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EFFECTS OF SINGLE PANEL REPLACEMENT ON A FULL SCALE, FULL
DEPTH, PRECAST CONCRETE BRIDGE SYSTEM

by

Jason Robert Perry

A thesis submitted in partial fulfillment
of the requirements for the degree

of

MASTER OF SCIENCE

in

Civil and Environmental Engineering

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Logan, Utah

2012

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ABSTRACT

Effects of Single Panel Replacement of a Full-Scale, Full-Depth, Precast Concrete Bridge Deck System

by

Jason Robert Perry, Master of Science

Utah State University, 2012

Major Professor: Dr. Marvin Halling
Department: Civil and Environmental Engineering

The use of precast concrete deck panels is becoming increasingly popular for bridge construction and rehabilitation in the state of Utah and across the country. It allows for the use of full depth concrete deck panels but removes the long construction times of traditional cast-in-place methods. One of the challenges to the use of precast deck panels is the transverse deck panel joints that exist between the panels. These joints are unreinforced using traditional methods and therefore are the weakest section of the bridge. In many situations the joint will fail and water seeps through and can damage the bridge superstructure.

Post-tensioning of precast decks has become the standard. The post-tensioning provides reinforcing through the joints, reducing the cracking that occurs. Additionally, the post-tensioning provides pressure along the joint and closes cracks that have occurred, therefore preventing water from leaking through to the superstructure and

damaging it. The Utah Department of Transportation uses post-tensioning cables that run along the length of the bridge deck, applying pressure on the joints. One of the problems with using this method is it does not allow for the replacement of a single deck panel should the need arise. Utah State University has been researching a new post-tensioned connection that would allow for the replacement of a single deck panel. The “curved bolt” connection connects each deck panel to adjacent panels, providing reinforcement and post-tensioning along the joint. Laboratory testing was undertaken to investigate the effects of single panel bridge rehabilitation on the existing deck system.

(52 pages)

PUBLIC ABSTRACT
**Effects of Single Panel Replacement of a Full-Scale, Full-Depth,
Precast Concrete Bridge Deck System**
By Jason R. Perry

As rehabilitation and replacement becomes for frequent due to an aging infrastructure. Reducing delays caused to drivers is becoming of utmost importance. The development and use of precast concrete has facilitated the use of accelerated bridge construction methods. Precast components are constructed off-site and then transported and installed on-site. The process allows rehabilitation and replacement projects to be completed in days instead of weeks. Leaking at panel connections has shown to cause damage to other bridge components. The use of post-tensioning has allowed for more sophisticated methods to be applied to reduce the leaking at panel connections.

The proposed investigation will evaluate the effects of using panel-to-panel post-tensioned connections in bridge rehabilitation and compare the results to those of a similar connection for original construction. A four panel test specimen will be tested in negative flexure in a four step process. Two of the joints of the specimen will be tested to determine a reduction in capacity for the system. Load, cracking behavior, steel strain and displacement will be used to determine the joint capacity. Both joints were found to behave adequately during the testing process. Both cracked at approximately 500 kn-m (369 k-ft). Although single panel replacement slightly reduced the behavior of the joint the specimen performed as predicted in previous research.

Results were compared to a theoretical values based on current bridge design specifications, a control data set that was obtained from the original construction as well as compared to data reported in previous research and a previously reported finite element model. The research confirmed previously reported data in strength analysis of the system. Current design codes conservatively model the expected capacity of the panel-to-panel connection.

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Without the example and support of my parents, James and Wanda Perry, I would not be the man I am today. I would like to thank them for teaching me intangible principles that have helped me to be successful on this project.

I would also like to thank my wife's family for the tireless support helping to raise my children and teach them valuable lessons while I have been working on this project and my other schoolwork. I would like to especially thank my brother-in-law, Ramón, for helping me in the lab and the encouragement he gave me.

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Jason R. Perry

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CHAPTER I

INTRODUCTION

The use of precast concrete deck panels is becoming increasingly more popular for bridge construction and rehabilitation in the state of Utah and across the country. It allows for the use of full-depth concrete deck panels but removes the long construction times of traditional cast in place methods. One of the challenges to the use of precast deck panels is the transverse deck panel joints that exist between the panels. These joints are unreinforced using traditional methods and therefore are the weakest section of the bridge. In many situations the joint will fail and water seeps through and can damage the bridges superstructure.

Post-tensioning of precast decks has become the standard. The post-tensioning provides reinforcing through the joints reducing the cracking that occurs. Additionally, the post-tensioning provides pressure along the joint and closes cracks that have occurred therefore preventing water from leaking through to the superstructure and damaging it. The Utah Department of Transportation (UDOT) uses post-tensioning cables that run along the length of the bridge deck applying pressure on the joints. One of the problems with using this method is it does not allow for the replacement of a single deck panel should the need arise. Replacing one panel would be more cost efficient than replacing the entire bridge deck. Utah State University has been developing a new post-tensioned connection that would allow for the replacement of a single deck panel. The “curved bolt” connection connects each deck panel to adjacent panels providing reinforcement and post-tensioning along the joint. Experimental findings have shown that the “curved

bolt” connection is showing flexural capacity that is similar to the standard post-tensioning system.

One of the differences between the curved cable system and the standard systems is the curved cable allows for individual panels to be replaced instead of replacing the entire deck system. This paper gives a report on laboratory testing of a full scale specimen that is constructed and tested in negative bending. The testing was completed in two phases, original construction and post panel replacement. During both tests load vs. deflection curves will be recorded. During the post panel replacement experiment a cracking moment and ultimate moment will also be tested. Punching shear capacity on the panel and the joint will also be tested.

CHAPTER II

LITERATURE REVIEW

Accelerated Bridge Construction (ABC) is becoming more popular for bridge construction and rehabilitation. ABC reduces construction times allowing bridges to be constructed in days instead of months. Precast concrete panels are cast and cured in a plant and transported to the construction site. The panels are connected to bridge girders using shear pockets to ensure composite action. Deck panels are often connected together using post-tensioning to reduce cracking at the joint.

The use of precast decks for bridge rehabilitation and construction began in the late 1960s to early 1970s. The state of Indiana in 1970 contracted Purdue University to perform testing on precast decks. The reasons of interest in precast decks were the quality control and excellent durability of precast concrete. Indiana used the precast decks on two bridges, one was new construction and the other was for bridge rehabilitation. The bridges were instrumented for performance monitoring. The performance of the precast decks has been reported as successful (Biswas 1986).

Bridge rehabilitation is becoming more important as the infrastructure ages. Bridges over 25 years old show significant bridge deterioration. Traffic, weather, and chemicals used for ice control are main causes for bridge damage. Patching of damaged locations and the use of overlays are sufficient in the early stages of deterioration but eventually deck replacement becomes necessary (Biswas 1986).

