Figure 5.2-37 Construction of additional exterior box-beam.
5.3 **Test Program Phase**

The strain and the load distribution tests conducted during the test program are described in this section. The data acquisition system and the different types of sensors employed in this study are also presented herein.

5.3.1 **Data Acquisition System**

The bridge model was instrumented with a range of sensors connected to a commercially available data acquisition system “P8048” supplied by the PROSIG Company (see Figure 5.3-1). The data acquisition system was connected to a Central Processing Unit (CPU) and subsequently linked to a monitor. The software used to run the data acquisition system was Dats for Windows 6.1, Data Acquisition V4.03.12c. This test setup was designed for forty-eight channels. The sensors were connected to terminal blocks, which were subsequently connected the data acquisition system.

![Figure 5.3-1 Data acquisition system.](image)

5.3.2 **Instrumentation of the Bridge Model**

The three main types of sensors used in this study were strain gages, linear motion transducers, and load cells. The type of strain gages used for the test program was N2A-06-20CBW-350, supplied by the Vishay Micro-Measurements, Inc., and were installed on the top surface of the deck slab in the transverse direction. A total of twenty seven strain gages were installed on one-
half of the deck slab along the shear-key locations, where the longitudinal cracks usually develop as observed in the field, as shown in Figure 5.3-2. The length of the strain gage used was 2 in., similar to the designed top width of the grouted shear-keys. Four linear motion transducers were attached at the mid-span to the underside of each beam to measure the vertical deflection of each box-beam during loading, as shown in Figure 5.3-2. Load cells were attached to the unbonded TPT CFCC to monitor the applied TPT forces.

The transverse strain gages were labeled by creating a skew grid from three axes parallel to the longitudinal direction, from “A-A” to “C-C”, and nine axes parallel to supports lines, from “1-1” to “9-9”. A typical transverse strain gage was labeled as “A-1”, when the strain gage was on the intersection between longitudinal axis “A-A” and transverse axis “1-1”. As shown in Figure 5.3-3, the direction of the transverse strain gages was made perpendicular to the longitudinal axes of the box-beams to measure the effective transverse strains developed due to the application of the TPT forces. Before installing the strain gages, the area was ground and subsequently smoothed using sand paper. M-Prep Conditioner “A” was used as a degreaser to clean the surface of the gage area and moderate amount of M-Prep Neutralizer 5A was also added to naturalize the acidic content of the concrete surface. The strain gages were placed on a chemically cleaned board using PCT-2M 310009 gage installation tape and partially placed on deck slab. M-Bond AE-10 Adhesive was prepared and placed on both the strain gages locations on the deck slab and the bonding side of the strain gages. After the strain gages were attached to the top surface of deck slab, a dead weight was used to provide uniform pressure of 5 psi on the installed strain gage for a period not less than 24 hours as recommended by the supplier.

5.3.3 Tests Conducted on the Bridge Model

The test program was conducted to study the effect of number of transverse diaphragms and the level of TPT forces on the behavior of the bridge model in the transverse direction. As mentioned earlier, the two main tests conducted throughout the test programs were the strain and load distribution tests. Details of these tests are presented in the following sections.
5.3.3.1 Strain Distribution Test

A parametric study was conducted by applying different levels of TPT forces at different number of diaphragms (different arrangements of TPT) and the corresponding transverse strains developed at the top surface of the deck slab were measured. The CFCC used for applying TPT forces are shown in Figure 5.3-4(a). The dead and live-ends for the TPT system are also shown in Figure 5.3-4(b) and Figure 5.3-4(c), respectively. The TPT forces were applied using a center-hole hydraulic cylinder, manufactured by the ENERPAC. The hydraulic cylinder has a maximum capacity of 132 kip and a maximum stroke of 12 in., as shown in Figure 5.3-4(d). The elongations of the CFCC were measured at three arbitrary levels of the TPT force; 10.8, 22.9 and 35 kip. The determination of the elongation was based on the numerical difference between the final and the initial measured readings of the elongation of the CFCC, as shown in Table 5.3-1. The initial reading corresponds to the elongation at a TPT force of 0.2 kip while the final reading corresponds to the elongation at the respective level of the TPT force. Steel spacers were used at the live-end to compensate for the expected elongation in the CFCC.
Figure 5.3-3 Layout of the strain gages installed on the deck slab.
One center-hole load cell, with a maximum capacity of 70 kip, was attached at the dead end of each CFCC to monitor the levels of the TPT force, as shown in Figure 5.3-4. Details of a typical dead-end for TPT system showing the load cells and the position of the CFCC are shown in Figure 5.3-5. In order to avoid possible local crushing of concrete due to the TPT forces, steel bearing plates, 7 in. wide, 8 in. high and 1.5 in. thick, were attached to all ends of the transverse diaphragms (ten plates). Rubber sheets of 0.25 in. thick were provided with the same width and height as the steel plates and attached to the bearing surface of the steel plates to ensure uniform distribution of the TPT forces on the bearing surfaces. The different arrangements of the strain distribution test are shown in Figure 5.3-6. The strain distribution test was intended to study the effect of different number of diaphragms and different levels of the TPT forces on the transverse strains as follows. The sequence of the strain distribution test is outlined as follows.

Case of all diaphragms:

The TPT forces were applied to all diaphragms at three different levels of 80, 40, and 20 kip/diaphragm.

Case of four diaphragms:

The TPT forces were applied to all diaphragms except the mid-span diaphragm at three different levels of 80, 40, and 20 kip/diaphragm.

Case of three diaphragms:

The TPT forces were applied to the end and the mid-span diaphragms only at three different levels of 80, 40, and 20 kip/diaphragm.

Case of no diaphragms:

In this case, no TPT force was applied to the bridge model. This case was used as the reference case throughout the test program.
Table 5.3-1 Elongation of CFCC at three different levels of TPT force.

<table>
<thead>
<tr>
<th>TPT Force, kip</th>
<th>Initial Reading, in.</th>
<th>Final Reading, in.</th>
<th>Elongation, in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>10.8</td>
<td>5.44</td>
<td>5.69</td>
<td>0.25</td>
</tr>
<tr>
<td>22.9</td>
<td>5.44</td>
<td>6.00</td>
<td>0.56</td>
</tr>
<tr>
<td>35</td>
<td>5.44</td>
<td>6.31</td>
<td>0.87</td>
</tr>
</tbody>
</table>

Figure 5.3-4 Details of the unbonded TPT system.
Figure 5.3-5 Typical dead-end of TPT system.
Figure 5.3-6 Arrangements for strain distribution test.

(a) Case of five diaphragms

(b) Case of four diaphragms

(c) Case of three diaphragms
5.3.4 Load Distribution Test

The load distribution test was conducted by applying a single point load at the mid-span of each box-beam one box-beam at a time and the corresponding deflections were measured using linear motion transducers. In order to avoid a possible premature cracking of the bridge beams, a service load of 15 kip was selected below the calculated cracking load of 16 kip for the entire bridge model. The vertical load of 15 kip was applied on each box-beam with different arrangements of the TPT forces, similar to the arrangements used for conducting the strain distribution test. The vertical load was applied by using a hydraulic cylinder, manufactured by the Templeton Kenly & Company, Inc., which has a maximum capacity of 220 kip and a maximum stroke length of 6 in. The hydraulic cylinder was attached to the bottom flange of an overhead transverse steel I-beam by four wheels to ensure easy movement from one location to the other, as shown in Figure 5.3-7. The single point load was applied using a steel cylinder with a diameter of 10 in. and height of 5 in. The 10 in. diameter was chosen to avoid the punching shear failure of the oval-shape ducts at mid-span underneath the loading locations. Figure 5.3-8 through Figure 5.3-11 show the load distribution test sequence.

(a) Application of vertical load at mid-span
(b) Details of load distribution test setup

Figure 5.3-7 Load distribution test setup.
Figure 5.3-8 Load distribution test with the application of TPT forces at five diaphragms.
Figure 5.3-9 Load distribution test with the application of TPT forces at four diaphragms.
Figure 5.3-10 Load distribution test with the application of TPT forces at three diaphragms.
Figure 5.3-11 Load distribution test without applying TPT forces at any diaphragm.
5.3.5 Stages of Test Program

5.3.5.1 Uncracked Deck Slab

The reference point for the investigation was the condition before longitudinal cracks occur. This stage simulated a typical newly constructed highway bridge as stated earlier. The load and strain distribution tests were both conducted at this stage of the bridge model.

5.3.5.2 Cracked Deck Slab

At this stage, the cracked deck slab and the shear-keys were partially cracked. The purpose was to simulate the longitudinal cracks developed between the box-beams over the shear-keys due to a combination of vertical loads and thermal effects. The creation of longitudinal cracks was made in three steps. The vertical load was applied to one box-beam while partially restraining the other three box-beams from the top and the bottom to prevent the entire bridge from rotating as it tends to act as a one unit.

In order to crack the exterior shear-key on axis C-C, the load was applied at the mid-span of beam B-4 and beams B-1, B-2, and B-3 were partially restrained at the top and the bottom using steel supports, as shown in Figure 5.3-12. A reversed arrangement was adopted to crack the other exterior shear-key on axis A-A by applying the load at beam B-1 and partially restraining beams B-2, B-3, and B-4 from rotation and translation. However, in order to crack the interior shear-key on axis B-B, the load was applied to beam B-3 and beams B-1, B-2, and B-4 were restrained at the top and the bottom, as shown in Figure 5.3-13. The crack widths were measured using the crack comparator and compared with the cracks pattern provided by the Michigan Department of Transportation (MDOT) for the box-beam bridge number S03-63022 built in 1999. The bridge carries South Hill Road over I-96. The strain distribution test was not conducted at this stage because of the discontinuity of the deck slab in the transverse direction as a result of the longitudinal cracks. The load distribution test was the only test conducted after cracking the deck slab and the results were compared to the stage of uncracked deck slab. Details of the cracks pattern developed in the deck slab are shown in Chapter 6.
Figure 5.3-12 Setup for cracking exterior shear-key C-C.
5.3.5.3 Damaged Beam Replacement

This stage was developed also to demonstrate the merits of the use of unbonded TPT CFCC instead of bonded TPT steel strands in the construction of side-by-side box-beam bridges. It is a well known fact that replacing a damaged beam in a highway bridge, when bonded TPT steel strands is used, is challenging. Therefore, studying the effect of the replacement of a damaged beam on the behavior of the bridge model in the transverse direction can quantify the efficiency of the replacement process. In this project, the effect is evaluated under the eccentric loading...
with the application of different TPT arrangements. Emphasis was also placed on the development of a more practical and simple approach for replacing the damaged box-beam. Only the load distribution test was conducted at this stage of the test program. The experimental work of this stage consisted of three major phases as follows.

**Phase (I):** Saw cutting of the damaged exterior beam.

1. The unbonded TPT CFCC were removed from the oval-shape ducts. This was followed by the removal of all the steel bearing plates from the bridge model.

2. The surface of the shear-key was cleaned. The cutting line was marked with a straight line along the entire span at the mid-width of the exterior shear-key C-C, which connected beam B-3 and beam B-4.

3. A full-depth cut of the shear-key was made over the marked line by using a circular saw provided by Fmg® Concrete Cutting, Inc. to separate the assumed damaged beam B-4 from the remaining portion of the bridge model (Figure 5.3-14).

**Phase (II):** Placement of the new exterior beam.

1. The first task undertaken at this phase was to remove the remaining FSSC grout from the shear-key C-C, as shown in Figure 5.3-15.

2. Horizontal holes of 0.75 in. diameter and 12 in. deep were drilled into the deck slab using electric hammer drill to accommodate the new transverse reinforcement bars, as shown in Figure 5.3-16(a). The spacing of the holes was uniformly maintained at 6 in. The holes were drilled at the mid-depth of the deck slab 1.5 in. from the top surface of the deck slab.

3. Styrofoam gaskets were then attached to the face of beam B-3 around the oval-shape ducts, as shown in Figure 5.3-16(b). This was to ensure continuity of the diaphragms when the new beam B-5 was placed and to prevent possible leakage of the shear-key grout inside the ducts.

4. The new box-beam was placed on the elastomeric bearing pads on the steel supports.

5. The formwork for the new shear-key was constructed by sealing the underside of the shear-key with plywood and the ends of the shear-key with a Styrofoam.
6. The shear-key was cleaned and the FSSC grout was placed in the keyway, as shown in Figure 5.3-17.

7. The newly constructed shear-key was cured under wet burlap and plastic sheets.

(a) Saw cutting of beam B-4  
(b) Separation of beam B-4

Figure 5.3-14 Separation of exterior beam B-4 by saw cutting through shear-key C-C.

(a) Remained portion of shear-key  
(b) Grout removed from shear-key

Figure 5.3-15 Removal of the remaining grout from the bridge model.
Phase (III): Reinforcement and casting of the deck slab.

1. The longitudinal reinforcements were placed and tied at equal spacing of 6 in.

2. The transverse reinforcements were also placed by using a high performance epoxy resin called Sikadur Injection Gel/AnchorFix-4 listed in the MDOT’s “Material Source Guide” Specification Number 603.02B. The procedures followed are as stated below:
a. The drilled holes were cleaned by blowing air through them using air-jet and water-jet accordingly to ensure that all dust particles are cleared to ensure bond between the bars and the concrete.

b. The nozzle of the dispensing gun was inserted into the drilled holes and the epoxy resin was pumped to fill the hole completely, as shown in Figure 5.3-18.

c. The reinforcement bars #3 were inserted gradually while rotating them to ensure that the rebar was coated in the epoxy.

d. The reinforcement bars #3 were left undisturbed for a duration of 24 hours until the epoxy resin cured and attained the required bond strength.

e. The deck slab formwork was then constructed. This was followed by casting of the deck slab with the same concrete mix design used for casting the old deck slab, as shown in Figure 5.3-19. The deck slab was cured under wet burlap and plastic sheets.

(a) Epoxy resin kit  (b) Insertion of epoxy into holes

Figure 5.3-18 Materials used and insertion of epoxy into the drilled holes.
(a) Reinforcing of new deck slab  
(b) Casted new deck slab

Figure 5.3-19 Construction of formwork and casting of deck slab.

5.3.5.4 Ultimate Load Test

The objectives of the ultimate load test were to determine the ultimate load-carrying capacity of the bridge model and to evaluate the efficiency of using two unbonded CFCC through the oval-shape duct at each diaphragm as a TPT system. The bridge model was loaded eccentrically at the mid-span of beam B-2 using a two-point loading frame (spreader), as shown in Figure 5.3-20. The two-point loading frame consisted of a 10 × 10 in. structural square steel tube with a total span of 7.5 ft. A TPT force of 80 kip was applied to all five diaphragms prior to testing. The TPT forces were monitored during the test using load cells attached to the dead-end of the CFCC. Four linear motion transducers were also installed at the mid-span of each of the beams to monitor the corresponding deflections of the four beams, as shown in Figure 5.3-21. In order to monitor the strains developed on the deck slab, two strain gages were installed in the longitudinal direction at the mid-span of beams B-2 and B-5. Five loading and unloading cycles were conducted before failing the bridge model in order to separate the elastic and inelastic energies. Each loading and unloading cycle was conducted by increasing and releasing the applied load at a rate of 15 kip/minute. The maximum applied loads in the five cycles were 20, 40, 60, 80, and 100 kip.
Figure 5.3-20 Experimental setup and instrumentation of the bridge model.

Figure 5.3-21 Ultimate load test setup.
CHAPTER 6: EXPERIMENTAL RESULTS AND DISCUSSIONS

6.1 Introduction

This chapter provides a comprehensive report and discussion of the results obtained during the experimental phases of this research project. The first section discusses the transverse strains induced on the deck slab due to varied levels of TPT forces applied to different number of diaphragms. The load distribution results obtained at different stages of deck slab condition are discussed in the second section. In the final section, the ultimate response of TPT system is evaluated using the ultimate load test.

The results related to the construction phase are presented in Appendix A. The appendix contains information on 1) the pre-tensioning forces during the curing period of the concrete, 2) continuous monitoring of the camber for the four individual box-beams, and 3) transfer length of the pre-tensioning forces.

