KEY WAY JOINT STRENGTH OF PRECAST BOX-BEAM BRIDGES

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Mohamed Habouh
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KEY WAY JOINT STRENGTH OF PRECAST BOX-BEAM BRIDGES

Mohamed Habouh
Dissertation

Approved:

Advisor
Dr. Anil Patnaik

Accepte:

Department Chair
Dr. Wieslaw K. Binienda

Committee Member
Dr. Ping Yi

Interim Dean of the College
Dr. Mario R. Garzia

Committee Member
Dr. David Roke

Dean of the Graduate School
Dr. Chand Midha

Committee Member
Dr. Xiaosheng Gao

Date

Committee Member
Dr. Nao Mimoto

ii
ABSTRACT

Satisfactory performance of non-composite adjacent box-beam bridges depends on the effectiveness of the key way, waterproofing membrane, tie rods, and the related construction practices. Development of cracks at the longitudinal joints of such bridges is often a recurring problem that causes water leakage at the joints and corrosion of the embedded prestressing strands. The primary objective of this study was to identify the sources, causes and effects of inadequate joint performance in adjacent box-beam bridges, and to develop prevention measures. The structural performance of key way joints with the existing and new grout materials were evaluated and correlated with field measurements under traffic loading. Observation of construction practices and an investigation of a bridge that was in service for 32 years at the time of its demolition were also included. This study revealed that shear transfer strength of key way joints under symmetric loading can be increased compared to the joints that use the current ODOT-approved grouts and key way details through proper selection of grout material, adjustment to the key way geometry, and joint surface preparation. In beam configurations, this increase can be by a factor of up to 7.3 prior to the occurrence of first crack. Key way joints with a combination of the currently specified ODOT geometry and ODOT-approved grouts are incapable of carrying any shear loads in conjunction with out-of-plane moments. However, with suitable modifications, it is possible to increase the shear strength of these joints under eccentric loading. From the limited site inspections
done in this project, the practices followed at construction sites seem to be seriously flawed and may be largely contributing to water leakage problems in box-beam bridges. New key way geometries and the grouts that were developed and tested in this project are suitable for implementation.
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CHAPTER I

INTRODUCTION

1.1 Background and Problem Statement

Precast concrete adjacent box-beam bridges are a common type of bridges deployed by ODOT for short spans (30 to 60 ft.) and medium spans (60 to 110 ft.). Such bridges are popular because of the low depth to span ratio which allows better clearance under the bridge than stinger supported bridge decks. The design of box-beam bridges is relatively simple and has been standardized by ODOT. The life cycle cost of such bridges is low because of the superior performance, rapidness of erection, and ease of construction. Precast concrete industry has readily accepted box-beam bridge designs because of the fast turnover and economical methods of fabrication. An individual beam within a box-beam bridge can be replaced quite readily as demonstrated in a recent ODOT rehabilitation project by (Wood, 2008)

Adjacent box-beams need to work together for a bridge to function effectively as a single unit. Structural performance of non-composite box-beam bridges is greatly dependent on the shear key, the connection details including the grout, waterproofing, and the tie rods. ODOT standard drawings specify box-beams of 36" or 48" width that can be tied together to form the superstructure of a bridge Figure 1.1.
There are over 6000 prestressed box-beam bridges in the state of Ohio. About one sixth of all bridges built annually in the nation on public roads are adjacent concrete box-beam bridges (Huckelbridge 1997). State DOT’s will likely use box-beam bridges more frequently as an economical option under FHWA Highways for Life program (Hanna 2009). These statistics make this study all the more important because of the impact the outcomes of this research can have in Ohio and nationally. The primary objectives of the project are in line with the Department’s Strategic Research Plan for 2012-2014, and address one of the strategic research focus areas “Transportation Asset Management”.

In Ohio, adjacent box beams are connected using partial depth grouted key ways along the longitudinal joints. Development of cracks at the interface of the grout and the box beam key ways is often a recurring problem. Water leakage at longitudinal joints is generally believed to be due to the failure of the key ways and waterproofing membranes. Seeping water is one of the primary causes for corrosion of the prestressing strands and non-prestressed steel embedded in box beams (Figure 1.2).

Key way recesses are formed in precast concrete box beams at the time of casting. Box beams are usually installed next to each other and are tied together with mild steel tie rods or post-tensioned strands. The tie rods or prestressing forces from the strands provide a clamping force normal to the joint. The key ways along the longitudinal joints are then grouted with cement mortar. The hardened grout in the key way is expected to transfer the interacting forces between the adjacent box beams to enable the assembly of box beams to
act as one unit. The use of cement-based mortar grouts is a common practice for adjacent box beam bridges. One layer of waterproofing membranes is provided on the deck surface to prevent water from entering and leaking through the longitudinal joints. A wearing course of 2 to 4-inch asphalt concrete is provided on the top of the waterproofing membrane.

Figure 1.2: Deterioration of bottom Side for Box Beam Girders

Prevention of water leakage is critical to minimize corrosion related deterioration at the longitudinal joints of adjacent box-beams. Any cracking along the joints and differential deflection of adjacent beams causes water proofing membrane to get damaged making water leakage inevitable. Most times the seeping water is contaminated with chloride from deicing materials which makes the concrete susceptible to corrosion related damage such as cracking and spalling.

The performance and strength of the joint can be affected by many factors such as the geometry of key ways, properties of grout material, interface characteristics, the amount of transverse force applied to the girder, type of the applied loads, and construction practices.
The use of cement-based mortar is a common practice in construction especially when pre-cast concrete elements are used; grouts are used to join the precast reinforced concrete structural elements. A wide range of grout materials available in the market; the non-metallic, non-shrink cement based grouts are specified by the departments of transportation in most of the states to reduce the shrinkage cracks and to avoid the corrosion due to exposure to weather conditions. The ultra-high performance concrete is a good alternative for the traditional grouts with higher bond strength with the concrete girders and higher resistance to the chloride penetration, less porosity because of the fine aggregates used in the mix design. Polymer grouts and Magnesium phosphate based grouts were introduced to this application by manufacturers and employed in several projects.

Key ways are formed in the structural elements to act as shear key when filled with grouts. The grout is expected to have a perfect bond with the precast concrete units and to transfer the stresses from one girder to another to have the structure acting as one unit. The key ways can be in the longitudinal direction joining adjacent beams in the case of pre-cast pre-stressed box beam girders or in the transverse direction connecting beams in long multiple span bridges.

Transverse ties or prestressing strands are used to provide clamping force in the transverse direction in the case of the adjacent pre-cast pre-stressed box beam girders to ensure the integrity of the structure in transferring the loads between the adjacent girders.

Joints performance and strength could be affected by many factors such as geometry of the key way, properties of grout material, interface characteristics, the amount of transverse force applied to the girder, type of applied loads, and construction practice.
The longitudinal joints in the adjacent prestressed box beam girders can be reinforced with transverse rebar or unreinforced concrete joints the capacity of the unreinforced joints can be significantly low in compare to the reinforced joints, the overall structural design of the bridges including the joints is to assure that the structure acts as one unit. The exposure to weather conditions of failed joints is usually accompanied with failure in the waterproofing membranes, water leakage throughout the key way rusts the rebar and prestressing strands leading to spalling of the concrete cover in the girders, this problem was reported by the department of transportation in most of US states.

A progressive failure is anticipated in case of failed joints because the beams will act individually under the applied loads resulting on higher vertical deflection and larger horizontal movements that might cause the reported waterproofing membrane failure, usually the cracks propagates to the pavement on the surface. The rusted strands will suffer reduced cross section and higher stresses leading to higher deflection than expected (Figure 1.3, and Figure 1.4).

In this research we investigated the vertical shear strength for key ways considering key way geometry, surface roughness, grout material, and the construction practice. Also the performance of key way joints under possible loading condition.
1.2 Objectives

The objective of this study was to develop insight into the performance of longitudinal joints with a particular reference to cracking and differential deflection that is
believed to cause the bridge to fail, and to develop preventive measures through careful evaluation of alternatives

1.3 Research Methodology

In order to address the problem and to develop potential solutions, a systematic study was conducted in this project to include the following major tasks:

(i) Structural performance of key way joints

(ii) Study of grout material

(iii) Field measurements of vertical differential deflection and separation of longitudinal joints

(iv) Beam assembly tests with symmetric loading

(v) Analysis for eccentric loading and structural tests

(vi) Observation of construction practices

(vii) Investigation of a bridge that was in service for 32 years at the time of its demolition

The above distinct parts of the project are described in this study. A tentative set of conclusions and recommendation for implementations are also presented.
CHAPTER II

LITERATURE REVIEW

2.1 ODOT Current Practice

Key way geometry, Grout material, and the transverse forces are the main three components of the joint. Atypical reported damages are often recorded in the longitudinal joints at the abutment locations of the simply supported box beams, the current practice for ODOT are described in the following section, the support conditions are discussed in later section.

2.1.1 Key Way Geometry

Ohio department of transportation has a standard designs for the box beam girders with standard key way geometry. For all 36” and 48” wide box beams with composite or non-composite beams, when the beam height is 12”, 17”, 21”, and 27” the key way is 3” deep and 3/4” wide for the top opening to place the grout then 4” of depth with 1.5” width. The key way depth is increased from total of $3 + 4 = 7”$ to $6 + 6 = 12”$ in case of deeper box beams with height of 33” or 42”, a standard slope of 1:1 for the chamfers to change width.
2.1.2 Grout Material

A wide range of grout materials with variable strengths levels are available in the industrial market. The Polymer and Magnesium-Phosphate based grouts are competitive readily available grouts and being used in different counties, however the cementatious grouts are the commonly approved and specified grouts in most of the DOT’s. ODOT requires a non-shrink, non-corrosive, non-metallic cementitious grout material to be used for the application of shear key. A list of some of the approved grouts and material properties are shown in Table 2.1

2.1.3 Transverse Forces

ODOT requires a one inch diameter tie-rod through two inches hole in the transverse direction through the beams to provide a normal force of 15 Kips results from torque of 250 kip-ft, for our test specimens we used Grade B7 Alloy Steel for the threaded rod with 1/2" diameter, 20 threads per inch, the hole in the concrete units for the tie-rod was 1” diameter, the applied torque was varied from 100 lb-in, 200 lb-in, and 230 lb-in, the tie rod is placed at six inches from the top of the girder.
<table>
<thead>
<tr>
<th>#</th>
<th>Company</th>
<th>Product – Non-shrink, Noncorrosive, Non-metallic Cementitious grout</th>
<th>Flowable Compressive Strength @ 1-7-28 days (psi)</th>
<th>ASTM C827 Early Height Change</th>
<th>Yield per 50 lb (22.7 kg)</th>
<th>Set Time (min)</th>
<th>Expansion - ASTM C-1090 1/7-28 DAYS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>BASF Building System</td>
<td>Masterflow 928</td>
<td>4,000 / 6,700 / 8,000 flexure 1000 1050 1150</td>
<td>Modulus of elasticity (2.82 / 3.02 / 3.24) * 10E6</td>
<td>180 / 300</td>
<td>0.3 %</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Bonsal American</td>
<td>Construction grout</td>
<td>2,500 / 5,700 / 7,000</td>
<td></td>
<td></td>
<td>270 / 450</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Chem Master</td>
<td>kemset</td>
<td>4,400 / 7,400 / 8,300</td>
<td>0.90%</td>
<td>0.43</td>
<td>42 / 117</td>
<td>0.3 %</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Conset grout</td>
<td>2,590 / 5,260 / 6,870</td>
<td>1.20%</td>
<td>0.43</td>
<td>132 / 210</td>
<td>0.13 %</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Gorilla grout</td>
<td>3,870 / 8,740 / 10,400</td>
<td>1.70%</td>
<td>0.45</td>
<td>165 / 240</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Conspec Marketing &amp; Mfg Co.</td>
<td>Endure 50 grout</td>
<td>4500 / 5800 / 8000</td>
<td></td>
<td></td>
<td>0.43</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Dayton Superior Corp</td>
<td>Sure Grip high performance grout</td>
<td>5,000 / 8,000 / 1,0000</td>
<td>0.42</td>
<td>0.3 %</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sure Grip utility grout</td>
<td>2,500 / 6,000 / 8,000</td>
<td>0.43</td>
<td>0.3 %</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Euclid Chemical Kaufman Product Inc</td>
<td>Sure Grout - 106</td>
<td>2,400 / 6,400 / 7,600</td>
<td>0.68%</td>
<td>(C - 157)</td>
<td>0.45</td>
<td>0.3 %</td>
</tr>
<tr>
<td>7</td>
<td>Kuhlman Construction Products</td>
<td>Kuhlman 1107 grout</td>
<td>3,300 / 6,200 / 7,000</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>L&amp;M Crystex</td>
<td>L&amp;M CRYSTEX</td>
<td>4,600 / 8,160 / 1,0150</td>
<td></td>
<td>0.40%</td>
<td>0.45</td>
<td>0.2%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Duragrount</td>
<td>2,300 / 7,000 / 8,300</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>Nox Crete Product Groups</td>
<td>NoxCete Construction Grout</td>
<td>3,300 / 6,200 / 7,000</td>
<td></td>
<td>0.40%</td>
<td>0.45</td>
<td>0.2%</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>Non-shrink General Purpose Grout</td>
<td>3,000 / 8,000 / 9,000</td>
<td>0.30%</td>
<td>0.45</td>
<td></td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>Quickrete</td>
<td>Quickrete Non-Shrink Precision Grout</td>
<td>3,000 / 9,500 / 12,500</td>
<td>0.40%</td>
<td>0.45</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Non-shrink General Purpose Grout</td>
<td>3,000 / 8,000 / 9,000</td>
<td>0.30%</td>
<td>0.45</td>
<td></td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>Sika Corp</td>
<td>Sikagrount 212</td>
<td>3,500 / 5,700 / 6,200</td>
<td></td>
<td>0.43</td>
<td>240 / 390</td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>SpecChem</td>
<td>SC PRECISION GROUT</td>
<td>3,500 / 7,600 / 10,275</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>Vexon Chemical</td>
<td>Certi - Grout 1000</td>
<td>4,025 / 7,700 / 10,250</td>
<td></td>
<td>0.05%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>W R Meadows, Inc</td>
<td>Sealtight 558-10k</td>
<td>4,500 / 6,500 / 9,200</td>
<td>0.43 : 0.64</td>
<td>180 / 300</td>
<td>0.1 : 0.13 : 0.14</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>sealtight CG-86</td>
<td>3,000 / 5,500 / 7,000</td>
<td>0.43</td>
<td>0.30%</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
2.1.4 Evaluation of Grouts

The compressive strength of the grout material is usually used as an indicator to qualify the grout for the key way application, shrinkage and freeze-thaw resistance are also considered for crack control under the severe environmental conditions. A List of ASTM Tests Related to Grouts are shown in Table 2.2

Table 2.2: List of ASTM Tests Related to Grouts

<table>
<thead>
<tr>
<th>#</th>
<th>Test Name</th>
<th>Test #</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Slant shear bond</td>
<td>ASTM C882</td>
</tr>
<tr>
<td>2</td>
<td>Volume change, % expansion</td>
<td>ASTM C 827</td>
</tr>
<tr>
<td>3</td>
<td>Volume change</td>
<td>ASTM C-1090</td>
</tr>
<tr>
<td>4</td>
<td>Expansion</td>
<td>ASTM C - 157</td>
</tr>
<tr>
<td>5</td>
<td>Freeze-thaw resistance</td>
<td>ASTM C666 Procedure A</td>
</tr>
<tr>
<td>6</td>
<td>Bond strength, hardened concrete to plastic grout</td>
<td>ASTM C-882 modified</td>
</tr>
<tr>
<td>7</td>
<td>Flexure strength - resist vibration</td>
<td>ASTM C-348</td>
</tr>
<tr>
<td>8</td>
<td>Bleeding of concrete</td>
<td>ASTM - C232</td>
</tr>
<tr>
<td>9</td>
<td>Flexure strength</td>
<td>ASTM - C 78</td>
</tr>
<tr>
<td>10</td>
<td>Yield, density, and air content</td>
<td>ASTM - C 138 % C 138M</td>
</tr>
<tr>
<td>11</td>
<td>Modulus of elasticity</td>
<td>ASTM C 469, modified</td>
</tr>
<tr>
<td>12</td>
<td>splitting tensile</td>
<td>ASTM C 496</td>
</tr>
<tr>
<td>13</td>
<td>Punching shear strength</td>
<td>BASF Method</td>
</tr>
</tbody>
</table>

The evaluation of grouts strength was of importance to ODOT to investigate the cracks that appears in key ways for the box beam bridges (ODOT). Current ODOT specification requires that three 3in. × 6 in. cylinders be made and sent to Office of
Materials Management lab for testing. The minimum required compressive strength of the cylinders is 5,000 psi before allowing construction or vehicular traffic on the structure. However, ASTM specification C1107, “Standard Specification for Packaged Dry, Hydraulic-Cement Grout (non-shrink)” pertains to restrained cube molds.

ASTM specification 1107 refers to restrained cube molds. This study was conducted to see if the strength of any of the other methods used is comparable to the strength of restrained cube molds. However the bond strength between the old concrete surface and fresh grouts, and the influence of the construction practices was not considered.

2.2 State of Practice for Box Beam Bridges (PCI Committee on Bridges, 2009)

This information was gathered primarily from a survey of state highway agencies through the AASHTO Highway Subcommittee on Bridges and Structures and a review of the AASHTO LRFD Bridge Design Specifications.

2.2.1 Transverse Forces

The locations for the ties were at the ends, mid-span, quarter-points, and third points, depending on the number of ties. About 70% of the respondents reported that the ties were placed at mid-depth. If two strands or bars were used at one longitudinal location, they were placed at the third points in the depth. Other responses included specific location depths.
Eighty-one percent of states and 89% of the respondents to the survey stated that they did not make any design calculations to determine the amount of transverse ties between box beams. Some respondents provided information about the post-tensioning force used for each transverse tie and the spacing of ties. Based on this information, the average transverse force per unit length along the span for various numbers of ties was calculated. Figure 11 shows the results for 11 states. Where a single horizontal line is shown, it is based on the specified maximum spacing between ties. If the ties are closer than the minimum, the force will be higher than shown in the figure. Some states presented a range of forces as these states used a fixed number of ties for a range of span lengths. These are shown as a vertical band of color. A design chart to determine the required
effective transverse post-tensioning force is provided in the PCI bridge design manual 4. This chart is based on the work of El-Remailey et al.

The current practice of ODOT requires one tie-rod at the top third of the girder height; that might act as a rotational pin, a better application in (PCI, 2009)When two tie rods are used to prevent separation in the tensile zone. In “direct” vertical shear the increase of the load will reduce the friction contribution due to lateral separation and vertical slip as found in all test specimens, unless the concrete blocks are supported in the horizontal direction to maintain full surface contact which is the common practice in the case of precast-prestressed box beam girders used in bridges design when ties or horizontal prestressing strands are used to provide horizontal support in the case of using ties or to provide lateral compression in the case of tension prestressed strands. The tie elevation and
the compressive force applied to the box beams will influence the friction contribution and crack width between the units.

2.2.3. State of Practice for key way Geometry

Key ways are formed to join the girders in the longitudinal direction. The geometries in Figure show examples of key way configurations that was recently employed by Illinois department of transportation and Figure 2.6 shows that some states use a full depth key way and other states use a partial depth key way with wide range of width.
The key way configuration has no design guide line to select the proper configuration based on the available strength of the employed geometry or the stresses imposed to the key way under the applied loads. A survey conducted by University of Cincinnati Department of Civil and Environmental Engineering to investigate the practices for each state and the design procedure for the used design (Russell H. G., 2011)
2.3 Residual Strength of Deteriorated Box Beams

The residual strength of deteriorated box beam bridges is a great concern for the public safety. The residual strength of box beams was tested by many researchers. Two 54-ft box beams [36in. wide × 27 in. deep] were removed and subjected to load test after being in service for 27 years. The first beam was removed from the center of the bridge with no signs of deterioration and the second beam showed a minor concrete cracking and spalling (Chandu et al., 1991) the measured ultimate flexural strength exceeded the required strength at factored loads. Load-deflection response closely followed the predicted response.

A severely distressed fascia 2-cell box beam was removed from the Hawkins Road Bridge in Jackson County, Michigan and tested and the bridge was found to be safe to operate in these conditions (Upul et al., 2005), Figure 2.7: shows the underside of the bridge (left) and the top surface of the bridge (right)

![Figure 2.7: 50-Year-Old Tested 2-Cell Box Beam (Upul et al, 2005)](image)

A fatigue tests were conducted to 27-year-old box beams (Chetana Rao et al, 1996) were removed from a deteriorated multi-beam bridge and subjected to fatigue testing. Visually, the beams appeared in good condition but showed signs of water leaking through the longitudinal shear keys and some corrosion of reinforcement. One beam, cycled to a nominal bottom tension stress level of $6\sqrt{f_c}$ the beam retained excellent performance after
1,500,000 cycles. The other beam was loaded to reach $9\sqrt{f_c}$, the strength was reduced and caused fatigue failure after 145,000 load cycles.

Minor science of deterioration, cracks, water leaking stains, and spalling of concrete cover in the underside of box beams, T beam girders and I shaped girders after 27 to 50 years of service does not affect the flexure strings or the ductility of the girders under service loads and the prestressed losses are within the AASHTO predictions for effective prestressed forces [(Shenoy, 1991), (Tabatabi, 1993), (Halsey, 1996), (Pessiki, 1996), (Rao, 1996), (Eder, 2005), (Czaderski, 2006), and (Attanayake, 2011)]. Corroded and snipped strands reduce the load carrying capacity by up to 50 percent (Steinberg, 2011).

2.4 Rehabilitation of Box Beam Girders (Wood, 2008).

It is convenient to replace the deteriorated Box beam girders at reasonable cost and short time as reported by the state of OHIO when two girders were replaced at MOT-35-1.55, rout 35 in Jacksontownship, Montgomery county, Ohio U.S. the visual inspection of the bridge indicated the need of rehabilitation Figure 2.8, the cost for the possible treatments were estimated as shown in Table 2.3

Table 2.3: Cost Estimate to Maintain Deteriorated Box-Beam Bridge (Wood 2008)

<table>
<thead>
<tr>
<th>Option</th>
<th>Preliminary Estimate</th>
<th>Estimate with Inflation</th>
</tr>
</thead>
<tbody>
<tr>
<td>New Non-Composite Superstructure</td>
<td>$174,606</td>
<td>$221,565</td>
</tr>
<tr>
<td>New Composite Superstructure</td>
<td>$200,286</td>
<td>$254,151</td>
</tr>
<tr>
<td>Replace Beams 5 and 6</td>
<td>$54,433</td>
<td>$69,072</td>
</tr>
<tr>
<td>Repair Beams 5 and 6</td>
<td>$55,266</td>
<td>$70,129</td>
</tr>
<tr>
<td>Repair and Strengthen Beams 5 and 6 with FRPC</td>
<td>$77,000</td>
<td>$97,709</td>
</tr>
</tbody>
</table>
In one week and a $100,000 cost the two middle girders were replaced Figure 2.9 and no signs of deteriorations appeared in the bridge after 1 year Figure 2.10.
2.5 Alternative Grouts

A laboratory study by (Gulyas 1995) to compare component material tests and composite grouted key way specimens using two different grouting materials: non-shrink grouts and magnesium ammonium phosphate mortars. Comparative composite specimens were tested in vertical shear, longitudinal shear, and direct tension. Results indicate significant differences in performance between the materials. Composite testing of the grouted key way assemblies, rather than component materials testing, was shown to be a more accurate way to evaluate the performance of the grouting material. The author emphasized that there is no requirement in the specification for important properties of a high quality key way grout: maximum allowable shrinkage; minimum bond strength.