The New York State Thruway Authority (NYSTA) began researching the use of precast decks in 1973. They used the decks on three bridges, the Amsterdam Interchange Bridge in 1973, the Krum Kill Road Bridge in 1977, Harriman Interchange Ramp in

1979. The Amsterdam Interchange bridge was constructed under close supervision of NYSTA authorities. They had issues connecting the precast deck panels to the steel stringers to ensure composite action but those were overcome. The bridge performed satisfactorily after the rehabilitation. The Krum Kill Road Bridge had cracks develop over the reinforcing bars in the precast panels. The cracks were filled with epoxy and durability was not further weakened. The bridge has been performing satisfactorily though leaking has been observed at the connections of the precast deck. The NYSTA had experience with precast deck construction but the contractor for the Harriman Interchange Ramp did not. The bridge had a complex geometry to the panels but more critical issue was the epoxy was not mixed with the correct proportions and the leaking and cracking has been observed at panel connections (Biswas 1986).

In 1978, the California Department of Transportation undertook a bridge rehabilitation project on CA-17 High Street Overhead, in Oakland. The bridge consists of two bridges, left and right, the only part of the bridge that was rehabilitated was one of the southbound lanes on the left bridge. The use of precast deck panels allowed for the bridge to remain open during the rehabilitation. The deck panels were connected to the steel girders using nelson studs and the panels were connected using high strength cement mortar and concrete (Biswas 1986).

Precast panels can be used to rehabilitate all types of bridges. Issa et al. (1995a) sent out surveys to 53 departments of transportation (DOTs) across the United States and in parts of Canada. The results from that survey show that the use of precast deck panels to rehabilitate aging bridges has become more common since the 1970s. Different DOT's have used the precast panels in different methods. The use of precast panels decreases

construction time, decreases the interruption of traffic without compromising strength of the bridge (Issa et al. 1995a).

The use of precast panels has a variety of challenges that must be addressed to use them effectively. The panels must be connected to each other and they must be connected to the girders to ensure composite behavior. Issa et al. performed a series of bridge inspections across the United States and in Toronto, Canada. Of the bridges inspected, many used precast deck sections that were performing satisfactorily. Two bridges of significant mention are the 03200 Waterbury Bridge and the Route 235 Bridge over Dogue Creek (Issa et al. 1995b).

The 03200 Waterbury Bridge in Connecticut was rehabilitated with precast deck sections using female-to-female joints. The joints were post-tensioned to 1 MPa (150 psi) at simple spans and a 2MPa (300 psi) at more critical larger spans. The bridge had no leaking or cracking in the deck (Issa et al. 1995b).

The Dogue Creek Bridge was also rehabilitated using precast deck sections. Except on this bridge, no joint post-tensioning was used. The bridge had cracking and leaking develop especially at the joint locations (Issa et al. 1995b).

Bridges deteriorate rapidly if the deck cracks and leaks. The water will leak through the deck and begin to oxidize the steel used in the bridge. As with the Dogue Creek Bridge once the deck started to leak many of the bridge components began to deteriorate.

Issa et al. (2000) tested three different two-panel bridge specimens using full-depth precast concrete decks on steel stringers. The three specimens were tested to identify capacity and behavior under cyclic fatigue loading. The first specimen was not

post-tensioned while the latter two specimens were post-tensioned at different levels. The post-tensioning reduced cracking in the concrete deck and allowed the deck to perform under higher loads. The post-tensioning also reduced the amount of permanent deflection. With the increase of post-tensioning pressure cracking was further delayed.

Harshbarger et al. (2007) discussed general rules for when bridges can be rehabilitated versus when they should be replaced. The paper was focused on rehabilitation of historic bridges more than non-historic bridges. Several components of a bridge can be replaced without significant cost such as a bridge deck. Usually bridge rehabilitation is not considered on short span bridges. On longer bridges or multi-span bridges rehabilitation is more common.

Wayne et al. (2009) explains several of the advantages that accelerated bridge construction has compared to traditional construction method, such as: less impact on the environment, minimizing traffic interruptions, while improving safety for workers. Wayne et al. (2009) developed a connection that uses the reinforcing bars from the panels that protrude out of the panels. The bars are spliced and cast in place concrete is poured to splice the bars together and connect them. The panel to panel connection was found to function satisfactorily with a failure load of 178 kN (40 kips).

Carter et al. (2007) worked on a multiyear project to design, test and build a bridge using ABC for the Wisconsin Department of Transportation (WisDOT). The I-90 bridges at Door Creek outside of Madison, Wisconsin had experienced severe deterioration of the concrete road due to the harsh winter conditions and heavy truck traffic. The rehabilitation project included a bridge widening project.

WisDOT in 2003 provided funding to the University of Wisconsin to develop full-depth precast concrete deck surface to use on the Door Creek bridges. The deck panels would be pretensioned to increase strength for moving and installation. The longitudinal joint connection was designed to protrude half of the pretensioning strand from the deck panels. The strand would be coupled to adjacent panels to provide post-tensioning across the joints. The joints were female-to-female joints filled with grout. Once the post-tensioning was applied it would place the joint in compression. Another design aspect of the panels was UW increased the shear pockets spacing from 610 mm (2 ft) to 1220 mm (4 ft) to increase the strength of the individual panels. The American Association of State Highway and Transportation Officials (AASHTO) design specification requires a spacing of 610 mm (2 ft) (Carter et al. 2007).

The bridge at Door Creek was constructed in 2005 and performance was monitored for a year. An inspection was performed a year after construction and found that the deck seemed in almost perfect condition. The bridge rehabilitated using the precast deck was found to be stiffer than the traditional bridges. The maintenance crews noted that the precast deck did not have as many issues with icing as the other cast-in-place bridges (Carter et al. 2007).

Badie and Tadros (2008) developed a connection that would reduce cracking and leaking at panel connections without the use of post-tensioning. The connection was constructed by installing a HSS section in the concrete panels. A traditional reinforcing bar runs through the panel with 127 mm (5 in.) of threaded bar protruding from the side of the panel. The threaded bar is inserted through a hole into the HSS and is bolted to the

wall of the HSS section. The HSS section is filled with grout. The connection was found to satisfactorily provide support across a deck panel connection.

The use of precast concrete decks for bridge rehabilitation has been used primarily on bridges that have steel girders for the support system. One issue that prevents the use of precast decks with concrete girders is the connectors to obtain composite action. The most common connectors for the shear pockets is using welded studs with steel or embedded stirrups for concrete. While welded studs can be applied at anytime with steel girders unless the stirrups are embedded from original construction they cannot be applied later (Issa et al. 2006).

Issa et al. (2006) designed a new shear connector to obtain full composite action for precast decks on concrete girders. The new connection was created by drilling 127 mm (5 in.) into the concrete girder. An epoxy grout is poured into the holes in the girder and a threaded rod is inserted into the hole with a twisting motion to allow for even distribution of epoxy in the threads. More epoxy was added to fill the drilled hole and the epoxy was allowed to setup.

A precast panel was placed on the girder with the rods in the shear pockets. The pockets were filled grout to connect the panel to the girder. A push-out test was run to test the capacity of the connection in shear. The number of rods, the length of the rods and position of the rods inside the pocket all varied to find an optimal configuration for rod placement (Issa et al. 2006).

The experimental results showed that rod position in the pocket had little to no effect on the strength of the connection. Longer rods typically had a higher shear resistance than shorter rods. The factor that resulted in the greatest effect on the shear

strength was the number of rods in the pocket. Failure was noted by first failure of the haunch and then the yielding of the threaded rods. The experimental results were compared to the AASHTO LRFD design specifications for shear design and found to be greater than design capacities. The use of the threaded rods installed in concrete girders were found to satisfactorily provide shear resistance and full composite action (Issa et al. 2006).

