 2.63 | 0.5 | |
| | (pit name, county): | Dell Rapids | ** Saturated Surface Dry Basis* | | |
| | (Section-Township-Range): | | | | |
| Cement | (brand, type, source): | GCC Type I/II Rapid City | 3.15 | | |
| Fly Ash | (brand, type, source): | Headwaters Coal Creek ND | 2.50 | | |
| Water | (source, location): | Brookings | 1.00 | | |
| | | | | oz/yd$^3$, oz/cwt, lb/yd$^3$ | |
| Admixture(s), etc | (brand, type): | Air Entrainment | | (5.0%-7.5% Air) | |
| | | Water Reducer | | (2"-3" Slump) | |

**DESIGN MIX PROPORTIONS:**

| W/C Ratio: | 0.41 (field max.) | lb/yd$^3$ | Abs. Vol. (ft$^3$) - † |
|---|---|---|---|
| Cement | | 460 | 2.34 |
| Fly Ash | | 115 | 0.74 |
| Fine Aggr. % | 40.0 | 1222 | 7.36 |
| Coarse Aggr. % | 44.0 | 1344 | 8.19 |
| Addit. Aggr. % | 16.0 | 490 | 2.99 |
| Water | | 235 | 3.77 |
| Air Content (structural, paving- 6.5%) | | 6.5% | 1.76 |
| TOTAL | | 3866 | 27.15 (≈27.0 ft$^3$) |

*%- Percent of Total Aggregate* *†- Absolute Volume= (lb. of product)÷[(Sp. Gr.)×(62.4)]*

**TRIAL MIX TEST DATA:** Attach Supporting Lab Test Documents - Aggregate: {sieve analysis, coarse % particles passing 200, absorption, fineness modulus, specific gravity, % particles less than 1.95 sp. gr., soundness, LA abrasion, flat and elongated, colormetric} Trial Batch: {batch weights, slump, air content, unit weight, actual aggregate moisture, actual w/c ratio, cylinder compressive strengths, strength gain curve}

**Concrete Purpose:**

**Comments:**

**Distribution:** Conc. Engr. - Area Engr. - Reg. Matl's Engr.

# APPENDIX B: FRESH AND HARDENED CONCRETE PROPERTIES

## Tolerances for Placement of Tie Bars in Portland Cement Concrete Pavement

Project SD2011-09

Tie Bar Misalignment Type: Aligned Specimen's (3 poured)

Testing by: SDSU, Walker Olson

**Concrete:**

| | |
|---|---|
| Supplier: | GCC Ready Mix |
| Mix Design: | Well Graded PCCP |
| Quantity ($yd^3$): | 2 |
| W/C Ratio: | 0.398 |
| Pouring Date: | Thursday, April 10, 2014 |
| Pouring Time: | 3:00:00 PM |
| Curing Method: | Wet burlap |

**FRESH CONCRETE PROPERTIES:**

| | Measured | Allowable |
|---|---|---|
| Air Temp. (°F) | 45 | - |
| Concrete Temp. (°F) | 85 | - |
| Unit Weight (lb/ft$^3$) | - | 143 .1 lb/ft$^3$ |
| Slump (in) | 2.5 | 2" to 3" |
| % Air Content | 7.5 | 5.0 to 7.5% |

**Cylinders:**

| | |
|---|---|
| Group Number: | WO - A |
| Number of Cylinders: | 18 |
| Cylinder Diameter (in): | 6.0 |
| Cylinder Length (in): | 12.0 |

**Beams:**

| | |
|---|---|
| Group Number: | WO - A |
| Number of Beams: | 8 |
| Beam width/height (in): | 6.0 |
| Span Length (in): | 18.0 |

**HARDENED CONCRETE PROPERTIES:**

| | | | |
|---|---|---|---|
| Compressive Strength | 3 - Day | 2740 | psi |
| | 7 - Day | 3973 | psi |
| | 28 - Day | 4785 | psi |
| Tensile Strength | 3 - Day | - | psi |
| | 7 - Day | 482 | psi |
| | 28 - Day | 434 | psi |
| Flexural Strength | 3 - Day | - | psi |
| | 7 - Day | 435 | psi |
| | 28 - Day | 647 | psi |
| Modulus of Elasticity | 3 - Day | - | psi |
| | 7 - Day | 3.78E+06 | psi |
| | 28 - Day | 4.01E+06 | psi |
| Average Concrete Unit Weight | | 142.3 | pcf |