The authors suggest that polymer modified materials are preferred over cementitious products because of the improved bond to the concrete united reduced
chloride permeability and the internal self-curing after initial moist curing improved freeze-thaw.

The author concluded that composite testing of grouted key way assemblies provides much more practical information than component testing of the materials. Effects of grouting materials, precast concrete member key way shapes, curing, substrate exposure, and texture can be evaluated.

2.5.1 Development of Ultra-High Performance Grout (Mohamed, H. 2015)

The existing options for Ultra-High Performance concrete (UHPC) are limited due to the need to use silica sand and quartz powder, which are both expensive. Traditionally, UHPC is produced with Ottawa sand having a pre-defined particle size distribution. The main reason for considering Ottawa sand is due to its mineralogy, particle shape and size distribution. Ottawa sand is siliceous almost with rounded to sub-rounded grains, and it has a smooth surface. The inherent properties of Ottawa sand are favorable when working with low water-to-cement ratios and when enhanced workability is needed. The use of standard Ottawa sand and quartz powder in UHPC production is restrictive where availability of these materials is limited. The main objective of this task is to use economical and readily available materials for developing a high performance grout based on UHPC. Consequently the sand used in our trials is conventional siliceous river sand with sub-rounded grains.

The use of large size gravel was not found to be suitable for high-performance grout. Different types of cements, undensified silica fume, water content, high-range water reducing agents, and steel fibers were used in our trials. Sand needed to be very clean, since contamination with clay and silt particles reduced the cement-aggregate bond strength.
Also, presence of clay and silt particles increased the water demand. The primary concern regarding the aggregate in the mix design for ultra-high performance grout is gradation, maximum particle size and strength. Large size particles are preferable, if workability can be achieved. The nominal size of sand ranged from 0.15 to 0.6 mm (0.006 inch to 0.024 inch).

Table 2.4: Mix Design for Ultra High Performance Grout without Coarse Aggregate

<table>
<thead>
<tr>
<th>UHP Grout</th>
<th>Cement</th>
<th>Silica fume</th>
<th>Sand</th>
<th>Water W/C = 0.23</th>
<th>S.P.</th>
<th>Steel fiber</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mix proportion by weight</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>lb/ft³</td>
<td>56</td>
<td>8.4</td>
<td>70</td>
<td>12.9</td>
<td>1.7</td>
<td>10</td>
</tr>
<tr>
<td>lb/yd³</td>
<td>1512</td>
<td>227</td>
<td>1890</td>
<td>348</td>
<td>46</td>
<td>270</td>
</tr>
<tr>
<td>kg/m³</td>
<td>900</td>
<td>135</td>
<td>1125</td>
<td>207</td>
<td>27</td>
<td>160</td>
</tr>
</tbody>
</table>

S.P. stands for super-plasticizer (high-range water reducer)

2.6 Studies of Transverse Forces

The transverse forces are usually applied to the box beams at the diaphragms after placing the beam assembly on the bridge location prior to grouting, the transverse forces does not generate any compression on the grouted joint, the effect of the transverse forces was found to be local at the diaphragm locations, the joint test (Habouh 2015) with tie rods. Ties or prestressing strands are used to provide clamping force in the transverse direction of adjacent precast prestressed box-beam bridges to ensure the integrity of the bridge in transferring the loads between the adjacent beams. The amount of transverse force applied to the beams is believed to increase the shear transfer strength and to provide lateral stability for the bridge assembly.

The test specimens comprise three small units with two key way joints having geometry similar to the one specified in ODOT standard drawings as shown in Figure
2.12 The three units in each specimen were tied together with a tie rod with three different levels of tie forces. The grout was ODOT-approved Kuhlman 1107 grout.

![Figure 2.11: Details of Joint Tests Specimen with a Tie Rod](image)

The results developed from the testing of the seven specimens and descriptions of the failure condition are shown in Fig. 2.12.

![Figure 2.12: Summary of Test Result](image)

Failure Modes: A sudden failure took place at the shear interface between the grout material and the concrete unit for the specimens without a tie rod; concrete units and grout
were not damaged or cracked. For specimens with a tie rod, the specimens could be loaded until the concrete units were cracked or the tie rod yielded. For the specimen size used in this study, the concrete failed before the tie rod failed, as shown in Fig. 2.13.

![Typical Failure Mode, Specimens without Ties (left) Specimen with Tie Rod (right)](image)

Figure 2.13: Typical Failure Mode, Specimens without Ties (left) Specimen with Tie Rod (right)

First Crack Load and Reserve Strength: A larger first crack load was recorded for the specimens with a tie rod compared to those without a tie rod. A load equal to 11,500 lb to 18,000 lb for specimens with tie rods was recorded as compared to 2,800 lb and 7,800 lb for the specimens without tie rod. The resulting higher load carrying capacity may be due to the contribution of the steel area of the tie rod crossing the two interfaces and larger frictional resistance at the interface due to the clamping force resulting from the tie rod torque. The reserve strength or the “post-cracking load” was larger for the specimens with a tie rod because the tie rod provides lateral stability, thereby preventing the excessive lateral spread of the units at the bottom.

Vertical Displacement: Using a tie rod with the resulting clamping force allows a larger vertical slip at first crack load. There was a trend of increasing vertical slip as the clamping force increased in all specimens. A vertical slip of 0.025 inch was the maximum
slip recorded for specimens without a tie rod, and the slip increased to 0.033 inch for specimen S7, which had a 100 in-lb torque. Specimens S5 and S6, with torque equal to 200 in-lb, had the same amount of slip (0.055”). For specimens S3 and S4 with 230 in-lb torque, vertical slips equal to 0.063” and 0.064” were recorded. Increasing the clamping force allowed a larger vertical slip at first crack load.

2.7 Eccentric Load Effects

When barriers and curb slaps are assembled with the exterior beams in bridges an asymmetric cross section should be considered in the box beam design, the effects of this eccentric loading in the transverse direction of the bridge assembly was studied by (Kasan, 2013), the researchers proposed a relationship to determine the capacity of the facial beams. The possibility of eccentric truck loading on an individual beam suggests out of plane moment and normal stresses on the key way joint.

2.7.2 Environmental Stresses

Shear key evaluation by testing girders under cyclic loads and environmentally induced loads were performed by (Miller R. H., 1998) in three phases test program, (a) testing of the current key way details using the currently specified non-shrink grouts, (b) testing the current key way details using epoxy grout, and (c) testing of modified key way details using non shrink grout in which the key way was moved to the neutral axis of the girder.

The author concluded that the Epoxy grouts are most resistant to cracking, the key ways experienced some cracking when placed at the neutral axis but it is more resistant to cracking than the current ODOT standard grouts, all cracks seems to be thermally induced.

Table 2.5: Test Program for Key Way Evaluation (Miller R. H., 1998)
2.8 Dimensional Tolerances and Construction Practices.

The poor performance of key way grout and joint details were of researcher’s interest. Joint detailing does not usually receive the required attention. This issue was raised by (Nottingham, 1995) because the intended joint design can’t be achieved due to construction tolerances as shown in Figure 2.15. The author emphasized that the details are as important to construction as they are to the required design performance. Often overlooked precast concrete element tolerance can lead to improper joint fit and incomplete
grouting. Some of the problems exhibited by joint details can be seen by the illustration; joints are never full strength and can be much weaker than envisioned by the designer.

Figure 2.14: Allowable Dimensional Tolerances Effect (Nottingham, 1995)

The author endorsed the joints in Figure 2.16 to be sufficiently large to handle panel tolerance and ensure maximum construction speed with full grout-to-panel contact. The author strictly highlighted that the bond capacity of grout will be greatly diminished if the grouting process didn’t follow a strict instructions such as; the precast surface must be sandblasted, pressure washed just prior to grouting, the grout should be thoroughly mixed with the minimum required mixing water for placement otherwise the delicate grouting operation will turn into a short of control chaos.
Figure 2.15: Recommended Design (Nottingham, 1995)
CHAPTER III

STRUCTURAL PERFORMANCE OF KEY WAY JOINTS

3.1 Objective

There are many factors that might affect the shear strength of key ways. Simplified test specimens were designed to determine the effects of these factors on shear strength of key way joints and to observe the associated failure modes, differential deflection, and lateral spread of the joints to down select the tentative characteristics for a larger scale test plan.

3.2 Test Procedure

A symmetric static load was used for this test to determine the vertical shear strength of the tested specimens.

3.2.1 Test Specimens

Three concrete units were cast separately and then joined together by filling the key ways with grout. The external units were 4”×4”×12” with key way formed in inner side, and the middle unit was 6”×4”×12” with two key ways formed in two outer sides as shown in Figure 3.1. The 28-day compressive strength of the concrete units was 6,800 psi.
In the test setup, the middle unit was supported against vertical movement, and the load was applied upward on the two external units. Six dial gages were installed to measure the vertical differential deflections and lateral spread. Roller supports were used in order to allow lateral spread.

3.2.2 Shear Test Setup and Procedure

The instrument used for the test was a hydraulic test machine with a capacity of 300 kips, whereas the maximum applied load to the specimens was 60 kips. A loading rate of 70 lb/sec was used in the static loading.

Dial gages were fixed at the top of the moving (left and right) concrete units to measure the upward slip relative to the middle fixed block and the corresponding applied load. The load and the corresponding vertical slip were recorded to develop a load-slip
diagram by calculating average slip from the two gages. The test machine and the test setup are shown in Fig. 3.2.

Figure 3.2: Testing Machine and Test Setup

Figure 3.3: Test Setup
3.2.3 Factors under Study

The factors considered under this study were:

(i) Key way geometries 

(ii) Grout material

(iii) Commonly used industrial chemical additives (use of bonding agents)

(iv) Cement slurry (use of cement slurry)

(v) Surface preparation (sandblasting)

3.2.3.1 Key Way Geometry

A test specimen prepared according to the current ODOT practice was considered as the control specimen and is referred to as “partial depth, narrow key way”. Three tentative geometries were investigated with combinations of wider and deeper key ways as shown in Figure 3.4. Set #1 had a key way conforming to the current practice of ODOT with ¾" opening at the top for the top 3 inches of depth, then 1.5” width for 4 inches of depth. Set #2 had a 1.5” top opening width followed by 3” of width with same height of key way as Set #1. Set #3 had the same width of the standard geometry and the same 3” of height for the top portion as Set #1 and #2, but the key way extended to the bottom of the concrete units, except for the last inch at the bottom to contain the grout material during casting. Set #4 had the height of the top opening that was the same as specified for the standard geometry, but it had a deeper and wider key way.
3.2.3.2 Grout Types:

The cement based grouts are often specified by the departments of transportation, however the use of alternative grout is introduced by the manufacturers and was considered in some bridges and considered in this research project.

3.2.3.2.1 Cement-Based grout: Pre-Manufactured ODOT-Approved Grout

This grout is a non-shrink, noncorrosive, non-metallic cementitious grout, with controlled aggregates, admixtures and Portland cement. It is a ready-to-use grout that can
be mixed by adding water. It has a setting time of 15 minutes as suggested by the manufacturer, and the mixing time is three to five minutes depending on the amount of mixing water.

3.2.3.2.2 Cement-Based Grout - Normal Strength Concrete (5,400 psi)

Normal strength concrete was made by using traditional concrete with maximum #8 coarse aggregates and Type I Portland cement (Table 3.1). This grout is a cheaper option with adequate strength, and it achieved proper compaction. The top opening of the key way was wider, preferably at least one inch, to allow for the insertion of a small vibrator.

Table 3.1: Mix Design for Normal Strength Concrete Used as Grout Material

<table>
<thead>
<tr>
<th>Mix Proportion</th>
<th>Description</th>
<th>lbs/yd3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>Type I</td>
<td>725</td>
</tr>
<tr>
<td>Coarse aggregate</td>
<td># 8 milestone</td>
<td>1523</td>
</tr>
<tr>
<td>Sand</td>
<td>River sand</td>
<td>1523</td>
</tr>
<tr>
<td>W/C</td>
<td></td>
<td>0.4</td>
</tr>
<tr>
<td>Water</td>
<td>Potable water</td>
<td>290</td>
</tr>
</tbody>
</table>

The setting time for normal strength concrete is around thirty minutes, and the average 28-day compressive strength was 5,400 psi.

3.2.3.2.3 Cement-Based Grout – High-Strength Concrete (9,800 psi)

This grout was made using traditional concrete with maximum #8 coarse aggregates and Type I Portland cement. Compaction was provided to the grout with a vibrator. The opening at the top of the key way is at least one inch to allow for vibration. The relevant mix design is shown in Table 3.2.
Mix Proportions

<table>
<thead>
<tr>
<th>Materials</th>
<th>Description</th>
<th>lbs/yd3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>Type I</td>
<td>1100</td>
</tr>
<tr>
<td>Coarse aggregate</td>
<td># 8 limestone</td>
<td>1596</td>
</tr>
<tr>
<td>Sand</td>
<td>River sand</td>
<td>1450</td>
</tr>
<tr>
<td>W/C</td>
<td></td>
<td>0.35</td>
</tr>
<tr>
<td>Water</td>
<td>Potable water</td>
<td>385</td>
</tr>
<tr>
<td>High range water reducing agent 100ml/100 lb of cement (SIKA 2100)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3.2: Mix Design for High-Strength Concrete Used as a Grout Material

Setting time was around thirty minutes and average twenty eight days compressive strength was around ten thousand pounds per square inch. Vibration is required for this concrete mix to ensure proper compaction for the grout material and proper curing after grouting to gain the desired strength and to reduce shrinkage cracks.

3.2.3.2.4 Cement-Based Grout: Ultra-High Strength Grout (19,000 psi)

The main objective of this task was to use economical and readily available materials for developing a high-performance grout based on UHPC. Sand used in the trials was conventional sieved siliceous river sand with sub-rounded grains with the least amount of contamination. Details regarding the mix design and mixing procedure are discussed later in this report.

3.2.3.2.5 Polymer-Based Grout

This is a polymer-based grout consisting of hydraulic binder with applied nanotechnology and premium mineral aggregates as per manufacturer’s description. This grout material is pre-manufactured; only the mixing water needs to be added to the dry grout. Mixing water is around 5.75 to 6.25 pound of water per 50 pounds of grout. Mixing time is three minutes, and setting time is 180 minutes with flowable, self-consolidating consistency that requires no vibration, and no curing at the wide range of operating temperature from 35 to 100° F (2 to 38° C). Polymer-based grout can also be used in cold
weather. Proper sealing of the key way is required to prevent leakage of the grout during casting considering the high flowability. The compressive strength exceeded 16,000 psi after twenty eight days.

3.2.3.2.6 Magnesium-Phosphate Grout:

Two formulations of magnesium-phosphate based grouts were also tested. It is a one-component magnesium-phosphate based mortar; the operating temperature of this grout is from 85 to 100°C (29 to 38°C) which might be a limitation in cold weather applications. The setting and workable time is ten minutes. One and a half minutes of mixing was recommended with mixing water of a maximum of 4.18 pounds per fifty pounds of grout. The manufacturer recommends no curing for these two components except for some protection from rain immediately after placing. Liquid-membrane curing compounds or plastic sheeting may be used to protect the early surface from precipitation.

3.2.3.3 Surface Preparation

The surface roughness effects on key way joints was studied by sandblasting the concrete surface using 120 psi air pressure mixed with sand passing a #30mesh. Figure 3.5 shows the sandblasted surface, and Figure 3.6 shows the as-cast concrete surface. Test specimens were tested for sandblasted surface with UHPG, HSC grout, with and without bonding agent, and with and without cement slurry.
Figure 3.5: Sand blasted surface for joint tests

Figure 3.6: As-cast Concrete Surface for Joint Tests
3.2.3.4 Shrinkage Cracks

The heat of hydration and/or hot weather might cause the mixing water to evaporate soon after placing the grout leading to shrinkage cracks at the surface that will propagate under the service loads (Miller R. H., 1998) Proper curing surface protection might reduce shrinkage cracks by preventing the mixing water from evaporation in the early age of the grout material. No curing, wet curing and curing compound were used separately in this study to monitor their effect on shrinkage cracks.

3.2.3.5 Cement Slurry Effect

The free water at the interface is required to develop bond strength between the fresh cementitious grout and the hardened concrete surface of the box beam girder at the interface. It was previously believed that providing cement slurry can help to increase bonding and shear strength at the interface.

Cement slurry mix was three parts by weight of cement mixed with two parts of water. It was poured from the top opening of the joint to flow onto the key way surface immediately before placing the grout material, as shown in Figure 3.7.
3.2.3.6 Bonding Agent Effect

Two types of bonding agent products are currently available and are commonly used in the construction industry. Type I is a liquid compound that should be applied by painting the surface prior to pouring the grout; this type is not practical for the key way application because of the small width of the key way and the narrow top opening. Type II is a liquid compound that should be mixed with the grout. Type II bonding agent was used in order to evaluate its effect on bond and shear strength of the joint.

3.3 Results and Discussion

The definition of failure, shear strength of the tested specimens, and the failure modes for the tested specimens are presented in this section.

3.3.1 Definition of Failure

The specimens were considered to have failed at the first crack load; further load was applied after the first crack until the specimens fell apart. Specimens were able to carry
load after the first crack because of the bearing strength at the bottom flange that results from the key way shape.

3.3.2 Shear Strength

The average strength of the three test specimens that were grouted with the ODOT-approved grout material and with the standard ODOT geometry was considered as a control strength. The average strength for each of the tested grouts and geometries were compared with the strength of the control test specimens. The average shear strength of the approved ODOT grout with standard key way geometry was 5,700 lb. The failure mode for this grout material was shear failure by debonding at the interface between the concrete units and the grout material.

The maximum average shear strength was 37,000 lb, which was obtained from the specimens grouted with polymer grout and narrow/full depth key way. The failure of these specimens was through the concrete units and not at the joint between the concrete and the grout. All test results are presented in Table 3.3 and Figure 3.8.

The maximum average shear strength for specimens with roller support was obtained using Ultra High Performance Grout with sand blasted surface, Polymer grout, and High Strength Concrete with sand blasted surface with average shear equal to 34,700 lb, 29,666 lb, and 29,367 respectively. All the failures for these specimens were shear failure through the concrete unit. All the specimens had a Wide-Full depth key way.
Table 3.3: % Average Change in First Crack Load

<table>
<thead>
<tr>
<th>Key Way Geometry</th>
<th>Grout Material</th>
<th>Cracking load</th>
<th>% Average change in first crack load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Partial / Narrow key way</td>
<td>ODOT Approved Grout</td>
<td>5,700</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>Magnesium Phosphate (1)</td>
<td>4,767</td>
<td>84</td>
</tr>
<tr>
<td></td>
<td>Magnesium Phosphate (2)</td>
<td>5,033</td>
<td>88</td>
</tr>
<tr>
<td></td>
<td>Polymer Grout</td>
<td>14,833</td>
<td>260</td>
</tr>
<tr>
<td>Full / Narrow key way</td>
<td>ODOT Approved Grout</td>
<td>15,950</td>
<td>280</td>
</tr>
<tr>
<td></td>
<td>Magnesium Phosphate (1)</td>
<td>5200</td>
<td>91</td>
</tr>
<tr>
<td></td>
<td>Magnesium Phosphate (2)</td>
<td>6,167</td>
<td>108</td>
</tr>
<tr>
<td></td>
<td>Polymer Grout</td>
<td>37,000</td>
<td>649</td>
</tr>
<tr>
<td>Partial / Wide key way</td>
<td>ODOT Approved Grout</td>
<td>8,800</td>
<td>154</td>
</tr>
<tr>
<td></td>
<td>Magnesium Phosphate (1)</td>
<td>5,800</td>
<td>102</td>
</tr>
<tr>
<td></td>
<td>Magnesium Phosphate (2)</td>
<td>4,850</td>
<td>85</td>
</tr>
<tr>
<td></td>
<td>Polymer Grout</td>
<td>17,167</td>
<td>301</td>
</tr>
<tr>
<td></td>
<td>Concrete (5,400 psi)</td>
<td>12,500</td>
<td>219</td>
</tr>
<tr>
<td>Full / Wide key way</td>
<td>ODOT Approved Grout</td>
<td>14,550</td>
<td>255</td>
</tr>
<tr>
<td></td>
<td>Magnesium Phosphate (1)</td>
<td>6,400</td>
<td>112</td>
</tr>
<tr>
<td></td>
<td>Magnesium Phosphate (2)</td>
<td>8,333</td>
<td>146</td>
</tr>
<tr>
<td></td>
<td>Polymer Grout</td>
<td>29,667</td>
<td>520</td>
</tr>
<tr>
<td></td>
<td>Concrete (5,400 psi)</td>
<td>16,500</td>
<td>289</td>
</tr>
<tr>
<td></td>
<td>HSC (9786 psi)</td>
<td>17,200</td>
<td>302</td>
</tr>
<tr>
<td></td>
<td>HSC (4282 psi) with bonding agent</td>
<td>17,439</td>
<td>306</td>
</tr>
<tr>
<td></td>
<td>HSC (4282 psi) with bonding agent and sand blasted surface</td>
<td>23,703</td>
<td>416</td>
</tr>
<tr>
<td></td>
<td>HSC (9786 psi) with sand blasted surface</td>
<td>29,367</td>
<td>515</td>
</tr>
<tr>
<td></td>
<td>UHPC - sand blast</td>
<td>34,700</td>
<td>609</td>
</tr>
<tr>
<td></td>
<td>UHPC - cement slurry</td>
<td>5,300</td>
<td>93</td>
</tr>
<tr>
<td></td>
<td>UHPC - sand blast + cement slurry</td>
<td>24,067</td>
<td>422</td>
</tr>
<tr>
<td></td>
<td>UHPC - no Sand blast - no cement Slurry</td>
<td>8,100</td>
<td>142</td>
</tr>
</tbody>
</table>
Figure 3.8: Summary of Test Results for Joint Test without Tie Rod
3.3.3 Failure Modes

Three failure modes were observed. Shear strength of the first failure mode (Type A) is limited by the shear strength of the joint at the interface, shear strength of the second failure modes (Type B) is limited by the strength of grout material, shear strength of and the third failure mode (Type C) is limited by the strength of the concrete units.