A variety of methods has been used to for panel-to-panel connections for use in ABC. Unreinforced connections, male-female connections, steel reinforcement splices, and post-tensioning are the common connections used. Utah State University has done a series of experiments for the Utah Department of Transportation (UDOT) to test different joint connections to use with precast concrete deck systems. Porter (2009) built several small test specimens to test different connection types for comparison. Five different connections were tested: a welded stud, welded rebar, post-tensioning using a straight threaded rod, post-tensioning using a 610 mm (24 in.) curved bolt, and post-tensioning using a 915 mm (36 in.) curved bolt.

Three of the connections have been used by UDOT on bridge projects. The latter two connections were variations of a new design being tested for use in bridge projects. The first two connections were not post-tensioned connections while the latter three are post-tensioned. The test specimens were small scale models a bridge to compare joint capacity (Porter 2009).

Two experiments were performed on the connections, a shear test and a flexure test. The results of the shear test show that the post-tensioned connections provide more ultimate shear resistance than the non post-tensioned connections. Although the

continuous welded stud connection provided 87% of the ultimate shear resistance of the post-tensioned connection (Porter 2009).

Each connection was also tested in cyclic testing for shear. The post-tensioned connections failed at higher loads than the welded stud connection. Unfortunately was difficult to measure the true capacity of the post-tensioned connections due to failure in the panel and not in the connection on three of the four post-tensioned specimens. The one specimen that failed at the connection failed at a higher load than the non post-tensioned connection (Porter 2009).

Each of the five connections were also tested in bending. The information from Porter et al. shows that the curved bolt has a stronger capacity than the other connections tested. One of the major advantages of the curved bolt system is that it would allow for bridge panels to be replaced individually instead of replacing the whole system as other systems require but still allow for the post-tensioning across the joints to utilize the added capacity of post-tensioning (Porter 2009).

Building on the research that was performed by Porter (2009), Utah State University further explored the strength of the curved bolt joint connection. Roberts tested the shear and moment capacity of the 910 Mm (36 in.) curved bolt in full-sized bridge specimens. The moment test specimens were built of two concrete panels that were 2440 mm (8 ft) long and 3660 mm (12 ft) wide. The panels were placed on two steel wide flange beams and were connected using nelson studs to obtain composite behavior. Two different longitudinal joint connections were tested. A 91 cm (36 in.) curved bolt connection and a straight post-tensioned system. Roberts (2011) also tested the connections for shear capacity by connecting two deck panels that were 1200 mm (4

ft) by 3660 mm (12 ft) and 223 mm (8.75 in.) thick. The shear panels were not connected to beams in order to fully analyze the capacity of the shear connection and not obtain skewed data.

Roberts (2011) reports that the curved bolt connection provided less flexural capacity than the straight post-tensioning connection by 16%. The shear specimens should crack at almost equivalent loads but the total shear capacity of the post-tension system was 70% greater than that of the curved bolt system.

To further improve upon the curved bolt, Wells (2012) addressed various points that Roberts recognized with the results. The test specimens were identical to those of Roberts (2011). The biggest difference was a change from curved bolts to using post-tensioning cables that were 3050 mm (10 ft) in length. The effective post-tensioning length was also increased to alternating 1830 mm (6 ft) and 1220 mm (4 ft). The conduit for the cables was embedded deeper into the panel and the bearing plate was also placed slightly deeper to prevent localized failure of the concrete behind the bearing plate. Wells (2012) increased the number of cables from six to ten and placed them in five coupled groups.

Wells (2012) built a flexure specimen and a shear specimen to compare to the findings of Roberts (2011). The results of the experiment showed that the curved cable connection has similar flexural capacity as the current UDOT standards connections.

CHAPTER 3
EVALUATION AND TESTING
SPECIMEN AND TEST DETAILS

The test specimen was constructed at the Systems, Materials, and Structural Health (SMASH) Lab at Utah State University. The specimen was designed based on the Utah Department of Transportation (UDOT) precast panel design specification (UDOT 2008). The specification requirements were used to determine panel thickness and required rebar reinforcement. Previous research (Roberts 2011; Wells 2012) had investigated the capacity for a curved post tensioning connection as compared with the approved standards already in use by UDOT. The current study investigates the effects of single panel rehabilitation on the bridge deck system. A full-scale, four-panel test specimen was constructed for investigation.

The constructed panels to be 3660 mm (12 ft) long, 2440 mm (8 ft) wide and 223 mm (8.75 in.) thick. The panels were reinforced with two mats of reinforcing steel in both the longitudinal and transverse directions. Number 19 bars (#6 bars) were used for all reinforcing steel, longitudinal steel was typically placed 300 mm (12 in.) on center as required in the UDOT specification. Transverse steel was placed 75 mm (3 in.) on center near the transverse joints with a spacing of 150 mm (6 in.) on center near the center of the panels. The top mat of steel was placed 95 mm (3.75 in.) below the deck surface while the bottom mat was placed 168 mm (6.625 in.) below the surface. Fig. 1 contains the detail of the steel placement.