**Compressive Strength (ASTM C39-14):**

Loading Rate: 35 psi/sec (35 ± 7 psi/sec = 989.6 ± 197.9 lb/sec)

| 3 - Day: Sunday, April 13, 2014 | | | 7 - Day: Thursday, April 17, 2014 | | | 28 - Day: Thursday, May 08, 2014 | | |
|---|---|---|---|---|---|---|---|---|
| Sample number | Modulus of Elasticity (ksi) | Compressive Strength (psi) | Sample number | Modulus of Elasticity (psi) | Compressive Strength (psi) | Sample number | Modulus of Elasticity (psi) | Compressive Strength (psi) |
| WO - A -1 | | 2307 | WO - A -4 | 3.83E+06 | 3939 | WO - A -16 | ~~8.29E+06~~ | 4956 |
| WO - A -2 | | 2486 | WO - A -5 | 3.84E+06 | 3721 | WO - A -17 | 4.01E+06 | 4803 |
| WO - A -3 | | 3427 | WO - A -6 | 3.75E+06 | 4145 | WO - A -18 | ~~7.63E+06~~ | 4596 |
| | | | WO - A -8 | 3.70E+06 | 4111 | | | |
| | | | WO - A -9 | 3.78E+06 | 3948 | | | |
| AVERAGE | | 2740 | AVERAGE | 3.78E+06 | 3973 | AVERAGE | 4.01E+06 | 4785 |

**Split Tensile Strength (ASTM C496):**

Loading Rate: 150 psi/min (100 to 200 psi/min)

| 3 - Day: Sunday, April 13, 2014 | | | 7 - Day: Thursday, April 17, 2014 | | | 28 - Day: Thursday, May 08, 2014 | | |
|---|---|---|---|---|---|---|---|---|
| Sample number | Ultimate Load (lb) | Tensile Strength (psi) | Sample number | Ultimate Load (lb) | Tensile Strength (psi) | Sample number | Ultimate Load (lb) | Tensile Strength (psi) |
| | | | WO - A -7 | 55640 | 492 | WO - A -12 | 41670 | 368 |
| | | | WO - A -10 | 54110 | 478 | WO - A -13 | 53380 | 472 |
| | | | WO - A -11 | 53800 | 476 | WO - A -14 | 45220 | 400 |
| | | | | | | WO - A -15 | 56000 | 495 |
| | | | | AVERAGE | 482 | | AVERAGE | 434 |

**Flexural Strength (ASTM C78):**

Loading Rate: 1800 lb/min (1500 - 2100 lb/min)

| 3 - Day: Sunday, April 13, 2014 | | | | 7 - Day: Thursday, April 17, 2014 | | | | 28 - Day: Thursday, May 08, 2014 | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Sample number | Ultimate Load (lb) | Failure Type* | Modulus of Rupture (psi) | Sample number | Ultimate Load (lb) | Failure Type* | Modulus of Rupture (psi) | Sample number | Ultimate Load (lb) | Failure Type* | Modulus of Rupture (psi) |
| | | | | WO - A -1 | 6430 | 1 | 536 | WO - A -6 | 7680 | 1 | 640 |
| | | | | WO - A -2 | 5090 | 1 | 424 | WO - A -7 | 8200 | 1 | 683 |
| | | | | WO - A -3 | 5450 | 1 | 454 | WO - A -8 | 7400 | 1 | 617 |
| | | | | WO - A -4 | 4970 | 1 | 414 | | | | |
| | | | | WO - A -5 | 4150 | 1 | 346 | | | | |
| | | | | | AVERAGE | | 435 | | AVERAGE | | 647 |