3.3.3.1 Failure Mode A

The first type of failure (Failure Mode A) occurs at the interface between the grout and the concrete unit. Failure occurs by large horizontal and/or vertical spread before cracking at the joint. This type of failure was observed in all test specimens with ODOT-approved grout material, normal strength concrete grout, and magnesium-phosphate based grout, UHP Grout without the application of either the cement slurry or sandblasting, and UHP Grout with cement slurry without sandblasting. Failure Mode is shown in Figures 3.9 to 3.12.

Figure 3.9: Failure Mode A
Figure 3.10: Failure Mode A

Figure 3.11: Failure Mode A
3.3.3.2 Failure Mode B

One specimen failed by the second type of failure mode (Failure Mode B). In this mode, failure occurs by shear failure through the grout at 17,167 lb for wide-partial depth key way and polymer grout. This failure mode, shown in Figure 3.13, can be avoided by increasing the depth of the key way. However, it should be noted that the load carried by the specimen before this mode of failure is high.
3.3.3.3 Failure Mode C

The failure occurs by diagonal crack in the middle (loaded) concrete unit. This failure was observed for 12 specimens with HSC grout, 11 specimens out of 12 with the polymer-based grout material, and 6 specimens with the UHPG with sandblasting with or without cement slurry. Details for each are as follows:

3.3.3.3.1 Failure in HSC Grout without Bonding Agent or Sandblasting:

Typical failure occurred by de-bonding and diagonal shear in the middle concrete unit for specimens with HSC grout with average cracking load of 17,200 lb with failure mode as shown in Figure. 3.14.
3.3.3.3.2 HSC Grout with Bonding Agent Failure in HSC Grout with Bonding Agent.

Typical failure occurred by de-bonding and diagonal shear in the middle concrete unit for specimens with HSC grout with bonding agent. Average cracking load was 17,439 lb with the failure mode as shown in Figure 3.15.
3.3.3.3 HSC Grout with Bonding Agent and Sandblasted Surface.

Typical failure occurred by de-bonding and diagonal shear in the middle concrete unit for specimens with HSC grout with bonding agent and sandblasted surface. Average cracking load was 23,703 lb with failure mode as shown in 3.16.
Figure 3.16: Failure Mode C - Failure in HSC Grout with Bonding Agent and Sandblasted Surface

3.3.3.4 HSC Grout without Bonding Agent and Sandblasted Surface.

Typical failure by de-bonding and diagonal shear in the middle concrete unit for specimens with HSC grout with sandblasted surface. Average cracking load = 29,367 lb as shown in Figure 3.17
3.3.3.5 UHPG with Cement Slurry and Sandblasted Surface

Typical failure occurred by de-bonding and diagonal shear in the middle concrete unit for specimens with UHP grout with cement slurry and sandblasted surface. Average cracking load was 24,067 lb with failure mode as shown in Figure 3.18.
3.3.3.3.6 UHPG with Sandblasted Surface

Typical failure for specimens with UHP grout with sandblasted surface. Failure occurred in the middle concrete unit, but not in the grout material. Average cracking load was 34,700 lb. The failure load was limited by the strength of the concrete units, and not the strength of the grout with failure mode as shown in Figures 3.19 to 3.21.
Figure 3.19: Failure Mode C - Failure in UHPG with Sandblasted Surface through Concrete and Grout Interface

Figure 3.20: Failure Mode C - Failure in UHPG with Sandblasted Surface through Concrete
3.3.3.7 Polymer Grout

For partial depth key way, the shear failure plane extended from the middle concrete unit to the bottom slope of the key way. The failure plane showed different slopes depends on the key way depth as seen in Figure 3.22 and Figure 3.23. For the case of partial depth, and in this figure, for full depth key way. The deeper the key way, the larger the ultimate load.
Figure 3.22: Failure Mode C - Failure in Polymer Grout, Narrow-Partial Depth Key Way

Figure 3.23: Failure Mode C - Failure in Polymer Grout, Narrow-Full Depth Key Way
3.3.3.4 Differential Deflection and Lateral Spread

Tension failure was observed in 3 specimens; one specimen with partial depth narrow key way and two specimens with partial depth wide key way. Figure 3.24 is a typical failure mode for the three specimens, failure load was 10,000 psi.

![Graph showing load vs. slip/movement for joints using ODOT-approved grout.](image)

**Figure 3.24: Joints Movements for ODOT-approved Grout vs. Load**
3.3.3.5 Support Conditions

The application of prestressed-precast box beam girders there is no lateral support for the bridge against lateral movements; a roller support was employed for 72 specimens out of the 78 test specimens to allow for lateral spread under symmetric axial loads as shown in Figure 3.25.

![Figure 3.25: Roller Support](image)

Six specimens with Polymer grout were tested using fixed support at the bottom, three specimens had Partial depth-Wide key way and three specimens with Full depth-Narrow key way as shown in Figure 3.26.
The Partial fixation of test specimens at bottom results from the friction between the concrete unit and the steel plates under the vertically applied load. The fixed support results in reduction in the lateral spread as shown in Table 3.4, at the same applied load of 12,500 pounds the lateral spread was reduced to zero with fixed support from 0.007” with roller support

Table 3.4: Support Condition vs. Lateral Spread

<table>
<thead>
<tr>
<th>Grout Material</th>
<th>Polymer Grout</th>
</tr>
</thead>
<tbody>
<tr>
<td>Applied Load</td>
<td>12,500 lb</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Key Way Geometry</th>
<th>Partial Depth - Narrow</th>
<th>Partial Depth - Wide</th>
</tr>
</thead>
<tbody>
<tr>
<td>Support</td>
<td>Roller</td>
<td>Fixed</td>
</tr>
<tr>
<td>Lateral Spread under Loading</td>
<td>0.007”</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>0.0075”</td>
<td>0.004”</td>
</tr>
</tbody>
</table>
The dial gage readings showed that the lower part of the interface undergoes tensile stresses indicated by the positive reading of the horizontal movements in most of the specimens (concrete units spread laterally at the bottom). Many of the recorded readings for the dial gages at the top of the specimen were negative for the Lateral movement, meaning that compressive strains and stresses may have developed at the top. It may be possible to reduce the failure triggered by the spreading of the concrete units at the bottom by placing a tie rod closer to the bottom.

3.3.3.6 Shrinkage Cracks

Shrinkage cracks appeared at the interface between the concrete units and the concrete grout with width ranging from 0.016” to 0.026” (0.4 mm to 0.65 mm).

Figure 3.27: Shrinkage Cracks for Cementitious Grout
3.4 Discussion

Key way geometry, bonding agent, cement slurry, surface roughness and grout material were found to have a critical impact on joints strength as discussed in the following section.

3.4.1 Effect of Key Way Geometry on Vertical Shear Strength

Tests results indicated that joints with deeper key ways have larger strength in resisting shear loads. Load transfer between the middle-loaded concrete unit to the external supported concrete units through the grout consists of two main components: shear resistance at the interface and bearing on the bottom flange. No failure was observed in the bottom flange in any of the test specimens (see Figure 3.28 and also Table 3.5).

![Key Way Geometry vs. Shear Strength](image-url)

**Figure 3.28: Key Way Geometry vs. Shear Strength**
Table 3.5: Key Way Geometry vs. Shear Strength

<table>
<thead>
<tr>
<th>Grout Material</th>
<th>First Cracking Load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Key Way</td>
</tr>
<tr>
<td></td>
<td>Partial / Narrow</td>
</tr>
<tr>
<td>ODOT-approved grout</td>
<td>5,700</td>
</tr>
<tr>
<td>Magnesium Phosphate #1</td>
<td>4,767</td>
</tr>
<tr>
<td>Magnesium Phosphate #2</td>
<td>5,033</td>
</tr>
<tr>
<td>Polymer grout</td>
<td>14,833</td>
</tr>
</tbody>
</table>

3.4.1.1 Increased Shear Strength When Contact Area Was Increased at the Interface

When Narrow/Partial depth key way was compared to Narrow/Full depth key way, the bearing area is constant with a width of 1.5 inches and the interface area was increased from 4” × 7” to 4” × 11”. The same trend was observed when Wide/Partial depth key way was compared to Wide/Full depth key way, with a constant bearing length of 3 inches and interface area increased from 4” × 7” to 4” × 11” as shown in Figure 3.29.

Figure 3.29: Key Way Depth vs. Shear Strength

3.4.1.2 Increased Shear Strength When Bearing Area Was Increased at the Bottom Flange

When Narrow/Partial depth key way was compared to Wide/Partial depth key way, the interface area is constant 4” × 7” with the bearing area increased from 1.5” to 3”. The
same trend was observed when Narrow/Full depth key way was compared to Wide/Full depth key way, the interface area is constant $4'' \times 11''$ with the bearing area increased from 1.5” to 3” as shown in Figure 3.30.

![Figure 3.30: Key Way Width vs. Shear Strength](image)

3.4.2 Bonding Agent, Cement Slurry, and Sandblasting vs. Shear Strength

Bonding agent reduced the strength for all test specimens with HSC grout. The maximum strength was obtained with sandblasted surface and without bonding agent. Cement slurry reduced the strength for all test specimens with UHP grout. The maximum strength was obtained with sandblasted surface without cement slurry as shown in Figure 3.31.
3.4.3 Grout Material

The tested grouts can be divided into three categories, i) Cementitious grouts, ii) Magnesium-phosphate based grouts, and iii) Polymer based Grout. The following observations were extracted from this test.

3.4.3.1 ODOT-approved Grout

For the specimens grouted with an ODOT-approved grout with the current practice for the key way geometry, strength can be increased by increasing the key way depth or increasing key way depth and width.

3.4.3.2 Normal Strength Concrete

Even though surface cracks appeared at the interface for normal strength concrete and the 28-day compressive strength of the concrete used as a key way grout was 5,400 psi, the strength was higher than what was obtained with ODOT-approved grout material. For Wide/Partial depth key way, the average strength was 42% higher than the strength obtained for specimens grouted with ODOT-approved grout.
For Wide/Full depth key way, the average strength was 13.4% higher than the specimens grouted with ODOT-approved grout.

3.4.3.3 High Strength Concrete

For Wide/Full depth key way, the average strength for HDS was 18.2% higher than the specimens grouted with ODOT-approved grout.

For Wide/Full depth key way, with sandblasted surface, the average strength was 102% higher than the specimens grouted with ODOT-approved grout.

This grout is a very good alternative to ODOT-approved grout.

3.4.3.4 Polymer Grout

This grout material is the best option to avoid shear failure and de-bonding failure at the interface because of good bond with concrete units for axially loaded applications.

3.4.3.5 Two Formulations of Magnesium Phosphate Based Grout

This grout material has specific requirements that can be difficult to satisfy in practical conditions. These specifications are:

- Must be mixed, placed, and finished within 10 minutes under normal temperatures (71°F or 21°C).
- No curing is required, but it must be protected from rain immediately after placing.
- Liquid-membrane curing compounds or plastic sheeting may be used to protect the early surface from precipitation, but wet curing should never be done.

3.4.3.6 UHP Grout

For full depth - wide key way and without sandblast or cement slurry, the average strength of UHP grout was 44% lower than the specimens grouted with ODOT-approved grout.
For full depth - wide key way and with sandblasted surface, the average strength of UHP grout was 140% higher than the specimens grouted with Kuhlman 1107 grout. This grout is also a great alternative to Kuhlman 1107.

3.5 Summary

The following conclusions were drawn from the joint tests without ties:

(i) Joint strength can be increased using deeper and/or wider key way.
(ii) Cement slurry and bonding agent does not enhance shear strength.
(iii) Sandblasting the interface can increase the shear strength of the joint significantly.
(iv) Polymer grout has a strong bond and shear strength under shear loads.
(v) Magnesium phosphate grouts have poor performance that can get worse if exposed to humidity and environmental conditions.
(vi) HSC grout is a better option compared to ODOT-approved grouts.
(vii) UHPG has the highest compressive strength and shear strength. Rough interface surface increases the bond and the corresponding shear strength allowing the key way to utilize the high compressive strength of UHPG.

Shear transfer strength of key way joints can be increased by a factor of up to 5.5 compared to the joints that use the current ODOT-approved grouts and ODOT-recommended key way details through proper selection of grout material, adjustment to the key way geometry, and surface preparation of the interface.
CHAPTER IV

STUDY OF GROUT MATERIALS

4.1 Evaluation of Approved Grout Materials

The influence of water content on the workability and compressive strength of one the ODOT-approved commercially available grouts was studied. The manufacturer recommends three ranges of water content:

- Three quarts of water per 50 lb of grout - the manufacturer describes the consistency as “plastic”
- Three and one-fifth quarts of water per 50 lb of grout - the manufacturer describes the consistency as “flow-able”.
- Three and a half quarts of water per 50 lb of grout - the manufacturer describes the consistency as “pump-able”.

The placing of pre-manufactured grouts with the specified mixing water content was difficult even in laboratory conditions for a ¾-inch opening of the key way. Workability of grout was not adequate to place the grout in key ways. Longer working time and more flowability are preferred to ensure that grout can be placed through the ¾-inch opening and the grout flows to the bottom to completely fill the key way. In controlled laboratory tests, the grout for small joint tests was wet cured at room temperature for 28 days, with
no load application on the joints before testing. The highest recommended water content 
(3.5 qt/50 lb) was not adequate for the narrow opening of ¾ inch. Vibration was needed 
to ensure proper compaction of the grout material in the key way in laboratory conditions. 
However, construction sites might not allow ideal conditions. Pavers and rollers driving 
over a newly constructed bridge before 28 days while the grout is curing can load the 
grouted joint prematurely. Actual compressive strengths of grouts at early ages and the 
effect of increased mixing water were therefore studied in this project.

In this task, the influence of water content on the workability and compressive 
strength was studied for the three water contents recommended by the manufacturer and 
three higher ranges of water content: 4, 4.5, and 5 quarts per 50 lb bag of grout. Twelve 
standard test cubes were made for each of the six water content ratios and tested at one, 
three, seven, and 28 days to develop the relation between the water content and the 
compressive strength. The reduction in compressive strength of grouts is significant with 
the addition of water during the entire curing time of the grout. Testing 2” cubes as per 
ASTM C-109 for 1, 3, 7, and 28 days with different proportions of mixing water shows the 
effect of higher amounts of mixing water on the strength to obtained flowability as shown 
in Figure 4.1.
4.2 Performance Improvement of Approved Grout Material

A high-range water reducing agent “SIKA 2100” was used to improve the workability and the compressive strength of grout materials to achieve the desired workability. A slump cone of 2”x4”x6” size was used to measure the spread. With 3 quarts per 50 lb grout, the spread was limited to the diameter of the test cone. The spread could be increased to 7.5 inches using larger amount of mixing water 4.5 quarts per 50 lb grout.

Three mixes were made with a super-plasticizer dose equal to 40, 80, 160 ml per 50 lb grout and three quarts of mixing water per 50 lb grout, and the spread was increased to 14, 15.5, and 17 inches as shown in Table 4.1 and Figure 4.2. These tests provide a solution to the problem of low workability for key way joint application, which can be improved with a suitable dosage of super-plasticizers.
Table 4.1: Increasing Mixing Water vs. the Use of Super-plasticizer

<table>
<thead>
<tr>
<th>Mix #</th>
<th>Superplasticizer dose (ml/50 lb grout)</th>
<th>Water content (quarts)</th>
<th>Spread (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>3</td>
<td>2</td>
</tr>
<tr>
<td>2</td>
<td>0</td>
<td>4.5</td>
<td>7.5</td>
</tr>
<tr>
<td>3</td>
<td>40</td>
<td>3</td>
<td>14</td>
</tr>
<tr>
<td>4</td>
<td>80</td>
<td>3</td>
<td>15.5</td>
</tr>
<tr>
<td>5</td>
<td>160</td>
<td>3</td>
<td>17</td>
</tr>
</tbody>
</table>

Figure 4.2: Inadequate Workability with 3 Quarts Water per 50 lb of Grout (Left); Improved Workability with 3 Quarts Water and 40 ml Super-plasticizer per 50 lb Grout (Right)

4.3 Development of High Compressive Strength Grout

It was determined from the joint tests that the higher the compressive strength of the grout, the higher the shear transfer strength of joint. Therefore, a high strength concrete grout with #8 aggregate was developed with mixture proportions as shown in Table 4.2. This grout performed much better than ODOT-approved grout both in terms of workability and compressive strength.
A traditional concrete mix design was developed through trials to achieve a flowable high strength concrete mix with 10,000 psi compressive strength, the mix proportions are shown in Table 4.2.

Table 4.2: Mix Proportions for High Strength Concrete Grout with #8 Aggregate

<table>
<thead>
<tr>
<th>HSC Grout</th>
<th>Cement</th>
<th>Sand</th>
<th>Aggregates</th>
<th>Water W/C = 0.35</th>
<th>S.P.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mix proportion by weight</td>
<td>1</td>
<td>1.32</td>
<td>1.45</td>
<td>0.35</td>
<td>0.03</td>
</tr>
<tr>
<td>lb/ft³</td>
<td>41</td>
<td>54</td>
<td>59</td>
<td>14.5</td>
<td>1.7</td>
</tr>
<tr>
<td>lb/yd³</td>
<td>1100</td>
<td>1450</td>
<td>1596</td>
<td>385</td>
<td>46</td>
</tr>
</tbody>
</table>
CHAPTER V

FIELD MEASUREMENTS OF VERTICAL DIFFERENTIAL DEFLECTIONS AND SEPARATION OF LONGITUDINAL JOINTS UNDER TRUCK LOADING

5.1 Objective

The objective of measuring differential vertical deflections and horizontal separation at the longitudinal joints of typical box beam bridges was to determine the movements that can be expected in an actual bridge when subjected to traffic loading. An understanding of the magnitudes and the nature of these movements would help to evaluate key ways and to understand the load paths and the stress condition. Based on the extent of the measured differential deflections and separations, the meaning of failure for these joints in actual bridges can be defined. The differential deflections and the horizontal movements of the laboratory test specimens can be correlated to the actual site measurements in order to define “failure” in our laboratory test specimens and develop strategies to minimize failures.

5.2 Methodology

Joints with visible and invisible cracks at the surface of the concrete asphalt were selected for the differential deflection measurements, the bridge and the locations of measurements are described in the flowing sections.
5.2.1 Bridge Selection

The database provided by ODOT for the entire state was reviewed and several bridges were physically inspected for down selection for detailed measurements of differential vertical deflections and horizontal movements. Some of the bridges inspected were not found to be suitable for obtaining detailed measurements. The water leakage problem does not appear to have a geographical or statistical trend. Lack of water tightness, cracking at longitudinal joints, and joint failures seem to be common problems in the bridges on highways throughout the state and many county roads.

Bridge ASD-42-12.49 in ODOT District 3 was selected for measuring vertical differential deflections and horizontal differential movements under truck loading. This bridge is a non-composite bridge having a 60-ft. span, thirteen precast-prestressed box beams tied with three sets of tie rods in the transverse direction, and an asphalt concrete wearing course. One set of tie rods was provided at an intermediate location as shown in Fig. 5.1.

Figure 5.1: Path of Truck Wheel for Deflection Measurements ASD-42-12.49
This bridge was recognized by the bridge engineers to be defective, and the key way joints between adjacent box beams were documented to have longitudinal cracks. These cracks appeared on the surface of the asphalt concrete layer as seen in Fig. 5.2. Cover spalling and corrosion for the reinforcement was observed at the underside of the box beams of the bridge.

The bridge had the following desired characteristics to be selected for this testing:

- It had around 12 ft of clear height under the bridge for the ease of installing dial gages at the bottom of the girders.
- The bridge is located on a straight highway to allow the loaded test truck to pick up speed (up to 70 mph) before reaching the bridge and to slow down after driving over the bridge without the need for traffic control.
- There was easy access to the area below the bridge to allow work under the bridge, i.e., no dense vegetation, deep water in the stream, or steep side slopes.

5.2.3 Measurements

The locations of interest for recording deflections were carefully selected based on the visual inspection of the bottom of the bridge to identify the locations where the water leakage damage had occurred and where cracks were clearly visible on the surface of the asphalt concrete overlay. Four longitudinal joints were selected out of the twelve joints between the box beams. Dial gages were installed on the underside of the beams to measure the movements at 11 locations as shown in Figure 5.2 and Figure 5.3.
Figure 5.2: ASD-42-12.49 Bridge and Measurements Layout

Dial gages were installed to measure the movement at 11 locations as listed in Table 5.1 and shown in Figure 5.4
The locations of the joints between the box beams were located and measured at the bottom of the bridge and marked on the top on the asphalt concrete surface by carefully transferring the corresponding points from the bottom of the bridge. A truck was loaded to
a total load of 67.4 kips (including the self-weight of the truck) and was driven over the bridge at speeds of 50 or 70 mph following the marked paths. The loaded truck was guided to drive on the beam next to the longitudinal joint where the gage readings were taken. The differential deflections and the lateral spread were videotaped and later analyzed to obtain measurements of the maximum movements.

The intermediate tie rod was at 40 ft from the south pier and 18ft-8in from the north pier. The measurements were obtained from gages at 1, 4, and 7 at the middle tie rod location. Points 4, 5, 8 were located at half the distance between the middle tie rod and the north pier. Points 3, 6, and 9 were located at half the distance between the tie rod and the south pier to monitor the effects of the clamping forces provided by the tie rod and the effects of the spacing between tie rods. Points 10 and 11 were located where severe cracks were visible. The 67.4 kips loaded truck was guided to drive on the girder next to the joint and the differential deflection and the lateral spread was videotaped and analyzed to find the maximum movement with 50 and 70 mph speeds.