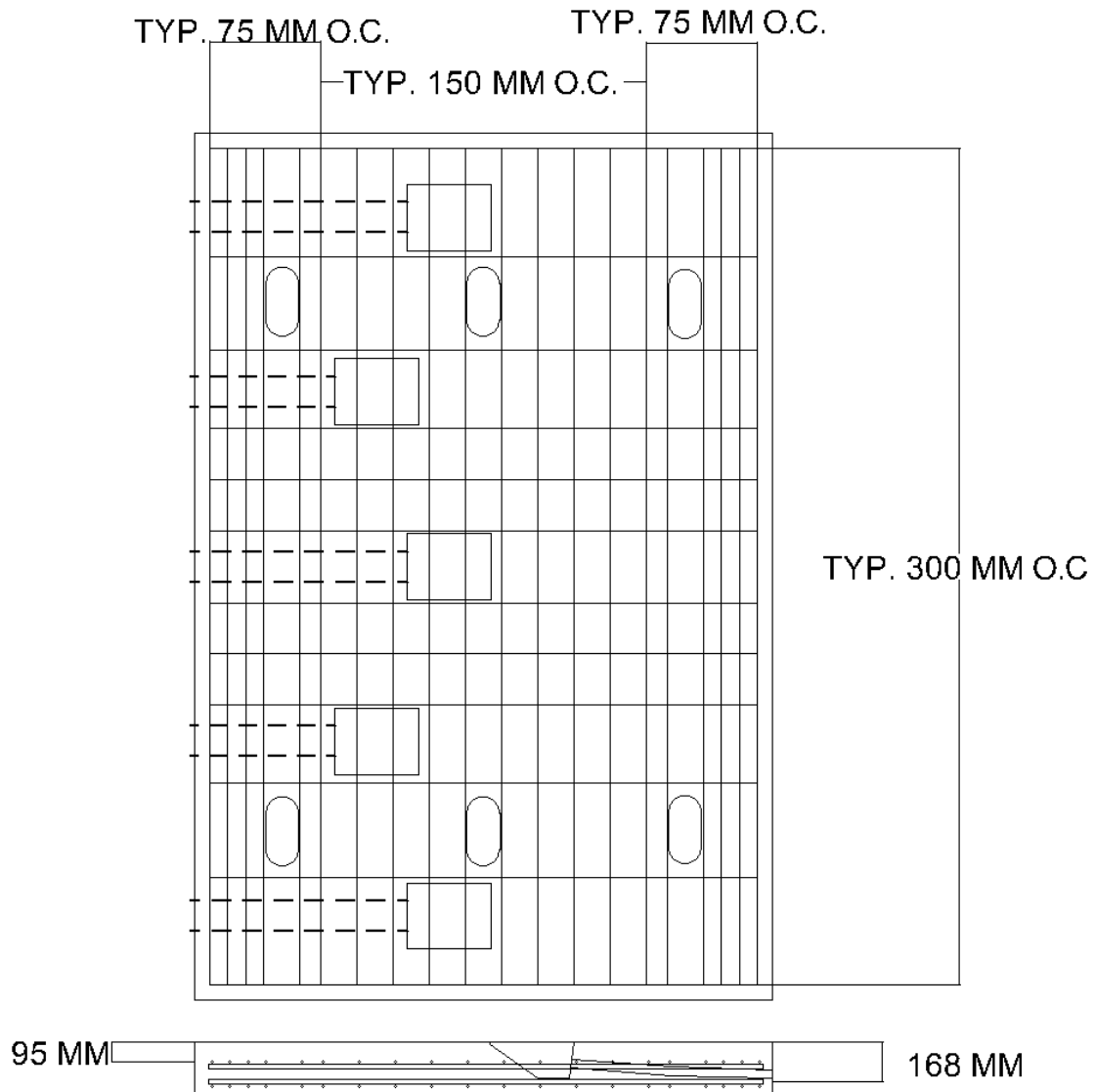


Fig. 1 Panel design detail

AASHTO requirement for post-tensioning of transverse joints requires 1.7 MPa (250 psi) across the joint. Due to the nature of post-tensioning and the losses associated UDOT suggests a pressure of 2 MPa (300 psi) be applied across the joint ensuring the requirement of 1.7 MPa (250 psi) is satisfied. The post-tensioning cable used for the project was 15 mm (0.6 in.) diameter 7 strand pre-stressing cable. The cable has a yield strength of 1860 MPa (270 ksi). To ensure that cables were not yielded during the post-

tensioning process ten cables were used and placed in five groups of two. The cable groups were spaced with a spacing of 740 mm (29 in.) between pockets.

After the panels had been formed, reinforced and the concrete had been cast and cured. The panels were placed on two W530X300 (W21X122) steel girders. To ensure composite action between the deck panels and the girders nelson studs were welded to the girders through the shear pockets in the panels. The nelson studs were 180 mm (7 in.) long and 22 mm (0.875 in.) in diameter. Three studs were placed in each shear pocket.

After the panels were placed on the girders, the transverse joint was formed up and the cables were placed in the 38 mm (1.5 in.) aluminum conduit that was cast in the deck panels. Masterflow 928 grout was used in the transverse joint between the panels. After the grout had cured the cables were stressed to apply the pressure across the joint. To measure the load in the cables load cells were placed between the chucks for the cables and the bearing plates. A hydraulic monostrand stressing jack was used to apply the load to the cables. One of the obstacles to using a curved cable is if the bearing surface is not perpendicular to the cable additional seating losses due to rotation occur. Therefore, the cables were jacked to an additional load to ensure that sufficient pressure would be applied at the joint. Each cable was stressed to approximately 90% of the yielding load for the cables. After the seating losses the load cells measured a load of 156 kN (35 kips) per cable. That load would correlate with a joint pressure of 1.9 MPa (277 psi) across the joint. The cables were stressed beginning in with the middle pocket, then edge pockets and lastly the inside pockets were stressed to ensure an equal pressure distribution along the joint. Upon completion of cable stressing, the shear pockets and haunch were grouted in.

Test Setup and Schedule

In order to determine the effect of single panel replacement the specimen was tested in four sessions. Two of the joints were tested in negative bending with the initial construction and also post replacement (PR) of a single panel. Fig. 2 shows the different test setups, in elevation view. All joints and panel are identified. The first two test setups are for obtaining a control data set for the specimen and the second two setups are to monitor the behavior of the specimen after panel replacement.

The specimen was placed on three supports, one directly under the joint to be tested, one under the middle joint and a third joint on the end not being tested for stability. The supports are comprised of bearing steel and spherical bearings to allow the specimen to rotate during testing but still be safely supported. A steel girder was placed on the middle joint and bolted to the strong floor using 5 cm (2 in.) bolts to prevent uplift of the specimen. The load was applied on the overhanging end by two hydraulic rams. Fig. 3 is a picture of the test setup.

A datalogger was used to record the data measured on the specimen during testing. Each panel was instrumented with eight strain gages along the length of the panel. The gages were placed on longitudinal reinforcing 1400 mm (55 in.) from the edge of the panel. Fig. 4 shows the location for the strain gages for each panel. Four gages were installed along bars in each mat of steel. Load cells were used to measure the force being applied on the specimen. Fig. 5 shows the locations of the strain gages installed on the girders, five underneath each joint being tested, in a vertical line to determine the neutral axis of the section. Fig. 6 is a picture of the installed gages.

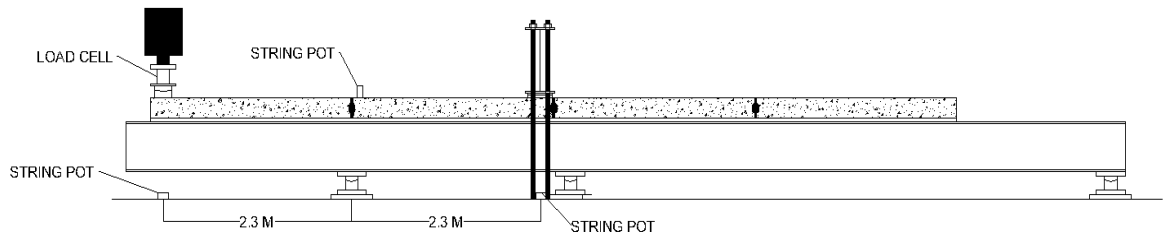
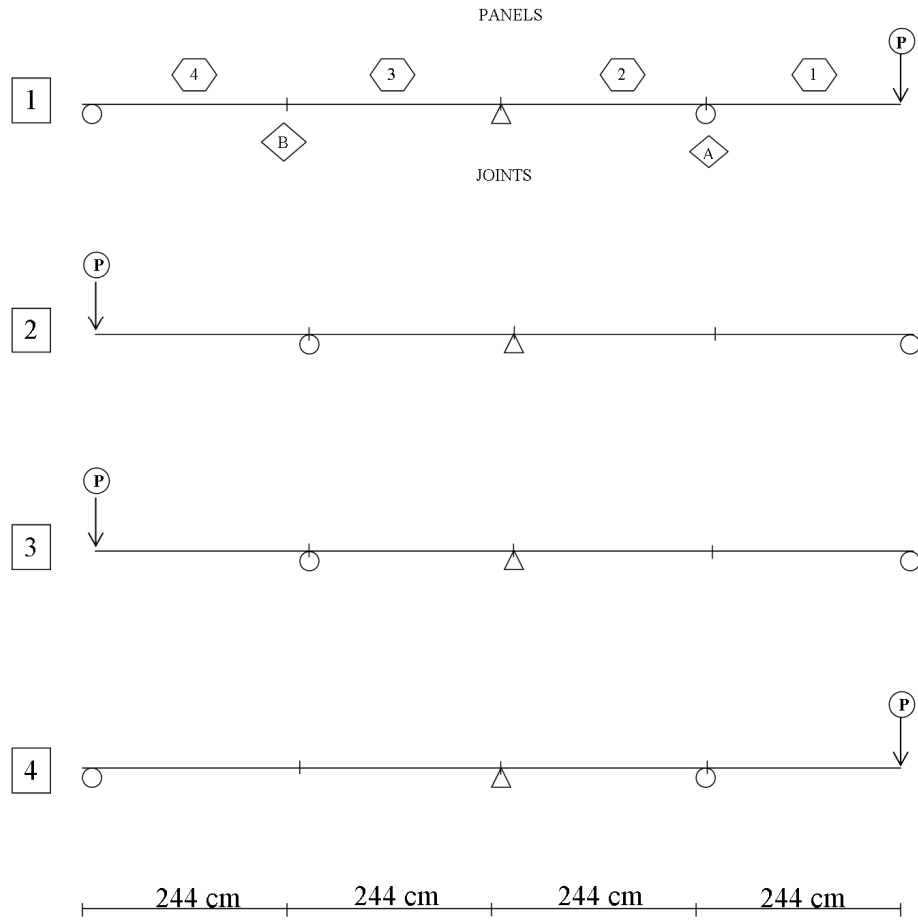