6.2 Transverse Strain Distribution

6.2.1 Introduction

The transverse strain distribution test was conducted on the bridge model by varying the number of transverse diaphragms and the levels of TPT forces. Three different levels of TPT forces were applied to each diaphragm. These levels were 20, 40, and 80 kip. The forces were applied using two unbonded CFCC located at each diaphragm. Each CFCC was post-tensioned by half of the TPT force provided at each diaphragm. A total of five transverse diaphragms were employed for the application of TPT forces. Three different numbers of diaphragms were selected for the parametric study. These were three, four, and five diaphragms. The transverse strains were measured perpendicular to the longitudinal axis of the bridge model using twenty seven transverse strain gages mounted on the top surface of the deck slab directly above the shear-keys.

This section is divided into three sub-sections. The first sub-section presents the variation of the transverse strains as a result of different arrangement of TPT (different number of diaphragm and different levels of TPT force) along the span of the bridge model. The effect of the level of TPT forces and the number of diaphragms on the transverse strain at various
locations are presented in the second and the third sub-sections, respectively. In addition, American Association of State Highway and Transportation Officials Load and Resistance Factor Design (AASHTO LRFD, 2004) transverse prestress recommendation was evaluated. In this study, the transverse diaphragms with TPT forces are denoted post-tensioned diaphragms and the transverse diaphragms without any TPT force are denoted none post-tensioned diaphragms.

6.2.2 Longitudinal Variation of Transverse Strains

In order to investigate the effect of different arrangements of TPT on the transverse strains developed on the surface of the deck slab, the transverse strains were plotted against the distance from the bridge end for one-half of the bridge span, as shown in Figure 6.2-1 through Figure 6.2-9. Three data series are plotted in each figure indicating the strains measured over the shear-keys. Figure 6.2-1 through Figure 6.2-3 show the variation of the transverse strains when applying different levels of TPT forces to all five diaphragms. Figure 6.2-4 through Figure 6.2-6 show the variation when applying different levels of TPT forces to four diaphragms. Figure 6.2-7 through Figure 6.2-9 show the variation when applying different levels of TPT forces to three diaphragms.

The deck slab experienced the highest transverse strains at the locations aligned with the post-tensioned diaphragms. It is clear that the strains decrease towards zero in the areas between the post-tensioned diaphragms. For instance, as shown in Figure 6.2-2, point C-5, located over quarter-span diaphragm, experienced transverse strain of -98 $\mu$ε due to TPT force of 80 kip applied to four diaphragms. At the same time, point C-7, located between the quarter-span and mid-span diaphragms, experienced transverse strain of -29 $\mu$ε. In the case of applying TPT force of 40 kip to three diaphragms, the transverse strain developed at point A-9, located over the post-tensioned mid-span diaphragm, was -69 $\mu$ε, compared to the the transverse strain of 20 $\mu$ε at point A-5 located over the none post-tensioned quarter-span diaphragm. This is shown in Figure 6.2-8.

The variation of the transverse strains along the longitudinal axis defined by the shear-keys demonstrates that the effect of post-tensioning is localized and indicates uneven distribution of the TPT forces to the deck slab and shear-keys. Furthermore, the transverse strain distribution variations along the shear-keys are similar, yet reduced, when the TPT force is reduced from 80
kip to 40 or 20 kip. This observation is common for the different investigated arrangements of post-tensioned diaphragms. While the majority of the cases demonstrate localized strain levels below the recommended levels by AASHTO LRFD (2004), only the case of five diaphragms with 80 kip TPT force was able to maintain the recommended strain level along most of the length of the bridge model.

The effect of the skew angle of the bridge on the transverse strain distribution led to a concentration of the transverse strains near the acute corner of the bridge model. For the same arrangement of TPT forces, the points located near the acute corner (shear-key C-C) experienced higher transverse strains than those points located near the obtuse corner (shear-key A-A) for the same transverse axis. In addition, the points located at the mid-width of the bridge model (shear-key B-B) experienced transverse strains lower than those developed at the points located near the acute corner and higher than those developed at the points located near the obtuse corner. For example, the transverse strains developed due to TPT force of 80 kip applied to three diaphragms for points A-1, B-1, and C-1, located on the same transverse axis “1”, were -34, -82, and -113 µε, respectively, as shown in Figure 6.2-7. However, the difference in the transverse strains between the points located at the same transverse axis reduced towards mid-span. For example, the transverse strains developed due to TPT force of 80 kip applied to three diaphragms for points A-9, B-9, and C-9, located on the same transverse axis “9”, were -95, -102, and -112 µε, respectively, as shown in Figure 6.2-7.

The results also show a systematic difference in the transverse strains developed at the end-diaphragm and the mid-span diaphragm. The transverse strains developed near the end of the bridge model were lower than the strains near the mid-span. The lower strain levels at the end span are likely due to the increased volume of concrete of the end-blocks, which reduced the strains. As an example, the transverse strains, due to TPT force of 40 kip applied to five diaphragms for points C-9 and C-1, which are located at mid-span and end-diaphragms, were -143 and -90 µε, respectively (see Figure 6.2-2). Similar behavior was observed when TPT force of 20 kip was applied to three diaphragms where the transverse strains for points A-9 and A-1 located at mid-span and end-diaphragms were -44 and -21 µε, respectively (see Figure 6.2-9).

AASHTO LRFD Bridge Design Specification (2004) recommends that the minimum transverse prestress developed due to TPT forces should not be less than 250 psi. However, the
AASHTO LRFD (2004) did not provide any additional information about this limit. It did not clearly specify the region or area at which the limit should be maintained. By assuming full integrity on the behavior of the bridge model in the transverse direction, it is expected that the lowest transverse compressive strains due to the application of the TPT forces at diaphragms locations should develop at the top surface of the deck slab (extreme top fiber). This is because of the eccentric resultant of TPT force with respect to the total depth of the entire bridge model (box-beams and deck slab). In addition, the bearing plate is placed directly on the side face of the box-beams and there was no direct contact between the bearing plate and the deck slab. Therefore, the effect of the TPT forces would transfer from the box-beams to the deck slab only through the composite action. Since the longitudinal cracks in side-by-side box-beam bridges are usually observed on the deck slab between the adjacent box-beams, the strain gages mounted on the deck slab of the bridge model above the shear-keys were used to evaluate the AASHTO LRFD (2004) limit.

In order to compare the measured transverse strains with the AASHTO LRFD (2004) limit, the AASHTO LRFD (2004) limit was converted to an equivalent limit in micro-strains. The determination of the equivalent strains was based on the compressive strength of deck slab concrete during the test (5,000 psi), the American Concrete Institute (ACI 318-05) equation for predicting the modulus of elasticity of concrete, and the elasticity theory (Hooke’s Law) as follows.

AASHTO LRFD (2004) minimum prestress limit ($f$)

$$f = -0.25 \text{ ksi}$$

Compressive strength of deck slab concrete during the test ($f'_{c}$)

$$f'_{c} = 5,000 \text{ psi}$$

ACI 318-05 equation for determining the modulus of elasticity for concrete ($E_{c}$) [Section 8.5.1]

$$E_{c} = 57,000\sqrt{f'_{c}} = 57,000\sqrt{5,000} = 4,030 \times 10^{3} \text{ psi} = 4,030 \text{ ksi}$$

(6.1)

Using the elastic theory for determining the equivalent strains ($\varepsilon$)

$$\varepsilon = \frac{f}{E_{c}} = \frac{-0.25}{4,030} = -62 \mu \varepsilon$$

(6.2)
The aforementioned strain limit presents the equivalent strain of the AASHTO LRFD (2004) prestress limit. The equivalent limit was superimposed on all the transverse strain distribution curves (Figure 6.2-1 through Figure 6.2-20). Figure 6.2-1 through Figure 6.2-9 show that at least eight points experienced transverse strains below the equivalent AASHTO LRFD (2004) limit for each arrangement of TPT forces. Furthermore, all the twenty seven points experienced transverse strains less than that of the equivalent AASHTO LRFD (2004) limit in some arrangements of TPT forces, such as applying TPT force of 20 kip at five, four, and three diaphragms (Figures 6.2-7, 6.2-8, and 6.2-9, respectively).

6.2.3 Effect of Number of Diaphragms

The effects of the number of diaphragms on the transverse strain distributions are evaluated in this section. Different number of diaphragms resulted in different spacing between the transverse diaphragms. Nevertheless, in all cases TPT forces were applied to the end-diaphragms. In the case of three diaphragms, the spacing between the three diaphragms was 15 ft. In the case of four diaphragms, two different spacings were introduced; 7.5 ft between the end and the intermediate-diaphragms and 15 ft between the intermediate-diaphragms. In the case of five diaphragms, the spacing was 7.5 ft. Furthermore, the bridge systems are not physically identical when the number of diaphragms is changed. Therefore, these results are compared using histograms.

The rate of increase in the transverse strains due to the corresponding increase in the TPT forces reflects the sensitivity of the transverse strains developed at any point to the applied TPT forces. It is based on the assumed linear relation between the TPT forces and the corresponding transverse strains for a given bridge system. Table 6.2-1 shows the rate of increase in the transverse strains due to different number of diaphragms.
Figure 6.2-1 Transverse strain distribution due to applying 80 kip at five diaphragms.
Figure 6.2-2 Transverse strain distribution due to applying 40 kip at five diaphragms.
Figure 6.2-3 Transverse strain distribution due to applying 20 kip at five diaphragms.
Figure 6.2-4 Transverse strain distribution due to applying 80 kip at four diaphragms.
Figure 6.2-5 Transverse strain distribution due to applying 40 kip at four diaphragms.
Figure 6.2-6 Transverse strain distribution due to applying 20 kip at four diaphragms.
Figure 6.2-7 Transverse strain distribution due to applying 80 kip at three diaphragms.
Figure 6.2-8 Transverse strain distribution due to applying 40 kip at three diaphragms.
Transverse strain gages

Figure 6.2-9 Transverse strain distribution due to applying 20 kip at three diaphragms.
The transverse strain is evaluated along the longitudinal axis and the results along shear-key A-A is presented in Figure 6.2-10 through Figure 6.2-12. Figure 6.2-10 shows the histogram of the transverse strains at five points (A-1, A-3, A-5, A-7, and A-9), located above the same shear-key A-A, due to TPT forces of 80 kip applied to different numbers of diaphragms. The horizontal axis represents different number of diaphragms and the vertical axis represents the corresponding transverse strains. As discussed in Section 6.2.1, the transverse strain decreases as the distance from the post-tensioned diaphragm increases. Point A-9, located at the mid-span diaphragm, experienced high transverse strains of -95 \( \mu \varepsilon \) due to TPT force of 80 kip at three diaphragms as the mid-span diaphragm was post-tensioned. This strain increased to -192 \( \mu \varepsilon \) for the bridge system with five diaphragms. However, as the mid-span diaphragm was not post-tensioned in the case of four diaphragms a low tensile transverse strain of 6 \( \mu \varepsilon \) was observed.

The results also show that as the distance from the point evaluated to the post-tensioned diaphragms increases, the transverse strains decreases towards zero and consequently, transverse tensile strains may indeed develop in the deck slab. Point A-5, located at the quarter-span diaphragm, experienced low transverse tensile strains of 23 \( \mu \varepsilon \) due to TPT force of 80 kip applied to three diaphragms and transverse compressive strains of -27 and -87 \( \mu \varepsilon \) when the same TPT force was applied at four and five diaphragms, as shown in Figure 6.2-10.

Point A-1, located at the end-diaphragm, experienced high transverse strains of -34, -48, and -61 \( \mu \varepsilon \) due to TPT force of 80 kip applied to three, four, and five diaphragms, respectively. It is observed that increasing the number of diaphragms increases proportionally the transverse strains as the total TPT forces acting on the bridge model increases. The results in Figure 6.2-10 also show that points A-3 and A-7 located between the post-tensioned diaphragms only experience small compressive stresses in all cases of number of post-tensioned diaphragms and level of TPT forces. Strain levels at these points ranged from 1 to about 29 \( \mu \varepsilon \). The variation of transverse strains on the deck slab along shear-key A-A was also observed for the other two shear-keys defined as B-B and C-C.

In general, it could be noted that the rate of increase of transverse strains depends on the location of the point with respect to the post-tensioned diaphragms. The points located on the post-tensioned diaphragms experienced higher rates of increase in the transverse strains than the points located between the diaphragms or on the none post-tensioned diaphragms. Furthermore, points located 22.5 in. away from the centerline of the diaphragms (on the
transverse axes “2”, “4”, “6”, and “8”) experienced higher rates of increase than those points located at the mid-distance between the diaphragms on the transverse axes “3” and “7” (45 in. away from the centerline of the diaphragms). Therefore, it could be concluded that closer the point to the diaphragm, the more sensitive the transverse strain would be to any change of force at the diaphragm.

For example, the rates of increase in the transverse strain at points A-9, A-5, and A-1 due to TPT forces applied to five diaphragms were -2.4, -1.08, and -0.76 \( \mu \varepsilon/\text{kip} \), respectively. However, the rates of increase at points A-7 and A-3 were -0.36 and -0.03 \( \mu \varepsilon/\text{kip} \), respectively for the same number of diaphragms. The rates of increase for points A-9, A-5, and A-1 were higher than those for points A-7 and A-3 because of the location of the points with respect to the post-tensioned diaphragms. Points A-9, A-5, and A-1 were located directly over the post-tensioned diaphragms, while points A-7 and A-3 were located at the mid-distance between the post-tensioned diaphragms. In addition, point A-8 experienced higher rate of increase in the transverse strains (-0.69 \( \mu \varepsilon/\text{kip} \)) due to different levels of TPT forces at three diaphragms than point A-7, which experienced rate of increase in the transverse strains of -0.11 \( \mu \varepsilon/\text{kip} \) due to different levels of TPT forces at the same number of diaphragms.

In the case of four diaphragms, points A-5 and A-1 experienced high rates of increase in the transverse strains (-0.34 and -0.61 \( \mu \varepsilon/\text{kip} \), respectively) compared to point A-9 (0.07 \( \mu \varepsilon/\text{kip} \)) since points A-5 and A-1 were located over post-tensioned diaphragms, while point A-9 was located over the mid-span diaphragm (none post-tensioned diaphragm). The close to zero positive rate of increase at point A-9 indicates the insignificant effect of TPT forces at that location. Similar behavior was observed at the shear-keys B-B and C-C.
Table 6.2-1 Rate of increase in the transverse strains in $\mu\varepsilon$/kip.

<table>
<thead>
<tr>
<th>Number of Diaphragms</th>
<th>Longitudinal Axis</th>
<th>Transverse Axis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>Five</td>
<td>A</td>
<td>-0.76</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>-1.49</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>-1.88</td>
</tr>
<tr>
<td>Four</td>
<td>A</td>
<td>-0.61</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>-1.31</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>-1.84</td>
</tr>
<tr>
<td>Three</td>
<td>A</td>
<td>-0.43</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>-1.03</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>-1.41</td>
</tr>
</tbody>
</table>
Figure 6.2-10 Transverse strains for shear-key A-A due to TPT force of 80 kip at different number of diaphragms.
Figure 6.2-11 Transverse strains for shear-key A-A due to TPT force of 40 kip at different number of diaphragms.
Figure 6.2-12 Transverse strains for shear-key A-A due to TPT force of 20 kip at different number of diaphragms.
6.2.4 Effect of Levels of TPT Forces

The effect of TPT forces on the transverse strains along the same longitudinal axis as well as locally is demonstrated in Figure 6.2-13 through Figure 6.2-15 and Figure 6.2-16 through Figure 6.2-20, respectively. Figure 6.10 shows the transverse strains developed at five points (A-1, A-3, A-5, A-7, and A-9), located at the same shear-key A-A, due to different levels of TPT forces applied to five diaphragms. Points A-1, A-5, and A-9 were located directly over the mid-span, quarter-span, and end-diaphragms, respectively. Points A-3 and A-7 were located at the mid-distance between the diaphragms. It was observed that by increasing the TPT forces, the transverse strains increased proportionally at all points located at the post-tensioned diaphragms. For example, point A-9 experienced transverse strains of -192, -94, and -56 µε due to TPT forces of 80, 40, and 20 kip, respectively.