5.3 Test Results and Discussion

The measured vertical differential deflection and lateral relative spread between the adjacent girders were the least at the middle tie rod location, with values equal to 0, 0 for Points 2, 0.002, 0.0018 for Points 5, and 0.001, 0.001 for Point 8 (Table 5.1). The maximum vertical differential deflection was recorded at the middle of the length between the middle tie rod and the south pier with 40 ft spacing between the tie rods. Fig. 5.3 shows typical vertical differential deflections at three locations. The maximum recorded vertical differential deflection was 0.0045 inch. The maximum horizontal separation at the underside of the box beams was measured to be 0.015 inch. Fig. 5.5 shows the vertical
differential deflection at Points 7, 8, and 9. For all locations, the movements were the largest at the time when the truck was driven right above the measured point.
Table 5.1: Points Significance and Measured Values

<table>
<thead>
<tr>
<th>Test point</th>
<th>Point significance</th>
<th>Vertical differential deflection (in)</th>
<th>Horizontal differential deflection (in)</th>
<th>Truck speed (mph)</th>
<th>Truck load (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>At middle distance between the north pier and the middle tie rod</td>
<td>0.001</td>
<td>---</td>
<td>50</td>
<td>67.4</td>
</tr>
<tr>
<td>1</td>
<td>At middle distance between the north pier and the tie middle rod</td>
<td>0.002</td>
<td>---</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>At the middle tie rod location</td>
<td>0.0</td>
<td>0.0</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>At middle distance between the south pier and the tie middle rod</td>
<td>0.001</td>
<td>0.0</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>At middle distance between the north pier and the middle tie rod</td>
<td>0.0015</td>
<td>---</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>At middle distance between the north pier and the middle tie rod</td>
<td>0.0025</td>
<td>---</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>At the middle tie rod location</td>
<td>0.002</td>
<td>0.0018</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>At middle distance between the south pier and the middle tie rod</td>
<td>0.003</td>
<td>0.015</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>At middle distance between the north pier and the middle tie rod</td>
<td>0.004</td>
<td>0</td>
<td>70</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>At the middle tie rod location</td>
<td>0.001</td>
<td>0.001</td>
<td>70</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>At middle distance between the south pier and the middle tie rod</td>
<td>0.0045</td>
<td>0.0095</td>
<td>70</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>Cracks on the surface aligned with the joint location</td>
<td>0.001</td>
<td>0.0015</td>
<td>70</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>Cracks on the surface aligned with the joint location</td>
<td>0.0015</td>
<td>0.001</td>
<td>70</td>
<td></td>
</tr>
</tbody>
</table>
5.4 Summary of Findings

The following findings were noted, based on field measurements obtained for bridge ASD-42-12.49:

- Tie rod clamping-force reduces the relative movements between the adjacent box beams.

- The larger the distance between the tie rod locations, the larger the relative movements between the adjacent box beams in a bridge.
CHAPTER VI

BEAM ASSEMBLY TESTS FOR SYMMETRIC LOADING

6.1 Objective

Joint tests described in Chapter III of this report were conducted with a large number of variables that include a wide range of geometries and grouts. The joint tests provided a basis for selecting the parameters for further testing and evaluation at a larger scale. The symmetric beam assembly tests conducted in the laboratory in this project are a simplified representation of three adjacent box beam units tied together to act as a single unit. The primary objective of the beam assembly tests was to study the joint strength and behavior under symmetric loading, for the application of new bridge construction and rehabilitation of existing bridges with failed joints while maintaining the current practice of key way geometry.

6.2 Test Procedure

Each beam assembly comprised three concrete units with two longitudinal key ways. Load was applied to the middle unsupported beam unit at the midspan, and the two external beams were symmetrically supported at the ends. The middle beam was not supported at the ends. This support condition ensured that the load applied to the middle beam at the midspan is transferred to the end supports.
through the two longitudinal joints symmetrically. The current ODOT practice in
terms of key way geometry and grout material was used to develop the baseline test
results as a starting point.

6.3 Test Specimen Configurations

Seven sets of beam assemblies were tested according to the following schedule

- Set # 1 and 2 had standard key way geometry with ODOT approved grout
  and as-cast concrete surface at the interface, (new, unused assemblies)
- Set # 3 had standard key way geometry with Polymer grout and as-cast
  concrete surface at the interface, (re-grouted assembly).
- Set # 4 had full depth – wide key way with ODOT approved grout and as-
  cast concrete surface at the interface, (new, unused assembly)
- Set # 5 had full depth – wide key way with HSC grout and as-cast concrete
  surface at the interface, (new, unused assembly)
- Set # 6 had full depth – standard width key way with Polymer grout and as-
  cast concrete surface at the interface, (new, unused assembly)
- Set # 7 had full depth – wide key way with HSC grout and sand blasted
  concrete surface at the interface, (re-grouted assembly).

Set # 1 and 2 had the standard key way geometry, set # 3 was a remanufactured
assembly; after testing set # 1 the assembly was taken apart, the old grout was removed
from the joints, the concrete surface at the interface was sandblasted, and then re-grouted
using polymer grout. Set # 4, 5, and 6 were made out of unused concrete units. Set # 7 was
remanufactured using set # 4 with the same procedure of remanufacturing set # 3 and
grouted with HSC grout. Set#5 and set#6 had damages in the concrete units during testing so that; it could not be remanufactured.

Typical details of the beam assembly test specimens are shown in Figure 6.1, Figure 6.2, and Figure 6.3. The complete details for each test specimens are presented in the next section.

![Diagram of Test Specimen](image)

**Figure 6.1: Test Specimen for Set#1, 2, and 3**
Figure 6.2: Test Specimen for Set#4, 5, and 7

Test specimen for:
- set # 4 ODOT approved grout - as cast concrete surface
- set # 5 HSC grout - as cast concrete surface
- set # 7 HSC grout - sand blasted surface

Figure 6.3: Test Specimen for Set#6

Test specimen for set # 6
Polymer grout
6.4 Specimen Design

Each of the six sets comprise three beams tied together using 1” tie rod at the ends of the beams through 2” holes. A torque of 250 ft-lb was applied to the tie rods to provide 15 kips of clamping force at each end of the beam to match the applied force in ODOT current practice. Lengths of 1 foot at each end of each unit were formed without key way, and the rest of the joints had different key way geometries. Supports were provided to the external beams only at both ends of each beam. LVDT’s(Linear Variable Differential Transducers) and dial gages were provided to measure the vertical deflection, lateral movements, and the vertical movements for the middle “unsupported” beam at the ends (near the tie rods). Strains in the internal reinforcing bars and concrete top surface at the midspan were recorded using strain gages and a data acquisition system.

6.5 Section Design

The three concrete units were designed to fail in a tension control mode using a strain-compatibility design method. Actual stress–strain curve for the reinforcing steel bar was used to determine the load-carrying capacity of the beams. Each beam was designed to support a point load of 74 kips at the midspan. The estimated moment capacity of the beam at failure was 278 ft-kips. The overall length of the beam was 16 feet and the simply supported span was 15 feet. The 28-day average compressive strength of concrete units was 10,000 psi. The design mix of the concrete units and reinforcement details for a typical cross section are shown in Table 6.1 and Fig. 6.4.
6.5.1 Concrete Mix Design

Table 6.1 shows the mix design used for the concrete units. #8 limestone was the maximum size of the aggregates to ensure proper compaction and consolidation for the concrete in the heavily reinforced beams.

Table 6.1: Mix Design of Concrete Units

<table>
<thead>
<tr>
<th>Material</th>
<th>Amount</th>
</tr>
</thead>
<tbody>
<tr>
<td>#8 Limestone</td>
<td>1500 lb/yd^3</td>
</tr>
<tr>
<td>Sand</td>
<td>1320 lb/yd^3</td>
</tr>
<tr>
<td>Type I Cement</td>
<td>750 lb/yd^3</td>
</tr>
<tr>
<td>Micro Silica Fume</td>
<td>50 lb/yd^3</td>
</tr>
<tr>
<td>Air Entrainment</td>
<td>0.20 oz/cwt</td>
</tr>
<tr>
<td>Viscocrete 2100</td>
<td>5.0 (±)oz/cwt</td>
</tr>
<tr>
<td>Potable Water</td>
<td>280 lb/yd^3</td>
</tr>
</tbody>
</table>

The concrete cross section of the middle loaded beam was 6” X 22”.

6.5.2 Concrete Section

![Figure 6.4: Section Design of Concrete Units](image)
Width of beam \( b = 6 \text{ in} \)

Over all height of concrete \( h = 22 \text{ in} \)

Depth from top concrete fibers to the centroid of rebar
\[ d = 20.5 \text{ in} \]

Depth to extreme bottom fibers of steel for \( \Phi \) factor calculation
\[ d_t = 21.25 \text{ in} \]

Concrete compressive stress
\[ F'_c = 10,000 \text{ psi} \]

Crushing strain at concrete
\[ \epsilon_{cu} = 0.00421 - 0.00138 \times ((f'_c - 1450) / 5800) = 0.0021757 \text{ in/in} \]

6.5.3 Reinforcement

Two #3 bars were used as top reinforcement to hang the stirrups only and not considered for strength calculation, the bottom rebar was 6 bars #6 in two layers, 60 ksi steel with modulus of elasticity \( E_s = 29,000 \text{ ksi} \)

Area of bottom reinforcement
\[ A_s = 6 \times 0.44 = 2.64 \text{ in}^2 \]

Steel strain at the end of linear stage
\[ \epsilon_s = 0.00138 \text{ in/in} \]

Steel strain at failure
\[ \epsilon_s = 0.009623 \quad \text{Assumed based on trial and error} \]

Tensile stresses in bottom rebar
\[ F_s = 67620 - (38.12 + \epsilon_s) = 63658.657 \text{ psi} \]

Reinforcement ratio
\[ \rho = \frac{A_s}{A_c} = 6 \times 21.25 \div 2.64 = 0.0207059 \text{ in}^2/\text{in}^2 \]

\[ \alpha_u = 0.961 - 0.167 \times \left( (f'_c - 1450) \div 5800 \right) = 0.714819 \]

\[ \beta_u = 0.474 - 0.085 \times \left( (f'_c - 1450) \div 5800 \right) = 0.3486983 \]

Tension force in bottom reinforcement \[ T = A_s \times F_s = 2.64 \times 63658.657 = 168058.86 \text{ lb} \]

Depth to neutral axe

\[ C = T \div (\alpha_u \times f'_c \times d) = 63,658.657 \div (0.715 \times 10,000 \times 20.5) = 3.918448 \text{ in.} \]

Check the assumption of \( \epsilon_s \) using similar triangles from strain diagram

\[ = \epsilon_{cu} \times \frac{d - c}{c} = 0.0021757 \times \frac{21.5 - 3.918448}{3.918448} = 0.0096 \]

Strain at extreme bottom fiber of steel at failure

\[ \epsilon_t = (\epsilon_{cu} \times d_t + c) - \epsilon_{cu} = 0.009 > 0.005 \]

Tension controlled failure with \( \Theta = 0.9 \)

\[ M_n = A_s \times f_s \times (d - \beta_u \times c) = 2.64 \times 63658.66 \times (20.5 - 0.35 \times 3.92) \]

\[ = 3,341,622.45 \text{ lb} - \text{in} = 278.47 \text{ kip - ft} \]

Maximum load for individual beam failure

\[ P = 4 \times M_n \div L = 4 \times 278.47 \div 15 = 74.26 \text{ kips} \]

The assembly as unit will stand up to \( 3 \times 74.26 = 222 \text{ kips} \) of testing load if joints didn’t fail at earlier load.

6.5.4 Shear strength of individual beam

\[ V_u = Vc \div 2 + Vs = 2\sqrt{f'_c} + Av \times f_y \times \frac{d}{s} \]

\[ = 2\sqrt{10,000} \div 2 + 0.22 \times 60,000 \times 20.5 \div 6 = 45,200 \text{ lb} \]

Maximum allowed load
\[ P = 2V_u = 2 \times 45,200 = 90,400 \text{ lb} \]

Allowed load for shear failure of individual beam = 90,400 lb > Allowed load for flexure failure = 74,258 lb.

The concrete units were cast in the lab under controlled room temperature of 70 degrees using plywood for the formwork that is painted with form oil to protect the surface from damage during de-molding, the total weight of the assembly after grout were estimated to be around 8,800 lb. Figure 6.6 and Figure 6.7, show the formwork for nine concrete units used to assemble set#4, 5, 6, and 7 before and after placing the concrete.
Figure 6.6: Form Work for Concrete Units

Figure 6.7: Cast Concrete Units for Set #4 to Set #7

Figure 6.8 shows the concrete units after assembly before grouting for set#1 with top opening of 0.75 inch and key way total depth of 7 inches, the first foot at each end did not
have key way to match the current practice where the tie rod and the transverse loads are applied in solid diaphragm. The applied torque was 250 lb-ft that results in 15 kips of transverse clamping force.

![Figure 6.8: Beam Assembly for Set# 1 before Grouting](image)

6.6 Curing and Shrinkage Cracks

Sets #1 and 2 were wet cured for 7 days without protecting the surface with burlap or plastic cover. Shrinkage cracks appeared on the top surface shortly after grouting. The width of the shrinkage cracks were 0.006 and 0.02 for Set#1, and Set #2 respectively, as shown respectively as shown in Figure 6.9, and Figure 6.10.
Figure 6.9: Crack Width - Set# 2

Figure 6.10: Crack Width - set# 1
Set#3 through set#6 were wet-cured for seven days and covered with wet burlap and plastic sheets, no shrinkage cracks appeared in the curing period, very minor cracks appeared after wet curing was stopped. Set#7 was cured once using curing compound immediately after grouting and covered with plastic sheet, shrinkage cracks did not appear at all as shown in Figure 6.11.

Figure 6.11: Curing Compound Eliminates Shrinkage Cracks

Beam Assembly Units Re-grouted with High Strength Concrete with #8 Aggregate after Sandblasting are shown in Figure 6.12.
6.7 Loading

A static load was applied at the mid-span of the middle unsupported concrete unit using a closed-loop hydraulic actuator (For sets#1, and set#2 with capacity of 55 kips). For Sets # 3 through 6, load was applied using a 300-kip hydraulic jack with a 200-kip capacity load cell attached to record the loading as shown in Figure 6.13. Set#7 was tested using 560-kips hydraulic jack. The load was applied at a rate of 70 lb/sec until the joint failed.
6.8 Test Setup, Instrumentation, and Measurements

Concrete strain gages, rebar strain gags, dial gages, and LVDT’s were used to monitor the strains and the movements of the concrete units to determine the load sharing between the loaded middle-unit and the supported external-units.

6.8.1 Strain Gages

Two steel reinforcing bar strain gages were attached to the bottom reinforcement in the tension zone of each unit of the three-beam assembly to measure the strains in the tensile reinforcing bars. A strain gage was attached on the top surface at the top compression fiber to measure the compressive strain on the surface of the concrete. The strain gage data were acquired through a data acquisition system during testing.
6.8.2 Dial Gages and LVDTs

LVDTs were attached to the concrete surface at locations shown in Figure 6.14 to measure the vertical deflection and the lateral spread of the concrete units during testing. The total lateral spread at the quarter span at the bottom of the two external beams was measured by adding the lateral movements of the two external beams acquired from LVDT 1 and LVDT 2.

Dial gages and LVDTs were deployed as follows:

- Gages $G_2$ and $G_3$ measured the total lateral spread at the top at the quarter span
- $LVDT\#3$ and $LVDT\#4$ measured the lateral spread at midspan at the bottom of the concrete beams
- $LVDT\#5$ and $LVDT\#9$ measured the lateral spread at midspan at the top of the concrete beams
• LVDT#10 and LVDT#11 measured the lateral spread at the ¾ span at the bottom of the concrete beams

• Gages $G_6$ and $G_7$ measured the total lateral spread at the top at the ¾ span

• Gage G4 measured the vertical differential deflection between the middle loaded concrete unit and one of the external concrete units at the quarter span

• Gage G5 measured the vertical differential deflection between the middle loaded concrete unit and the other external concrete unit

• Gage G8 measured the vertical differential deflection between the middle loaded concrete unit and one of the external concrete units at the ¾ spans

• G9 measured the vertical differential deflection between the middle loaded concrete unit and the other external concrete unit

• The vertical differential deflection between the middle loaded concrete unit and the external units was measured by subtracting the deflections at LVDT#6 from LVDT#7 on one side and LVDT#8 from LVDT#7 on the other side

• The vertical differential deflections at supports were measured from LVDT #12 and G1 directly as the external units were prevented from vertical movements with roller supports, as shown in Fig. 6.15.
The base holder for G4, G5, G8, and G9 were rigidly attached to the middle concrete unit so that they obtained the vertical differential deflection readings directly, as shown in Figures 6.16 and 6.17.
Figure 6.16: Typical Cross-Section of Beam Assembly at Quarter Spans

Figure 6.17: Typical Cross-Section of Beam Assembly at Three Quarter Spans
The rebar strain gages are exposed to excessive disturbance during casting concrete and vibrating the fresh concrete in the form work; two gages were installed in each beam for the expected damage to some of the rebar strain gages as shown in Figure 6.18. The concrete strain gages were exposed at the surface. We had chance to replace the damaged gages before testing.

Figure 6.18: Beam Assembly’s Cross-section at Midspan
6.9 Test Results

The reported test loads in this section are the shear strength of the two joints in the three concrete units assembly. Load sharing was determined based on the strains and relative movements between the units.

6.9.1 Load Carrying Capacity

Table 6.2 and Figures 6.19 and 6.20 present the results of the symmetric loading tests of the beam assembly specimens. The following observations were made from these tests:

- For Set # 1 and Set # 2 with ODOT-approved grout and standard geometry which are considered as control test specimens, the first cracking load was 37,500 lb and 38,000 lb for these two specimens, respectively. The load carrying capacity was increased by 95% when a deeper key way of 21 inches was used with a top opening of 1” and 2” wide key way below the top opening using the same grout material.

- Comparing the results of Set # 1 with those of Set # 3, a 223% increase in the first cracking load was obtained for the same key way geometry when polymer grout was used, with an increase in the load from 37,500 lb to 121,140 lb. This suggests that polymer grout could be considered for rehabilitation of existing structures.

- For full depth key way (21 inch), the HSC grout was stronger than the currently used grout with a load capacity of 107,027 lb compared to 72,994 lb. This reflects a 46% increase from Set # 4 to Set # 5.

- Full depth key way (21 inch) with polymer grout in Set # 6 had a superior load carrying capacity, when compared to all the grout materials tested in this project,
with a strength of 131,191 lb. This is a 250% increase in first crack load when compared to a strength of 37,500 lb obtained for Set # 1.

- For HSC grout the sand blasted surface could increase the first cracking load from 107,026 lb in set#5 to 275,000 lb in set#7 with 157% increase and the overall increase compared to the 37,500 lb from set#1 was 633%

<table>
<thead>
<tr>
<th>Set#</th>
<th>Grout Material</th>
<th>Cracking Load (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>ODOT-Approved Grout As-Cast Concrete Surface</td>
<td>37,500</td>
</tr>
<tr>
<td>2</td>
<td>ODOT-Approved Grout As-Cast Concrete Surface</td>
<td>38,000</td>
</tr>
<tr>
<td>3</td>
<td>Polymer Grout As-Cast Concrete Surface</td>
<td>121,140</td>
</tr>
<tr>
<td>4</td>
<td>ODOT-Approved Grout As-Cast Concrete Surface</td>
<td>72,994</td>
</tr>
<tr>
<td>5</td>
<td>High Strength Concrete As-Cast Concrete Surface</td>
<td>107,026</td>
</tr>
<tr>
<td>6</td>
<td>Polymer Grout As-Cast Concrete Surface</td>
<td>131,191</td>
</tr>
<tr>
<td>7</td>
<td>High Strength Concrete Sandblasted Surface</td>
<td>275,000</td>
</tr>
</tbody>
</table>
Figure 6.19: First Crack Load of Tested Assemblies
Figure 6.20: Percentage Change in the Strength of the Tested Assemblies

6.9.2 Lateral Separation

For set#1 the assembly was loaded until failure, the first cracking load was 37.5 kips, large horizontal movement was recorded at the top of the assembly at mid-span, change from 0.002 in to 0.016 was recorded before and after the failure load as shown in
Figure 6.21, the assembly was unloaded, the lateral spread had 0.006 inch of permanent spread, the assembly was reloaded again to capture the behavior of the assembly with failed joint, the lateral spread at 37.5 kips was 0.0185 inch. The lateral spread significantly higher that the spread before failure at first crack, hence the first crack was convenient to define the failure of the joint.

![External beams (B1 + B3) @mid-span (LVDT # 5+9) - set#1](image)

**Figure 6.21:** Set#1 - Lateral Spread vs. Load at Midspan

For set#1 the first cracking load was 37.5 kips. The differential deflection at mid span at first crack load was 0.12 inch as shown in Figure 6.21 which is greater than the vertical differential deflection measured at site with 67.4 kips moving vehicle at 70 mph; that was 0.045 inch.
6.9.3 Differential Deflection

Figure 6.22: Set# 1 - Differential Deflection at Mid-span
6.9.4 Failure Modes

Typical failure occurred in the tested assemblies by cracking along the joint at the middle of the beam length under the applied loads. These cracks did not extend to the supports at the end of the beams. This may be due to the local failure from stress concentration under the applied load and/or the clamping force of the tie rods at the ends that may have prevented the lateral spread and reduced the vertical differential deflection.

It was previously stated in this report that the tie rods increase the shear strength and reduce the vertical slip locally at its location due to the clamping force and the shear resistance provided by the cross sectional area of the tie rod itself.

The beams were completely disassembled after testing in order to completely break the bond between the grout and the concrete units at locations away from the cracked regions. The disassembly of the beams was very hard and needed lots of efforts. The assemblies were taken apart without damaging the concrete units for Set # 1 as shown in Figure 6.23. These units were re-grouted to form Set # 3 to test the polymer grout to simulate rehabilitation/repair process.
In Set # 2, the grout was firmly attached to the concrete units. It was not possible to separate the grout from the concrete units without damaging the concrete units, as shown in Fig. 6.24. Therefore, it was not possible to reuse the concrete units from Set # 2 for further rehabilitation or testing.
During the testing of the beam assembly with polymer grout (Set # 6), the first cracking was detected at a load of 131,191 lb. The joints between the concrete beam units had longitudinal cracks at the middle of the joint length under the applied loads. The grout detached from the middle beam as shown in Figure 6.25.

![Figure 6.25: Set # 6 Middle Concrete Unit after Failure](image)

However, the polymer grout in Set # 6 was still intact and well bonded with one of the external concrete units at the same location as shown in Fig. 6.26, which indicates that a local failure of the joint may have occurred at one side only near the middle cracked portion of the joint.
Grout was removed from the tested beam assemblies using Jack hammer without damaging concrete beam units or the joint surfaces for Set # 4. The restored concrete units were used to assemble Set # 7 to test to determine the effects of sand blasting with full depth key way with HSC grout as shown in Fig. 6.27.
• The beam assembly originally grouted with ODOT-approved grout failed at 73 kips (Set #4) without damage to the concrete units. It was possible to remove the grout, sandblast the surface, and re-grout this beam assembly with HSC containing #8 aggregate.

• The beam assembly originally grouted with HSC with #8 maximum aggregate size failed at 107 kips (Set #5) with damage to the bottom flange and this beam assembly could not be reused after its failure.

• The assembly originally grouted with polymer grout failed at 131 kips (Set #6) with crushing of the top compression zone and wide cracks in the bottom flange. It was not possible to reuse this beam assembly.

• Set# 7, the remanufactured assembly using set #4: The beam assembly was loaded to 200 kips, then unloaded, the tie rods was removed from both ends of the assembly, then reloaded to 275 kips to failed by flexure cracks in the middle concrete unit. The joint was intact and excessive force was needed to separate the three concrete units. The concrete cover was separated with the grout material, the reinforcement was exposed as shown in Figure 6.28 and Figure 6.29.
Figure 6.28: Flexure Failure in the Middle Concrete Unit
6.9.5 Load Sharing Between the Concrete Units Before and After Cracking

Concrete compressive strains and the tensile strains in the bottom reinforcing bars indicated equal load distribution between the loaded and un-supported middle concrete unit and the external supported concrete units.