Fig. 2 Test setup



Fig. 3 Picture of the elevation view of test setup

Fig. 7 is a picture of the end of the specimen looking down the length of the girder. Here the load cells measuring the load from the hydraulic rams and the string pots measuring the deflection directly under the load are seen. Four string pots were connected to the specimen to measure deflection under the load and also the uplift measured at the girder serving as an uplift restraint. The opening of the transverse joint was also monitored by a string pot.

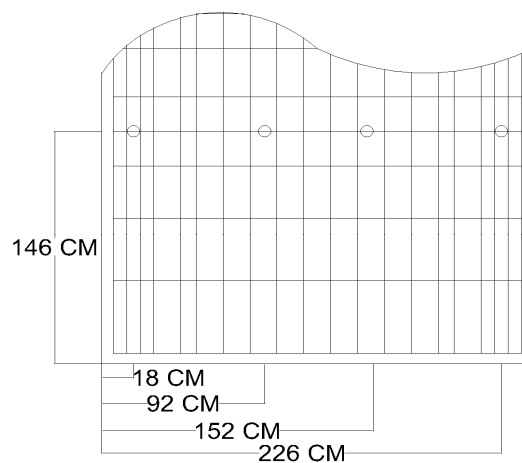


Fig. 4 Strain gage location detail

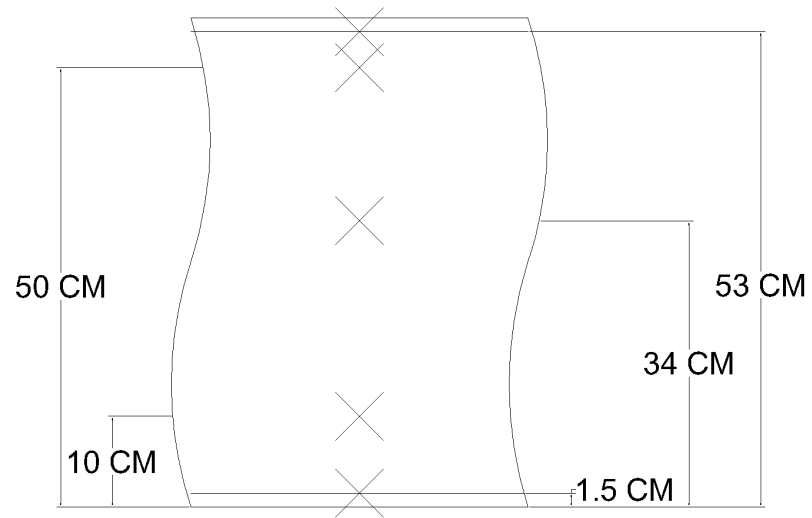


Fig. 5 Location of strain gages installed on girders



Fig. 6 Picture of strain gages installed on girder

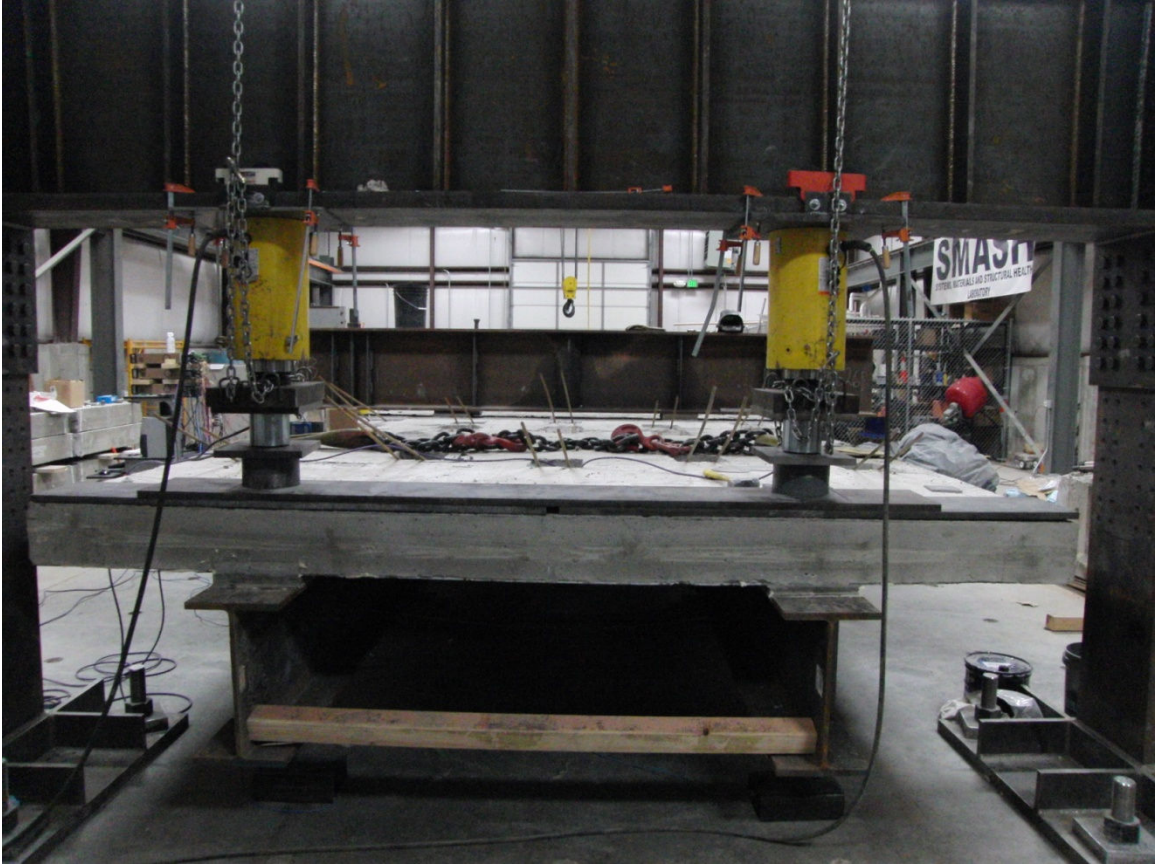


Fig. 7 Picture of end view of test setup

Previous research reported a cracking moment of 353 kN-m (260 k-ft) (Wells 2012). Therefore, a control data set was obtained by inducing a moment of 307 kN-m (225 k-ft) on the specimen. Testing began with test setup 1 with joint A being induced and control data was recorded.