* See ASTM C76. If failure initiates in the tension face of the Middle 1/3 **enter 1.** If failure initaites in the tension face outside of the middle 1/3 by more than 0.9" **enter 2.** If failure initiates in the tension face within 0.9" of the middle 1/3 **enter value of "a" in inches.** (a = average distance between line of fracture and the nearest support)

# Tolerances for Placement of Tie Bars in Portland Cement Concrete Pavement

Project SD2011-09

Tie Bar Misalignment Type: Vertical Translation (8 Specimens)

Testing by: SDSU, Walker Olson

Concrete:

| | |
|---|---|
| Supplier: | GCC Ready Mix |
| Mix Design: | Well Graded PCCP |
| Quantity ($yd^3$): | 4.5 |
| W/C Ratio: | 0.409 |
| Pouring Date: | Thursday, April 24, 2014 |
| Pouring Time: | 10:00 |
| Curing Method: | Wet burlap |

**FRESH CONCRETE PROPERTIES:**

| | Measured | Allowable |
|---|---|---|
| Air Temp. (°F) | 50 | - |
| Concrete Temp. (°F) | 62 | - |
| Unit Weight ($lb/ft^3$) | - | 143 .1 $lb/ft^3$ |
| Slump (in) | 2.5 | 2" to 3" |
| % Air Content | 6.2 | 5.0 to 7.5% |

Cylinders:

| | |
|---|---|
| Group Number: | WO - VT |
| Number of Cylinders: | 18 |
| Cylinder Diameter (in): | 6.0 |
| Cylinder Length (in): | 12.0 |

Beams:

| | |
|---|---|
| Group Number: | WO - VT |
| Number of Beams: | 8 |
| Beam width/height (in): | 6.0 |
| Span Length (in): | 18.0 |

**HARDENED CONCRETE PROPERTIES:**

| | | | |
|---|---|---|---|
| Compressive Strength | 3 - Day | 4562 | psi |
| | 7 - Day | 5357 | psi |
| | 28 - Day | 6635 | psi |
| Tensile Strength | 3 - Day | - | psi |
| | 7 - Day | 565 | psi |
| | 28 - Day | 629 | psi |
| Flexural Strength | 3 - Day | - | psi |
| | 7 - Day | 460 | psi |
| | 28 - Day | 699 | psi |
| Modulus of Elasticity | 3 - Day | - | psi |
| | 7 - Day | 4.60E+06 | psi |
| | 28 - Day | 4.81E+06 | psi |
| Average Concrete Unit Weight | | 146.0 | pcf |

## Compressive Strength (ASTM C39-14):

Loading Rate: 35 psi/sec (35 ± 7 psi/sec = 989.6 ± 197.9 lb/sec)

**3 - Day:** Sunday, April 27, 2014

| Sample number | Modulus of Elasticity (ksi) | | Compressive Strength (psi) |
|---|---|---|---|
| WO - VT -1 | | | 4624 |
| WO - VT -2 | | | 4409 |
| WO - VT -3 | | | 4654 |
| AVERAGE | | | 4562 |

**7 - Day:** Thursday, May 01, 2014

| Sample number | Modulus of Elasticity (psi) | | Compressive Strength (psi) |
|---|---|---|---|
| WO - VT -5 | 4.72E+06 | | 5489 |
| WO - VT -9 | 4.41E+06 | | 4966 |
| WO - VT -8 | ~~9.31E+06~~ | | 5406 |
| WO - VT -6 | 4.67E+06 | | 5568 |
| AVERAGE | 4.60E+06 | | 5357 |

**28 - Day:** Thursday, May 22, 2014

| Sample number | Modulus of Elasticity (psi) | | Compressive Strength (psi) |
|---|---|---|---|
| WO - VT -11 | 4.53E+06 | | ~~5076~~ |
| WO - VT -7 | ~~7.11E+06~~ | | 6526 |
| WO - VT -12 | 5.00E+06 | | 6663 |
| WO - VT -4 | 4.91E+06 | | 6717 |
| AVERAGE | 4.81E+06 | | 6635 |

## Split Tensile Strength (ASTM C496):

Loading Rate: 150 psi/min (100 to 200 psi/min)