For Set # 6 with first cracking load of 131 kips, the bond failure between beam units B3 and B2 accompanied with drop in concrete strains at gage C3 and an increase in the stains in C1 and C2 indicated that B3 has released loads while B2 and B1 retained and attracted more loads at the time of the release from B3 as shown in Figure 6.30.
For Set # 5, the compressive strain in concrete at the top fibers was identical up to a failure load of 107 kips. The drop in concrete strain gage C1 and increase in stains C2 and C3, indicating bond failure in one joint only. The drop in C1 indicated that the load was released from the detached unit and the increased strain in C2 and C3 indicated that they attracted more load after joint failure as shown in Fig. 6.31.
At first crack (73 kips), the load sharing between the beams in Set # 4 was significantly reduced; strain in the bottom tensile reinforcement increased for R2 and R3. However R1 had reduced strain, indicating de-bonding between beam units B1 and B2, no failure between B2 and B3 as shown in Figure 6.32.
6.9.6 Differential Deflection and Lateral Spread of Beam Assemblies

The conclusion of local failure from the previously discussed failure modes can be supported by investigating the differential deflection behavior of the tested assemblies. In Set # 3 with polymer grout and standard key way geometry, the vertical differential deflection at the quarter span measured by gages G4 and G5 indicated no vertical differential deflection occurred up to 100 kips of load. At that load, a sudden increase in strain in gage G4 occurred before reaching the failure load and sudden increase from 0.00 to 0.001 in gage G5 at the failure load of 121 kips occurred as shown in Fig. 6.33.
For Set # 6 with polymer grout and full depth key way with 131 kips of cracking load, the vertical slip at midspan started at 60 kips of load, which is less than half the cracking load. Polymer grout could accommodate up to ±0.05 inch of differential deflection before failure as seen in Figure 6.34. This ability to accommodate the differential deflection is high when compared to Set # 5 with HSC used as a grout material with a failure load of 107 kips with the same key way depth and with vertical differential deflection of 0.02 inch.
Figure 6.34: Set# 6 Differential Deflection at Midspan
In Set #5, where HSC was used as a grout material with failure load of 107 kips, no lateral spread occurred at the quarter span before the failure at both joints. The increase in readings in G5 indicates that there is separation and joint failure between B1 and the middle beam B2, while B2 and B3 are still intact. The separation and the vertical differential deflections took place before the actual crack at a load of 92 kips, which is slightly lower than the first cracking load of 107 kips as seen in Fig. 6.35. Similar behavior was observed before and after failure at the three-quarter span as shown in Fig. 6.36.
Figure 6.36: Differential Deflection at Quarter Span

(Concrete grout) Differential deflection @ L/4 - set #5

Vertical Slip (in)

Load (lb)

Differential deflection @1/4 Span G5 (B2 - B1)

Differential deflection @1/4 Span G4 (B2 - B3)
For Set # 3 at the quarter span, no lateral spread was recorded up to a load of 100 kips, and at the three-quarter span. The negative lateral spread at the midspan indicated compressive stresses at the top fibers before failure while the joint was still intact. At higher loads and at failure, that compression was released when the joint broke and allowed the units to move separately as shown in Figure 6.38.

Figure 6.37: Set # 5 Differential Deflection at Three Quarter Span
Figure 6.38: Set #3 Lateral Spread at Top

Figure 6.39: Set #7 Differential Deflection at Midspan
6.10 Summary

The following observations were made based on the laboratory tests of box-beam assemblies:

1. The polymer grout has a superior performance with flowable self-consolidating properties that do not need vibration or widening of the key way opening. Polymer grout has three hours’ working time after mixing over a wide range of temperatures that can provide flexibility in terms of temperature condition at the time of grouting. Polymer grout had 3.2 times the strength of the currently ODOT-approved cementitious grout using ODOT standard key way geometry. Therefore, polymer grout is a potentially implementable grout.

2. HSC concrete with a maximum #8 aggregate size is a better option for the higher shear strength obtained with full depth key way. The shear strength obtained with HSC concrete was 107 kips compared to the currently ODOT-specified grout that had 73 kips of shear strength. The shear strength obtained with HSC concrete with sandblasted surface was 275 kips compared to the currently ODOT-specified grout that had 73 kips of shear strength.

3. Sandblasted key way surface can increase the shear strength by 157% for the same grout material and same key way geometry.

4. Deeper key way can increase the shear strength by 95% for the same grout material.

5. Shear failure is local under the applied loads, and joint cracks do not extend throughout the length of the beam.

6. The failed joints may undergo higher deflections after failure when subjected to further loading beyond the time at which the first crack appears.
7. The assembled units had perfectly symmetric load sharing before cracking under the symmetric applied loading.
CHAPTER VII

ANALYSIS FOR ECCENTRIC LOADING

7.1 Introduction

The joint tests and the beam assembly tests presented in Chapters III and VI were designed to determine the shear transfer strengths at the joints with standard and modified geometries and to determine the suitability of the grouts under symmetric loading conditions. In reality, the load application under traffic loading condition on the box beams is not symmetric. Multiple adjacent box beams need to act together in sharing the loads from axle loading so that the load is transferred over more than one beam. For this interaction to happen between box beams, the longitudinal joints between adjacent box beams need to transfer the load resulting from the beams located within the effective width of the loading.

The interacting forces at the joints depend on the joint details, the bond that is mobilized between the grout and the box beam key way recess, and the joint response to the internal forces developed at the interface. It is generally well accepted that the interacting forces can cause shear loading at the longitudinal joints. However, the positioning of wheel loads is expected to cause out-of-plane moments at the joint concurrent with the shear loading.
The out-of-plane moments are developed due to the eccentricity of the loading relative to the centerline of individual box beam. This aspect of longitudinal joint behavior was studied using different finite element models and modeling approaches and is presented in this chapter and chapter XII.

7.2 Objective

The primary objective is to understand and to determine the interacting forces for various combinations of in-plane shear loading and out-of-plane moments through structural or finite element analysis of adjacent box beams under design truck wheel loading.

Suitable structural analyses were also needed to design the test specimens to represent the stress conditions caused by the combinations of out-of-plane moments and the joint shears acting concurrently at the key way joints in bridges that use box beams conforming to ODOT standard dimensions and typical spans.

7.3 Assumptions for New Analyses

Box beam bridge models were developed with SAP2000, a structural analysis program that also has finite element analysis features. The models presented in this report used the finite element modules of the program. The following assumptions were made in the new analyses:

- The key way grout and the box beam concrete have linearly elastic material properties
- The torsional rigidity of the box beams are automatically included in the analysis because finite elements using plates or solid elements are able to include the rigidity of box section due to geometry alone
• Elastomeric bearing provide stiffness in compression and shear, but not in tension. Other aspects of modeling elastomeric bearings are given in a later section

• Two anchor dowel bars of 1-inch diameter are provided at the two ends of each box beams, i.e., one anchor dowel bar at each end. The nominal shear strength of about 28 kips for each dowel bar of 1-inch diameter ($\approx 0.6 \times f_y \times A_b = 0.6 \times 60 \times 0.79 = 28$ kips) is not adequate to prevent longitudinal and transverse sliding of the beam ends. Therefore a roller support condition is more realistic at the ends for modeling the current practice, various support conditions were considered in this study

• A small-deflection linearly elastic structural or finite element analysis is suitable because of the small strains resulting from the loading considered in these analyses

• The longitudinal joints were assumed to be uncracked

• The use of fine mesh captures joint behavior adequately to predict the state-of-stresses at the joints. Validity of this assumption is presented in other section

• Two adjacent box beams were modeled for the load case causing maximum out-of-plane moment, but no shear (Load Case I) Figure 7.3. Three box beams were modeled together for the load case that causes simultaneous shear and moment (Load Case II) Figure 7.4

• Pre-tensioned tie rods prior to grouting the key way will not influence the stress condition in hardened grout of the longitudinal joints

• The interaction and the effects of the beams next to the two beams (Load Case I) or the three beam arrangement (Load Case II) are minimal on box beams included in the analysis models considering more than one truck moving in line next to each other
• Stresses are transferred at the longitudinal joints through the grout only. That means, the adjacent box beams are not in contact with each other below the grouted joint

7.3.1 Design Truck

The H-25 highway design truck loading configuration was employed to determine the stress conditions at the longitudinal joints of typical bridges according to AASHTO LRFD 2012 Bridge Design Specifications (6th edition). Load configurations (Figure 7.1) were selected to maximize the number of axle loads on the box beam models. Only standard design truck wheel load configurations were considered because the lane load is expected to provide a symmetric loading condition on adjacent box beams and therefore may not cause any differential vertical movements or torsional effect.

Figure 7.1: Design Truck Loads

For HS25 loading, the front axle is 10 kips and two following axles were spaced at 14 ft with 40 kips each. The width of the standard ODOT box beams will allow the placement of only one set of wheel loads because the width to be considered in the axle
loads is 6 ft. (72 inch) and the standard widths of the box beams can be either 36 inch or 48 inch. Both these standard widths are less than 72 inch and therefore there is no chance of both wheel loads from an axle occurring over the width on a single box beam. One set of wheels with unfactored loading condition can therefore be 5 kips followed by two 20 kip loads as shown in Figure 7.1.

In this report, the worst case scenario of the joints in box beams with a maximum simple span of 90 ft. simple span was considered. The positions of the set wheel loads to maximize the effects on the joint are shown in Figure 7.3.

7.3.2 Position of Loads in Transverse Direction Relative to the Longitudinal Joint

Wheel loads can be eccentric with respect to the center line of an individual box beam. Two critical load cases were considered to maximize the eccentric load effects.

Load Case I simulates two adjacent trucks moving next to each other at the edges of the adjacent girders remote from the joint, where the joint under study is subjected to stresses from the out-of-plane moments caused by the wheel loads on the longitudinal joints. The key way needs to have adequate strength to transfer the stresses resulting due to the out-of-plane moment at the joint between two adjacent beams.

Figure 7.2: Loads in Longitudinal Direction
This load case is expected to result in tension at the top and compression at the bottom over the key way depth. The maximum tension will occur when two trucks are moving in line next to each other. The shear force at the joint is zero due to symmetric geometric and loading conditions of the two adjacent box beams as shown in Figure 7.3. Load Case II applies when one set of the wheels of the trucks are placed at the edge of the girder next to the joint. In this load configuration, the wheel loads cause out-of-plane moment at the longitudinal joints causing tensile stresses at the bottom of the joint and compressive stresses at the top of the joint (Figure 7.4) simultaneous with shear.
This load case simulates the situation when two adjacent trucks are driving next to each other at the edges of two adjacent box beams separated by a middle box beam.

7.3.3 Tie Rod Effects

Tie rod effects were found to be local at the diaphragm location from previous studies (Henry, 2011). Tie rod forces are applied to the girders before grouting, and therefore there are no stresses imposed on the grout material by the tie rod force. The tie rod effects were not considered in this analysis and discussed later in section 7.6.4. Tie rods are needed to provide lateral stability to the adjacent box beams for structural integrity. Spacing between tie rods is currently specified by ODOT to be no more than 25 feet. Tie rods are provided at the locations of the diaphragms within the box beams. They need to be tensioned to 15 kips of transverse clamping force, which will translate to 250 ft-lb torque.
7.3.4 Support Conditions

The following three aspects of support conditions were considered and are being reported:

(i) The uncertainty related to the stiffness of the elastomeric bearing pads

(ii) the role of the bearing pads in allowing or restraining rotation of the beam ends, and

(iii) the effects of any reaction provided by the anchor dowel bars at the ends of the

These effects were determined using two models with intermediate diaphragm with 36 inch deep key way for Load Case 1.

Figure 7.5: Boundary Conditions at the Beam Ends with Anchor Dowel Bar (a & b) and without Anchor Dowel Bars (c & d)
7.4 ODOT Design Specifications for Box Beam Ends

Prestressed box beam members are supported on two elastomeric bearings at each support (BDM ODOT, 2010 section 302.5.1) as shown in Figure 7.6 (PSBD-02-07) according to section 711.23 of ODOT- construction and material specification manual.

One inch dowel bar is inserted in the center of the box-beam through 2-in. hole, at 6” from the edge of the beam (Figure 7.7) by drilling a 12-in. hole in the abutment.

Preformed expansion joint filler (Section 705.03 ODOT BDM), to be installed with the same thickness as the elastomeric bearing, is installed under the box beam, around each anchor dowel bar to prevent the grout or sealer from leaking through to the beam seat. Because the beam ends are seated on the bearing pads, no vertical load is transferred from the beam ends into the abutments. The anchor dowel bars are not expected to provide any vertical reaction after the grouting or filling the annular space around anchor bars with a sealer. The preceding explanation leads to an important aspect of modeling the beam ends in structural and finite element analyses.

The box beam ends are set on four elastomeric bearing pads. All of the beam self-weight is transferred through the four bearing pads soon after the beams are set in position. We believe that four bearing pads for each beam (two at each end) must be modeled as vertical springs. These pads also provide some lateral shear stiffness as claimed by their manufacturers. We do not believe that anchor dowel bars will provide any vertical support at the beam ends. The shear strength of 1-inch diameter of Grade 60 bars is about 28 kips.
Bearing pads must satisfy the requirements of slip, shear, compressive stress, deflection, and rotation requirements of AASHTO LRFD Bridge Design Specification, 6th Edition (2012). Typical stress-strain curves of elastomers are shown in Figure 7.8 which shows that these pads need to deform significantly to provide the adequate reaction at the beam ends.
7.5 Modeling Approaches used for the Revised Analyses

The following two computer analysis software programs were used to determine the state of stresses: (i) SAP2000 and (ii) SpaceGass. Service loads were applied. The required joint strengths under the eccentric load effects due to the out-of-plane moments were determined for the two load cases for the load arrangements described previously using three independent analysis approaches with i) Shell elements for the box beam and the diaphragms and beam elements for grout, ii) Shell elements for the box beam and the diaphragms and springs for the grout for symmetric loading when no shear forces are transferred across the longitudinal joint, and iii) Eight-node elements

7.5.1 Spring Factors in Compression

For the purpose of structural analysis and finite element (FE) analysis, the live load deformation of the elastomer was assumed to be 0.125-in, at the upper limit, for 2.5-in
thick elastomer with strain = 0.125/2.5 = 5% and corresponding stress of 1.4 ksi (Figure 9), for 90 in² bearing pad.

\[ K = \frac{(\sigma \times A)}{(\varepsilon \times L)} = \frac{(1.4 \text{ ksi} \times 90 \text{ in}^2)}{(0.05 \text{ in/in} \times 2.5 \text{ in})} \approx 1,000 \text{ ksi} \]

7.5.2 Spring Factors in Shear

Shear Modulus \( G_{73} \) at 73°F is less than \( G_0 \) at 0°F, therefore, a value of \( G_{73} = 95 \text{ psi} \) was used. For shape factor = 12, and 90 in² elastomer area

\[ K_2 = G \times \frac{A}{h_t} = 0.095 \times 90/2.5 = 4 \text{ ksi}. \]

7.5.3 Modeling with Shell and Beam Elements

Beam elements known as frame elements in SAP 2000 with 6 degrees of freedom were used for modeling the hardened grout within the key way recess of box beams with a very fine mesh. Shell elements were used for modeling the walls of the box beams and for modeling the diaphragms in SAP 2000.

A width of box beams of 36” or 48” is specified in the standard ODOT drawings ranging from 20 ft to 90 ft. Typical sections specified in ODOT standard drawings (PSBD-2-07) are shown in Figure 7.9 for 48 inch wide sections.

Figure 7.9: Standard Sections and the Details of the Key Way Geometry
7.5.3.1 Modeling of Box Beam with Shell Elements (CSI manual)

The shell element is a type of area object that is used to model membrane, plate, and shell behavior in planar and three-dimensional structures. A non-layered, homogeneous material with constant thickness was used to define the shell element. A homogeneous quadrilateral elements with four-node formulation that combines membrane and plate-bending behavior was used to form the box beam plates and walls. Plate-bending behavior includes two-way, out-of-plane, plate rotational stiffness components and a translational stiffness component in the direction normal to the plane of the element. You may choose a thin-plate (Kirchhoff) formulation that neglects transverse shearing deformation, or a thick-plate (Minlin/Reissner) formulation which includes the effects of transverse shearing deformation. Out-of-plane displacements are cubic. The use of the full shell behavior (membrane plus plate) is recommended for all three-dimensional structures. The Surface Pressure Load is used to apply external pressure loads upon any of the six faces of the Shell element.

Box beams were modeled as shells with a thickness of 5.5 inch, the girder cross section in SAP analysis with plates was 42.5” wide by 36.5” height to account for the thickness of the shell elements forming the girder. Wheel loads were divided by equivalent tire contact area to retain an eccentricity of 14 inch between the center of the load area and the centroid of the girder while accounting for the reduced girder width due to the 5.5” thickness of the shell elements. Typical cross sections used in the analysis for 48-inch-wide box beam sections are shown in Figure 7.10.
7.5.3.2 Modeling of Diaphragms with Shell Elements

ODOT requires two end diaphragms and intermediate diaphragms at the tie rod locations in the transverse direction. At least one intermediate diaphragm should be provided for spans less than 50-ft and two intermediate diaphragms for spans larger than 50-ft and less than 75-ft, and three intermediate diaphragms for spans larger than 75-ft (Standard drawing PSBD 02-07). Diaphragm was modeled as a series of two dimensional plates (shell elements) spaced at 3-in. in the longitudinal direction with thickness of each plate equal to 1.5-in for the two exterior plates and a thickness of 3-in. for the interior plates. Figure 7.11 shows typical modeling of end diaphragm and interior diaphragms.
7.5.3.3 Modeling of Grout with Frame Elements (CSI manual)

The grout material was modeled using Frame elements that can be used to model beams, columns, braces, and trusses in-planar and three-dimensional structures. Linear material properties were used to define the cementitious grout. The Frame element uses a general, three-dimensional, beam-column formulation, which includes the effects of biaxial bending, torsion, axial deformation, and biaxial shear deformations. A prismatic section of the element was used to have linear variation of the axial, shear, torsional, mass, and weight properties over each segment. The Frame element has six degrees of freedom at both of its connected joints. The frame element uses only isotropic material properties.

The longitudinal joints between the box beams were modeled using link beams with 10,000 psi compressive strength and the corresponding concrete material properties. These link beams were spaced at 3 inches in both longitudinal and vertical directions. The length of the link beams was one inch in the top opening region and two inches below the top opening in order to match the recommended key way widths for the test specimens. The
stiffness of the link beams was determined based on the length, loading area, and the type of stresses.

The modulus of elasticity for concrete in compression \((E_C)\) was used considering the stiffness calculations for link beams in the expected compressive stress regions, and the modulus of elasticity for concrete in tension \((E_T)\) was considered for the link beams in the expected tensile regions. The tensile modulus of elasticity was approximately half of the compressive modulus of elasticity.

Figure 7.12 shows the link beam distribution along the height of the key way for beams with 42 inches of actual height. However, to match the centerline dimensions for models with plate elements for box beams, the dimensions were 42.5 inch by 36.5 inch as shown in the figure.
In Load Case I with tension expected above the neutral axis and compression expected below the neutral axis of the vertical joint section with a depth of 33.5 inch, the stiffness of the link beams was calculated as follows:

At height = 3”

\[ K_1 = E_c \times A \times L = 5,700 \text{ ksi} \times (1.5 \text{ in} \times 3.0 \text{ in}) \div 2 \text{ in} = 12,825 \text{ kip/in} \]

\[ E_c = 57\sqrt{f'_c} = 57\sqrt{10,000} = 5,700 \text{ ksi} \]

\[ E_T = 2,500 \text{ ksi} \text{ (assumed as a minimum)} \]

At height = 6” to 18”

\[ K_2 = E_c \times A \times L = 5,700 \text{ ksi} \times (3.0 \text{ in} \times 3.0 \text{ in}) \div 2 \text{ in} = 25,650 \text{ kip/in} \]

At height = 21” to 30”

\[ K_3 = E_T \times A \times L = 2,500 \text{ ksi} \times (3.0 \text{ in} \times 3.0 \text{ in}) \div 2 \text{ in} = 11,250 \text{ kip/in} \]

At height = 33”

\[ K_4 = E_T \times A \times L = 2,500 \text{ ksi} \times ((3.0 \text{ in} \times 1.5 \text{ in}) \div 2 \text{ in} + (3.00 \text{ in} \times 1.5 \text{ in}) \div 1) = 16875 \text{ kip/in} \]

At height = 36.5”

\[ K_5 = E_T \times A \times L = 2,500 \text{ ksi} \times (4.25 \text{ in} \times 3.0 \text{ in}) \div 1 \text{ in} = 31,875 \text{ kip/in} \]

Adjusted cross sectional areas were calculated accordingly to match the calculated stiffness value. For example, the cross-section area of elements at a height of 21 inch to 30 inch was calculated to provide 11,250 kip/in stiffness:

\[ A = K \times L \div E = 11,250 \text{ kip/in} \times 2 \text{ in} \div 5700 \text{ ksi} = 3.947 \text{ in}^2 \]

7.5.4 Solid Modeling with Three-dimensional Eight-node Elements for the Grout and the Box Beams (CSI manual)

Eight- node solid elements were used for the box beams and the grout material. This element is suitable for modeling three- dimensional structures and solids. It is based
upon an iso-parametric formulation that includes nine optional incompatible bending modes. The Solid element activates the three translational degrees of freedom at each of its connected joints. Rotational degrees of freedom are not activated. This element contributes stiffness to all of these translational degrees of freedom. The Solid element models a general state of stress and strain in a three-dimensional solid. All six stress and strain components are active for this element.

Figure 7.13 shows a cross-section of the modeled beam in the longitudinal direction

![Cross-Section at the middle of the modeled Box Beam](image)

Figure 7.13: Cross Section at the Middle of the Modeled Box Beam

Three dimensional finite element models were developed with 3D-solid. All the models were developed for 90-ft. span simply supported box beam bridges with B42-48 ODOT standard section dimensions.