Table 1 Testing Schedule

Test Setup	Control or Post Replacement (PR)	Joint	Moment Induced (kN-m)
1	Control	A	307, Cracking
2	Control	B	307, Cracking
3	PR	B	307, Cracking, 1024, 2048, 3072
4	PR	A	307, Cracking, 1024, 2048, 3072

After the data was obtained the test apparatus was moved to setup 2 to obtain the control data for joint B. A similar moment was induced on the joint B for the control data. Table 1 contains a testing schedule for the induced moments.

Initial Construction Test Results

Control data was obtained from testing of differing loads. Initially, the joints were induced to a moment just below the reported cracking value. The data from this test was used to obtain a load vs. deflection curve also strains were measured in the panel to determine locations of likely cracking. Lastly, the neutral axis was measured to compare to theoretical values. A second phase of testing was performed on each joint to determine the cracking moment. Data was also collected to compare load vs. deflection curves.

Fig. 8 shows the cracking loads for both joints A and B.

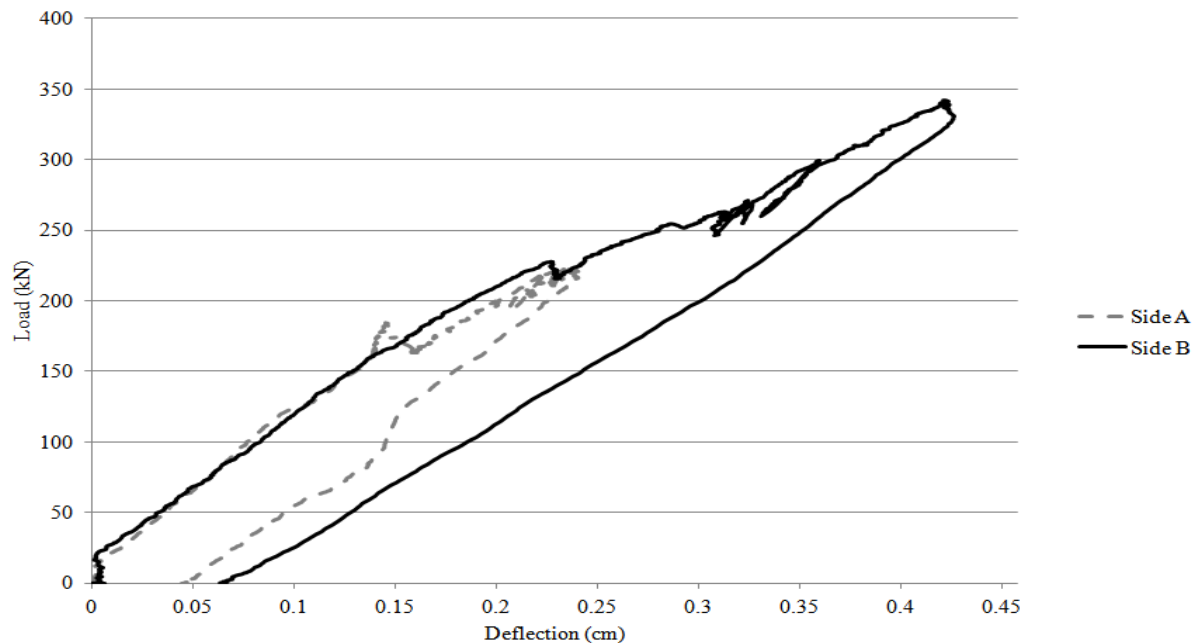


Fig. 8 Cracking load

Visual observation, an audible cracking, and data analysis determined cracking. The data shows that as the load increases on the specimen there is a point where the load decreases but the deflection continues to increase. This is the reported value for the cracking moment. For Side A the reported cracking moment is 427 kN-m (315 kip-ft) and for Side B 580 kN-m (427.5 kip-ft).

In a traditional concrete structure, a reduction in stiffness happens after the cracking moment is reached. Fig. 8 shows that even though the cracking moment was reached and the signs are in the data and through observation, the slope of the load versus deflection curve remains mostly constant. Even though the section is cracked the post-tensioning allows the member to maintain stiffness. Applying the control load both before and after cracking confirms that the stiffness of the section remains constant. The load versus deflection curves if adjusted for permanent deflection are similar.

Fig. 9 compares the load versus deflection curves for the control data obtained from both joints and previous research performed by Wells (2012) and the ANSYS computer modeling by James (2012). This figure shows that the joints behaved more closely to the model and showed more stiffness than the previous research.

The neutral axis of the specimen was calculated from theoretical calculations. As the section begins to yield the neutral axis will move closer to the neutral axis of the steel girders, which shall fail last. Fig. 10 shows the data from the strain gages on the girders that were used to locate the neutral axis. A linear trendline was fit to the data and the equation was displayed to obtain the y-intercept or the neutral axis location. Both axes had similar locations for the elastic neutral axis. Fig. 11 contains a microstrain distribution for control values for both sides of the test specimen.