**3 - Day:** Sunday, April 27, 2014

| Sample number | Ultimate Load (lb) | | Tensile Strength (psi) |
|---|---|---|---|

**7 - Day:** Thursday, May 01, 2014

| Sample number | Ultimate Load (lb) | | Tensile Strength (psi) |
|---|---|---|---|
| WO - VT -18 | 65680 | | 581 |
| WO - VT -13 | 63230 | | 559 |
| WO - VT -10 | 62730 | | 555 |
| | AVERAGE | | 565 |

**28 - Day:** Thursday, May 22, 2014

| Sample number | Ultimate Load (lb) | | Tensile Strength (psi) |
|---|---|---|---|
| WO - VT -16 | 72300 | | 639 |
| WO - VT -17 | 70650 | | 625 |
| WO - VT -14 | 71480 | | 632 |
| WO - VT -15 | 70270 | | 621 |
| | AVERAGE | | 629 |

## Flexural Strength (ASTM C78):

Loading Rate: 1800 lb/min (1500 - 2100 lb/min)

**3 - Day:** Sunday, April 27, 2014

| Sample number | Ultimate Load (lb) | Failure Type* | Modulus of Rupture (psi) |
|---|---|---|---|

**7 - Day:** Thursday, May 01, 2014

| Sample number | Ultimate Load (lb) | Failure Type* | Modulus of Rupture (psi) |
|---|---|---|---|
| WO - VT -4 | 6020 | 1 | 502 |
| WO - VT -3 | 5120 | 1 | 427 |
| WO - VT -1 | ~~3450~~ | 1 | ~~288~~ |
| WO - VT -5 | 5410 | 1 | 451 |
| | AVERAGE | | 460 |

**28 - Day:** Thursday, May 22, 2014

| Sample number | Ultimate Load (lb) | Failure Type* | Modulus of Rupture (psi) |
|---|---|---|---|
| WO - VT -6 | 9450 | 1 | 788 |
| WO - VT -7 | 7500 | 1 | 625 |
| WO - VT -8 | 8210 | 1 | 684 |
| | AVERAGE | | 699 |

* See ASTM C76. If failure initiates in the tension face of the Middle 1/3 **enter 1.** If failure initaites in the tension face outside of the middle 1/3 by more than 0.9" **enter 2.** If failure initiates in the tension face within 0.9" of the middle 1/3 **enter value of "a" in inches**. (a = average distance between line of fracture and the nearest support)

# Tolerances for Placement of Tie Bars in Portland Cement Concrete Pavement

Project SD2011-09

Tie Bar Misalignment Type: Vertical Skew (8 Specimens) | Testing by: SDSU, Walker Olson

**Concrete:**

| | |
|---|---|
| Supplier: | GCC Ready Mix |
| Mix Design: | Well Graded PCCP |
| Quantity ($yd^3$): | 4.5 |
| W/C Ratio: | 0.39 |
| Pouring Date: | Tuesday, May 13, 2014 |
| Pouring Time: | 3:00 |
| Curing Method: | Wet burlap |

**FRESH CONCRETE PROPERTIES:**

| | Measured | Allowable |
|---|---|---|
| Air Temp. (°F) | 50 | - |
| Concrete Temp. (°F) | 63 | - |
| Unit Weight ($lb/ft^3$) | - | 143 .1 $lb/ft^3$ |
| Slump (in) | 2.75 | 2" to 3" |
| % Air Content | 5 | 5.0 to 7.5% |

**Cylinders:**

| | |
|---|---|
| Group Number: | WO - VS |
| Number of Cylinders: | 18 |
| Cylinder Diameter (in): | 6.0 |
| Cylinder Length (in): | 12.0 |

**Beams:**

| | |
|---|---|
| Group Number: | WO - VS |
| Number of Beams: | 8 |
| Beam width/height (in): | 6.0 |
| Span Length (in): | 18.0 |