Similar behavior was observed at point A-5, which experienced transverse strains of -87, -25, and -10 µε as a result of applying TPT forces of 80, 40, and 20 kip at five diaphragms, respectively. Similar behavior was observed at the cases of four and three diaphragms for the shear-key A-A as well as the points located at the shear-keys B-B and C-C. For example, point B-1 experienced transverse strains of -105, -48, and -27 µε due to TPT forces of 80, 40, and 20 kip applied to four diaphragms, respectively. Also, point C-9 experienced transverse strains of -102, -91, and -44 µε due to TPT forces of 80, 40, and 20 kip applied to three diaphragms, respectively.

On the other hand, the points located at the mid-distance between the diaphragms, such as those belong to the transverse axes “3” and “7”, experienced insignificant change in the transverse strains due to different levels of TPT forces. As shown in Figure 6.2-13, point A-3 experienced transverse strains of -2, 0.2, and 2 µε due to TPT forces of 80, 40, and 20 kip applied to five diaphragms. Also, point A-7 experienced transverse strains of -29, -28, and -15 µε due to TPT forces of 80, 40, and 20 kip applied to five diaphragms. Figures 6.2-14 and 6.2-15 indicate similar behavior for points A-3 and A-7 with four and three diaphragms cases. In addition, the same behavior was observed at the points located between the diaphragms in the other shear-keys B-B and C-C. The low transverse strains at these points located between the diaphragms confirmed the uneven distribution of the transverse stresses and that these strains were lower than the recommended levels as set by AASHTO LRFD (2004).
Figure 6.2-16 through Figure 6.2-20 show the levels of TPT force on the vertical axis and the corresponding transverse strains on the horizontal axis for points B-1, B-3, B-5, B-7, and B-9, which are located on the same shear-key B-B. Each plot presents three lines corresponding to three different numbers of diaphragms. It is observed that the relation between the TPT forces and the corresponding transverse strains is linear for any selected number of diaphragms. The linear relation between the level of the TPT forces and the corresponding transverse strains reflects the elastic behavior of the concrete when TPT forces were applied.

The average transverse strains and the standard deviation were calculated for each arrangement of TPT forces based on the average of the twenty seven strain readings. The results are as shown in Table 6.2-2. It is expected that increasing the level of TPT will increase the average transverse strains. However, the results also demonstrated that increasing the level of TPT also increased the variation of strains along the top of the deck slab at the shear-key locations. The effect of different levels of TPT on the average transverse strains as well as the standard deviation was evaluated for each number of diaphragms. From Table 6.2-2, it is evident that for the same number of diaphragms, increasing the level of TPT force resulted in a corresponding increase in the average transverse strain. In the case of five diaphragms, the average transverse strains were -95, -52, and -26 µε due to TPT forces of 80, 40, and 20 kip, respectively. Same behavior was noted in the case of four and three diaphragms. In addition, increasing the TPT forces resulted in corresponding increase in the standard deviation for each number of diaphragm case. For instance, the standard deviations in five diaphragm case are 60, 36, and 19 due to TPT forces of 80, 40, and 20 kip, respectively. Similar behavior was observed at the four and three diaphragm cases.

It was observed that only the case of applying 80 kip at five diaphragms resulted in average transverse strain of -95 µε that is higher than the equivalent AASHTO LRFD (2004) limit of -62 µε. All the other TPT arrangements resulted in average transverse strains lower than the equivalent AASHTO LRFD (2004) limit. In other words, providing adequate spacing between the diaphragms, together with the application of 80 kip/diaphragm, could lead to a better distribution of the TPT forces along the longitudinal direction, and consequently satisfies the equivalent AASHTO LRFD (2004) limit.
Table 6.2-2 Average transverse strains.

<table>
<thead>
<tr>
<th>Number of Diaphragms</th>
<th>Level of TPT Force, kip</th>
<th>Average Transverse Strains, με</th>
<th>Standard Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Five</td>
<td>80</td>
<td>-95</td>
<td>60</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>-52</td>
<td>36</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>-26</td>
<td>19</td>
</tr>
<tr>
<td>Four</td>
<td>80</td>
<td>-37</td>
<td>39</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>-16</td>
<td>19</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>-7</td>
<td>11</td>
</tr>
<tr>
<td>Three</td>
<td>80</td>
<td>-35</td>
<td>41</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>-20</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>-9</td>
<td>14</td>
</tr>
</tbody>
</table>
Figure 6.2-13 Transverse strains for shear-key A-A due to applying different levels of TPT force at five diaphragms.
Figure 6.2-14 Transverse strains for shear-key A-A due to applying different levels of TPT force at four diaphragms.
Figure 6.2-15 Transverse strains for shear-key A-A due to applying different levels of TPT force at three diaphragms.
Figure 6.2-16 Transverse strains at point B-9 due to different levels of TPT force.
Figure 6.2-17 Transverse strains at point B-7 due to different levels of TPT force.
Figure 6.2-18 Transverse strains at point B-5 due to different levels of TPT force.
Figure 6.2-19 Transverse strains at point B-3 due to different levels of TPT force.
Figure 6.2-20 Transverse strains at point B-1 due to different levels of TPT force.
6.3 Load Distribution

6.3.1 Introduction

The fundamental function of the TPT system in side-by-side box-beam bridges is to tie the beams together to act as one unit, which when functioning properly provides the mechanism for the uniform distribution of service loads across the width of the bridge model. The behavior of the box-beam bridge model was investigated using the vertical deflections of the box-beams across the width of the bridge model. The deflections as described in Chapter 5 were measured by linear motion transducers installed at the mid-width of each individual box-beam. The load distribution test was based on the deflections induced by the application of a single point load of 15 kip subjected to the four individual beams. The single point load was applied over a cylindrical steel block of diameter, 10 in., which resulted in a contact stress of 191 psi on each loaded beam. The selection of the 15 kip single point load was to avoid major flexural cracking, as the theoretical first cracking load was 16 kip.

The load distribution test was conducted first in the presence of TPT force of 80 kip applied to all five diaphragms. Subsequently, the TPT forces per diaphragm were decreased to 40 and 20 kip. Similarly, the number of diaphragms were also reduced to four, three, and finally to the case of “No TPT” (where no TPT force was applied to any diaphragm). The corresponding deflections for all box-beams were recorded for each loading case investigated. The load distribution test was conducted in the three stages of the bridge model. These stages represent 1) uncracked deck slab, 2) cracked deck slab and shear-keys, and 3) repaired deck slab and shear-key due to the replacement of exterior box-beam. In this section, the results are presented and discussed from the three stages of the bridge model in terms of the load-deflection response of the bridge model and the load fraction ($LF$) across the width of the bridge model. Furthermore, the percentage improvement ($PI$) in the load distribution due to the investigated TPT arrangements is also presented. Definitions of the $PI$ as well as the $LF$ are also provided.

6.3.2 Derivation of Load Fraction and Percentage Improvement

The percentage of the applied load carried by each box-beam in the bridge model was evaluated using a deflection-based load fraction ($LF$) as defined by Grace et al. (2000) and Klaiber et al. (2001). It quantifies the ability of the bridge model to distribute an externally applied load across the width of the bridge model. The $LF$ presented herein was based on the
deflection experienced by the loaded box-beam relative to the summation of deflections of all four box-beams, as shown by equation 6.3. A smaller difference in $LF$ of the loaded box-beam relative to the unloaded box-beams confirms effective distribution of the applied load across the bridge width. Figure 6.3-1 shows a typical bridge model with no initial deflection prior to the application of any external load. However, the deflection between the loaded box-beam and the far exterior box-beam increased during loading as illustrated in Figure 6.3-2.

The $PI$ was introduced to determine the effect of the TPT arrangement on the bridge model’s ability to respond as a monolithic structural unit. The effect of the TPT arrangements is expected to have only minor influence on the bridge model behavior as long as the bridge deck and shear-keys remained uncracked. Whereas the TPT arrangement is expected to have significant effect on the bridge model behavior in the case of cracked deck slab and shear-keys.

Figure 6.3-1 Bridge model in the uncracked stage with no external Load.

Figure 6.3-2 Deflection of the bridge model when loaded.

The load fraction is defined as follows.
\[ LF = \frac{\Delta_i}{\sum_{i=1}^{4} \Delta_i} \times 100 \]  

(6.3)

where \( i \) is the box-beam for which the load fraction is calculated, and \( \Delta_i \) is the deflection of the \( i^{th} \) beam. From the \( LF \), the \( PI \) is evaluated as follows.

\[ PI = \frac{(LF_w - LF_{TPT})}{(LF_w)} \times 100 \]  

(6.4)

where \( LF_w \) = the load fraction of the loaded beam in the absence of TPT force (i.e. no TPT force applied to any diaphragm), \( LF_{TPT} \) = the load fraction of the loaded beam when TPT arrangement is present.

### 6.3.3 Cracking of Deck Slab

After conducting the strain and load distribution tests on the uncracked bridge model, the bridge model was partially cracked at the shear-key joints between the adjacent box-beams, as mentioned earlier in Chapter 5. In general, the cracking of the shear-keys and deck slab was conducted by applying a vertical load at the mid-span of a single box-beam while partially restraining the other three box-beams from possible displacement and transverse rotation. Two types of cracks were initiated at this stage: shear-key cracks and other cracks. The shear-key cracks propagated as longitudinal cracks over the three shear-key locations, while the other cracks developed on the top surface of deck slab between the shear-keys especially at the locations of the applied load. In addition, cracking of the deck slab led to the development of transverse flexural cracks at the mid-span for the individual box-beams. The methodology for developing the longitudinal cracks is described in details in Appendix B. Figures 6.3-3 and Figure 6.3-4 show the shear-key cracks and other cracks developed on the top surface of the deck slab.
Figure 6.3-3 Cracks of deck slab at mid-span.
Figure 6.3-4 Cracks pattern of deck slab.

* 23"/0.005" = crack length / maximum crack width

Shear-key cracks
Other cracks
6.3.4 Effect of Number of Diaphragms

6.3.4.1 Introduction

In this section, the effect of the number of transverse diaphragms on the behavior of the bridge model in terms of load distribution is evaluated through three different stages: uncracked, cracked, and repaired stage. The cases of three, four, and five diaphragms were investigated. For simplicity, the uncracked stage of the bridge model is designated as \( UC \), while the cracked and the repaired stages are denoted as \( C \) and \( R \), respectively. Comparative analyses are conducted on how different number of transverse diaphragms influenced the deflection, load fraction, and percentage improvement of the bridge model.

6.3.4.2 Deflection and Load Fraction of the Bridge Model

As expected, the loaded beam experienced the largest deflection among the four beams in all cases of loading. In addition, the loaded exterior box-beams experienced larger deflections than the loaded interior box-beams. For instance, in the cracked stage, when the exterior beam B-1 was loaded with the application of TPT force of 80 kip to all five diaphragms, the deflections for the box-beams B-1, B-2, B-3, and B-4 were 0.30, 0.29, 0.28, and 0.26 in., respectively. On the other hand, when the interior beam B-2 was loaded under the same conditions, the deflections recorded were 0.29, 0.29, 0.28, and 0.27 in., respectively.

In the uncracked stage, the differential deflection between the adjacent box-beams were insignificant regardless of the number of diaphragms, as shown in Figure 6.3-5 through Figure 6.3-8. For example, the deflections of the beams, when a TPT force of 80 kip was applied to three transverse diaphragms and beam B-1 was loaded, were 0.26, 0.24, 0.22, and 0.20 in. for beams B-1, B-2, B-3, and B-4, respectively. Very similar deflections were observed when four or five diaphragms were used and the deflections in those cases were 0.25, 0.23, 0.22, and 0.21 in., and 0.24, 0.23, 0.21, and 0.21 in., respectively.

As anticipated, when the deck slab was partially cracked, larger deflections were observed compared to the uncracked stage, as shown in Figure 6.3-5 through Figure 6.3-8. For instance, in the cracked stage when beam B-1 was loaded and the TPT force of 80 kip was applied to three diaphragms, the deflections observed were 0.31, 0.30, 0.29, and 0.26 in. for beams B-1, B-2, B-3, and B-4, respectively. In the uncracked stage with the same TPT arrangement, the deflections observed were 0.26, 0.24, 0.22, and 0.20 in.
The case of “No TPT” in the cracked stage resulted in a larger differential deflection between the adjacent box-beams compared to other cases of number of diaphragms (three, four, and five). For example, the deflections for beams B-1, B-2, B-3, and B-4, when beam B-1 was loaded, were 0.41, 0.37, 0.26, and 0.21 in. in the case of “No TPT”. However, the deflections observed were 0.31, 0.30, 0.29, and 0.26 in. for the case of three diaphragms, 0.32, 0.30, 0.29, and 0.25 in. for the case of four diaphragms, and 0.30, 0.29, 0.28, and 0.26 in. for the case of five diaphragms. The larger deflections observed in the cracked stage was partly due to the longitudinal cracks developed in deck slab and shear-keys as well as the flexueral cracks developed on the beams during the cracking process. This lead to the reduced stiffness of the bridge model.

Figure 6.3-5 Load-deflection response of the bridge model while loading beam B-1.
Figure 6.3-6 Load-deflection response of the bridge model while loading beam B-2.

Figure 6.3-7 Load-deflection response of the bridge model while loading B-3.
As mentioned earlier, the repaired stage was simulated by replacing an exterior beam (B-4 with B-5) and repairing the adjacent shear-key and deck slab. The goal was to determine if the structural function could be restored. When the assumed damaged box-beam B-4 was replaced in the repaired stage, lower deflections were observed for beams B-3 and B-5 compared to the cracked stage. However, beams B-1 and B-2 experienced similar deflections in both stages, as shown in Figure 6.3-9 through Figure 6.3-12. Considering these results, it can be stated that the stiffness of the bridge model was restored. For example, the deflections observed when a TPT force of 80 kip was applied at three diaphragms and beam B-1 was loaded were 0.33, 0.30, 0.29, and 0.27 in. for B-1, B-2, B-3, and B-5, respectively. When compared to the cracked stage, the deflections recorded were 0.31, 0.30, 0.29, and 0.26 in. Furthermore, by increasing the number of transverse diaphragms to four, the deflections observed in the repaired stage were 0.34, 0.31, 0.29, and 0.27 in. for B-1, B-2, B-3, and B-5, respectively, while the deflections observed in the cracked stage were 0.32, 0.30, 0.29, and 0.25 in.

The $LF$ was determined to evaluate the fraction of the load carried by each box-beam. When the $LF$ of the box-beams was evaluated, it was generally noted that, the portion of the
load carried by the loaded box-beams is always greater than the other box-beams. For example, when the beam B-1 was loaded in the cracked stage with a TPT force of 80 kip applied to all five diaphragms, the LF observed were 27%, 26%, 24%, and 23% for beams B-1, B-2, B-3, and B-4, respectively. This trend in the LF was consistent regardless of the number of transverse diaphragms and the stage of the bridge model.

As the number of transverse diaphragms increased, the LF of the loaded beams decreased while the LF for the other beams increased in the cracked and repaired stages. For instance, when beam B-1 was loaded with TPT force of 80 kip applied to four diaphragms in the cracked stage, the LF for beams B-1, B-2, B-3, and B-4 were 27.5%, 25.9% 24.9%, and 21.4%, respectively. On the other hand, when the number of diaphragms was increased to five with the same level of TPT force, the LF noted were 26.7%, 26%, 24.4%, and 23.1%. Similarly, in the repaired stage, the LF for beams B-1, B-2, B-3, and B-5 were 28.1%, 25.6% 24.0%, and 21.8%, respectively, in the case of four diaphragms. Moreover, when the number of diaphragms was increased to five with the same level of TPT force, the LF observed were 27.7%, 25.0%, 24.1%, and 23.0%. This means that by increasing the number of diaphragms, the distribution of the applied load across the bridge width is improved. On the contrary, similar LF were observed in the uncracked stage irrespective of the number of diaphragms. The LF observed, when loading beam B-1 with TPT force of 80 kip, were 27.5%, 25.2%, 24.1%, and 23.2% for four diaphragms and 27.6%, 25.7%, 23.4%, and 23.2% for five diaphragms, respectively.
Figure 6.3-9 Load-deflection response for beam B-1 after repairs.