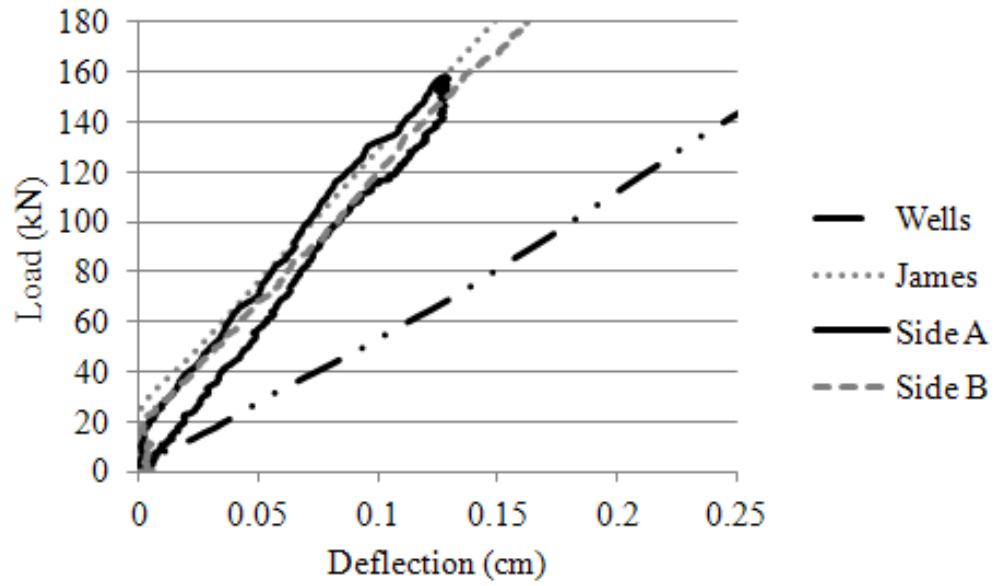


Fig. 9 Control data comparison chart

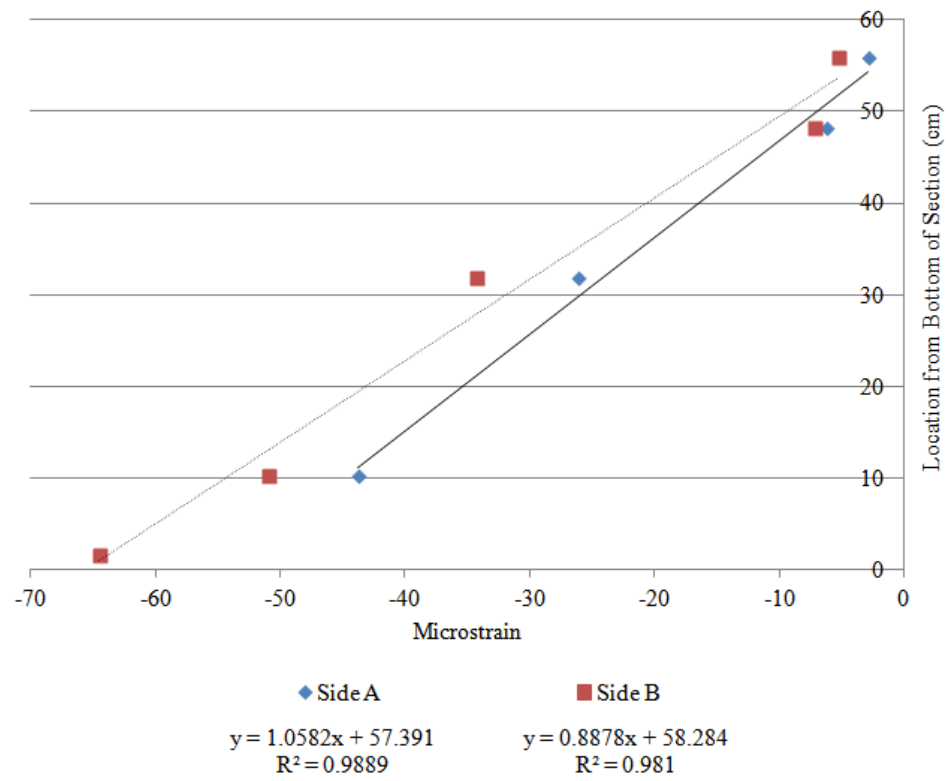


Fig. 10 Control data neutral axis location chart

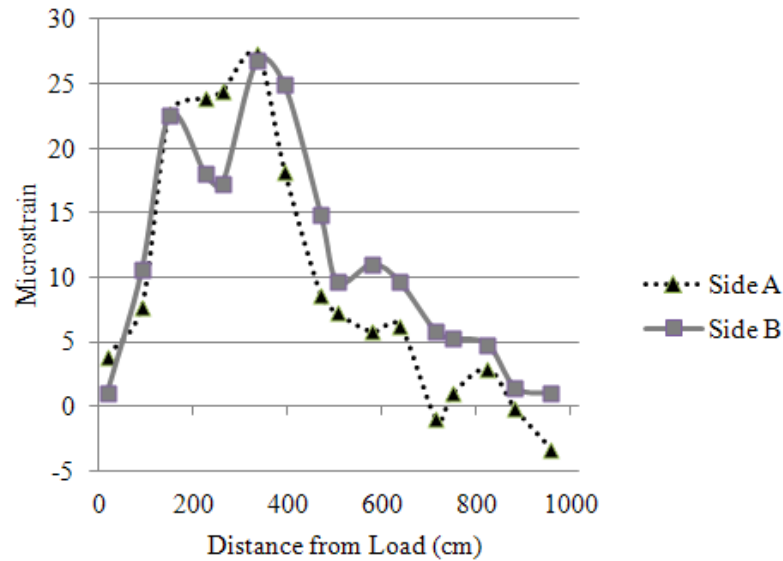


Fig. 11 Strain distribution for control data

Strain distributions show likely cracking positions. The strain distributions have two locations of strain peaks at 1500 mm (60 in.) and 3350 mm (132 in.) from the load location. These positions correspond with the location of the post-tensioning pockets in the deck panels. In a prismatic beam the max strain location should correspond to the location of maximum moment. The data shows a lower strain next to the joint than next to the panels. The post-tensioning induces a negative strain on those bars therefore, they read a lower strain than the gages outside the post-tensioning area.

Comparison to Previous Research and Theoretical Values

By comparing the values for the control data shows that the specimen was performing at expected values. The cracking moment for the control data set was found to be 427 kN-m (315 kip-ft) for Side A and 580 kN-m (427.5 kip-ft) for Side B. Wells (2012) reported the cracking moment for the section to be 475 kN-m (350 kip-ft) while the ANSYS model reported cracking at 514 kN-m (379 kip-ft). Showing that the

specimen was showing an expected behavior. The load vs. deflection curves for the joints showed act more stiff than previous laboratory research and were similar to the ANSYS models reported stiffness. Table 2 contains a summary of specimen behavior versus previous research.

The test specimen had joint cracking at similar loads as previous research, Joint A cracked at 90% of the load while Joint B cracked at 22% load increase. Deflection was measured to be 43% and 47% less than previous laboratory testing. The deflections for both joints were almost identical to that predicted by the ANSYS model. The computer model cracked at an 8% increase compared to the previous research. The cracking moment was determined by a change in slope in the load versus deflection curve for the test for this investigation and reported values from previous research.

Theoretical calculations were computed for the test specimen using a transformed section analysis. The components of the specimen were transformed into equivalent area of concrete for moment of inertia calculation. Table 3 contains a summary of theoretical versus measured values for the control data.

Table 2. Control Data Comparison

Research	Cracking Moment (kN-m)	Deflection (cm) at 133.5 kN
Wells	475	0.23
ANSYS	514	0.1
Joint A	427	0.1
Joint B	580	0.11

Table 3. Control Data vs. Theoretical Calculations

Test Setup	Cracking Moment (kN-m)	Deflection at 133.5 kN (mm)	Neutral Axis Location from Bottom of Section (cm)
Theoretical	970	0.61	57.4
1	427	1.0	57.4
2	580	1.1	58.3

Both joints cracked before the theoretical cracking moment. This is attributed to the definition of cracking for this research was the opening of the joint, which resulted from the grout in the transverse joint debonding from the concrete of the adjacent deck panels instead of rupturing the concrete of a monolithic pour. The maximum microstrain recorded from the test specimen was 27, the required strain to crack the concrete is 131 microstrain. Therefore, concrete rupturing was not observed and should not have been observed.

The test specimen deflected more than expected based on theoretical calculations. The calculations were based on a single prismatic member; the test specimen was constructed of different materials with different properties. The analysis was a transformed section that assumes full composite action between the members. Even though composite action was achieved because of the installation of the nelson studs the section did not appear to act fully compositely therefore had a larger deflection than calculated. The elastic neutral axis of the specimen was reported to be very similar to the theoretical values.