**HARDENED CONCRETE PROPERTIES:**

| | | | |
|---|---|---|---|
| Compressive Strength | 3 - Day | 4383 | psi |
| | 7 - Day | 5261 | psi |
| | 28 - Day | 6216 | psi |
| Tensile Strength | 3 - Day | - | psi |
| | 7 - Day | 478 | psi |
| | 28 - Day | 567 | psi |
| Flexural Strength | 3 - Day | - | psi |
| | 7 - Day | 644 | psi |
| | 28 - Day | 738 | psi |
| Modulus of Elasticity | 3 - Day | - | psi |
| | 7 - Day | 4.59E+06 | psi |
| | 28 - Day | 4.89E+06 | psi |
| Average Concrete Unit Weight | | 147.0 | pcf |

## Compressive Strength (ASTM C39-14):

Loading Rate: 35 psi/sec (35 ± 7 psi/sec = 989.6 ± 197.9 lb/sec)

| 3 - Day: Friday, May 16, 2014 | | | 7 - Day: Tuesday, May 20, 2014 | | | 28 - Day: Tuesday, June 10, 2014 | | |
|---|---|---|---|---|---|---|---|---|
| Sample number | Modulus of Elasticity (ksi) | Compressive Strength (psi) | Sample number | Modulus of Elasticity (psi) | Compressive Strength (psi) | Sample number | Modulus of Elasticity (psi) | Compressive Strength (psi) |
| WO - VS -1 | | 4268 | WO - VS -7 | 4.71E+06 | 5453 | WO - VS -14 | 4.84E+06 | 6201 |
| WO - VS -2 | | 4552 | WO - VS -5 | 4.53E+06 | 5420 | WO - VS -15 | 5.02E+06 | 6324 |
| WO - VS -3 | | 4328 | WO - VS -6 | ~~9.42E+06~~ | 5146 | WO - VS -13 | 4.82E+06 | 6124 |
| | | | WO - VS -8 | 4.54E+06 | 5026 | | | |
| AVERAGE | | 4383 | AVERAGE | 4.59E+06 | 5261 | AVERAGE | 4.89E+06 | 6216 |

## Split Tensile Strength (ASTM C496):

Loading Rate: 150 psi/min (100 to 200 psi/min)

| 3 - Day: Friday, May 16, 2014 | | | 7 - Day: Tuesday, May 20, 2014 | | | 28 - Day: Tuesday, June 10, 2014 | | |
|---|---|---|---|---|---|---|---|---|
| Sample number | Ultimate Load (lb) | Tensile Strength (psi) | Sample number | Ultimate Load (lb) | Tensile Strength (psi) | Sample number | Ultimate Load (lb) | Tensile Strength (psi) |
| | | | WO - VS -10 | 53340 | 472 | WO - VS -17 | 61910 | 547 |
| | | | WO - VS -9 | 51630 | 457 | WO - VS -16 | 63340 | 560 |
| | | | WO - VS -11 | 61210 | 541 | WO - VS -18 | 67040 | 593 |
| | | | WO - VS -12 | 50250 | 444 | | | |
| | | | | AVERAGE | 478 | | AVERAGE | 567 |

## Flexural Strength (ASTM C78):

Loading Rate: 1800 lb/min (1500 - 2100 lb/min)

| 3 - Day: Friday, May 16, 2014 | | | | 7 - Day: Tuesday, May 20, 2014 | | | | 28 - Day: Tuesday, June 10, 2014 | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Sample number | Ultimate Load (lb) | Failure Type* | Modulus of Rupture (psi) | Sample number | Ultimate Load (lb) | Failure Type* | Modulus of Rupture (psi) | Sample number | Ultimate Load (lb) | Failure Type* | Modulus of Rupture (psi) |
| | | | | WO - VS -4 | 7490 | 1 | 624 | WO - VS -3 | 8420.00 | 1 | 702 |
| | | | | WO - VS -1 | 8130 | 1 | 678 | WO - VS -5 | 9370.00 | 1 | 781 |
| | | | | WO - VS -2 | 7560 | 1 | 630 | WO - VS -6 | 8790.00 | 1 | 733 |
| | | | | | AVERAGE | | 644 | | AVERAGE | | 738 |