Figure 6.3-10 Load-deflection response for B-2 after repairs.
Figure 6.3-11 Load-deflection response for B-3 after repairs.

Figure 6.3-12 Load-deflection response for B-4 after repairs.
6.3.4.3 Improvement in the Behavior of the Bridge Model

This section presents the effect of the number of transverse diaphragms on improving the behavior of the bridge model in terms of $PI$ in each stage of the bridge model. Varying the number of transverse diaphragms in the uncracked stage did not have significant impact on the $PI$ of the bridge model with a maximum of 4% improvement observed when loading beam B-1. For instance, the $PI$ in the uncracked stage when a TPT force of 80 kip was applied and B-1 was loaded were 1.2%, 3.1%, and 3.0% for three, four, and five diaphragms, respectively. The bridge model did not experience any definite trend in the $PI$ at the uncracked stage, as shown in Figure 6.3-13 through Figure 6.3-16.

As the bridge deck experienced longitudinal cracks in the cracked stage, the effect of the number of transverse diaphragms was more pronounced as shown by an increase in the $PI$ relative to the uncracked stage. For instance, the $PI$ observed for three, four, and five diaphragms when loading beam B-1 in the cracked stage with TPT force of 80 kip were 17.4%, 15.6%, and 18.2%, respectively.

The rehabilitation of the “assumed damaged” beam B-4 and the shear-key resulted in an improvement in the stiffness of the bridge model and consequently led to a marginal reduction in the $PI$ for beams B-3 and B-5 in the repaired stage relative to the cracked stage, as shown in Figure 6.3-13 through Figure 6.3-16. For instance, the $PI$ observed for three, four, and five diaphragms when loading beam B-4 in the cracked stage with TPT force of 80 kip were 17.4%, 16.5%, and 21.85%, respectively. However, the $PI$ observed for three, four, and five diaphragms when loading beam B-5 in the repaired stage with the same TPT force were 12.8%, 11.8%, and 14.4%, respectively. The reason is that the “No TPT” condition in the repaired stage performed better than the “NO TPT” condition of the cracked stage (see Figure 6.3-9 to Figure 6.3-12).

The $PI$ for the interior box-beams was relatively lower than the exterior box-beams, as shown in Figure 6.3-13 and Figure 6.3-16. For example, the $PI$, when TPT forces of 20 and 40 kip are applied to all five diaphragms and beam B-1 was loaded in the cracked stage, were 14.3% and 15.2%, respectively. However, when beam B-2 was similarly loaded, the $PI$ evaluated were only 9.5% and 10.2% for 20 and 40 kip, respectively. Furthermore, the $PI$ observed for beam B-2 was lower than that of beam B-3, as shown in Figure 6.3-14 and Figure 6.3-15. As an example, the $PI$, when TPT forces of 20 and 40 kip were applied to all five
diaphragms in the cracked stage and beam B-2 was loaded, were 9.5% and 10.2%, respectively. However, when B-3 was loaded, the \( PI \) obtained were 14.5% and 15.7% for 20 and 40 kip, respectively. This was because the intensity of the cracks initiated during the cracked stage along the shear-key A-A (25% of the entire bridge span) was less than the intensity of the cracks initiated along the shear-key C-C (75% of the entire bridge span).

It was observed that the case of five diaphragms outperformed the three diaphragm case in terms of effectively distributing the applied vertical load especially in the cracked and the repaired stages of the bridge model. For instance, the \( PI \), when loading beam B-2 with TPT force of 80 kip applied to three and five diaphragms, were 10.7% and 12.8% in the cracked stage while the \( PI \) were 9.4% and 12.3% in the repaired stage (see Figure 6.3-14). This was because the distance between the diaphragms increased from 7.5 ft in the case of five diaphragms to 15 ft in the case of three diaphragms. In addition, the case of four diaphragms resulted in a lower improvement in terms of load distribution compared to the three and five diaphragms. For example, the \( PI \) decreased to 7.3% and 7.8% in the cracked and repaired stages, respectively.

\[
PI = \frac{LF_T - LF_W}{LF_W} \times 100
\]

\( PI \) = Percentage of improvement

\( LF_T \) = Load fraction in the presence of TPT

\( LF_W \) = Load fraction in the absence of TPT

Figure 6.3-13 Effect of number of diaphragms on \( PI \) for loading beam B-1.
$PI = \frac{(LF_w - LF_{TPT})}{(LF_w)} \times 100$

- $PI$ = Percentage of improvement
- $LF_{TPT}$ = Load fraction in the presence of TPT
- $LF_w$ = Load fraction in the absence of TPT

**Figure 6.3-14** Effect of number of diaphragms on $PI$ for loading beam B-2.

**Figure 6.3-15** Effect of number of diaphragms on $PI$ for loading beam B-3.
6.3.5 Effect of Levels of TPT Forces

6.3.5.1 Introduction

In this section, the effect of varying the level of TPT forces on the behavior of bridge model in the transverse direction is discussed. Three different levels of TPT forces were selected: 20, 40, and 80 kip while the number of transverse diaphragms remained constant. For each level of TPT force, the load distribution test was conducted and the corresponding deflections of the four beams were recorded. Similar to the previous section, the effect of level of TPT forces was evaluated in all the three stages of the bridge model; uncracked, cracked, and repaired stages. Comparative analyses were made on how different levels of TPT force affect the deflection, load fraction, and percentage improvement of the bridge model.

6.3.5.2 Deflection and Load Fraction of the Bridge Model

The deflections of the loaded interior beams were lower than the loaded exterior beam irrespective to the level of TPT force and the stage of the bridge model, as shown in Figure 6.3-18, Figure 6.3-19, Figure 6.3-22, and Figure 6.3-23. The deflections observed, when TPT
forces of 0, 20, 40, and 80 kip applied to five diaphragms, were 0.39, 0.34, 0.32, and 0.31 in. for the loaded exterior beam B-1 and 0.36, 0.32, 0.31, and 0.29 in. for the loaded interior beam B-2.

Similar deflections were observed for the loaded beam in the uncracked stage with different levels of TPT forces. The deflections recorded for the loaded beam B-1 were 0.27, 0.25, 0.25, and 0.24 in. for TPT force of 0, 20, 40, and 80 kip, respectively. Unlike the uncracked stage, the deflection of the loaded box-beam decreased as the level of the TPT forces increased in the cracked and the repaired stages of the bridge model. For example, the deflections observed for the loaded beam B-1 in the cracked stage were 0.41, 0.34, 0.33, and 0.30 in. corresponding to TPT force of 0, 20, 40, and 80 kip applied to all five diaphragms. Similarly, the deflections observed with the same beam loaded in the repaired stage were 0.39, 0.34, 0.32, and 0.31 in.

In the cracked and repaired stages, the deflections of the loaded beam B-1 was evaluated with different levels of TPT forces applied to three diaphragms. It was observed that the deflection reduced significantly as the level of TPT force increased from zero to 20 kip. However, only a slight reduction in deflections was observed as the level of TPT force increased from 20 to 80 kip. For example, the deflections in the cracked stage for beam B-1 with TPT forces of 0, 20, 40, and 80 kip applied to three diaphragms were 0.41, 0.35, 0.32, and 0.31 in., respectively. Also, the deflections in the repaired stage for beam B-1 with TPT forces of 0, 20, 40, and 80 kip applied to three diaphragms were 0.39, 0.34, 0.33, and 0.33 in., respectively. Relatively high difference in deflections was observed between 0 and 20 kip (0.06 in.) compared to 20 and 40 kip (0.03 in.) or 40 and 80 kip (0.01 in.). This shows that a significant improvement in the load distribution was observed as the level of TPT force increased from zero to 20 kip. However, the effect of level of TPT forces on the improvement in the load distribution of the bridge model was insignificant beyond TPT force of 40 kip.
Figure 6.3-17 Load-deflection response for beam B-1 at different level TPT force.

Figure 6.3-18 Load-deflection response for beam B-2 at different level TPT force.
Figure 6.3-19 Load-deflection response for beam B-3 at different level TPT force.

Figure 6.3-20 Load-deflection response for beam B-4 at different level TPT force.
When the $LF$ was evaluated in each stage of the bridge model, it was generally observed that, applying different levels of TPT forces improved the distribution of the applied load. It was observed that the $LF$ carried by the loaded beam decreased, while the $LF$ carried by the other beams increased with increasing the level of TPT force. For instance, in the cracked stage, when beam B-1 was loaded in the absence of TPT forces, the $LF$ of beam B-1 was 32.6% while that for beam B-4 was 16.8%. However, as the level of TPT force was increased to 20 kip at five diaphragms, the $LF$ of beam B-1 decreased to 27.9% and that for beam B-4 increased to 22.1%. Similar trend in behavior was observed in the repaired stage when the levels of TPT forces were increased in a similar manner.

The $LF$ for the loaded beam B-4/B-5 was monitored in the three stages of the bridge model with different levels of TPT forces applied to five diaphragms. In the case of No TPT, the $LF$ for the loaded beam B-4 increased from 27.8% in the uncracked stage to 33.1% in the cracked stage due to the development of the longitudinal cracks in the deck slab and the shear-keys. However, the $LF$ for the loaded beam B-5 with no TPT forces reduced to 31.0% in the repaired stage because of the repair of longitudinal deck crack and shear-key. Furthermore, similar $LF$ was observed for beam B-4/B-5 in the three stages as the level of TPT forces was increased. For instance, in the case of five diaphragms, the $LF$ observed for the loaded beam B-4/B-5 in the uncracked, cracked, and repaired stages were 27.7%, 27.0%, and 27.2% for TPT force of 20 kip and 27.4%, 27.3%, and 27.6% for TPT force of 40 kip. This clearly shows that, applying TPT forces improved the load distribution among the adjacent beams significantly.
Figure 6.3-21 Load-deflection response for beam B-1 after repairs at different TPT forces.

Figure 6.3-22 Load-deflection response for beam B-2 after repairs at different TPT forces.
Figure 6.3-23 Load-deflection response for beam B-3 after repairs at different TPT forces.

Figure 6.3-24 Load-deflection response for beam B-4 after repairs at different TPT forces.
6.3.5.3 Improvement in the Behavior of the Bridge Model

The $PI$ was also determined to evaluate the effect of varying the level of TPT forces on the bridge model behavior. In the uncracked stage, $PI$ was not affected by the level of the TPT force, as shown in Figure 6.3-25 through Figure 6.3-28. For instance, when beam B-1 was loaded, the $PI$ for beam B-1 were 2.6%, 0.8%, and 3.0% corresponding to the TPT forces of 20, 40, and 80 kip applied to all five diaphragms, respectively.

In the cracked stage, the $PI$ increased significantly relative to the uncracked stage. For example, in the cracked stage, when beam B-1 was loaded and the level of TPT forces were increased, the $PI$ for beam B-1 were 14.3%, 15.2%, and 18.2% for TPT forces of 20, 40, and 80 kip applied to all five diaphragms, respectively.

No significant changes in the $PI$ were observed in the repaired stage for the beams B-1 and B-2, as shown in Figure 6.3-25 through Figure 6.3-28. However, slight reduction was observed in the $PI$ when beam B-3 and B-4/5 were loaded, as shown in Figure 6.3-27 and Figure 6.3-28. For example, in the cracked stage when beam B-3 was loaded, the $PI$ was 14.5%, 15.7%, and 17.0% for TPT forces of 20, 40, and 80 kip at all five diaphragms, respectively. However, in the repaired stage and with the same TPT arrangements, the $PI$ reduced slightly to 10.7%, 12.1%, and 12.7%.

Furthermore, it was noted that in the cracked and the repaired stages, the $PI$ increased accordingly when the TPT increased from zero to 20 kip. However, insignificant improvement was observed in $PI$ as the TPT force was increased to 40 and 80 kip, as shown in Figure 6.3-25 to 6.3-28. For example, the $PI$ observed for beam B-3 due to TPT forces of 20, 40, and 80 kip were 14.3%, 15.1%, and 16.2% in the cracked stage and 8.6%, 10.7%, and 11.4% in the repaired stage. Based on this, it could be deduced that, applying TPT force of 40 kip is adequate to hold the adjacent beams to act as a one unit when the bridge model was subjected to the vertical load although it did not satisfy the AASHTO LRFD (2004) prestress limit of 250 psi.
\[ PI = \left( \frac{LF_{TPT} - LF_{W}}{LF_{W}} \right) \times 100 \]

- **\( PI \)** = Percentage of improvement
- **\( LF_{TPT} \)** = Load fraction in the presence of TPT
- **\( LF_{W} \)** = Load fraction in the absence of TPT

**Figure 6.3-25** Effect of varying levels of TPT forces on \( PI \) for beam B-1.

**Figure 6.3-26** Effect of varying levels of TPT forces on \( PI \) for beam B-2.
Figure 6.3.27 Effect of varying levels of TPT forces on $PI$ for beam B-3.

Figure 6.3.28 Effect of varying levels of TPT forces on $PI$ for beam B-4/5.
6.4 Ultimate Load Test

6.4.1 Introduction

The function of TPT arrangement is to ensure that the bridge model acts as a monolithic system where the differential deflection between the adjacent box-beams are limited. The efficiency of the unbonded TPT arrangement has been demonstrated and discussed in the previous sections for the case of service and repair conditions. This section presents results on the evaluation of the efficiency of the unbonded TPT arrangement during failure as induced by an eccentric load. The bridge model was subjected to TPT forces of 80 kip at each of the five diaphragms during this test.

This prestressed bridge model reinforced with steel was designed according to the American Concrete Institute (ACI 318-05) as an under-reinforced bridge. Therefore, the three expected failure modes for the bridge model under the eccentric load were: 1) ductile flexural failure at maximum moment, 2) brittle shear failure between adjacent box-beams due to shear failure of the unbonded TPT CFCC, and 3) a combination between the first two failure modes.

The eccentric load was applied at the mid-span of the interior box-beam B-2. The load was applied using a symmetrical two-point loading frame. The distance between the two loading points was 7.5 ft, which was the same as the spacing between the diaphragms. Therefore, the two points of the loading frame were located exactly at the mid-distance between the mid-span and quarter-span diaphragms. The selected locations for the loading points experienced the lowest effect of TPT forces along the span as noticed in the transverse strain distribution test results. The load was applied on beam B-2 as the deck cracks were located evenly over the shear-keys whereas the deck cracking had propagated into the center of box-beam B-3. It was determined that the risk of local shear punchure failure would be less likely for box-beam B-2.

The eccentric load was applied in five different loading/unloading cycles with maximum load of 20, 40, 60, 80, and 100 kip before reaching the ultimate load. Throughout the test, the rates of loading and unloading were kept at an average of 15 kip/min. The following sections present details of the observed failure mode of the bridge model, load-deflection response for the four beams, concrete compressive strains at the extreme top fiber of the bridge deck, and the effect of the applied load on the TPT forces at the various diaphragms. Finally, comparison between the theoretical and experimental ultimate loads is also presented.
6.4.2 Load-Deflection Response

The deflection for each box-beam was measured using linear motion transducer attached at the mid-width of each beam. Figure 6.4-1 through Figure 6.4-4 show the load-deflection responses of the four beams for the various loading and unloading cycles. Prior to conducting the ultimate cycle, residual deflections of 2.07, 2.02, 2.02, and 1.97 in. were observed at the four box-beams B-1, B-2, B-3 and B-5, respectively. The residual deflections for the four box-beams were almost the same with a slight increase towards beam B-1. The bridge model acted as a single plate subjected to eccentric vertical load, which cause the plate to deflect with a slight rotation towards the location of the applied load.