The following results were extracted from these models for Load Case I and Load Case II:

(i) Normal stresses perpendicular to the longitudinal joint surface and shear stresses in the plane of the longitudinal joints

(ii) Deflections at key points

(iii) Reactions in the elastomeric bearing pads

(iv) Shear force transferred at the joints along the length of the beams
Graphs and contours of normal stresses and shear stresses

Nine models were developed to study the beam behavior considering the following factors: (i) models with and without intermediate diaphragms (ii) models using full depth key ways and partial depth key ways. (iii) Models using cracked and un-cracked grout. Figure 7.14 shows these different models used in this study. All the analyses were done for service load conditions without any load factors. However, because of the assumption regarding linear elastic material properties and linear elastic analysis with uncracked condition, the stresses for factored load conditions can be obtained by multiplying suitable load factors to the results obtained in this report particularly when strength limit state checks are needed.
Figure 7.14: Developed Models with 3D-Solid Elements in SAP 2000
7.6 Analysis Results

Effect of support conditions are presented in this chapter, normal stresses concentration was observed at diaphragm locations and local shear stresses at wheel load locations. Also Stresses were nonlinear along the depth of the grout joint. Contour lines for normal and shear stresses are presented in the appendix.

7.6.1 Effects of Modeling Support Reactions Considering Bearing Pads and Anchor Dowel Bars

The support reactions including the spring forces representing the reactions provided by elastomeric bearing pads at the beam ends for different load cases and different models are shown in Table 1 through Table 4. The following conclusions were drawn from the results of the analyses.

(i) At any end, the two springs representing the two bearing pads along with one anchor dowel bar resist all the end reaction force transferred from the corresponding beam end. Removing the dowel bar from the analysis models allows the redistribution of the reaction forces into the springs, thereby increasing the axial force resisted by the two springs at each end. However, any redistribution of the end reactions between the springs and the dowel bar did not have any effect on the normal stresses or shear stresses at the longitudinal joint. This is possibly because there seems to be no significant rotational displacements at the beam ends and therefore the bearing pads were attracting almost equal amounts of vertical compressive reactions. The two pairs of bearing pads at either end of each box beam were seen to be settling by approximately same amount similar to what was observed from the analyses with pinned or roller support simulating the end anchor.
dowel bars. Therefore, there is absolutely no change in the state of stresses at the longitudinal joints.

(ii) The axial spring stiffness representing the bearing pads used in the analyses had absolutely no effects on the state of stresses at the longitudinal joint. A larger spring stiffness results in reduces amount of settlement at the pads and smaller spring stiffness increases the amount of settlement at the pads. As long as there is no differential settlement, there are no changes observed in the state of stresses at the joint. A typical comparison of normal stresses at the top support for Load Case I of a 36 inch key way with intermediate diaphragms is shown in Figure 16 for two conditions (i) with vertical displacement prevented at the anchor dowel bar location and (ii) with vertical displacement not prevented at the anchor dowel bar location. The two curves for the two conditions are identical and are overlapping except over a small length of about 3 ft. at the beam ends. This observation is particularly significant because it proved that any inaccuracies in assuming stiffness constants in the analysis to represent the bearing pads have no effect on the normal and shear stresses developed at the longitudinal joints.
Figure 7.15: Effect of Support Conditions
The support reactions at the key points for different models for different load cases are shown in Tables 1-4.

Table 7.1: Support Reaction with Constraint at the Dowel Locations – Load Case I – Without Intermediate Diaphragm

<table>
<thead>
<tr>
<th>Beam</th>
<th>Support</th>
<th>Load Case I 1 - 36&quot; deep with intermediate diaphragm</th>
<th>Load Case I 2 - 12&quot; deep with intermediate diaphragm</th>
</tr>
</thead>
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<td>F1 (kip): 0.81*</td>
</tr>
<tr>
<td></td>
<td></td>
<td>F2 (kip): 0.00</td>
<td>F2 (kip): -0.02</td>
</tr>
<tr>
<td></td>
<td></td>
<td>F3 (kip): 5.82</td>
<td>F3 (kip): 6.21</td>
</tr>
<tr>
<td>South</td>
<td>S2</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>North</td>
<td>H1</td>
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<td>F1 (kip): NR*</td>
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<tr>
<td></td>
<td></td>
<td>F3 (kip): 51.32</td>
<td>F3 (kip): 50.94</td>
</tr>
<tr>
<td>South</td>
<td>H2</td>
<td>F1 (kip): -1.60</td>
<td>F1 (kip): -1.60</td>
</tr>
<tr>
<td></td>
<td></td>
<td>F2 (kip): NR</td>
<td>F2 (kip): NR</td>
</tr>
<tr>
<td></td>
<td></td>
<td>F3 (kip): 47.33</td>
<td>F3 (kip): 46.98</td>
</tr>
<tr>
<td>North</td>
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<td>F1 (kip): 0.80*</td>
<td>F1 (kip): 0.81*</td>
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<td>F2 (kip): 0.00</td>
<td>F2 (kip): -0.02</td>
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<td>F3 (kip): NR</td>
<td>F3 (kip): NR</td>
</tr>
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<td></td>
</tr>
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<td>F1 (kip): 0.81*</td>
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<td>F2 (kip): 0.00</td>
<td>F2 (kip): -0.02</td>
</tr>
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<td></td>
<td></td>
<td>F3 (kip): NR</td>
<td>F3 (kip): NR</td>
</tr>
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<td>South</td>
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</tr>
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<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>North</td>
<td>H3</td>
<td>F1 (kip): NR*</td>
<td>F1 (kip): NR*</td>
</tr>
<tr>
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<td></td>
<td>F2 (kip): NR</td>
<td>F2 (kip): NR</td>
</tr>
<tr>
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<td>F3 (kip): 50.94</td>
</tr>
<tr>
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<td>H4</td>
<td>F1 (kip): -1.60</td>
<td>F1 (kip): -1.60</td>
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<tr>
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<td></td>
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<td>F3 (kip): 46.98</td>
</tr>
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<td>S7</td>
<td>F1 (kip): 0.80*</td>
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<td></td>
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</tr>
<tr>
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<td>F1 (kip): 0.00</td>
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<td></td>
<td>F2 (kip): 0.00</td>
<td>F2 (kip): 0.00</td>
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<td></td>
<td>F3 (kip): 219.5</td>
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* Movement in the longitudinal direction = 0.2 in

NR – Not restrained.

Table 7.2: Support Reaction with Constraint at the Dowel Locations – Load Case II – With Intermediate Diaphragm
## Load Case II

### 3-36” depth with intermediate diaphragm

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<th>F2 (kip)</th>
<th>F3 (kip)</th>
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<tr>
<td>North H1</td>
<td>0.22</td>
<td>1.37</td>
<td>4.83</td>
</tr>
<tr>
<td>South H2</td>
<td>NR**</td>
<td>1.06</td>
<td>3.97</td>
</tr>
<tr>
<td>North S1</td>
<td>0.00</td>
<td>0.00</td>
<td>19.69</td>
</tr>
<tr>
<td>South S2</td>
<td>-0.54**</td>
<td>0.00</td>
<td>18.97</td>
</tr>
<tr>
<td>North S3</td>
<td>0.00</td>
<td>0.00</td>
<td>20.08</td>
</tr>
<tr>
<td>South S4</td>
<td>-0.54**</td>
<td>0.00</td>
<td>19.33</td>
</tr>
<tr>
<td>North H3</td>
<td>0.88</td>
<td>NR</td>
<td>12.58</td>
</tr>
<tr>
<td>South H4</td>
<td>0.00**</td>
<td>NR</td>
<td>10.41</td>
</tr>
<tr>
<td>∑ F</td>
<td>0.00</td>
<td>2.43</td>
<td>109.9</td>
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** Movement in the longitudinal direction = 0.14 in  
NR – Not restrained.

### 4-12” deep with intermediate diaphragm

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<tr>
<td>North H1</td>
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</tr>
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<tr>
<td>North S1</td>
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<td>0.00</td>
</tr>
<tr>
<td>South S2</td>
<td>-0.55**</td>
<td>0.00</td>
</tr>
<tr>
<td>North S3</td>
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</tr>
<tr>
<td>South S4</td>
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<tr>
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<td>South H4</td>
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<td>NR</td>
</tr>
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<td>∑ F</td>
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Table 7.3: Support Reaction with Constraint at the Dowel Locations – Load Case I – Without Intermediate Diaphragm

<table>
<thead>
<tr>
<th>Support</th>
<th>F1 (kip)</th>
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<tr>
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<td>S1</td>
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<td>5.83</td>
</tr>
<tr>
<td>South</td>
<td>S2</td>
<td>0.00</td>
<td>0.00</td>
<td>5.30</td>
</tr>
<tr>
<td>North</td>
<td>H1</td>
<td>NR*</td>
<td>NR</td>
<td>51.32</td>
</tr>
<tr>
<td>South</td>
<td>H2</td>
<td>-1.61</td>
<td>NR</td>
<td>47.33</td>
</tr>
<tr>
<td>North</td>
<td>S3</td>
<td>0.81*</td>
<td>0.00</td>
<td>NR</td>
</tr>
<tr>
<td>South</td>
<td>S4</td>
<td>0.00</td>
<td>0.00</td>
<td>NR</td>
</tr>
<tr>
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<td>NR</td>
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<th>F1 (kip)</th>
<th>F2 (kip)</th>
<th>F3 (kip)</th>
<th>Load Case I 5-36&quot; depth without intermediate diaphragm</th>
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</tr>
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<td>6.24</td>
</tr>
<tr>
<td>South</td>
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<td>NR</td>
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<td>H2</td>
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<td>46.95</td>
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</tr>
<tr>
<td>South</td>
<td>S4</td>
<td>0.00</td>
<td>0.00</td>
<td>NR</td>
</tr>
<tr>
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<td>S5</td>
<td>0.82*</td>
<td>0.00</td>
<td>NR</td>
</tr>
<tr>
<td>South</td>
<td>S6</td>
<td>0.00</td>
<td>0.00</td>
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<td>NR*</td>
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<td>0.00</td>
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<tr>
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<td>0.82*</td>
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<tr>
<td>Σ F</td>
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<td>0.00</td>
<td>219.5</td>
<td></td>
</tr>
</tbody>
</table>

* Movement in the longitudinal direction = 0.2 in
NR – Not restrained.
Table 7.4: Support Reaction without Vertical Constraint at the Dowel Locations – Load Case I – With Intermediate Diaphragm

<table>
<thead>
<tr>
<th>Support</th>
<th>F1 (kip)</th>
<th>F2 (kip)</th>
<th>F3 (kip)</th>
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<tr>
<td>North</td>
<td>S1</td>
<td>0.80</td>
<td>0.00</td>
</tr>
<tr>
<td>South</td>
<td>S2</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>North</td>
<td>H1</td>
<td>NR*</td>
<td>NR</td>
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<tr>
<td>South</td>
<td>H2</td>
<td>-1.60</td>
<td>NR</td>
</tr>
<tr>
<td>North</td>
<td>S3</td>
<td>0.80</td>
<td>0.00</td>
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<tr>
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<td>S4</td>
<td>0.00</td>
<td>0.00</td>
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<tr>
<td>North</td>
<td>S5</td>
<td>0.80</td>
<td>0.00</td>
</tr>
<tr>
<td>South</td>
<td>S6</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>North</td>
<td>H3</td>
<td>NR*</td>
<td>0.00</td>
</tr>
<tr>
<td>South</td>
<td>H4</td>
<td>-1.60</td>
<td>0.00</td>
</tr>
<tr>
<td>North</td>
<td>S7</td>
<td>0.80</td>
<td>0.00</td>
</tr>
<tr>
<td>South</td>
<td>S8</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>∑ F</td>
<td>0.00</td>
<td>0.00</td>
<td>219.54</td>
</tr>
</tbody>
</table>

* Movement in the longitudinal direction = 0.2 in  
NR – Not restrained.
7.6.2 Normal Stresses along the Length of the Box beams

Tensile and compressive stresses in the key way joint was extracted from the model by averaging the stresses at the eight nodes of each element and plotted at the centroid of the element.

7.6.2.1 Comparison of Results Obtained from Eight node Solid Element Models with the Results Obtained from Shell and Beam Element Models

The peak normal stresses obtained from eight node solid elements are lot smaller at the diaphragm location but slightly higher at other locations (Figure 7.16). From here on, only the results obtained from the finite element models with eight node solid elements are presented.
7.6.2.2 Load Case I

The normal stresses along the length of the longitudinal joints of adjacent box beams at the top of the key way are plotted in Figure 7.17. This figure shows the tensile stresses for Load Case I for models with 12-in deep key way, and 36-in deep key way, with and without diaphragms.
7.6.2.2.1 Top Stresses

The following observations were made based on the normal stresses within the longitudinal joints at the top surface of the box beams using six models for the two load cases:

![Diagram of normal stresses at the top surface along the length of the longitudinal joint for Load Case I with intermediate diaphragms.](image)

Figure 7.17: Normal Stresses at the Top Surface along the Length of the Longitudinal Joint for Load Case I with Intermediate Diaphragms
7.6.2.2.1.1 Box Beams with Intermediate Diaphragms (Load Case I)

The load configuration in this load case as seen in Figure 7.3 is expected to cause tension at the top edge of the grouted longitudinal joint and compression at the bottom edge of the joint. Because of symmetric nature of the geometry, stiffness, and loading, the shear stresses at the joint are expected to be zero. Therefore, the peak normal stresses within the longitudinal joints are expected to occur when there are no shear stresses at the joint.

(i) The normal stresses on the joints within the diaphragm width were much larger than the normal stresses obtained at locations away from the diaphragms suggesting that much of the out-of-plane moment is transferred between the box beams primarily through the diaphragms.

(ii) The peak normal stresses occur at the diaphragm locations. Large normal stresses also occur at the wheel load locations.

(iii) Increase in the depth of the joint from 12-inch to 36-inch decreases the normal stresses at the diaphragm location at the quarter and three-quarters spans by a factor of 2.0 at the mid-span diaphragm and by a factor of 1.85.

(iv) Remote from the diaphragms, there are larger normal stresses at the location of the wheel loads. These normal stresses are slightly smaller than those at the diaphragm locations, but larger than everywhere else. Decrease in normal stresses at the wheel loads is also observed with an increase in the joint depth, but the decrease occurs to a smaller extent.

(v) The occurrence of large normal stresses at the location of the wheel loads other than at the diaphragms seem to suggest that these peaks can occur at any point along the length of the joint because the moving wheel loads can occur at any point.
Therefore, there is an equal chance that the peak normal stresses at the wheel loads will occur all along the joint length because of the moving nature of truck axle loads. However, the normal stresses at the diaphragms are the largest particularly when the wheel loads are at the locations of the diaphragm.

(vi) Increase in joint depth from 12 inch to 36 inch reduced the normal stresses close to the beams ends by a factor 3 to 4 where the sections are solid (for the 39 inch-length at each end).

(vii) Similar trend was observed for normal stresses at the bottom of the joint. However, the normal stresses at the bottom are compressive and therefore of less concern when high strength grout is used.

(viii) As expected, the shear stresses at the joint were close to zero.

7.6.2.2.1.2 Box Beams without Intermediate Diaphragms

For Load Case I without intermediate diaphragms, the normal tensile stresses in the joints at the top surface occur at the location of the wheel loads. The magnitudes of stresses are less than about 75 psi for 12 inch deep joint and less than 60 psi for 36 inch deep joint, suggesting a reduction of stresses up to about 25% when diaphragms are not provided.

(i) The normal stresses in box beams without intermediate diaphragms have peaks only at the locations of the wheel loads. In the absence of the intermediate diaphragms, these peaks follow the wheel loads along the length of the joint. Therefore, these peak normal stresses occur over the entire length of the joint due the moving nature of the wheel loads.
(ii) Reduction in normal stresses due to the increase in joint depth from 12 inch to 36 inch is only about 15% under the different wheel loads applied along the length of the joint.

(iii) While the peak normal tensile stresses occur at the locations of wheel loads when there are no intermediate diaphragms, the magnitudes of the peaks are smaller than those obtained for the corresponding load conditions for both 36 inch and 12 inch joint depths for box beams with intermediate diaphragms. From Figure 18, the difference between the normal stresses for 36 inch deep key way and those for 12 inch deep key way is not as large as it is for box beams with diaphragms. However, compared to the normal tensile stresses at the diaphragm locations, the corresponding stresses at the wheel loads are smaller by a factor of at least 2.0 for 36 inch deep key way joint.

(iv) Removing the intermediate diaphragms from the current practice and using full depth key way may result in a more uniform peak normal stresses along the entire length of the longitudinal joint and will eliminate the stress concentration at the intermediate diaphragm location. For 12-inch key way removing the intermediate diaphragms reduces the stresses significantly along the span except for the end diaphragms locations. End diaphragms cannot be removed in prestressed box beams, it is needed to transfer the prestressing forces form the strands to the concrete section.
7.6.2.2.2 Bottom Stresses

The normal stresses at the bottom of the joint are generally low for both depths (12 inch and 36 inch) when diaphragms are not provided. The peak stresses still occur at the location of the wheel loads as shown in Figures 7.18 and 7.19.

Figure 7.18: Normal Stresses at the Bottom Surface along the Length of the Longitudinal Joint for Load Case I without Intermediate Diaphragms - 36” vs. 12” Key Way
Figure 7.19: Normal Stresses at the bottom Surface along the Length of the Longitudinal Joint for Load Case I without Intermediate Diaphragms - 36” vs. 12” Key Way

7.6.2.3 Load Case II

The load configuration in this load case as seen in Figure 7.4 is expected to cause compression at the top edge of the grouted longitudinal joint and tension at the bottom of the joint. In this load case, the joint is not at the axis of symmetry. Therefore, the joint will be subjected to both normal stresses and shear stresses. The normal stresses are expected to be smaller in this load case than those in Load Case I. The shear stresses at the joint for Load Case I were zero.

(i) As shown in Figure 7.20, the normal tensile stresses on the joints at the bottom edge within the diaphragm width were much larger than the normal stresses obtained at
locations remote from the diaphragms suggesting that much of the out-of-plane moment is transferred between the box beams primarily through the diaphragms.

(ii) For joints with 12 inch depth, there is a reversal of stresses at the diaphragms. While it is difficult to explain this reversal, it is believed that the amount of load transfer from the loaded box beam to the adjacent box beam is drastically reduced, and most of the load seems to be transferred to the beam ends longitudinally rather than transversely. Transverse distribution of loads results in better sharing of loads between adjacent box beams and better efficiency. Therefore 12-inch deep joint is less efficient than a 36-inch deep joint for this load case.

(iii) Similar trends can be observed for normal compressive stresses at the top of the joint as seen in Figure 7.20. However, the normal stresses at the top are mostly compressive and therefore of less concern.

(iv) Providing a deeper joint reduces the normal (tensile) stresses by a factor of about 3.0 for this case. Figure 7.20 shows that for Load Case II with intermediate diaphragms, the normal stresses at the top surface are mostly not influenced by the depth of the joint. The peak stresses still occur at the location of the wheel loads or at the diaphragm locations, but the stresses at the wheel load locations are larger.
Figure 7.20: Normal Stresses at the Top Surface along the Length of the Longitudinal Joint for Load Case II with Intermediate Diaphragms

Figure 7.21: Normal Stresses at the Bottom Surface along the Length of the Longitudinal Joint for Load Case II with Intermediate Diaphragms
7.6.3 Shear Force Transferred Through the Joints

The shear stresses developed at the joint are shown for Load Case II with 36 inch deep joint and 12 inch deep joint in Figures 7.22 and 7.23, respectively. The figures show shear force along the length of the joint in kips/in which is calculated by integrating the shear stresses over the depth at each location (36 inch or 12 inch). For beams with 36 inch deep joint, most of the shear is transferred at the wheel load locations only, regardless of the location of the diaphragm. With smaller joint depth of 12 inch (Figure 7.23), some of the shear force is also transferred through the joints outside of the locations of the wheel loads. The peak stresses for 36 inch deep joint (6 psi) are about two thirds smaller than for 12 inch deep joint (18 psi).

Figure 23 shows the shear force at the joint over the length of the box beams with intermediate diaphragms. The shear force is mostly transferred between adjacent box beams at the wheel load locations. The average shear stress at the diaphragm location is about 6 psi over a joint length of about 50 inches in the vicinity of the 20-kip wheel load and the depth of 36 inch. This suggests that the shear force at the wheel loads will govern the required strength throughout the length of the box beams because the wheel loads can occur anywhere along the length of the beams.
7.6.4 Effects of Tie Rods

The finite element models presented in this report did not include beam elements corresponding to the tie rods. However, the relative transverse displacements at the far edges of the adjacent beams at the level of the tie rods (14 inches from the top) provide an indication of how much load would be attracted by the tie rods and their effectiveness, if
they were included in the models. From Figure 7.24 it is clear that in all cases with 12 inch key way, and one case with 36 inch key way (Load Case II), the relative movements are compressive. Therefore, the tie rods will not participate in preventing the box beams from spreading outward because the beams are not even spreading. For the remaining two cases with 36 inch deep key way, the maximum tensile elongation at the level of the tie rod is 0.0024 inch (2 x 0.0012 inch) over a length of 96 inch. Considering $E_s = 29,000$ ksi which is the modulus of steel, and area of tie rod of 0.79 inch$^2$ for 1 inch diameter tie rod, the tensile force caused by the elongation over 96 inch length of the tie rods is 0.725 kip. This tie force is mostly insignificant. In this calculation of increased tie force due to the spreading of the beams, it is assumed that the tie rods are tensioned first, and the grout is placed after the tensioning of the tie rods. Therefore, there is no pre-compression developed in the hardened grout due to the tie rod tension.
7.6.5 Effects of Cracking at the Top Level of Key Ways

The effects of cracking at the top surface of the joints were determined using an approximate approach. Solid elements were removed from the top three inches of the finite element mesh (i.e., from 39 inch to 42 inch from the bottom) only at the locations of the maximum tensile stresses (at the diaphragms and the wheel loads). When the elements are removed, the physical condition is similar to when a crack occurs at that point.

Figure 7.25 shows the resulting normal tensile stresses at the top surface before and after the removal of the elements at the highly stressed locations (i.e., uncracked and cracked conditions). Figure 7.26 shows that the normal tensile stresses are redistributed to the elements surrounding those elements that have been removed from the mesh, particularly to those elements three inches below the top surface. The figure shows that...
substantial increase in tensile stresses will occur and therefore the cracks will propagate progressively deeper and deeper as the cracks extend downward.
Figure 7.25: Crack Effect on Normal Stresses at top Surface for 36” Key Way

Figure 7.26: Crack Effect on Normal Stresses below Top Surface 36” Key Way

Similar change in tensile stresses will occur in the box beams with 12 inch deep joints as shown in Figures 7.27 and 7.28. However, the increases in tensile stresses in the elements below the induced cracking are much more severe than what was observed for 36 inch deep joints.
Figure 7.27: Crack Effect on Normal Stresses at top Surface for 12” Key Way
7.6.4 Normal Stress and Shear Stress Contours

Many of the relevant normal stress and shear stress contours for six models showing stress distribution are included in the Appendix. These contours provide more information to support some of the conclusions drawn in this report.