* See ASTM C76. If failure initiates in the tension face of the Middle 1/3 **enter 1.** If failure initaites in the tension face outside of the middle 1/3 by more than 0.9" **enter 2.** If failure initiates in the tension face within 0.9" of the middle 1/3 **enter value of "a" in inches**. (a = average distance between line of fracture and the nearest support)

# Tolerances for Placement of Tie Bars in Portland Cement Concrete Pavement

Project SD2011-09

Tie Bar Misalignment Type: Lateral Translation (8 Specimens)

Testing by: SDSU, Walker Olson

Concrete:

| | |
|---|---|
| Supplier: | GCC Ready Mix |
| Mix Design: | Well Graded PCCP |
| Quantity ($yd^3$): | 4.5 |
| W/C Ratio: | 0.399 |
| Pouring Date: | Thursday, May 22, 2014 |
| Pouring Time: | 3:00 |
| Curing Method: | Wet burlap |

FRESH CONCRETE PROPERTIES:

| | Measured | Allowable |
|---|---|---|
| Air Temp. (°F) | 72 | - |
| Concrete Temp. (°F) | 81 | - |
| Unit Weight ($lb/ft^3$) | - | 143 .1 $lb/ft^3$ |
| Slump (in) | 2.75 | 2" to 3" |
| % Air Content | 5.7 | 5.0 to 7.5% |

Cylinders:

| | |
|---|---|
| Group Number: | WO - LT |
| Number of Cylinders: | 18 |
| Cylinder Diameter (in): | 6.0 |
| Cylinder Length (in): | 12.0 |

Beams:

| | |
|---|---|
| Group Number: | WO - LT |
| Number of Beams: | 8 |
| Beam width/height (in): | 6.0 |
| Span Length (in): | 18.0 |

HARDENED CONCRETE PROPERTIES:

| | | | |
|---|---|---|---|
| Compressive Strength | 3 - Day | 4297 | psi |
| | 7 - Day | 5241 | psi |
| | 28 - Day | 6320 | psi |
| Tensile Strength | 3 - Day | - | psi |
| | 7 - Day | 490 | psi |
| | 28 - Day | 553 | psi |
| Flexural Strength | 3 - Day | - | psi |
| | 7 - Day | 502 | psi |
| | 28 - Day | 608 | psi |
| Modulus of Elasticity | 3 - Day | - | psi |
| | 7 - Day | 4.79E+06 | psi |
| | 28 - Day | 4.32E+06 | psi |
| Average Concrete Unit Weight | | 145.3 | pcf |

## Compressive Strength (ASTM C39-14):

Loading Rate: 35 psi/sec (35 ± 7 psi/sec = 989.6 ± 197.9 lb/sec)

| 3 - Day: Sunday, May 25, 2014 | | | | 7 - Day: Thursday, May 29, 2014 | | | | 28 - Day: Thursday, June 19, 2014 | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Sample number | Modulus of Elasticity (ksi) | | Compressive Strength (psi) | Sample number | Modulus of Elasticity (psi) | | Compressive Strength (psi) | Sample number | Modulus of Elasticity (psi) | | Compressive Strength (psi) |
| WO - LT -1 | | | 4373 | WO - LT -6 | 4.65E+06 | | 5237 | WO - LT -5 | 4.49E+06 | | 6190 |
| WO - LT -2 | | | 4247 | WO - LT -8 | 4.74E+06 | | 5243 | WO - LT -9 | 4.72E+06 | | 6460 |
| WO - LT -3 | | | 4271 | WO - LT -4 | 4.99E+06 | | 5243 | WO - LT -11 | 3.38E+06 | | 6426 |
| | | | | | | | | WO - LT -12 | 4.70E+06 | | 6204 |
| AVERAGE | | | 4297 | AVERAGE | 4.79E+06 | | 5241 | AVERAGE | 4.32E+06 | | 6320 |