During the ultimate load cycle, the deflection of the bridge model was proportional to the applied load up to about 100 kip, the deflection afterwards increased rapidly under constant vertical load. This was an indication of yielding of the bottom steel reinforcements. The deflections at 100 kip were 8.51, 8.30, 8.24, and 8.11 in. for beams B-1, B-2, B-3 and B-5, respectively. The ultimate load for the bridge model was 104 kip and the corresponding deflections were 10.76, 10.55, 10.46, and 10.34 in. The post-peak behavior of beam B-3 showed that the concrete around the deflection sensor crushed at level of 83 kip of the applied load. However, similar behavior of the deflections for beams B-1, B-2 and B-5 was observed till the complete failure of the bridge model. The maximum observed deflection for the bridge model was 27.07 in. and occurred at beam B-2.

Figure 6.4-5 shows the deflections of the four beams at different levels of the applied load during the last load cycle leading to failure. The deflections are shown for the four beams illustrating the transverse behavior of the bridge model at loads of 0, 20, 40, 60, 80, 100, 104, and 83 kip. As mentioned earlier, the deflections for the four box-beams at zero load were residual deflections from the previous loading and unloading cycles. It was also noticed that all box-beams deflected as one unit regardless of the level of the applied load, coupled with a slight rotation due to the eccentric load up to the ultimate load 104 kip.

6.4.3 Failure Mode

The expected failure modes for the bridge model were: flexural failure, transverse shear failure or a combination between them. The observed failure mode was a typical flexural ductile failure with localized failure at the location of maximum moment. The failure started by the
initiation and propagation of flexural tensile cracks at the soffit of the beams around the mid-span. This was followed by the yielding of the bottom reinforcements, resulting in increasing deformation of the bridge model at constant load. The yielding of the bottom reinforcement was followed by crushing of the deck slab concrete across the entire width of the bridge model near the mid-span, as shown in Figure 6.4-6. No rupture of the bottom reinforcement was observed. However, buckling of the top reinforcement was observed at the location of the concrete crushing. Failure of the shear-keys was observed at the mid-span only. Propagation of the existing longitudinal cracks in the deck slab was not observed.
Figure 6.4-1 Load-deflection response of box-beam B-1 for different loading/unloading cycles.
Figure 6.4-2 Load-deflection response of box-beam B-2 for different loading/unloading cycles.
Figure 6.4 Load-deflection response of box-beam B-3 for different loading/unloading cycles.
Figure 6.4-4 Load-deflection response of box-beam B-5 for different loading/unloading cycle.
Figure 6.4-5 Deflection of the bridge model at different levels of applied load.
6.4.4 Concrete Compressive Strains

The concrete compressive strains at the top surface of the deck slab were monitored during the ultimate load cycle. Two longitudinal strain gages were attached to the deck slab and mounted exactly at mid-span directly over the box-beams B-2 and B-5, as shown in Figure 6.4-7. Figure 6.4-8 shows the variation of the compressive strains as a function of the applied load. Prior to the application of the load, both strain gages experienced residual strains resulted from the previous loading/unloading cycles (i.e. -379 and -226 µε for beams B-2 and B-5, respectively). The compressive strains were proportional to the applied load during the ultimate load cycle up to 100 kip, after which the compressive strains increased at a constant level of applied load that corresponds to the yielding plateau experienced during the ductile failure of the bridge model. The final crushing of deck slab concrete occurred close to the location of the strain gages. The maximum compressive strains observed by the strain gages were -2,582 and -2,062 µε for beams B-2 and B-5, respectively. These strain levels approaches the ACI recommendation for the maximum compressive strain at the concrete crushing of -3,000 µε.
6.4.5 Variation of TPT Forces

Variations in the TPT forces were expected during loading as the transverse rotation would cause an elongation at the level of the TPT strands. As noted earlier, the bridge model had five transverse diaphragms located at equal spacing of 7.5 ft. The layout for the transverse diaphragms and the CFCC is shown in Figure 6.4-9. For instance, the top TPT CFCC located at the end diaphragm is designated as E(1)-T, the bottom strand at the mid-span diaphragm is designated as M(3)-B. The TPT forces were monitored continuously during the ultimate load cycle for six of the CFCC. Two strands were located at the end-diaphragm 1), two strands were located at the quarter-span diaphragms 2), and two strands were located at the mid-span diaphragms 3). The six CFCC represented one-half of the bridge model.

A slight increase in the TPT forces was observed prior to conducting the ultimate load test at each CFCC due to the residual deformations of the bridge model resulted from the previous loading/unloading cycles. It was observed that the strands located at the mid-span diaphragm experienced the largest increase in the TPT forces. Furthermore, the bottom strands experienced a larger increase in the TPT forces compared to the top strands for the same diaphragm. For example, an increase in the TPT forces observed at the top and bottom strands located at end-diaphragms (1) were 0.04 kip and 0.07 kip, respectively. For the quarter-span diaphragms, the increase in the TPT forces observed at the top and bottom strands were 0.26 and 0.48 kip. Finally, for the mid-span diaphragm, the increase in the TPT forces observed at the top and bottom strands were 0.77 and 1.25 kip, respectively.

Figure 6.4-10 shows the variation of the TPT forces during the ultimate load cycle. The strands located at the end-diaphragms did not experience significant change in the TPT forces. The strands located at the mid-span diaphragms experienced the larger increase in the TPT forces. The largest increase of the TPT force was observed in the bottom strands located at the mid-span diaphragms, which increased by about 2.3 kip. This increase of about 9% above the TPT level corresponds to an elongation of 8%. However, the strand was still only stressed to 54% of its capacity. The top strand located at the mid-span diaphragms experienced an increase in the TPT force of 7% above the TPT level. The top and bottom strands located at the quarter-span diaphragms experienced increase in the TPT force of 1% and 2% of the TPT level, respectively. The strands located at the end diaphragms did not experience any increase in the TPT forces.
6.4.6 Theoretical and Experimental Load-Carrying Capacities

The load-deflection response of the bridge model show that the bottom reinforcement started to yield at a load of 100 kip and the ultimate load-carrying capacity was 104 kip. The deflections for all beams were similar at the ultimate load and the entire bridge was acting as a one unit when subjected to eccentric load. The theoretical load-carrying capacity was 88 kip and it was based on the strain compatibility approach (Appendix C). The failure of the bridge model in the theoretical calculations was assumed to be a ductile flexural failure for an under reinforced bridge model with a uniform distribution of the applied load in the transverse direction. The theoretical load-carrying capacity showed good agreement with the experimental capacity.

6.4.7 Ductility

The ductility is the ability of a structure to sustain load during increasing deformation. In this study, an energy based approach is used to quantify ductility (Grace et al., 1998). The energy based approach is based on the the area under the load-deflection curve where load and deflection are determined at the same points and in the same direction. A typical prestressed concrete bridge would experience two types of energies prior to the failure load. The first is the elastic energy \( E_{\text{elastic}} \) and second is the inelastic energy \( E_{\text{inelastic}} \). During increased loading, the structure creates new crack surfaces which create irreversible deformation upon unloading. Under several loading/unloading cycles of the vertical load, the structure stores the inelastic energy as newly formed cracks and releases the elastic energy. In the post-peak area, additional cracking and yielding cause additional inelastic energy which is denoted as \( E_{\text{in(adx)}} \). The total energy is the summation of the elastic and the inelastic energies. The ductility, based on these energies, can be expressed by the ductility index. The ductility index is the energy ratio between the absorbed inelastic energy and the total energy, as shown in equation 6.5 (Grace and Abdel-Sayed, 1998). The failure mode of a bridge could be classified as a ductile failure if the ductility index is greater than 75%. However, if the ductility index lies between 70 and 74%, the failure would be semi-ductile. The brittle failure is classified for those bridges which achieve ductility index less than 69%.

\[
\text{Ductility Index} = \frac{E_{\text{inelastic}}}{E_{\text{Total}}} = \frac{E_{\text{inelastic}}}{E_{\text{elastic}} + E_{\text{inelastic}} + E_{\text{in(adx)}}}
\]  

(6.5)
For the bridge model, several loading/unloading cycles were applied to separate the elastic and the inelastic energies. The loading cycles had maximum values of 20, 40, 60, 80, and 100 kip. The energy ratio was calculated based on the load-deflection curve of the loaded beam B-2, as shown in Figure 6.4-11. The inelastic energy observed by the bridge model prior to the ultimate load was 549 kip-in. and the elastic energy was 300 kip-in. The elastic energy was estimated by calculating the area under load-deflection curve for the unloading path of the fifth cycle 100 kip. The additional inelastic energy observed was 738 kip-in. and was estimated by calculating the area under the load-deflection curve subsequent to the ultimate load. The bridge model experienced ductility index of 81.1%, which lies within the range of the ductile failure.

### 6.4.8 Evaluation of TPT System

As mentioned earlier, the main objective of the ultimate load test was to evaluate the efficiency of the TPT system in distributing the eccentric load in the transverse direction. The TPT system consisted of five transverse diaphragms equally spaced each 7.5 ft. Each diaphragm contained two unbonded TPT CFCC placed in oval-shape ducts and prestressed with TPT force of 40 kip/strand. The following observations were made during the ultimate load test.

1. The failure mode for the bridge model was ductile flexural failure initiated by yielding of the bottom reinforcement and followed by crushing of concrete as a typical tension controlled prestressed concrete bridge.

2. No significant differential deflection was observed between the adjacent box-beams at any load levels prior to the ultimate load. The bridge was slightly rotating as a single unit subjected to an eccentric vertical load.

3. Furthermore, no new longitudinal cracks were initiated or propagated on the top surface of the deck slab between the adjacent box-beams.

4. No shear rupture of the TPT CFCC was observed during the ultimate load test and none of the transverse diaphragms failed even when complete failure occurred. The TPT strands located at the mid-span were able to transfer the vertical load in the transverse direction despite the failure of the shear-key and the deck slab at the mid-span.

5. The theoretical and experimental load-carrying capacity show good agreement. The theoretical load-carrying capacity was based on ductile flexural failure of the bridge
model. In addition, the energy ratio shows that the failure of the bridge model was ductile flexural failure.

Based on the previous observations, it could be noted that the TPT system was successful in distributing the eccentric vertical load to the adjacent box-beams, even after the failure of the shear-keys and the deck slab at the mid-span, which enabled the bridge model to act as a composite unit and not as individual beam elements.

(a) Strain gage over beam B-2  (b) Strain gage over beam B-5

Figure 6.4-7 Crushing of concrete near the location of the strain gages.
Figure 6.4-8 Concrete compressive strains at the top surface of deck slab during ultimate load cycle.
Figure 6.4-9 Layout of transverse diaphragms and CFCC.
Figure 6.4-10 Variation of TPT forces for the ultimate load cycle.
Figure 6.4-11 Elastic and inelastic energy bands of the bridge model during the ultimate load cycles.

- $E_{\text{elastic}} = 300 \text{ kip-in.}$
- $E_{\text{inelastic}} = 549 \text{ kip-in.}$
- $E_{\text{in (add)}} = 738 \text{ kip-in.}$
- Ductility Index $= \frac{E_{\text{inelastic}}}{E_{\text{Total}}} = 81.1\%$
CHAPTER 7: CONCLUSIONS AND RECOMMENDATIONS

Experimental and numerical studies were conducted to evaluate the number of transverse diaphragms and the level of transverse post-tensioning (TPT) forces in order to control the development of the longitudinal cracks in side-by-side box-beam bridges. This chapter is divided into two separate sections. The first section presents the conclusions and recommendations obtained from the experimental investigation while the second section provides the conclusions and the recommendations derived from the numerical investigations.

7.1 Findings and Conclusions from the Experimental Study

7.1.1 Introduction

A half-scale 30° skew bridge model was constructed, instrumented, and tested at the Center for Innovative Materials and Research (CIMR) at the Lawrence Technological University, Southfield, to investigate the effect of the number of the transverse diaphragms and the level of the TPT forces on the behavior of side-by-side box-beam bridge where unbonded TPT CFCC were used. Oval-shape ducts were used at each transverse diaphragm location to accommodate the unbonded TPT CFCC and to allow for the differential camber intentionally developed between the adjacent beams. Throughout the test program, three different types of transverse diaphragms configurations were used (three, four, and five diaphragms). Similarly, three different levels of TPT forces of 20 kip, 40 kip, and 80 kip were also used for each evaluated diaphragm configuration. The major tests conducted on the bridge model were 1) the transverse strain distribution test, 2) load distribution test, and 3) ultimate load test. The transverse strain distribution test primarily involved examining the transverse strains that developed on the top surface of the deck slab along the shear-key locations. Likewise, the load distribution test was also conducted to evaluate the efficiency of the different TPT arrangements in distributing the applied load across the width of bridge model. The purpose of the ultimate load test was to evaluate the response of the unbonded TPT CFCC in distributing the vertically applied load across the width of the bridge model.
7.1.2 Conclusions

7.1.2.1 General

The performance of the TPT system used in the experimental program was evaluated and the following conclusions are derived.

1. The use of unbonded CFCC is suitable for TPT application in side-by-side box-beam bridge system. The combination of TPT CFCC and ungrouted transverse diaphragms facilitates the replacement of damaged box-beam and allows restoration of the stiffness of the box-beam bridge. Furthermore, the flexural capacity of the repaired bridge exceeded the design flexural capacity (about 15%).

2. The use of oval-shape ducts to accommodate the unbonded TPT CFCC can overcome the construction problem observed when using the traditional circular ducts and the misalignment problem that arises as a results of differential camber between the adjacent box-beams.

7.1.2.2 Strain Distribution Test

Based on the results obtained from the strain distribution test conducted on the bridge model, the following conclusions are drawn.

1. Increasing the levels of TPT forces proportionally increases the transverse strains for all points located along the post-tensioned diaphragms. The linear relation between the level of the TPT forces and the corresponding transverse strains reflects the elastic behavior of the concrete when the TPT forces are applied.

2. Increasing the number of transverse diaphragms has insignificant influence on the transverse strain developed in the region between the diaphragms. The low transverse strains at these locations indicate a non-uniform distribution of the TPT forces along the entire span of the bridge model.

3. Regardless of the number of transverse diaphragms, and the levels of TPT forces experienced in this bridge model, the resulting transverse strain did not satisfy the transverse stresses limit of 250 psi in the deck slab recommended by the AASHTO LFRD (2004). However, 70% of the deck slab meets the criterion when TPT force of 80 kip was applied at five diaphragms.
7.1.2.3 Load Distribution Test

Based on the load distribution test, the following conclusions are drawn.

1. The level of TPT forces and the number of transverse diaphragms have no effect on the transverse behavior of the bridge model if the deck slab is uncracked.

2. The effect of TPT arrangements on the load distribution was significant when the deck slab experienced longitudinal cracks at the shear-key locations. Through the evaluation of the percentage improvement, it was observed that increasing the level of TPT forces improved the behavior of the bridge model.

3. The replacement of the damaged beam and reconstruction of the deck slab as well as the shear-key enhanced the behavior of the bridge model by reducing the deflection of the loaded new box-beam B-5 as well as the adjacent interior box-beam B-3. The effect increased with larger number of transverse diaphragms and higher levels of TPT forces.

4. The case of five diaphragms outperformed the case of three diaphragms in terms of percentage improvement of the load distribution for the cracked deck slab and the repaired stages. This was because, the spacing between the diaphragms increased from 7.5 ft (in case of five diaphragms) to 15 ft (in case of three diaphragms).

5. As the level of TPT force increased from zero to 20 kip, the behavior of the bridge model improved accordingly. However, at higher levels of the TPT force above 40 kip, the improvement in the behavior was reduced. This means that applying TPT force of 40 kip is adequate to hold the adjacent beams to act as a one unit when the bridge model was subjected to the vertical load although it did not satisfy the AASHTO LRFD (2004) prestress limit of 250 psi.

7.1.2.4 Ultimate Load Test

The failure mode experienced by the bridge model was ductile flexural failure. The failure started by yielding of the bottom steel reinforcements followed by crushing of deck slab concrete across the width of the bridge model. Failure of the shear-key and the crushing of the concrete were observed particularly at the mid-span of the bridge model. Prior and during the failure of the bridge model, no differential deflection was observed between the adjacent box-
beams, which substantiate the adequacy of the unbonded TPT CFCC along with the full-depth shear-keys in maintaining a composite behavior throughout the loading history.