Figure 7.28: Crack Effect on Normal Stresses below Top Surface for 12” Key Way
7.7 Conclusions

The following conclusions may be drawn from the results of the analyses presented in this chapter:

1. Any redistribution of the end reactions between the springs and the dowel bar did not have any effects on the normal stresses or shear stresses on the longitudinal joint as long as all bearing pads have the same stiffness properties. There is no change in the state of stresses in terms of normal stresses and shear stresses at the longitudinal joints due to the inclusion or exclusion of the end anchor dowel bars in modeling end supports.

2. The axial spring stiffness representing the bearing pads used in the analyses has no effects on the state of stresses at the longitudinal joint. As long as there is no differential settlement, there were no changes observed in the state of stresses at the joint. Any inaccuracies in assuming the stiffness constants in finite element analyses to represent the bearing pads have no effect on the normal and shear stresses at the longitudinal joints.

3. For box beams with intermediate diaphragms, most of the out-of-plane moment at the joint is transferred to the adjacent box beam at the locations of the diaphragms. The peak normal stresses occur at the diaphragm locations. However, the shear forces are transferred at the locations of the wheel loads.

4. Box beams with intermediate diaphragms and deeper key ways (depth of 36 inch) have smaller normal tensile stresses than those with 12 in deep key ways by a factor of about 2.0 in normal tensile stresses and a factor of 3 in shear stresses, suggesting that the joints with deeper key way will have smaller tendency to crack at the joint surface.
5. Box beams without intermediate diaphragms and deeper key ways (depth of 36 inch) also have smaller normal tensile stresses than those with 12 in deep key ways, but the reduction of normal stresses is not as much as that for box beams with diaphragms.

6. The effectiveness of tie rod clamping force when the tie rods are tensioned prior to the grouting is negligible. Therefore, there is no significant contribution of tie rods in controlling the spreading of adjacent box beams or tendency to crack at the joint.

7. Any cracking in the top 3 inches of a joint with 12 inch deep key way has significant effect on normal stresses in the vicinity of the cracks suggesting that the crack propagation will be more severe in such joints compared to that of deeper key way joints.
CHAPTER VIII

ECCENTRIC LOAD TESTS

8.1 Objective

The objective of this chapter is the determination of the strength of the joints under the possible concurrent action of shear loading and the corresponding out-of-plane moment using structural tests. A study of eccentric load effects at a full or large scale requires bigger beams than those used for symmetric loading (Chapter VII) with comparable cross-sections and spans. Such tests are expensive and time consuming. Therefore, simplified smaller test specimens with a length of 42 inches were designed to capture the behavior of full-scale box beams for the select key way geometries, interface conditions, and loading conditions. Structural tests were conducted in order to determine if the key way geometries and grouts developed in this study would satisfy the load carrying requirements at the longitudinal joints of box beam bridges with typical spans.

8.2 Test Specimens Set# 1

Simplified test specimens were designed to simulate the corresponding full-scale tests for different key way geometries and load combinations. The details of the test specimens and the typical test setup are shown in Figure 8.1. Table 8.1 shows the details of the specimens for the first set of test specimens.
Table 8.1: Details of Specimens for Eccentricities Load set#1

<table>
<thead>
<tr>
<th>Grout Material</th>
<th>Key Way Geometry</th>
<th>Surface Roughness</th>
</tr>
</thead>
<tbody>
<tr>
<td>ODOT-Approved Grout</td>
<td>Standard geometry - 7&quot; deep</td>
<td>Sandblasted</td>
</tr>
<tr>
<td></td>
<td>(0.75&quot; - 1.5&quot; wide)</td>
<td></td>
</tr>
<tr>
<td>ODOT-Approved Grout</td>
<td>Wider key way - 21&quot; deep</td>
<td>Sandblasted</td>
</tr>
<tr>
<td></td>
<td>(1&quot; - 2&quot; wide)</td>
<td></td>
</tr>
<tr>
<td>ODOT-Approved Grout</td>
<td>Wider key way - 21&quot; deep</td>
<td>As-Cast</td>
</tr>
<tr>
<td></td>
<td>(1&quot; - 2&quot; wide)</td>
<td></td>
</tr>
<tr>
<td>Polymer Grout</td>
<td>Standard width key way - 21&quot; deep</td>
<td>Sandblasted</td>
</tr>
<tr>
<td></td>
<td>(0.75&quot; - 1.5&quot; wide)</td>
<td></td>
</tr>
<tr>
<td>UHPG</td>
<td>Wider key way - 21&quot; deep</td>
<td>Sandblasted</td>
</tr>
<tr>
<td></td>
<td>(1&quot; - 2&quot; wide)</td>
<td></td>
</tr>
<tr>
<td>HSC with Maximum #8 Aggregates Size</td>
<td>Wider key way - 21&quot; deep</td>
<td>Sand blasted</td>
</tr>
<tr>
<td></td>
<td>(1&quot; - 2&quot; wide)</td>
<td></td>
</tr>
<tr>
<td>HSC with Maximum #8 Aggregates Size</td>
<td>Wider key way - 21&quot; deep</td>
<td>As-Cast</td>
</tr>
<tr>
<td></td>
<td>(1&quot; - 2&quot; wide)</td>
<td></td>
</tr>
</tbody>
</table>
Figure 8.1: Load Configuration for Eccentric Load Tests

In the test setup, one unit was fixed to a rigid support, while the other unit was loaded under eccentric loading to simulate the out-of-plane moment on the joint. The test variables used in this study are summarized in Figure 8.2.

Figure 8.2: Test Combinations and Key Way Geometry for Eccentric Load Tests
8.3 Factors Studied in the Eccentric Load Tests

Grout material, Surface roughness, and eccentric loading are the three factors that were studied in this test and discussed in the following section. Bond enhancing chemicals were not considered based on the outcomes from the joint tests from chapter III. All specimens were cast, grouted, properly cured and stored outdoor in the summer.

8.3.1 Grout Material

Based on the joint tests and the symmetric load tests on beam assemblies, the polymer-based grout, ultra-high performance grout (UHPG), and high-strength concrete (HSC) were selected as the tentative grouts that are suitable for possible implementation. With high shear strength, polymer-based grout and UHPG were found to be very flowable and self-consolidating, with no vibration needed for compaction. The HSC concrete with maximum # 8 coarse aggregate size required some amount of vibration for proper consolidation. The ODOT-approved Kuhlman grout was considered for comparison and the possibility of achieving the desired strength using a rough sandblasted surface with a deeper and wider key way. The current ODOT-specified key way geometry was also used in the test plan for the sake of comparison.

8.3.2 Surface Roughness and Interface Sandblasting

Based on the joint tests, roughening of the joint surface by sandblasting improved the shear strength of the joint significantly. For UHPG, the average joint shear strength was found to have increased by a factor of about 3.3. For the HSC with # 8 aggregates, the joint strength was increased by a factor of 1.7. As-cast concrete surfaces and roughened concrete surfaces by sand blasting were considered in this study to determine the joint strength under eccentric loads for both surface conditions. Sand blasting was performed using a 3,500 psi
pressure washer and fine sand with a wet blasting kit (Figure 8.3). An air pressure of 120 psi may provide similar roughness if air blasting were to be used.

Figure 8.3: Sandblasted Surface Preparation for Eccentric Load Tests

8.3.3 Eccentric Loads

The applied loads were designed to result in tension and compression normal to the joint surface and shear forces parallel to the joint surface to match the required stresses obtained from finite element analyses. One unit of the test specimen was firmly attached to the testing frame using turnbuckles and/or tie rods to prevent any rotation or translation in the transverse direction. It was supported so that there was no vertical movement. The other unit of the specimen was hanging free of the supports and was connected to the supported unit only through the grout in the key way. The loading brackets were attached to the free concrete unit to apply axial eccentric load at 18 inches of eccentricity to result in tension, compression, and shear forces (Figure 8.4).
Table 8.2: Summary of Test Plan for Eccentric Load Tests

<table>
<thead>
<tr>
<th>Grout Material</th>
<th>Surface Preparation</th>
<th>No. of specimens</th>
<th>Width of the opening (in)</th>
<th>Key Way Dimensions Width (in)</th>
<th>Key Way Dimensions Depth (in)</th>
<th>Comments</th>
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</thead>
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<td>Kuhlman 1107</td>
<td>Sand blasted</td>
<td>2</td>
<td>0.75</td>
<td>1.5</td>
<td>4</td>
<td>Standard Geometry</td>
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<tr>
<td>Master Flow 4316 (Polymer)</td>
<td>Sand blasted</td>
<td>2</td>
<td>0.75</td>
<td>1.5</td>
<td>18</td>
<td>Vibration not Needed</td>
</tr>
<tr>
<td>Kuhlman 1107</td>
<td>Sand blasted</td>
<td>2</td>
<td>1</td>
<td>2</td>
<td>18</td>
<td>Wider opening to allow vibration</td>
</tr>
<tr>
<td>Kuhlman 1107</td>
<td>As Cast</td>
<td>2</td>
<td>1</td>
<td>2</td>
<td>18</td>
<td>Wider opening to allow vibration</td>
</tr>
<tr>
<td>HSC</td>
<td>Sand blasted</td>
<td>2</td>
<td>1</td>
<td>2</td>
<td>18</td>
<td></td>
</tr>
<tr>
<td>HSC</td>
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<td>1</td>
<td>2</td>
<td>18</td>
<td></td>
</tr>
<tr>
<td>UHPG</td>
<td>Sand blasted</td>
<td>2</td>
<td>1</td>
<td>2</td>
<td>18</td>
<td></td>
</tr>
<tr>
<td>Total No. of Test Specimens</td>
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<td>14</td>
<td></td>
<td></td>
<td></td>
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</tr>
</tbody>
</table>
8.4 Results for Eccentric Load Tests set# 1

For specimen with full depth key way and as-cast concrete surface using ODOT-approved grout, the joint failed under its own self-weight plus the weight of the loading brackets before applying any external loads. The joint failed through the entire length and entire depth by de-bonding and separation at the interface without any damage to the concrete units or the grout material as seen in Figure 8.5.
For specimen with partial depth key way and sandblasted surface using ODOT-approved grout, the joint failed after applying 500-lb of eccentric load and its self-weight, and the weight of the loading brackets. The joint failed through the entire length and entire depth by de-bonding and separation at the interface with minor damage to the grout seen in Figure 8.6.
For specimens with a sandblasted surface and a full-depth key way, the joint did not fail. For these tests, the concrete unit failed at 10,000 lb of tension by splitting at the tie rod location as shown in Figure 8.7 and Figure 8.8. The strength of the concrete units was less than the required strength to fail the joint.

Figure 8.7: Typical Failure in Concrete Units
8.5 Test Specimens Set# 2

It was not possible to determine the failure load and joint strength for different key way geometries and grout materials because the concrete test specimens failed locally before joint failure could occur. A second set of test specimens with larger strength was needed to determine the available strength of the tentative grouts. A new set of 8 test specimens with larger width were cast and grouted to determine the available strength of each of the 4 grouts under study (Polymer, High strength Concrete, ODOT-approved grout, and UHPG) with sandblasted surface. The concrete units were designed to avoid the tensile split under eccentric loads, the reinforcement details for set#1 and set#2 are shown in Figure 8.9. The formwork and rebar for set#2 is shown in Figure 8.10.
Eight test specimens were cast and tested under eccentric loads. The eccentric load was applied at 27” from the key way surface.
8.6 Results for Eccentric Load Tests Set # 2

The failure loads are shown in Table 8.3. The largest bond strength between the grout and the concrete units was obtained when HSC was used as grout material with average eccentric load of 32 kips.

Table 8.3: Test Results for Set # 1 and Set # 2

<table>
<thead>
<tr>
<th>Specimen #</th>
<th>Grout Material</th>
<th>Key Way Surface</th>
<th>Failure Load (lb)</th>
<th>Average Failure Load (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>High Strength Concrete</td>
<td>Sandblasted</td>
<td>30,000</td>
<td>32,000</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td></td>
<td>34,000</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Odot-Approved Grout</td>
<td>Sandblasted</td>
<td>18,000</td>
<td>20,000</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
<td>22,000</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Polymer 4316</td>
<td>Sandblasted</td>
<td>12,000</td>
<td>13,000</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td></td>
<td>14,000</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>Ultra High Performance Grout</td>
<td>Sandblasted</td>
<td>8,000</td>
<td>8,500</td>
</tr>
<tr>
<td>8</td>
<td></td>
<td></td>
<td>9,000</td>
<td></td>
</tr>
</tbody>
</table>

An elastic behavior of the grout before failure was assumed to develop the stress diagram for the test specimens to determine the maximum tensile stress at the extreme top fibers at failure. Figure 8.11 and Figure 8.12 shows linear elastic analysis for the tested grouts.
Figure 8.11: Linear Elastic Analysis for the Tested Specimens - I
8.6.1 Failure Modes for set# 2

A sudden failure was observed in all the test specimens. A monolithic failure pattern was observed only when the HSC was used as grout material with average failure load of 32 kips as shown in Figure 8.13. Sandblasting the concrete surface improved the strength of the test specimens for all the tested grouts. ODOT-approved grout failed at average load of 20 kips with superior performance to the polymer and UHPG with failure load of 13 kips, and 8.5 kips respectively.

Failure mode for each grout type is shown in Figures 8.13 to Figure 8.16
Figure 8.13: Failure for HSC Grout

Figure 8.14: Failure for Polymer Grout
A summary of the test results for set# 1 and set# 2 are presented in table 8.4

Table 8.4: Summary of Test Results for Set# 1 and Set# 2
<table>
<thead>
<tr>
<th>Key Way Depth</th>
<th>Grout Material</th>
<th># of Specimes</th>
<th>Surface Roughness</th>
<th>Set #</th>
<th>Applied Moment at Failure (kip-ft)</th>
<th>Grout Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Partial depth</td>
<td>ODOT-Approved List</td>
<td>2</td>
<td>Sandblasted Surface</td>
<td>1</td>
<td>1.175</td>
<td>4.4</td>
</tr>
<tr>
<td>Full depth</td>
<td>ODOT-Approved List</td>
<td>2</td>
<td>As-C ast</td>
<td>1</td>
<td>0.425</td>
<td>1.6</td>
</tr>
<tr>
<td>Full depth</td>
<td>ODOT-Approved List</td>
<td>2</td>
<td>Sandblasted Surface</td>
<td>1</td>
<td>15.425</td>
<td>Not Determined</td>
</tr>
<tr>
<td>Full depth</td>
<td>ODOT-Approved List</td>
<td>2</td>
<td>Sandblasted Surface</td>
<td>2</td>
<td>35.9</td>
<td>140</td>
</tr>
<tr>
<td>Full depth</td>
<td>HSC-Grout</td>
<td>2</td>
<td>As-C ast</td>
<td>1</td>
<td>15.425</td>
<td>Not Determined</td>
</tr>
<tr>
<td>Full depth</td>
<td>HSC-Grout</td>
<td>2</td>
<td>Sandblasted Surface</td>
<td>1</td>
<td>15.425</td>
<td>Not Determined</td>
</tr>
<tr>
<td>Full depth</td>
<td>HSC-Grout</td>
<td>2</td>
<td>Sandblasted Surface</td>
<td>2</td>
<td>56.9</td>
<td>221</td>
</tr>
<tr>
<td>Full depth</td>
<td>UHPG</td>
<td>2</td>
<td>Sandblasted Surface</td>
<td>1</td>
<td>15.425</td>
<td>Not Determined</td>
</tr>
<tr>
<td>Full depth</td>
<td>UHPG</td>
<td>2</td>
<td>Sandblasted Surface</td>
<td>2</td>
<td>15.78</td>
<td>61</td>
</tr>
<tr>
<td>Full depth</td>
<td>Polymer Grout</td>
<td>2</td>
<td>Sandblasted Surface</td>
<td>1</td>
<td>15.425</td>
<td>Not Determined</td>
</tr>
<tr>
<td>Full depth</td>
<td>Polymer Grout</td>
<td>2</td>
<td>Sandblasted Surface</td>
<td>2</td>
<td>23.65</td>
<td>92</td>
</tr>
</tbody>
</table>

8.6 Discussion and Summary from Eccentric Load Tests

The test specimen with the currently used ODOT key way geometry with ODOT-approved grout failed under the self-weight of the test specimen unit, proving that the current key way with currently used grout is totally incapable of carrying any shear load in conjunction with out-of-plane tension or moment. For this reason, changes in key way
geometry details and grout material specifications are unavoidable. Increasing the key way depth alone without proper surface preparation did not improve the strength of the joint when using ODOT-approved grout.

The shear transfer strength of specimens with a sandblasted surface and full-depth key way and with high strength grout or ODOT-approved grout was significantly larger.

The structural tests to determine the joint strength under eccentric loads identified a serious inadequacy of the current key way geometry and ODOT-approved grout. Increasing the depth of the key way to full depth but with no sandblasting was not adequate to improve the joint strength under the simultaneous action of out-of-plane moment. However, the changes to the key way geometry and grout materials were found to improve the joint performance substantially.

The HSC concrete with #8 coarse aggregate had perfect bond with the concrete units with tensile strength.

The test results of eccentric loads and the beam assembly tests in chapter VI indicated that the compressive strength of the grout material is not the only measure to qualify the grout for all the applications, the bond strength is an important factor that might disqualify a high compressive strength grout, an accurate analysis of the bridge maybe required to select the grout based on the required load combinations.
CHAPTER IX

OBSERVATION OF CONSTRUCTION PRACTICES AT A BRIDGE CONSTRUCTION SITE

9.1 Objective

To understand the construction processes, the sequence of the activity, and the time frame for each activity, multiple site visits were made to a construction site where a new bridge was under construction in Shreve, Ohio, and the construction process was observed.

9.2 Construction Process

On the first working day, seven box beams were delivered to the site. Within 3 hours of the delivery, the beams were installed in position and tied together with tie rods tensioned to the specified torque of 250 lb-ft to provide the transverse clamping force of 15 kips at the tie rod location as shown in Figure 9.1. During the second working day, the six longitudinal joints were grouted by noon. The bridge was ready for the waterproofing membrane to be laid within two days.
The box beams for this bridge were B17”-48” with 17 inches of height and 48 inches of width with a 7-inch key way which was sandblasted according to ODOT specifications. The girders were a little out of straightness but within the allowable tolerance of one inch. Oakum (ropes) of one inch diameter was used to seal the key way joints between the box beams to prevent the grout from leaking into the stream from the bottom of the beams as shown in Figure 9.2.
The grouting process proceeded rapidly. After blowing the dust from the bridge deck and the joints, the grout was mixed in a grout mixer that was placed on the top of the bridge deck. Immediately after mixing, it was transported to the joints in trolleys. An approved grout material from ODOT QPL was used. The mixing water was around 5 quarts per 50 lb bag to obtain a flowable consistency and to accomplish the filling of the key way with wet grout in a reasonably short period of time. However, the amount of water added seemed excessive, and much more than what would be required to allow the development of the compressive strength to the specified level as shown in Figure 9.3.
9.3 Observations about the Installation

Beam $B_{17” - 48”}$ with 17 inches height and 48 inches width; the 7 inches key way was sand blasted according to ODOT specification, the girders were a little out of straightness within the allowable tolerance (Figure 9.5), one inch diameter oakum (Figure 9.6) was used to fill the gaps between the girders to prevent the grout from leaking into the stream from the bottom side of the girders
Figure 9.4: Abutment Formwork above Girder Level

Figure 9.5: Dimension Tolerance and Key Way Geometry
Figure 9.6: Oakum to Close Key Way Gaps
CHAPTER X

INVESTIGATION OF A BRIDGE THAT WAS IN SERVICE FOR 32 YEARS AT
THE TIME OF ITS DEMOLITION

Bridge RIC-42-12.34 in District 3, Mansfield, OH was constructed in 1983 and
scheduled for demolition in August 2015. The width of the bridge is 60 ft. and the span is
34 ft. (Figure 10.2). The primary objective of this task was to evaluate the condition of the
waterproofing membrane after the bridge was in service for 32 years. The asphalt concrete
overlay was carefully cut to extract waterproofing membrane samples for inspection and
water-tightness testing. The underside of the bridge had several locations of severe
corrosion of strands, spalling and deterioration as shown in Figure 10.1. Suitable locations
to remove the asphalt concrete overlay were selected to expose the membrane on the deck
surface based on the amount of corrosion, spalling and cracking on the underside of the
box beams.
Figure 10.1: Corrosion Damage of the Underside of Box Beams
A large area (about 250 to 300 ft\(^2\)) on the bridge deck had no waterproofing membrane at all. The most severe corrosion and spalling were found where no waterproofing membrane was present. The box beam surface at the key way interface was not sandblasted (Figure 10.4). Longitudinal cracks between the grout and the box beams were wide and were visible throughout the bridge deck (Figure 10.3).
Figure 10.3: Condition of a Typical Joint and Deck Surface

Figure 10.4: Box-Beam Conditions
CHAPTER XI

CONCLUSIONS

The following broad conclusions are drawn from the work completed in this research:

1. A small amount of tie force across the joint noticeably enhances the shear transfer strength of key way joints. It is expected that clamping stresses normal to the joint surface will increase the shear transfer strength of key way joints.

2. Joint failure is local under the applied load, the clamping forces provided by the tie rod increases the vertical shear strength locally and can be limited to the diaphragm width.

3. Cement-based high-strength concrete grout increases the strength of key way joints significantly. The use of polymer-based grout has also demonstrated excellent properties in vertical shear applications. Full-depth key way joints that are much deeper than the ODOT-specified key way joints, along with suitable surface preparation, will increase the shear transfer strength by a large margin compared to the current ODOT key way geometry and ODOT-approved grouts.

4. With suitable modifications to the key way geometries and judicious selection of grout material, it is possible to increase the shear transfer strength of longitudinal
5. Joints in box beam assemblies by a factor greater than 7.3 prior to the occurrence of first crack at one of the joints without providing any clamping force.

6. Key way joints with a combination of the currently specified ODOT geometry and ODOT-approved grouts are incapable of carrying shear loads in conjunction with out-of-plane moments. Suitable modifications to key way geometry and the grout materials that were developed in this research were found to provide adequate joint shear strengths and performance under the simultaneous action of out-of-plane moment and shear corresponding to 40-ft. and 90-ft. bridge spans.

7. The test results of eccentric loads and the beam assembly tests in chapter VI indicated that the compressive strength of the grout material is not the only measure to qualify the grout for all the applications, the bond strength is an important factor that might disqualify a high compressive strength grout, an accurate analysis of the bridge maybe required to select the grout based on the required load combinations.

8. The limited site inspections made in this project suggest that the construction practices followed at bridge construction sites are seriously flawed and will not prevent water leakage in box beam bridges. These practices need to be reviewed carefully and suitable modifications to the relevant specifications and inspection protocols developed in order to achieve reliable and consistent outcome of prevention of water leakage at the longitudinal joints in box beam bridges.
12.1 Introduction

A finite element analysis for 3-D models with 8-node solid elements with the same assumptions and load cases in the transverse direction from chapter VII was used to model bridge # RIC-TR037-0.21 over cedar fork, a tributary of the clear fork river on Shauck road, Richland County, OH for potential implementation. It is a box beam bridge with 39-ft of length and 28-ft width, with B17-48 cross-section for the box-beams. The section B17-48 is allowed for bridges with spans up to 40-ft. The beam was modeled with the maximum permitted span instead of the actual bridge span 39-ft to determine the maximum normal and shear stresses for this beam section.