## Split Tensile Strength (ASTM C496):

Loading Rate: 150 psi/min (100 to 200 psi/min)

| 3 - Day: Sunday, May 25, 2014 | | | | 7 - Day: Thursday, May 29, 2014 | | | | 28 - Day: Thursday, June 19, 2014 | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Sample number | Ultimate Load (lb) | | Tensile Strength (psi) | Sample number | Ultimate Load (lb) | | Tensile Strength (psi) | Sample number | Ultimate Load (lb) | | Tensile Strength (psi) |
| | | | | WO - LT -17 | 55530 | | 491 | WO - LT -7 | 62690 | | 554 |
| | | | | WO - LT -16 | 62560 | | 553 | WO - LT -13 | 61660 | | 545 |
| | | | | WO - LT -15 | 48290 | | 427 | WO - LT -10 | 60930 | | 539 |
| | | | | WO - LT -18 | 55180 | | 488 | WO - LT -14 | 64960 | | 574 |
| | | | | | AVERAGE | | 490 | | AVERAGE | | 553 |

## Flexural Strength (ASTM C78):

Loading Rate: 1800 lb/min (1500 - 2100 lb/min)

| 3 - Day: Sunday, May 25, 2014 | | | | 7 - Day: Thursday, May 29, 2014 | | | | 28 - Day: Thursday, June 19, 2014 | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Sample number | Ultimate Load (lb) | Failure Type* | Modulus of Rupture (psi) | Sample number | Ultimate Load (lb) | Failure Type* | Modulus of Rupture (psi) | Sample number | Ultimate Load (lb) | Failure Type* | Modulus of Rupture (psi) |
| | | | | WO - LT -4 | 6640 | 1 | 553 | WO - LT -1 | 7030 | 1 | 586 |
| | | | | WO - LT -3 | 5340 | 1 | 445 | WO - LT -5 | 7540 | 1 | 628 |
| | | | | WO - LT -2 | 6100 | 1 | 508 | WO - LT -6 | 6860 | 1 | 572 |
| | | | | | | | | WO - LT -7 | 6800 | 1 | 567 |
| | | | | | | | | WO - LT -8 | 8260 | 1 | 688 |
| | | | | | AVERAGE | | 502 | | AVERAGE | | 608 |

* See ASTM C76. If failure initiates in the tension face of the Middle 1/3 **enter 1.** If failure initaites in the tension face outside of the middle 1/3 by more than 0.9" **enter 2.** If failure initiates in the tension face within 0.9" of the middle 1/3 **enter value of "a" in inches.** (a = average distance between line of fracture and the nearest support)

## Tolerances for Placement of Tie Bars in Portland Cement Concrete Pavement

Project SD2011-09

Tie Bar Misalignment Type: Horizontal Skew (8 Specimens) Testing by: SDSU, Walker Olson

Concrete:

| | |
|---|---|
| Supplier: | GCC Ready Mix |
| Mix Design: | Well Graded PCCP |
| Quantity ($yd^3$): | 4.5 |
| W/C Ratio: | 0.402 |
| Pouring Date: | Tuesday, June 03, 2014 |
| Pouring Time: | 1:00 |
| Curing Method: | Wet burlap |

FRESH CONCRETE PROPERTIES:

| | Measured | Allowable |
|---|---|---|
| Air Temp. (°F) | 67 | - |
| Concrete Temp. (°F) | 80 | - |
| Unit Weight ($lb/ft^3$) | - | 143 .1 $lb/ft^3$ |
| Slump (in) | 2.75 | 2" to 3" |
| % Air Content | 6.7 | 5.0 to 7.5% |

Cylinders:

| | |
|---|---|
| Group Number: | WO - HS |
| Number of Cylinders: | 18 |
| Cylinder Diameter (in): | 6.0 |
| Cylinder Length (in): | 12.0 |

Beams:

| | |
|---|---|
| Group Number: | WO - HS |
| Number of Beams: | 8 |
| Beam width/height (in): | 6.0 |
| Span Length (in): | 18.0 |