None of the unbonded TPT CFCC ruptured and none of the transverse diaphragms failed during or after the complete failure of the bridge model. In addition, the TPT forces in the CFCC increased due to the deflection of the bridge model. The CFCC located at the mid-span of the bridge model experienced significant increase in the TPT forces (9% above the TPT level); whereas the TPT force in the CFCC located at the quarter-span diaphragms increased marginally (2% above the TPT level). Negligible increase in the TPT forces was observed in the strands located at the end-diaphragms. The used TPT system, coupled with the deck slab, jointly distributed the applied eccentric load in the transverse direction until the total collapse of the bridge model.

7.1.3 Recommendations for Field Implementations and Future Research

The results of this research project have yielded the following recommendations for field implementation and further consideration.

1. The oval-shape ducts, placed at the location of the transverse diaphragms, are successful in accommodating the differential camber between the adjacent beams in side-by-side box-beam bridges. The ducts were also useful to accommodate the two TPT strands at each diaphragm.

2. The unbonded CFCC were effective in the application of the TPT forces as they did not experience any rupture or permanent deformation during all tests conducted on the bridge model. Therefore, the CFCC can be implemented in the TPT system for side-by-side box-beam bridges since they have high corrosion resistance and higher tensile capacity than that of the conventional steel. In addition, the threaded anchorage system of the CFCC allowed for partial or full release of the TPT forces as needed.

The following issues require further investigations for a comprehensive evaluation of the box-beam bridge behavior:

1. It is recommended to examine the sensitivity of the vertical load to different locations along the entire span of the bridge model, since the applied load at the mid-span was sensitive to presence of the mid-span diaphragm.
2. In order to compare the load fraction determined from the AASHTO LRFD (2004) live load distribution factor equations, it is recommended to construct a bridge model of two lanes width and a larger number of beams (more than four) in order to examine the applicability range of these equations. Hence, the load distribution factors based on the experimental investigation cannot be directly compared with the predicted values obtained from the AASHTO equations.

3. The effect of different skew angles on the load and strain distribution of the bridge model was not investigated in this study. Thus, it is recommended that bridge models with different skew angles should be investigated.

7.2 Findings and Conclusions from the Numerical Study

7.2.1 Introduction

The numerical study presented herein was carried out in the Civil Engineering Computer Aided Design Laboratory at Lawrence Technological University and included generating and analyzing thirty-three extensive FE models for side-by-side box-beam bridges with spans of 50, 62, 100, and 124 ft and widths of 24, 45, 58, 70, and 78 ft. The FE study resulted in new recommendations for MDOT Specifications for the TPT arrangements for side-by-side box-beam bridges. The current specifications for TPT arrangements (MDOT Bridge Design Guide, 2006) were shown to be inadequate. All the FE bridge models with TPT arrangements following the current specifications experienced longitudinal deck slab cracking over the shear-key joints. It should be noted that the conclusions of this study is independent of the configuration of the shear-key and the TPT force was assumed to be applied in two stages: before and after pouring the deck slab. A summary of the results and recommendations are presented as follows:

1. With current layout of the TPT arrangements within the box-beams in side-by-side box-beam bridges, the AASHTO LRFD (2004) recommendation of 250 psi as minimum transverse prestress required throughout longitudinal joints in side-by-side box-beam bridges is impractical and unreachable. The stress level can be attained at the diaphragms. However, it can not be attained between the diaphragms.
2. Contrary to the findings of some previous studies (i.e. El-Remaily et al., 1996), live loads are not the major cause for developing longitudinal cracks in the slab.

3. The contributing factor of the positive temperature gradient, which had not been investigated in any of the previous studies, is the major contributing factor in the development of the longitudinal cracks in deck slabs of side-by-side box-beam bridges. When the bridge is exposed to a positive temperature gradient, the deck slab experiences compressive stresses in the longitudinal and transverse directions. However, the cracks do not develop because of the thermal stresses in the deck slab; instead, the cracks develop because of the relative movements of the box-beams when exposed to positive temperature gradient. The box-beams tend to separate from each other. This separation generates high tensile stresses in the deck slab bottom surface and consequently develops longitudinal cracks. This phenomenon is in agreement with observations made by Miller et al. (1999) when they conducted their experimental program to investigate the shear-key cracking problem in side-by-side box-beams bridges.

4. The positive temperature gradient combined with live loads develop longitudinal cracks in the deck slab if an adequate TPT arrangement is not provided. Thus, the design baseline for TPT arrangement is to ensure that maximum principal stresses developed in the deck slab when subjected to positive temperature gradient with live loads and Impact allowances are less than the cracking strength of the concrete.

5. Impact allowance of 75% recommended by AASHTO LRFD (2004) for the design of the transverse deck joints is appropriate for the design of the longitudinal joints as well.

6. The AASHTO LRFD (2004) should be updated to include the proper effect of the temperature gradient in the design of side-by-side box-beam bridge components.

**7.2.2 Adequate Number of Diaphragms**

Based on the analysis of 24 ft wide side-by-side box-beam bridge models of spans of 50, 62, 100, and 124 ft, using box-beams of widths of 36 and 48 in., the following conclusions are drawn:
1. The number of diaphragms specified in the MDOT Bridge Design Guide (2006) is not adequate to eliminate the longitudinal cracks in the deck slab over the shear-key joints.

2. In order to eliminate the longitudinal deck slab cracks in bridges constructed using side-by-side box-beams of a width of 48 in., the recommended number of diaphragms should be provided based on the bridge span as presented in Figure 4.2-41 Adequate number of diaphragms for bridges constructed using 48 in. wide beams. The Figure presents the minimum number of diaphragms required to prevent the longitudinal cracks. However, the research team recommends using one additional diaphragm over the minimum number to account for any unpredictable loads or concrete deterioration. The diaphragms should be equally spaced along the entire bridge span.

3. In the case of 36 in. wide side-by-side box-beams bridges, the number of diaphragms should be provided according to Figure 4.2-42. This Figure presents the minimum number of diaphragms required to prevent the longitudinal cracks.

4. Each number of diaphragms obtained from Figure 4.2-41 or Figure 4.2-42 is applicable for all bridges having the corresponding span regardless of their widths. However, to adjust for the effect of increasing the bridge width on the transverse tensile stresses, the TPT force should be adjusted as presented in the following section.

7.2.3 Appropriate TPT Force

The FE models for bridges of widths of 24, 45, 58, 70, and 78 ft and spans of 50, 62, and 100 ft were analyzed to investigate the influence of the bridge width on the proper level of the TPT force. The models in this phase of the analysis were provided with the minimum number of diaphragms as concluded from the preceding analysis phase using 24 ft wide bridges. Three classes of the deck slabs were investigated in the analysis; 1) deteriorated deck slabs (concrete of strength of 3,000 psi), 2) recently constructed deck slabs (concrete strength of 4,000 psi), and 3) higher strength deck slabs (concrete strength of 5,000 psi). The results can be summarized as follows:

2. The TPT force per diaphragm is independent of the bridge span because the number of diaphragms increases with increasing the bridge span.

3. The analysis of the models composed of 48 in. wide box-beams and those composed of 36 in. wide box-beams yielded the same results for the appropriate level of the TPT force. In other words, the appropriate level of TPT force for a certain bridge width is independent of the width of an individual box-beam.

4. The appropriate level of the TPT force increases when increasing the bridge width and slightly decreases when increasing the concrete strength of the deck slab. The appropriate level of TPT force can be obtained from Figure 4.3-13.
REFERENCES


ACI Committee (2005). “Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05)”, American Concrete Institute, Farmington Hills, Michigan.


APPENDIX A: TRANSFER LENGTHS, PRE-TENSIONING FORCES, AND DIFFERENTIAL CAMBER MONITORING

A.1 Introduction
The results related to the construction phase are presented in Appendix A. The appendix is divided into three sections. The first section involves a continuous monitoring of the pre-tensioning forces in the four box-beams. The second demonstrates the transfer length of the pre-tensioning forces at the end-blocks. The continuous monitoring of the camber for the four individual box-beams is presented in the third section.

A.2 Monitoring Pre-Tensioning Forces During the Curing of Concrete
The pre-tensioning forces in the steel strands were monitored continuously during the first three days of curing the concrete to study the effect of the curing on the levels of pre-tensioning forces. As mentioned in Chapter 5, the two exterior box-beams had average pre-tensioning force of 20.6 kip/strand while, the interior box-beams had average pre-tensioning force of 25 kip/strand. The levels of pre-tensioning forces were measured for each pre-tensioning strand using the center-hole load cells attached to the dead-end between the face of the bulkhead and the sleeve. Figure A-1 shows the variation in the average pre-tensioning forces during the curing period of the four box-beams. For all strands, the levels of pre-tensioning forces decreased gradually to reach their minimum level after approximately 17 hours from the end of casting the box-beams. The reduction of the pre-tensioning forces could be attributed to the thermal effects of the hydration process, which led to a volumetric expansion in the concrete and elongation of the steel strands. The elongated steel strands counteracted the pressure on the load cells by reducing the reactive forces on the bulkhead. However, the thermal and shrinkage contraction of the concrete during the subsequent hydration led to close to complete recovery of the pre-tensioning forces. The difference between the average pre-tensioning forces for the exterior and the interior box-beams was 3.91 kip at the age of three days.

The pre-tensioning forces recorded at the time of casting (initial levels of pre-tensioning forces) for the exterior beams were 21.69, 20.26, and 20.47 kip for box-beam B-1 and 20.83, 20.64, and 21.57 kip for box-beam B-4. The minimum levels of pre-tensioning forces obtained after 17 hours were 20.33, 18.65, and 19.05 kip for box-beam B-1 with a reduction of 6%, 8%,
and 7% of the initial levels of the pre-tensioning forces. Also, the minimum levels of pre-tensioning forces obtained after 17 hours were 19.01, 19.39, and 20.24 kip for box-beam B-4 with a reduction of approximately 9%, 6%, and 6% of the initial levels of the pre-tensioning forces. The levels of pre-tensioning forces after three days were 21.64, 19.98, and 20.15 kip for box-beam B-1, which represent 100%, 99%, and 98% of the initial levels of the pre-tensioning forces. In addition, the levels of pre-tensioning forces after three days were 20.61, 20.51, and 21.39 kip for box-beam B-4, which represent approximately 99%, 99%, and 100% of the initial levels of the pre-tensioning forces.

For the interior box-beams, the initial pre-tensioning forces were 25.78, 24.82, and 25.02 kip for box-beam B-2, and 25.07, 24.43, and 25.37 kip for box-beam B-3. The minimum levels of pre-tensioning forces after 17 hours were 24.20, 23.50, and 23.00 kip for box-beam B-2 with a reduction of approximately 6%, 5%, and 8% of the initial levels of the pre-tensioning forces. Also, the minimum levels of pre-tensioning forces after 17 hours were 23.65, 22.86, and 23.90 kip for box-beam B-3 with a reduction of 6%, 6% and 6% of the initial levels of the pre-tensioning forces. The levels of pre-tensioning forces after three days were 25.46, 24.43, and 24.33 kip for box-beam B-2, which represent approximately 99%, 98%, and 97% of the initial levels of the pre-tensioning forces. In addition, the levels of pre-tensioning forces after three days were 24.76, 24.02, and 24.74 kip for box-beam B-3, which represent 99%, 98%, and 98% of the initial levels of the pre-tensioning forces.
Figure A-1 Pre-tensioning forces in the steel strands for the exterior and the interior box-beams.
A.3 Transfer Length Evaluation

Transfer length \( (L_t) \) is defined as the length over which the applied prestressing force is transferred completely to the concrete (Mahmoud et al., 1999). In order to measure the transfer lengths in this bridge model, the demountable mechanical (DEMEC) points were installed on one side of the live and the dead-ends of the interior beams B-2 and B-3 along the location of the pre-tensioning strands, as shown in Figure A-2. Measurements between the successive DEMEC points were taken before and after the applied pre-tensioning force were released by cutting the prestressed steel strands. The computed compressive strains were plotted against the bridge span, as shown in Figure B-3 and B-4.

![Installed DEMEC points](image)

Figure A-2 Installation of DEMEC point at the live-end of beam B-2.

The transfer lengths were determined by using the 95% average maximum strain method which involved plotting a horizontal line at 95% of the maximum strain exerted by the prestressed strands on the concrete. The point of intercept between the horizontal line and the plotted measured strains represents the transfer length. Furthermore, the measured transfer lengths were compared with the theoretical using the following empirical equations.

\[
L_t = \left( \frac{f_{se}}{3,000} \right) d_b \text{ in inches (ACI 318, 2005)} \tag{A.1}
\]

where \( f_{se} \) is the prestress at transfer (psi) and \( d_b \) is the nominal diameter of the steel strand (in.).
\[ L_t = \frac{480 \, d_b}{\sqrt{f'_{ci}}} \]  
\text{in mm (Zou, 2003)} \quad (A.2)

\[ L_t = \frac{f_{pi} \, d_b}{\alpha_t \, f'_{ci}}^{0.67} \]  
\text{in mm (Mahmoud et al., 1999)} \quad (A.3)

where \( f_{pi} \) is the prestress at transfer (MPa), \( d_b \) is the nominal diameter of the strand (mm), and \( \alpha_t \) is transfer length coefficient (\( \alpha_t = 2.4 \), for steel strands) and \( f'_{ci} \) is the concrete compressive strength at transfer (MPa).

The transfer lengths at the live and dead-ends of beam B-2 were 20 in. and 21 in., respectively. However, the beam B-3 resulted in a slightly higher transfer length relative to B-2 with 23 in. and 22 in. at the dead and live-ends, respectively, as shown in Figure A-4. A good conformity was observed between the values of transfer length at the live and dead-end for both beams which depicts the fact that the prestressed forces were effectively transferred equally at both ends of the beams.

The average values between the dead and live-ends obtained during the experimental transfer length were compared with the theoretical values, as shown in Table A-1. The transfer length determined by using the equation proposed by American Concrete Institute (ACI-318, 2005) and Zou et al. (2003) overestimated the transfer length by about 20 to 50\%. However, good agreement was obtained with the transfer length predicted by the equation proposed by Mahmoud et al. (1999).
Figure A-3 Transfer length determination for beam B-2.

Figure A-4 Transfer length determination for beam B-3.
Table A-1 Transfer length of pre-tensioning steel strands of the box-beam bridge model.

<table>
<thead>
<tr>
<th>Box-Beam</th>
<th>Average Prestress, ksi</th>
<th>Concrete Strength at Transfer, ksi</th>
<th>Measured Transfer Length, in.</th>
<th>Calculated Transfer Length, in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-2</td>
<td>130</td>
<td>6.1</td>
<td>20</td>
<td>2</td>
</tr>
<tr>
<td>B-3</td>
<td>130</td>
<td>6.1</td>
<td>22</td>
<td>23</td>
</tr>
</tbody>
</table>
A.4 Monitoring of Differential Camber

Camber is the upward deflection that results from applying eccentric pre-tensioning forces below the center of gravity of the cross-section in prestressed concrete beams. Cambers for identical beams casted on the same day with the same conditions may vary according to different factors such as exposure to air and humidity and/or the non-homogenous behavior of the concrete. Long-term camber is affected primarily by relaxation of steel strands, which leads to change in the levels of pre-tensioning forces and/or creep of concrete, which leads to change in deflections due to sustained loads. Unlike long-term camber, short-term camber does not change significantly.

One of the major objectives of the project was to study the adequacy of using oval-shape ducts that accommodates two TPT CFCC to adopt the uneven differential cambers between adjacent box-beams. In order to study that, different levels of pre-tensioning forces were applied to create different levels of cambers among the individual box-beams. In particular, the two exterior box-beams B-1 and B-4 were prestressed with an average pre-tensioning force of 60 kip/beam; while the two interior box-beams B-2 and B-3 were prestressed with an average pre-tensioning force of 75 kip/box-beam. The camber was monitored continuously over a period of 24 days for the individual box-beams and the values are plotted with time, as shown in Figure A-5.