12.2 Objective

Eight models were analyzed to (i) study the key way performance for the proposed modifications for the joints under the two load cases in transverse direction as described in section 7.2.3 in chapter VII for both standard key way geometry and modified geometry, (ii) to determine the shear force in the dowel bars in the longitudinal and transverse directions for factored loads conditions and service load conditions, (iii) and to study the effect of using multiple dowel bars on the normal stresses at the end diaphragms.
12.3 Loads in Longitudinal Direction

The location of the truck that will maximize the moment on the simply supported box beam was determined using the following approach.

![Diagram of loads and reaction forces on a box beam](image)

**Figure 12.1: Load Location in Longitudinal direction**

i) Resultant force of (HL-93 truck $\times$ Dynamic Load Factor $= 16 \times 1.25 = 205$ kip)

$$R = 2 \times 20 + 5 = 45 \text{ kip}$$

ii) The location with respect to load $P_B$

$$d = 20 \times 14 + 20 \times 14 \div 45 = 4.66' = 4' - 8"$$

iii) the reaction at $D$

$$\Sigma M_E = 0 = -LR_D + R (L - x - d)$$

$$R_D = R (L - x - d)/L$$
The maximum location will occur when
\[ \frac{dM}{dx} = 0 = R \left( L - 2x - d \right) / L \]
Therefore, the maximum is when
\[ x = L/2 - d/2 \]
The maximum bending moment under the middle point load will occur when the center of the beam is midway between that load and the resultant of all loads acting on the span.

Figure 12.2: Loads in Longitudinal direction

The maximum vertical deflection and bending moment occurs the 20-kip load at point B as shown in figure 12.2 and calculated as follow,

\[ M = 19.8 \text{ kip} \times 17.167 \text{ft} - 5 \text{ kip} \times 14 \text{ ft} \approx 270 \text{ ft-kip} \]
12.4 Modeling of Elastomeric Bearing

Bearing pads must satisfy the requirements of slip, shear, compressive stress, deflection, and rotation requirements of AASHTO LRFD Bridge Design Specification, 6th Edition (2012). Typical stress-strain curves of elastomers are shown in Figure 12.3 which shows that these pads need to deform significantly to provide the adequate reaction at the beam ends.

![Diagram of Dowel Bar, Box Beam, Abutment, Elastomeric Bearings, and Spring Factors]

**Figure 12.3: Typical Modeling of Elastomers**
12.4.1 Spring Factors in Compression

For the purpose of structural analysis and finite element (FE) analysis, the live load deformation of the elastomer was assumed to be 0.1-in, at the upper limit, for 2.0-in thick elastomer with strain $= 0.1/2.0 = 5\%$ and corresponding stress of 1.05ksi (Figure 12.4), for 60-in$^2$ bearing pad for shape factor =

$$K = \frac{(\sigma \times A)}{(\varepsilon \times L)} = \frac{1.05\text{ksi} \times 60\text{ in}^2}{0.05\text{ in/in} \times 2.0\text{ in}} \approx 630\text{ksi}$$

Eighteen nodes were used for modeling each 60-in$^2$ elastomer with vertical compressive spring factor $= 630/18 = 35\text{ksi}$.

12.4.2 Spring Factors in Shear

Shear Modulus $G_{73}$ at 73°F is less than $G_0$ at 0°F, therefore, a value of $G_{73} = 95\ \text{psid}$ was used. For shape factor $= 7.5$, and 60 in$^2$ elastomer area
\[
K_2 = G \times A \div h_r = 0.095 \times 60 \div 2 = 2.85 \text{ ksi.}
\]

Eighteen nodes were used for modeling each 60-in\(^2\) elastomer with shear spring factor = \(2.85 \div 18 = 0.158 \text{ ksi, in both lateral directions.}\)

12.5 Analysis Results

Normal stresses concentration was observed at diaphragm locations and at wheel load locations. Local shear stresses at wheel load locations. Also Stresses were nonlinear along the depth of the grout joint.

12.5.1 Normal Stresses – Load Case I

The load configuration in load case I Figure 7.3 is expected to cause tension at the top edge of the grouted longitudinal joint and compression at the bottom edge of the joint.

- The peak normal stresses occur under the wheel load, the intermediate diaphragm attracts more load when the wheel load is close to the diaphragm location
- Increase in the depth of the joint from 7-inch to 11-inch decreases the peak normal stresses at the top of the joint at diaphragm locations and under the load remote from the diaphragms by a factor of 1.5
- Remote from the diaphragms, there are larger normal stresses at the location of the wheel loads. These normal stresses are slightly larger than those at the diaphragm locations
- The occurrence of large normal stresses at the location of the wheel loads other than at the diaphragms seem to suggest that these peaks can occur at any point along the length of the joint because the moving wheel loads can occur at any point. Therefore, there is an equal chance that the peak normal stresses at the wheel loads will occur all along the joint length because of the moving nature of truck axle loads
Similar trend was observed for normal stresses at the bottom of the joint. However, the normal stresses at the bottom are compressive and therefore of less concern when high strength grout is used.

Figure 12.5: Normal Stresses along the Top of Key Way Joint of 40-ft Bridge
12.5.2 Normal Stresses – Load Case II

The load configuration in this load case as seen in Figure 7.4 is expected to cause compression at the top edge of the grouted longitudinal joint and tension at the bottom of the joint. In this load case, the joint is not at the axis of symmetry. Therefore, the joint will be subjected to both normal stresses and shear stresses.

As seen in Figure 12.5 and Figure 12.6, the normal tensile stresses on the joints at the bottom edge within the diaphragm width were much larger than the normal stresses obtained at locations remote from the diaphragms suggesting that much of the out-of-plane moment is transferred between the box beams primarily through the diaphragms.

Similar trends can be observed for normal compressive stresses at the top of the joint as seen in Figure 12.6. However, the normal stresses at the top are mostly compressive and therefore of less concern.
12.5.3 Shear Stresses

Because of symmetric nature of the geometry, stiffness, and loading for load case I, the shear stresses at the joint are expected to be zero. For load case II the shear stresses are transferred to the adjacent box beams at the wheel load locations. The diaphragms attract large shear forces in both load cases.

![Cumulative Shear Force along the Depth of the Key Way Joint](image)

**Figure 12.7: Shear Force**

12.5.4 Support Reaction in Longitudinal direction

Twenty six percent reduction in the stresses at the end diaphragms by providing hinge supports between the box beams and the abutments under truck loads with dynamic magnification factor as shown in Figure 12.8.
Figure 12.8: Effect of Dowel Bars at Supports

The 3-D finite element analysis shows large horizontal reaction at the dowel bars in the longitudinal direction, these forces cannot be mobilized using 1-in diameter dowel bar, the dowel bars are expected to undergo large deformation to allow the longitudinal movements of the box beams. The box beams are seated on the abutments before installing the dowel bars; the self-weight of the box beams will not develop any stresses in the dowel bars. To control the stresses near the supports, the joint between the box beams and the abutment shall be designed to act as a hinge to develop strength to prevent the relative longitudinal movements between the adjacent box beams. The abutment design should account for the horizontal reaction in the dowel bars, the dowel bars shall be designed based on the stresses on each individual case depending on the span of the bridge. To reduce the amount of horizontal reaction on the abutments, the dowel bars may be designed to prevent the movements at service load conditions only with allowing the joint to crack under factored load conditions.
12.5.4.1 Required dowel bars strength to fully restrain the lateral and longitudinal movements in ultimate load condition

The design shear force due to the factored super imposed dead load, lane load, future wearing coarse, and the wheel load of HL-93 design truck with dynamic load factor was determined using 3-D model and were found to be $V_u = 240$ kip. The design shear strength of 1.5-in diameter dowel bar is

$$\Phi V_n = \Phi \times 0.6 \times f_y \times A_b = 0.75 \times 0.6 \times 1.76 \times 60 = 47.5 \text{ kip}$$

Number of required dowel bars $= 240 \div 47.5 = 5.05 \approx 6$ dowel bars

12.5.4.2 Required dowel bars strength to restraint only the lateral and longitudinal relative movements in service load condition

Under service super imposed dead loads, lane load, un-factored future wearing coarse, and HL-93 truck wheel load with dynamic load factor, the dowel bar reactions were determined using 3-D finite element analysis to be 148 kip. The required dowel bars to prevent longitudinal movement the shear strength of 1.5-in. dowel bar under service load is

$$V_n = (2 \div 3) \times 0.6 \times f_y \times A_b = (2 \div 3) \times 0.6 \times 60 \text{ ksi} \times 1.768 \text{ in}^2 = 42.43 \text{ kips}$$

Number of required dowel bars $= 148 \div 42.43 = 3.48 \approx 4$ dowel bars

12.6 Recommendation for Implementation

From the work completed in this research, three items have potential for implementation: (i) revised dowel bars design, (ii) revised key way geometry, (iii) type of grout material, and (iv) construction specifications.
12.6.1 Revised Dowel Bars Arrangements

Figure 12.9: Original Anchor Dowel Details (Top) Modified Anchor Dowels Details (Bottom)

12.6.2 Key Way Geometry

A new key way geometries as shown in Figure 12.10 that were successfully demonstrated to increase the shear transfer strength of grouted joints is recommended with Wide full-depth key way.
Figure 12.10: Original Geometry (Top) Modified Geometry (Bottom) for B17-48

12.6.3 Grout Material

The Cement-based grout and High-strength concrete (9,800 psi) grout was developed and tested in this study. It is recommended for implementation.
This grout is made using traditional concrete with maximum #8 coarse aggregates and Type I Portland cement. Compaction is achieved for this grout with a vibrator. The opening at the top of the key way needs to be at least one inch to allow for vibration. The key way geometry shown in Figure 12.10 is recommended to be used for high-strength concrete grout.

High-strength concrete grout needs to be flowable with a compressive strength of over 10,000 psi compressive strength. The mix proportions given in the following table were found to be satisfactory for the high strength grouts in a laboratory environment. However, other mixes with better optimized aggregate gradation and supplementary cementitious materials may also be used to improve the mix proportions and reduce cement content while maintaining similar minimum strength and performance. Typical mix proportions for high-strength concrete grout are shown in Table 12.1.

Table 12.1: Mix Proportions for High-Strength Concrete Grout with #8 Aggregate

<table>
<thead>
<tr>
<th>HSC Grout</th>
<th>Cement</th>
<th>Sand</th>
<th>Aggregates</th>
<th>Water W/C = 0.35</th>
<th>S.P.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mix proportion by weight</td>
<td>1</td>
<td>1.32</td>
<td>1.45</td>
<td>0.35</td>
<td>0.03</td>
</tr>
<tr>
<td>lb/ft³</td>
<td>41</td>
<td>54</td>
<td>59</td>
<td>14.5</td>
<td>1.7</td>
</tr>
<tr>
<td>lb/yd³</td>
<td>1100</td>
<td>1450</td>
<td>1596</td>
<td>385</td>
<td>46</td>
</tr>
</tbody>
</table>

S.P. stands for super-plasticizer (high-range water reducer)

12.6.4 Construction Specifications

1. Key ways should be grouted after erection of box beams. Generally, plastic rope or jute is installed into the bottom of the key way to block the grout from flowing out. Utmost care shall be taken to seal the bottom edge of the key way to prevent leaking of wet grout during and after the grouting process.
2. Ensure that the installation is done properly. Box beam keys have failed because of improper jute installation. However, suitable foam sealant may be used to seal the key way and make it watertight before the grouting operation begins.

3. The fabricator shall sandblast the key way surface within four days of shipment to the project’s site as specified in ODOT standard drawings PSBD-2-07. The sandblasting shall yield a visual appearance and texture equal or rougher than 100 grit sandpaper over the entire key way surface. When stains are visible before sandblasting the concrete, use a degreaser to ensure removal of grease, oils and other similar contaminate. The degreaser shall be water soluble so it can be removed before the blasting begins. Before mortaring, remove all dirt, dust, grease, oil and other foreign materials from surfaces using a high pressure wash of at least 1,000 psi at a delivery rate of 4 gal/min.

4. Grout should meet the material requirements of the Office of Structural Engineering's standard box beam drawings. Additional requirements for high performance grout which a high strength concrete with #8 maximum size aggregate with the following specifications
   - Minimum compressive strength of 10,000 psi needs to be achieved before allowing any construction equipment on the deck
   - The grout shall be designed to include well graded #8 coarse aggregate suitable for high strength concrete applications
   - The top surface of the grouted joints shall be cured with approved curing compound that is to be applied on the surface after one hour of grouting
   - The grout shall have workability that is adequate to fill the key way
Suitable needle vibrator shall be used to consolidate the grout

5. The manufacturer’s mixing instructions are required and should ensure that the grout is properly mixed, vibrated into the joints, cured, and sampled for testing

6. Grouting should not be allowed if there is construction traffic or erection still going on

7. The grout can be cracked by the vibration and deflection movements and make the key ways worthless

8. The design of the structure counts on the grout in the shear keys

9. Do not allow traffic on the deck before the grout has obtained the required strength of 10,000 psi. This includes construction traffic. This specification must be strictly followed.

12.6.5 Design Calculations for B17-48

A summary for design calculation for box beam bridge with 40-ft of length and 28-ft width are shown in Table 12.3. The recommended modifications for the key way geometry, as shown in Figure 12.3, will maintain the loading strength of the bridge for shear and moment strength in ultimate load conditions and within the allowed stress limitations by the design codes under service load conditions.
Table 12.2: Design Summary for Original and Modified Geometries

<table>
<thead>
<tr>
<th>Properties for box-beam sections</th>
<th>Original Beam</th>
<th>Modified Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area (in²)</td>
<td>594.20</td>
<td>624</td>
</tr>
<tr>
<td>Distance from the centroid to bottom fiber Yₜ (in)</td>
<td>8.40</td>
<td>8.43</td>
</tr>
<tr>
<td>Moment of inertia (in⁴)</td>
<td>18,825</td>
<td>19,022</td>
</tr>
<tr>
<td>Distance from the centroid to top fiber Cₜ (in)</td>
<td>8.56</td>
<td>8.57</td>
</tr>
<tr>
<td>Total weight of one box-beam including diaphragms (kips)</td>
<td>26.5</td>
<td>27.4</td>
</tr>
<tr>
<td>Total length (ft)</td>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td>Span between dowel bars (ft)</td>
<td>39</td>
<td>39</td>
</tr>
<tr>
<td>Initial prestressing force per strand (lb)</td>
<td>33,818</td>
<td>33,818</td>
</tr>
<tr>
<td>Final prestressing force per strand (lb)</td>
<td>26,378</td>
<td>26,378</td>
</tr>
</tbody>
</table>

Loads considered

Self-weight of the beam

3.5” Asphalt concrete overlay

Weight of the key way grout

Moving load form HS25 truck, and HL93 including lane load

Service loads stresses

| Initial stresses at the top fiber at mid-span (psi)                    | 115 (tensile) | 131 (tensile) |
| Initial stresses at the bottom fiber at mid-span (psi)                 | -2,149 (compressive) | -2,065 (compressive) |
| Final stresses at the top fiber at mid-span (psi)                      | -2,216 (compressive) | -2,198 (compressive) |
| Final stresses at the bottom fiber at mid-span (psi)                   | 599 (tensile) | 615 (tensile) |

Design flexure strength

\(\Omega M_n =\) 809.60 ft-kips 809.00 ft-kips

Design shear strength including the contribution of the shear reinforcement (was found to be adequate)

\(\Omega V_n =\) 94.8 kips 92.4 kips

Deflection and camber

| Initial camber (in)                                                   | 0.99          | 0.96          |
| Maximum deflection under service loads (in)                          | 0.48          | 0.64          |
12.6.6 Design Calculation for B42-48

A similar set of design calculation for box beam bridge with 103-ft of length and 24-ft width are shown in Table 12.3. The recommended modifications for the key way geometry, as shown in Figure 12.8, will maintain the loading strength of the bridge for shear and moment strength in ultimate load conditions and within the allowed stress limitations by the design codes under service load conditions.

Figure 12.11: Original Geometry (Top) Modified Geometry (Bottom) for B42-48
Table 12.3: Design Summary for Original and Modified Geometries

<table>
<thead>
<tr>
<th>Properties for box-beam sections</th>
<th>Original Beam</th>
<th>Modified Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area (in²)</td>
<td>873.95</td>
<td>874</td>
</tr>
<tr>
<td>Distance from the centroid to bottom fiber Y_b (in)</td>
<td>20.8</td>
<td>20.87</td>
</tr>
<tr>
<td>Moment of inertia (in⁴)</td>
<td>205,435</td>
<td>211,453</td>
</tr>
<tr>
<td>Distance from the centroid to top fiber C_t (in)</td>
<td>21.20</td>
<td>21.13</td>
</tr>
<tr>
<td>Total weight of one box-beam including diaphragms (kips)</td>
<td>106.7</td>
<td>106.1</td>
</tr>
<tr>
<td>Total length (ft)</td>
<td>103</td>
<td>103</td>
</tr>
<tr>
<td>Span between dowel bars(ft)</td>
<td>102</td>
<td>102</td>
</tr>
<tr>
<td>Initial prestressing force per strand (lb)</td>
<td>33,818</td>
<td>33,818</td>
</tr>
<tr>
<td>Final prestressing force per strand (lb)</td>
<td>26,378</td>
<td>26,378</td>
</tr>
</tbody>
</table>

Loads considered

Self-weight of the beam

3.5” Asphalt concrete overlay

Weight of the key way grout

Moving load form HS20 truck, and HL93 including lane load

Service loads stresses

<table>
<thead>
<tr>
<th>Initial stresses at the top fiber at mid-span (psi)</th>
<th>-686 (compressive)</th>
<th>-710 (compressive)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial stresses at the bottom fiber at mid-span (psi)</td>
<td>-2,240 (compressive)</td>
<td>-2,221 (compressive)</td>
</tr>
<tr>
<td>Final stresses at the top fiber at mid-span (psi)</td>
<td>-3,029 (compressive)</td>
<td>-3,043 (compressive)</td>
</tr>
<tr>
<td>Final stresses at the bottom fiber at mid-span (psi)</td>
<td>699 (tensile)</td>
<td>726 (tensile)</td>
</tr>
</tbody>
</table>

Design flexure strength

\( \Phi M_0 = \) 4,525 ft-kips

Design shear strength including the contribution of the shear reinforcement (was found to be adequate)

\( \Phi V_0 = \) 280 kips

Deflection and camber

| Initial camber (in) | 2.3 | 2.2 |
| Maximum deflection under service loads (in) | 2.7 | 2.9 |
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APPENDIX

STRESS CONTOURS
Load Case I – With Intermediate Diaphragm
36-in Key Way
Normal Stresses (S22) at Key Way Joint

X = 0 TO X = 540 inch

X=204 in. Under 20 kip Wheel Load
Far from the Diaphragm

X=540 in. Under 20 kip Wheel Load
At Diaphragm Location
Load Case II - With Intermediate Diaphragm
12-in Key Way
Normal Stresses (S22) at Key Way Joint

X = 0 TO X = 540 inch

X=204-in. Under 20 kip Wheel Load
Far from the Diaphragm

X=540-in. Under 20 kip Wheel Load
At Diaphragm Location
Load Case I - With Intermediate Diaphragm
12-in Key Way
Normal Stresses (S22) at Key Way Joint

X = 0.00 TO X = 45-ft

X=204-in. Under 20 kip Wheel Load
   Far from the Diaphragm

X=540-in. Under 20 kip Wheel Load
   At Diaphragm Location
Load Case I - Without Intermediate Diaphragm

36-in Key Way

Normal Stresses (S22) at Key Way Joint

X = 0 TO X = 540 inch

X=204-in. Under 20 kip Wheel Load
Far from the Diaphragm

X=540-in. Under 20 kip Wheel Load
At Diaphragm Location
Load Case I - Without Intermediate Diaphragm
12-in Key Way
Normal Stresses (S22) at Key Way Joint

X = 0 TO X = 540 inch

X=204-in. Under 20 kip Wheel Load
Far from the Diaphragm

X=540-in. Under 20 kip Wheel Load
At Diaphragm Location
Load Case I – With Intermediate Diaphragm
36-in Key Way
Shear Stresses (S23) at Key Way Joint

X = 0 TO X = 540 inch

X=204 in. Under 20 kip Wheel Load
Remote from the Diaphragm

X=540 in. Under 20 kip Wheel Load
At Diaphragm Location
Load Case I – With Intermediate Diaphragm
36-in Key Way
Shear Stresses (S23) – Cross Section

X=204in.

X=540in.
Load Case 1 - With Intermediate Diaphragm
12-in Key Way Shear Stresses (S23) at Key Way Joint

X = 0.00 TO X = 45-ft

X = 540-in. Under 20 kip Wheel Load at Diaphragm Location
X = 204-in. Remote from the Diaphragm
Load Case I – With Intermediate Diaphragm
12-in Key Way
Shear Stresses (S23) – Cross Section

X=204in.

X=504in.
Load Case I - Without Intermediate Diaphragm

12-in Key Way

Shear Stresses (S23) at Key Way Joint

X = 0 TO X = 540 inch

X=204-in. Under 20 kip Wheel Load
Remote from the Diaphragm

X=540-in. Under 20 kip Wheel Load
At Diaphragm Location
Load Case II - With Intermediate Diaphragm
36-in Key Way
Shear Stresses (S23) at Key Way Joint

X = 0 TO X = 540 inch

X=204 in. Under 20 kip Wheel Load
Remote from the Diaphragm

X=540 in. Under 20 kip Wheel Load
At Diaphragm Location
Load Case II – With Intermediate Diaphragm
36-in Key Way
Shear Stresses (S23) – Cross Section

X=204 in.

X=540 in.
Load Case II - With Intermediate Diaphragm
12-in Key Way
Shear Stresses (S23) at Key Way Joint

X = 0 TO X = 540 inch

X=204-in. Under 20 kip Wheel Load
Remote from the Diaphragm

X=540-in. Under 20 kip Wheel Load
At Diaphragm Location
Load Case II – With Intermediate Diaphragm
12-in Key Way
Shear Stresses (S23) – Cross Section

X=204in.

X=540in.
Load Case I - Without Intermediate Diaphragm
36-in Key Way
Shear Stresses (S23) at Key Way Joint

X = 0 TO X = 540 inch

X=204-in. Under 20 kip Wheel Load
Remote from the Diaphragm

X=540-in. Under 20 kip Wheel Load
At Diaphragm Location
Load Case I – Without Intermediate Diaphragm
36-in Key Way
Shear Stresses (ft kips) – Cross Section

X=204 in.  X=540 in.