HARDENED CONCRETE PROPERTIES:

| | | | |
|---|---|---|---|
| Compressive Strength | 3 - Day | 4103 | psi |
| | 7 - Day | 5297 | psi |
| | 28 - Day | 6384 | psi |
| Tensile Strength | 3 - Day | - | psi |
| | 7 - Day | 488 | psi |
| | 28 - Day | 563 | psi |
| Flexural Strength | 3 - Day | - | psi |
| | 7 - Day | 486 | psi |
| | 28 - Day | 766 | psi |
| Modulus of Elasticity | 3 - Day | - | psi |
| | 7 - Day | 4.44E+06 | psi |
| | 28 - Day | 4.51E+06 | psi |
| Average Concrete Unit Weight | | 83.7 | pcf |

### Compressive Strength (ASTM C39-14):

Loading Rate: 35 psi/sec (35 ± 7 psi/sec = 989.6 ± 197.9 lb/sec)

| 3 - Day: Friday, June 06, 2014 | | | | 7 - Day: Tuesday, June 10, 2014 | | | | 28 - Day: Tuesday, July 01, 2014 | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Sample number | Modulus of Elasticity (ksi) | | Compressive Strength (psi) | Sample number | Modulus of Elasticity (psi) | | Compressive Strength (psi) | Sample number | Modulus of Elasticity (psi) | | Compressive Strength (psi) |
| WO - HS -1 | | | 4044 | WO - HS -4 | 4.59E+06 | | 5453 | WO - HS | 4.56E+06 | | 6570 |
| WO - HS -2 | | | 4053 | WO - HS -6 | 4.34E+06 | | 5097 | WO - HS | 4.68E+06 | | 6627 |
| WO - HS -3 | | | 4213 | WO - HS -5 | 4.40E+06 | | 5341 | WO - HS | 4.50E+06 | | 6293 |
| | | | | | | | | WO - HS | 4.28E+06 | | 6046 |
| AVERAGE | | | 4103 | AVERAGE | 4.44E+06 | | 5297 | AVERAGE | 4.51E+06 | | 6384 |

* See ASTM C39 for the 6 typical failure types.

### Split Tensile Strength (ASTM C496):

Loading Rate: 150 psi/min (100 to 200 psi/min)

| 3 - Day: Friday, June 06, 2014 | | | | 7 - Day: Tuesday, June 10, 2014 | | | | 28 - Day: Tuesday, July 01, 2014 | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Sample number | Ultimate Load (lb) | | Tensile Strength (psi) | Sample number | Ultimate Load (lb) | | Tensile Strength (psi) | Sample number | Ultimate Load (lb) | | Tensile Strength (psi) |
| | | | | WO - HS -13 | 51960 | | 459 | WO - HS | 63100 | | 558 |
| | | | | WO - HS -12 | 58270 | | 515 | WO - HS | 63230 | | 559 |
| | | | | WO - HS -15 | 55480 | | 491 | WO - HS | 64610 | | 571 |
| | | | | | AVERAGE | | 488 | | AVERAGE | | 563 |

### Flexural Strength (ASTM C78):

Loading Rate: 1800 lb/min (1500 - 2100 lb/min)

| 3 - Day: Friday, June 06, 2014 | | | | 7 - Day: Tuesday, June 10, 2014 | | | | 28 - Day: Tuesday, July 01, 2014 | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Sample number | Ultimate Load (lb) | Failure Type* | Modulus of Rupture (psi) | Sample number | Ultimate Load (lb) | Failure Type* | Modulus of Rupture (psi) | Sample number | Ultimate Load (lb) | Failure Type* | Modulus of Rupture (psi) |
| | | | | WO - HS -1 | 5690 | 1 | 474 | WO - HS | 9230.00 | 1 | 769 |
| | | | | WO - HS -2 | 5580 | 1 | 465 | WO - HS | 9010.00 | 1 | 751 |
| | | | | WO - HS -3 | 6220 | 1 | 518 | WO - HS | 9330.00 | 1 | 778 |
| | | | | | AVERAGE | | 486 | | AVERAGE | | 766 |

* See ASTM C76. If failure initiates in the tension face of the Middle 1/3 **enter 1.** If failure initaites in the tension face outside of the middle 1/3 by more than 0.9" **enter 2.** If failure initiates in the tension face within 0.9" of the middle 1/3 **enter value of "a" in inches**. (a = average distance between line of fracture and the nearest support)