In this study, the initial camber refers to the cambers measured directly after placing the individual box-beams on the supports (the cambers at the beginning of the monitoring period). The final camber refers to the cambers measured after 24 days from placing the individual box-beams on the supports (the cambers at the end of the monitoring period). From the results, the initial cambers developed in the exterior box-beams B-1 and B-4 were 0.23 and 0.16 in., respectively. However, the initial cambers developed for interior box-beams B-2 and B-3 were 0.48 and 0.53 in., respectively. The previous variation in the values of the cambers show clearly that the cambers for identical prestressed concrete beams may vary significantly. The average initial differential camber developed between exterior and interior box-beams was 0.31 in.

A superimposed dead load of 1,000 lb was added on each of the two exterior box-beams for one week in order to increase the average differential camber between the exterior and the interior box-beams. The cambers were monitored and the effect of the applied superimposed
dead loads on the cambers was evaluated. After one week, the superimposed dead loads were removed from the two exterior box-beams and the cambers were monitored further for two weeks.

In order to describe the camber-time relationship in details, seven points in time were selected to define the camber profiles for the exterior box-beams as follows.

1. **Point A**: Initial camber developed for each box-beam due to the effect of pre-tensioning forces and self-weight of the beam.
2. **Point B**: Camber after 0.6 days (15 hours) from the initial camber, just before adding the superimposed dead loads.
3. **Point C**: Camber after 0.66 days (15.84 hours) from the initial camber, immediately after adding the superimposed dead loads.
4. **Point D**: Camber after 6.8 days (163 hours) from the initial camber, just before removing the superimposed dead loads.
5. **Point E**: Camber after 6.9 days (166 hours) from the initial camber, immediately after removing the superimposed dead loads, at which the deflection due to the superimposed dead loads was partially recovered.
6. **Point F**: Camber after 9.2 days (220 hours) and 10.2 days (245 hours) from the initial camber for beams B-1 and B-4, respectively, at which the deflection due to the superimposed dead loads was totally recovered and the variation of the camber became approximately negligible.
7. **Point G**: Cambers after 24 days (576 hours) from the initial camber (final camber), the last point at the end of the monitoring period.

**A.4.1 Exterior Box-Beams B-1 and B-4**

The initial camber for the exterior box-beam B-1 was 0.23 in. Before adding the superimposed dead load, the camber decreased with relatively small rate of 0.005 in./day. The superimposed dead load was removed from the exterior beam B-1 after one week. A rapid decrease in the camber occurred after adding the superimposed dead load at a steep rate of 3.05 in./day. The rate of reduction in camber became approximately twice the initial one (-0.009 in./day). When the superimposed dead load was removed, a rapid increase in camber occurred with rate of 1.635 in./day. Only 60% of the total displacement was recovered immediately after removing
the superimposed dead load. The remaining 40% of the recovered displacement was achieved gradually through a period of 2.3 days (55 hours). The total recovered displacement was 0.113 in. and the total reduction in camber due to adding the superimposed dead loads was 0.128 in. The difference between the reduction in camber and the recovered camber is 0.015 in., which was 6.5% of the initial camber. That can be attributed to the effect of creep of the concrete under sustained superimposed dead load. After recovering all the displacement, the rate of reduction in camber was 0.002 in./day, which was close (slightly lower) to the initial rate of reduction in camber.

When the superimposed dead loads were placed on the exterior beams, the values of the reduced cambers were close (0.015 in. and 0.016 in. for beams Bw1 and Bw4, respectively). It clearly shows that, the reduction in the camber due to the superimposed dead load depended on the added load and/or the period of the addition of the load rather than the levels of the initial cambers.

A.4.2 Interior Beams B-2 and B-3
The camber was monitored for the interior box-beams continuously through 24 days. The interior beams experienced higher initial cambers than the exterior box-beams due to higher pre-tensioning forces. No superimposed dead loads were added on the interior box-beams. Therefore, the rate of change in camber was approximately constant and very small through the period of monitoring the cambers (e.g. -0.002 in./day and 0.001 in./day for beam Bw2 and Bw3, respectively).

A.4.3 Average and Differential Cambers
Figure A-5 shows the average cambers for exterior and interior box-beams. For the exterior box-beams, the average initial camber was 0.195 in., while the final camber after three weeks was 0.107 in. The reduction of the average camber due to creep of concrete under sustained superimposed dead loads and relaxation of steel strands with time was 0.088 in., which represented 45% from the initial camber. On the other hand, the average initial camber for the interior box-beams was 0.505 in., while the average final camber was 0.484 in. The reduction of the average camber for the interior box-beams due to creep of concrete and relaxation of steel was 4.16% only. This obviously shows the effect of creep of concrete under sustained
superimposed dead loads on the camber. The addition of superimposed dead loads on the exterior box-beams for one week increased the average differential camber between the exterior and the interior box-beams by 21.6%.
Initial differential camber at release = 0.31 in.

Differential camber during the addition of superimposed load = 0.49 in.

Final differential camber = 0.38 in.

Figure A-5 Cambers for the exterior and the interior box-beams.
APPENDIX B: CRACKING OF DECK SLAB

B.1 Introduction

The bridge model was partially cracked at the shear-key joints between the adjacent box-beams. Additional structural boundary conditions were utilized to develop the required longitudinal cracks in the deck slab over the shear-keys. In general, these boundary conditions were based on applying a vertical load at the mid-span of one box-beam and partially supporting the other three beams from downward displacement and transverse rotation. This was to ensure that the difference in deflection (differential deflection) between the loaded beam and other box-beams would cause high shear stresses at the shear-key joints, and/or the rotation of the loaded beam would cause transverse moments that lead to the development of high transverse tensile stresses on the top surface of deck slab (extreme top fibers). The shear stresses developed at the shear-key joints caused partial shear failure while the transverse tensile stresses caused longitudinal cracks on the deck slab.

Cracking of the deck slab was carried out in three steps; each step was implemented in cracking one of the three longitudinal shear-keys A-A, B-B, and C-C. Two classes of cracks were initiated at this stage: shear-key cracks and other cracks. The shear-key cracks were the longitudinal cracks developed over the three shear-key locations, while the other cracks were the cracks developed on the top surface of deck slab at the locations of the application of the vertical load. In addition, cracking of the deck slab led to the development of transverse flexural cracks at the mid-span for the individual beams.

B.2 Cracking of Exterior Shear-Key C-C

The shear-key C-C was the first shear-key that was cracked in the bridge model. A vertical load was applied on the exterior box-beam B-4 and the other three box-beams were partially restrained from the vertical displacement and the transverse rotation using top and bottom steel supports, as shown in Figure B-1. A maximum vertical load of 89 kip was applied gradually at beam B-4 and the corresponding deflections for beams B-1, B-2, B-3, and B-4 were -0.17, 0.05, 0.3, and 0.6 in., respectively. Figure B-2 shows the deflections for the four box-beams.
under different levels of the vertical load applied to beam B-3. It was noticed that beams B-4 and B-3 experienced downward displacement as well as transverse rotation, while beam B-2 experienced slight rotation only in the transverse direction. However, beam B-1 experienced upward displacement. This shows that the bridge model was rotating as a rigid structure around beam B-2 (maximum observed deflection for beam B-2 was 0.05 in.).

The shear-key cracks on axis C-C appeared first on the top surface of the deck slab near the supports and at mid-span. By increasing the vertical load, partial debonding between the shear-key and beam B-3 was observed starting from the bottom and propagating up towards the deck slab. The cracks on the top surface of deck slab propagated to cover approximately 75% of the entire length of the bridge and the maximum crack width observed on deck slab over the shear-key C-C during the loading of beam B-4 was 0.02 in. Figures B-3 and B-4 show the deformed shape of the bridge model due to the maximum vertical load and the cracks developed at the end of the shear-key C-C, respectively.

**B.3 Cracking of Interior Shear-Key B-B**

After cracking the exterior shear-key C-C, the interior shear-key B-B was cracked. The shear-key was cracked by applying the vertical load on the interior beam B-3 and partially restraining the other box-beams from the vertical displacement and the transverse rotation, as shown in Figure B-5. The exterior beam B-3 was loaded at the mid-span by a maximum vertical load of 123 kip. The cracks were developed due to the partial debonding between the shear-key and beam B-2. The behavior of the bridge model under different levels of vertical load applied to beam B-3 is shown in Figure B-6. By increasing the vertical load, the cracks propagated towards the top surface of deck slab. The deflections at the maximum applied vertical load (123 kip) for beams B-1, B-2, B-3, and B-4 were 0.06, 0.16, 0.23, and 0.12 in., respectively. The levels of deflection were relatively small compared to the applied vertical load. Figure B-7 shows the deformed shape of the bridge model due to applying the maximum load on beam B-3. The cracks reflected on the deck slab were approximately 60% of the entire length of the bridge model. The maximum crack width observed during loading beam B-3 was 0.013 in. Figure B-8 shows the cracks developed at the end of shear-key B-B.
B.4  Cracking of Exterior Shear-Key A-A

The exterior shear-key A-A was the last shear-key that was partially cracked. Similar procedure adopted in the preceding shear-keys (B-B and C-C) were followed to crack the shear-key A-A by applying the vertical load at beam B-1, and partially restraining the other three box-beams from the vertical displacement and the transverse rotation using top and bottom steel supports, as shown in Figure B-9. The maximum applied vertical load was 80 kip and the corresponding deflections for beams B-1, B-2, B-3, and B-4 were 0.28, 0.15, 0.06, and 0.02 in., respectively (see Figure B-10). The deformed shape of the bridge model under the applied vertical load on beam B-1 is shown in Figure B-11. Only 25% of the entire length of the bridge was cracked at that shear-key and no partial debonding of the shear-key was observed at either ends. The maximum cracks width observed over the shear-key A-A was 0.013 in.
Figure B-1 Loading arrangements for cracking the exterior shear-key C-C.

Figure B-2 Deflection of bridge model due to applying vertical load at box-beam B-4.
Figure B-3 Bridge deformed shape due to applying vertical load at beam B-4.

Figure B-4 Cracks of exterior shear-key C-C.
Figure B-5 Loading arrangements for cracking the exterior shear-key B-B.

Figure B-6 Deflection of bridge model due to applying vertical load at box-beam B-3.
Figure B-7 Bridge deformed shape due to applying vertical load at beam B-4.

Figure B-8 Cracks of the interior shear-key B-B.
Figure B-9 Loading arrangements for cracking the exterior shear-key A-A.

Figure B-10 Deflection of bridge model due to applying vertical load at box-beam B-1.
Figure B-11 Bridge deformed shape due to applying vertical load at beam B-1.
APPENDIX C: FLEXURAL DESIGN OF THE BRIDGE MODEL

Introduction

The bridge model was designed as under-reinforced structure with tension controlling the failure. The design was based on a uniform distribution of the vertical load in the transverse direction. The designed bridge model consisted of:

1. Four 30° skew box-beams, 18 in. wide and 11 in. deep, were placed side-by-side. The total span of the proposed one half scale 30° skew box-beam was 31 ft. Stirrups were extended 1.5 in. above the top flange of box-beams to provide a composite action with deck slab (Section 7.02.18.B.6 of MDOT Bridge Design Manual, 2006). Figure C-1 shows the cross-section of the individual box-beam.

2. The depth of the shear-key between box-beams was 8 in. and the maximum width was 2.5 in. A 3 in. thick reinforced concrete deck slab was cast over the individual box-beams and the shear-keys in between. Both the shear-key and the deck slab were provided to connect the individual box-beams.

3. Two strands were placed in each diaphragm to facilitate the transverse post-tensioning (TPT) process. Oval-shape aluminum ducts, 6 in. high and 4.5 in. wide, was used for the TPT to facilitate the process in the presence of differential camber in the bridge model as a part of the objectives.
**C-I Materials Properties**

**A. Concrete**

1. **Box-Beam**

   Compressive strength, \( f'_{cb} = 6,268 \text{ psi} \) (uniaxial compression test of standard concrete cylinders after 28 days).

   Initial compressive strength, \( f'_{cbi} = 6,086 \text{ psi} \) (uniaxial compression test of standard concrete cylinders after 21 days).

   Modulus of elasticity, \( E_{cb} = 4,450 \text{ ksi} \) (uniaxial compression test of standard concrete cylinders after 28 days). Modulus of rupture, \( f_r = -7.5 \sqrt{f'_{cb}} = -7.5 \sqrt{6,268} = -594 \text{ psi} \) [ACI 318-05, Section 9.5.2.3, equation (9-10)]

   Unit weight of concrete, \( \gamma_{cb} = 151.6 \text{ lb/ft}^3 \)

2. **Deck slab**
Compressive strength, \( f'_{cs} = 4,600 \text{ psi} \) (uniaxial compression test of standard concrete cylinders after 28 days).

Modulus of elasticity, \( E_{cs} = 57,000 \sqrt{f'_{cs}} = 57,000 \sqrt{4,600} = 3,866 \text{ ksi} \) (ACI 318-05, Section 8.5.1)

Modular ratio, \( n = \frac{E_{cs}}{E_{cb}} = \frac{3,866}{4,450} = 0.87 \)

Unit weight of concrete, \( \gamma_{cs} = 150 \text{ lb/ft}^3 \)

**B. Steel Reinforcement**

1. **Non-Prestressing Steel Bars:**
   
   Yield strength, \( f_y = 60 \text{ ksi} \)

   Modulus of elasticity, \( E_s = 29,000 \text{ ksi} \)

2. **Prestressing Steel Strands:**
   
   Yield strength, \( f_{py} = 229.5 \text{ ksi} \)

   Ultimate tensile strength, \( f_{pu} = 270 \text{ ksi} \)

   Modulus of elasticity, \( E_{ps} = 28,500 \text{ ksi} \)

**C-II Properties of Cross-Section**

**A. Box-Beam**

Area of concrete, \( A = 140 \text{ in}^2 \)

Moment of inertia, \( I = 1,850 \text{ in}^4 \)

Distance from center of gravity to extreme top fiber, \( y_t = 5.5 \text{ in} \)

Distance from center of gravity to extreme bottom fiber, \( y_b = -5.5 \text{ in} \)

Bottom section modulus, \( Z_b = \frac{I}{y_b} = \frac{1,850}{5.5} = 336.36 \text{ in}^3 \)

Top central kern, \( k_t = -\frac{Z_b}{A} = -\frac{336.36}{140} = -2.4 \text{ in} \)

Area of non-prestressing steel (#4 bar), \( A_s = 0.2 \text{ in}^2 \)
Effective area of prestressing steel strands, \( A_{ps} = 0.153 \text{ in}^2 \)

Eccentricity of prestressing steel strands, \( e = 3.5 \text{ in.} \)

![Figure C-2 Simplified cross-section.](image)

**B. Composite Section**

Composite area, \( A_c = 194 \text{ in}^2 \)

Composite moment of inertia, \( I_c = 3,435.88 \text{ in}^4 \)

Distance from center of gravity to extreme top fiber of slab, \( y_{wc} = 5.41 \text{ in.} \)

Distance from center of gravity to extreme top fiber of beam, \( y_{wc} = 3.91 \text{ in.} \)

Distance from center of gravity to extreme top fiber of beam, \( y_{bc} = -7.09 \text{ in.} \)

Bottom section modulus, \( Z_{bc} = \frac{I_c}{y_{bc}} = \frac{3,435.88}{7.09} = 484.61 \text{ in}^3 \)

Transformed width of deck slab, \( b_{tr} = nb = 0.87 \times 18 = 15.66 \text{ in.} \)

Eccentricity of prestressing steel strands, \( e_c = 5.09 \text{ in.} \)
C-III Loads

A. Self-Weight of Box-Beam

\[ w_b = A \times \gamma_{cb} = \frac{140 \times 151.6}{1,000 \times 12^2} = 0.147 \text{ kip/ft} \]

B. Self-Weight of Deck Slab

\[ w_s = (A_c - A) \times \gamma_{cs} = \frac{(194 - 140) \times 150}{1,000 \times 12^2} = 0.06 \text{ kip/ft} \]

Total dead load, \( w_i = w_b + w_s = 0.147 + 0.06 = 0.207 \text{ kip/ft} \)

Effective span, \( l_e = 30 \text{ ft} \)

Total moment due to dead load, \( M_{DL} = \frac{w_i l_e^2}{8} = \frac{0.207 \times 30^2}{8} = 23.29 \text{ kip/ft} \)

C. Prestressing Forces

1. Exterior Beam

Jacking prestressing force, \( F_j = 20 \text{ kip/strand} \) (60 kip/beam)

Assume instantaneous losses occurs in prestressing forces (friction, elastic shortening, and anchorage losses) \( \approx 15\% F_j \)

Initial prestressing force, \( F_i = 17 \text{ kip/strand} \) (51 kip/beam)
2. **Interior Beam**

Jacking prestressing force, \( F_j = 25 \) kip/strand (75 kip/beam)

Assume instantaneous losses occurs in prestressing forces (friction, elastic shortening, and anchorage losses) \( \approx 15\%F_j \)

Initial prestressing force, \( F_i = 21.25 \) kip/strand (63.75 kip/beam)

Average initial prestressing force, \( F_i = 19.125 \) kip/strand (57.375 kip/beam)

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**C-IV Check of Normal Stresses at Concrete (Unshored Action)**

**A. Allowable Stresses (at transfer)**

Initial tensile stresses at mid-span, \( f_{a} = -3\sqrt{f_{cbi}'} = -3\sqrt{6,086} = -234 \) psi (ACI 318-05, Section 18.4.1).

Initial tensile stresses at supports, \( f_{a} = -6\sqrt{f_{cbi}'} = -6\sqrt{6,086} = -468 \) psi (ACI 318-05, Section 18.4.1).

Initial compressive stresses, \( f_{ci} = 0.6f_{cbi}' = 0.6 \times 6,086 = 3,652 \) psi (ACI 318-05, Section 18.4.1).

**B. Calculation of Stresses at Transfer (before placing deck slab)**

1. **At Supports**

Top fiber stress, \( f_t = \frac{F_i}{A} - \frac{F_i y_t e}{I} = \frac{57.375}{140} - \frac{57.375 \times 5.5 \times 3.5}{1,850} = -0.187 \text{ ksi} \geq -0.468 \text{ ksi} \)

Bottom fiber stress, \( f_b = \frac{F_i}{A} - \frac{F_i y_b e}{I} = \frac{57.375}{140} - \frac{57.375 \times (-5.5) \times 3.5}{1,850} = 1.0 \text{ ksi} \)

\( \leq 3,652 \text{ ksi} \)
2. *At Mid-Span*

Top fiber stress,

\[
f_t = \frac{F_i}{A} - \frac{F_{y_i} e}{I} + \frac{w_{y} l^2 y_i}{8I} = \frac{57.375}{140} - \frac{57.375 \times 5.5 \times 3.5}{1850} + \frac{0.147 \times (30 \times 12)^2 \times 5.5}{12 \times 8 \times 1850}
\]

\[= 0.41 \text{ ksi } \leq 3.625 \text{ ksi}
\]

Bottom fiber stress,

\[
f_b = \frac{F_i}{A} - \frac{F_{y_b} e}{I} + \frac{w_{y} l^2 y_b}{8I} = \frac{57.375}{140} - \frac{57.375 \times (-5.5) \times 3.5}{1850} + \frac{0.147 \times (30 \times 12)^2 \times (-5.5)}{12 \times 8 \times 1850} = 0.41 \text{ ksi } \leq 3.625 \text{ ksi}
\]

C. *Calculation of Stresses (after placing deck slab)*

1. *At Supports*

Top fiber stress,  

\[
f_t = \frac{F_i}{A} - \frac{F_{y_i} e}{I} = \frac{57.375}{140} - \frac{57.375 \times 5.5 \times 3.5}{1850} = -0.19 \text{ ksi } \geq -0.468 \text{ ksi}
\]
Bottom fiber stress, \[ f_b = \frac{F_i}{A} - \frac{F_i y_b e}{I} = \frac{57.375}{140} - \frac{57.375 \times (-5.5) \times 3.5}{1850} = 1.0 \text{ ksi} \]
\[ \leq 3.625 \text{ ksi} \]

2. **At Mid-Span**

Top fiber stress,
\[ f_t = \frac{F_i}{A} - \frac{F_i y_t e}{I} + \frac{w_i l^2}{8 I} = \frac{57.375}{140} - \frac{57.375 \times 5.5 \times 3.5}{1850} + \frac{0.207 \times (30 \times 12)^2 \times 5.5}{12 \times 8 \times 1850} \]
\[ = 0.64 \text{ ksi} \leq 3.625 \text{ ksi} \]

Bottom fiber stress,
\[ f_b = \frac{F_i}{A} - \frac{F_i y_b e}{I} + \frac{w_i l^2}{8 I} = \frac{57.375}{140} - \frac{57.375 \times (-5.5) \times 3.5}{1850} + \frac{0.207 \times (30 \times 12)^2 \times (-5.5)}{12 \times 8 \times 1850} \]
\[ = 0.18 \text{ ksi} \leq 3.625 \text{ ksi} \]

### C-V Elongation

Effective cross-sectional area of strand, \( A = 0.153 \text{ in}^2 \)

Modulus of elasticity, \( E = 28,500 \text{ ksi} \)

Length of each strand, \( L = 56 \text{ ft} = 672 \text{ in} \)
1. **For Interior Box-Beams (B-2 and B-3)**

Prestressing force, \( P = 25 \) kip

From the elasticity theory, \( \frac{P}{A} = E \frac{\Delta L}{L} \)

\[ \Delta L = \frac{PL}{EA} \]

\[ \Delta L = \frac{25 \times 672}{28,500 \times 0.153} = 3.85 \text{ in.} \]

2. **For Exterior Box-Beams (B-1 and B-4)**

Prestressing force, \( P = 20 \) kip

From the elasticity theory, \( \frac{P}{A} = E \frac{\Delta L}{L} \)

\[ \Delta L = \frac{PL}{EA} \]

\[ \Delta L = \frac{20 \times 672}{28,500 \times 0.153} = 3.08 \text{ in.} \]

**C-VI Cracking Moment \((M_{cr})\)**

Cracking moment in excess of the total self-weight, \( \Delta M_{cr} \)

\[ \Delta M_{cr} = \frac{Z_{b}}{Z_{c}} \left[ F_{i} (e - k_{i}) - M_{DL} \right] - f_{c} Z_{k_{c}} \text{ (Naman 2004, equation 9.28)} \]

\[ \Delta M_{cr} = \frac{484.61}{336.36} \left[ 57.375(3.5 + 2.4) - 23.39 \times 12 \right] - \left( \frac{-594}{1000} \right) (484.61) = 371.18 \text{ kip-in.} \]

\[ = 30.93 \text{ kip-ft} \]

Total cracking moment for a single beam, \( M_{cr} \)

\[ M_{cr} = \Delta M_{cr} + M_{DL} = 30.93 + 23.39 = 54.3 \text{ kip-ft (Naman 2004, equation 9.29)} \]

Cracking load for load distribution test for single beam, \( \frac{4M_{cr}}{l} = \frac{4 \times 30.93}{31} = 4 \text{ kip} \)

Cracking load for the bridge model, \( p_{cr} = 4 \times 4 = 16 \text{ kip} \)

**C-VII Nominal Moment Capacity \((M_{n})\) Using Strain Compatibility Approach**

A. **First estimation of ultimate stresses in prestressing steel strands**

\[ f_{ps} = f_{pu} \left[ 1 - \frac{\gamma_{p}}{\beta_{f}} \left( \rho_{p} \frac{f_{pu}}{f_{c}} + \frac{d}{d_{p}} (w - w') \right) \right] \]

\( \text{ (ACI 318-05, equation 18-3)} \)
Specified tensile strength of prestressing steel, \( f_{pu} = 270 \text{ ksi} \)

Factor for type of prestressing strands, \( \gamma_p = 0.28 \)

Distance from extreme compression fiber to centroid of prestressing steel, \( d_p = 12 \text{ in.} \)

Distance from extreme compression fiber to centroid of longitudinal tension non-prestressing reinforcement, \( d = 8.25 \text{ in.} \)

Distance from extreme compression fiber to centroid of longitudinal compression non-prestressing reinforcement, \( d' = 2 \text{ in.} \)

Prestressing reinforcement ratio, \( \rho_p = \frac{A_{sp}}{bd_p} = \frac{3 \times 0.153}{15.66 \times 12} = 2.443 \times 10^{-3} \)

Tension reinforcement index, \( w = \frac{A_y f_y}{bd f'_c} = \frac{8 \times 0.2 \times 60}{15.66 \times 8.25 \times 4.6} = 0.1615 \)

Compression reinforcement index, \( w' = \frac{A'_y f'_y}{bd f'_c} = \frac{3 \times 0.11 \times 60}{15.66 \times 8.25 \times 4.6} = 0.03317 \)

Factor relating depth of equivalent rectangular compressive stress block to neutral axis depth, \( \beta_i = 0.85 - 0.05(f'_c - 4) = 0.85 - 0.05(4.6 - 4) = 0.82 \)

\[
\rho_p \frac{f_{pu}}{f'_c} + \frac{d}{d_p} (w - w') = 2.443 \times 10^{-3} \frac{270}{4.6} + \frac{8.25}{12} (0.1615 - 0.03317) \\
= 0.231 \geq 0.17 \quad \text{(ACI 318-05, Section 18.7.2)}
\]

\[
f_{ps} = 270 \left[ 1 - \frac{0.28}{0.82} \frac{0.17}{0.17} \right] = 248.79 \text{ ksi}
\]
B. First Estimation of the Depth of Neutral Axis (from equilibrium)

Tension Force ($T$)

1. Prestressing Steel Strands ($d_p = 12$ in.)

Initial prestressing strain (locked in strain due to prestress),

$$\varepsilon_{ps} = \frac{F_i}{A_p E_{ps}} = \frac{19.125}{0.153 \times 28,500} = 4.386 \times 10^{-3} \leq 0.0086$$

Decompression strain,

$$\varepsilon_o = \frac{F_i}{A_p E_c} \left[ 1 + \left( \frac{e^2}{r^2} \right) \right] = \frac{19.125 \times 3}{194 \times 4,450} \left[ 1 + \left( \frac{5.09^2}{4.208^2} \right) \right] = 1.637 \times 10^{-4} \leq 0.0086$$

Prestressing tension force, $T_{ps} = f_{ps} \times A_{ps} = 248.79 \times 3 \times 0.153 = 114.19$ kip

2. Non-Prestressing Bars

(a) First layer [top reinforcement] ($d = 4.5$ in.)

Strain, $\varepsilon_{top} = \left( \frac{4.5 - c}{c} \right) 0.003$ (from similar triangles)
Tension force, \( T_1 = \varepsilon_{sup} \times E_s \times A_s = \left( \frac{4.5-c}{c} \right)0.003 \times 29,000 \times (4 \times 0.20) = 64.2 \left( \frac{4.5-c}{c} \right) \)

(b) Second layer [bottom reinforcement] \((d = 12 \text{ in.})\), assume \( f_s = f_y \)
Tension force, \( T_2 = f_y \times A_s = 60 \times (4 \times 0.20) = 48 \text{ kip} \)

Total tension force,
\[
T = T_{ps} + T_1 + T_2 = 114.19 + 64.2 \left( \frac{4.5-c}{c} \right) + 48 = 162.19 + 64.2 \left( \frac{4.5-c}{c} \right)
\]

**Compression Force (C)**

1. **Concrete Compressive Stress Block**

Assume the depth of neutral axis, \( c < 3 \text{ in.} \) (neutral axis lies on the deck slab)

\[
C_c = 0.85 f'_c b \beta_c = 0.85 \times 4.6 \times 15.66 \times 0.82 \times c = 50.21c
\]

2. **Deck Slab Steel Reinforcements**

Strain, \( \varepsilon_{deck} = \left( \frac{c-2}{c} \right)0.003 \) (from similar triangles)

Compression force, \( C_s = \varepsilon_d \times E_s \times A_s = \left( \frac{c-2}{c} \right)0.003 \times 29,000 \times (3 \times 0.11) = 28.71 \left( \frac{c-2}{c} \right) \)

Total compression force, \( C = C_s + C_c = 50.21c + 28.71 \left( \frac{c-2}{c} \right) \)

**Equilibrium**

\[
T = C
\]

\[
50.21c^2 - 69.28c - 346.32 = 0
\]

\( c = +3.41 \text{ in.} \) or \( c = -2.71 \text{ in.} \)

\( \therefore \) First depth of neutral axis, \( c = +3.41 \text{ in.} \)
C. Second Estimation of Ultimate Stresses in Prestressing Steel Strands

Total prestressing strain = Initial prestressing strain + Decompression strain + Ultimate strain at level of prestressing strands

\[ \varepsilon_{ps} = \varepsilon_{pi} + \varepsilon_o + \varepsilon_{si} \]

\[ \varepsilon_{ps} = 4.386 \times 10^{-3} + 1.637 \times 10^{-4} + 7.557 \times 10^{-3} = 0.01211 > 0.0086 \]

\[ f_{ps} = 270 - \frac{0.04}{\varepsilon_{ps} - 0.007} \quad \text{(PCI Design Handbook (2003), Section 11.2.5)} \]

\[ f_{ps} = 270 - \frac{0.04}{0.01211 - 0.007} = 262.17 \text{ ksi} \]

D. Second Estimation of the Depth of Neutral Axis (from equilibrium)

By repeating the same procedures, we get

\[ 50.21c^2 - 75.43c - 346.62 = 0 \]

\[ c = +3.48 \text{ in. or } c = -1.98 \text{ in.} \]

\[ \therefore \text{Depth of neutral axis, } c = +3.48 \text{ in.} \]
D. Third Estimation of Ultimate Stresses in Prestressing Steel Strands

Total prestressing strain = Initial prestressing strain + Decompression strain + Ultimate strain at level of prestressing strands

\[ \varepsilon_{ps} = \varepsilon_{ps} + \varepsilon_{i} + \varepsilon_{si} \]

\[ \varepsilon_{ps} = 4.386 \times 10^{-3} + 1.637 \times 10^{-4} + 7.345 \times 10^{-3} = 0.0118947 > 0.0086 \]

\[ f_{ps} = 270 - \frac{0.04}{\varepsilon_{ps} - 0.007} \]

\[ f_{ps} = 270 - \frac{0.04}{0.0118947 - 0.007} = 261.82 \text{ ksi (close enough to the previous iteration)} \]

E. Check of Forces Equilibrium

Total Tension Force

\[ T = T_{top} + T_{bottom} = 168.17 + 18.8 = 186.97 \text{ kip} \]

Total Compression Force

\[ C = C_{c} + C_{s} = 174.73 + 12.21 = 186.94 \text{ kip} \]
Total tension force ≈ Total compression force

**E. Nominal Moment Capacity \( (M_u) \) [single box-beam]**

By calculating the moment about the extreme top (compression) fiber,

Depth of compression stress block, \( a = \beta_i \times c = 0.82 \times 3.48 = 2.85 \) in.

\[
M_u = T_{bottom} \times 12 + T_{top} \times 4.5 - C_c \times 1.42 - C_s \times 2
\]

\[
M_u = 168.17 \times 12 + 18.82 \times 4.5 - 174.73 \times 1.42 - 12.21 \times 2 = 1830 \text{ kip-in.}
\]

\[
= 152.52 \text{ kip-ft}
\]

**F. Ultimate vertical load \( (P_u) \)**

\[
M_{LL} = M_u - M_{DL} = 152.52 - 23.29 = 129.23 \text{ kip-ft}
\]

By using two-point loading frame having a longitudinal spacing of 7.5 ft,

\[
M_{LL} = \frac{P_u \left( \frac{L}{2} - \frac{l}{2} \right)}{2} \Rightarrow P_u = \frac{4M_{LL}}{(L-l)} = \frac{4 \times 129.23}{(31-7.5)} = 22 \text{ kip}
\]

Load-carrying capacity of single box-beam, \( P_u = 22 \text{ kip} \)
By assuming equal distribution of the ultimate vertical load ($P_u$) in the transverse direction, the load-carrying capacity of the bridge model would be:

$$P_u = 22 \times 4 = 88 \text{ kip}$$