Panel Replacement Feasibility

Upon completion of initial testing, panel two was removed and replaced. The first step was to remove the post-tensioning. The post-tensioning cables were relaxed by heating them thus allowing the chucks to be cut and the cables removed. An acetylene torch was used to cut the chucks and the cables. Fig. 12 shows the chuck being cut. Fig. 13 shows the cable and chuck after cut and removed. Each panel was connected to the steel girders by six shear pockets. To remove the shear pockets an electric jackhammer was used to remove the grout from the shear pockets on the panel. The nelson studs in each pocket must have all existing grout removed from them thus allowing the new panel to achieve composite action. Fig. 14 shows the shear pocket after the grout removal. Transverse joints between panels had shear keys on them to increase the shear capacity of the joint. Care was used to remove the grout from the panels without damaging the shear key. The jackhammer was used to break the grout in the middle while the hammer drill was used to effectively remove the grout from the shear key without causing damage to the shear key. When the two transverse joints were cleared, an overhead crane was used to lift the panel off the girders and the haunch. The panel was lifted using lifting straps on either side of the panel connected with a chain. Upon removing the panel from the steel girders residual grout from the haunch was cleared from the bottom of the panels and the girder. The panel was replaced by lifting a new panel of similar construction with the same lifting method. Fig. 15 shows the specimen with the panel removed. Overall, the panel removal required 45 man-hours of labor, using one jack hammer and one hammer drill.



Fig. 12 Chuck and cable being removed



Fig. 13 Cable and chuck after being removed from specimen



Fig. 14 Shear pocket after grout removal



Fig. 15 Specimen with the panel removed

Alternative methods for removal would reduce the time. One method would be using a concrete saw to cut the transverse joint, therefore allowing the residual grout to be removed more easily. The benefit of single panel replacement is that only one panel was removed as opposed to removing all the panels with the current deck systems used by UDOT.

The installation of the new panel was similar to the original construction. The panel was placed over the nelson studs according the construction drawings. The transverse joint was formed and grouted using Masterflow 928 grout. Once the grout had cured the cables were again stressed to 156 kN (35 kips) after instantaneous losses. Upon completion of the cable stressing, the shear pockets and haunch were grouted in. Fig. 16 shows the panel being replaced and Fig. 16 shows the joint being grouted.



Fig. 16 Panel being replaced



Fig. 17 Grouting of replaced joint.

During stressing, an observation was noted about the difference between the original construction and the rehabilitation. Initially, the panels were not connected to the girder therefore free to move with the stressing, but after the reconstruction three of the panels were acting compositely with the steel girders. Thus, stressing was resisted by the shear pockets which may have affected the ability to get full pressure along the joints being stressed. Fig.18 shows the cables being stressed after panel replacement.

Post Replacement Test Results

Upon completion of the panel replacement and deck system reconstruction, the test apparatus was positioned to test setup number 3. As shown in the testing schedule the specimen was tested to similar loads as the control data and also the additional loads of 445, 890 and 1335 kN. Upon completion of testing Joint B, the apparatus was moved to test Joint A.



Fig. 18 Cables being stressed post panel replacement

The purpose of this research was to investigate the effect of single panel replacement on the bridge deck system. The data received from test sessions 3 and 4 was compared to the initial values for the specimen obtained during test sessions 1 and 2. Fig. 19 shows the stiffness comparison between the initial construction and the post replaced system.

The load versus deflection curve for Joint B after the panel replacement is almost identical to the load versus deflection curve obtained initially. The same slope was recorded as well as a very similar deflection during loading. However, Joint A had observed a significant reduction in stiffness as shown in Fig. 8. One of the reasons for the reduction in performance may be attributed to the difficulty in obtaining full pressure across the joint. The cables were stressed to the same level as before but the shear pockets from adjacent panels may have reduced the effective pressure across the joint.

Fig. 20 contains a strain distribution for the specimen in both testing sessions. Side B demonstrated a similar strain distribution as the control data. The difference between the control data and the post replacement measured a higher strain than the control data set. The strains for Joint A are showing the same trend of peak values in the same positions. The values are lower than the control values. There is an additional peak that appears in panel 3; this is attributed to the Joint B already having been induced to higher loads thus the cracking of the concrete at that location caused an higher strain in the steel reinforcement.

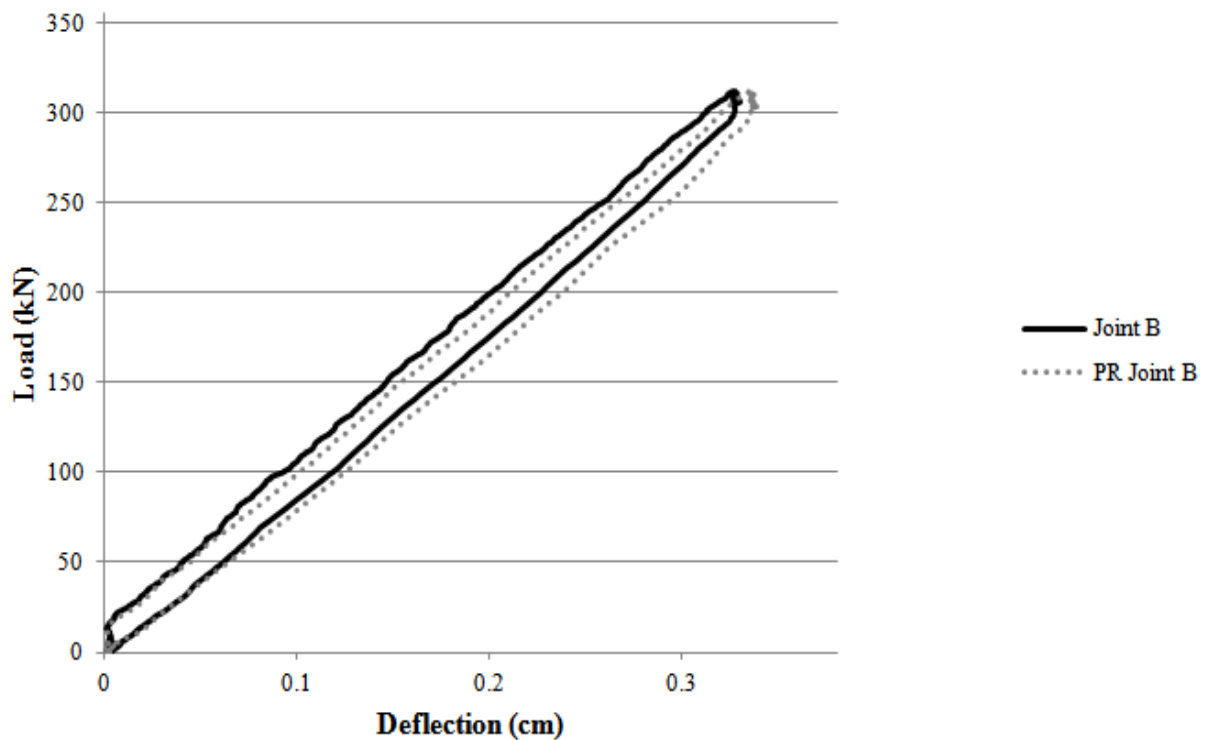


Fig. 19 Control vs. PR load deflection curves

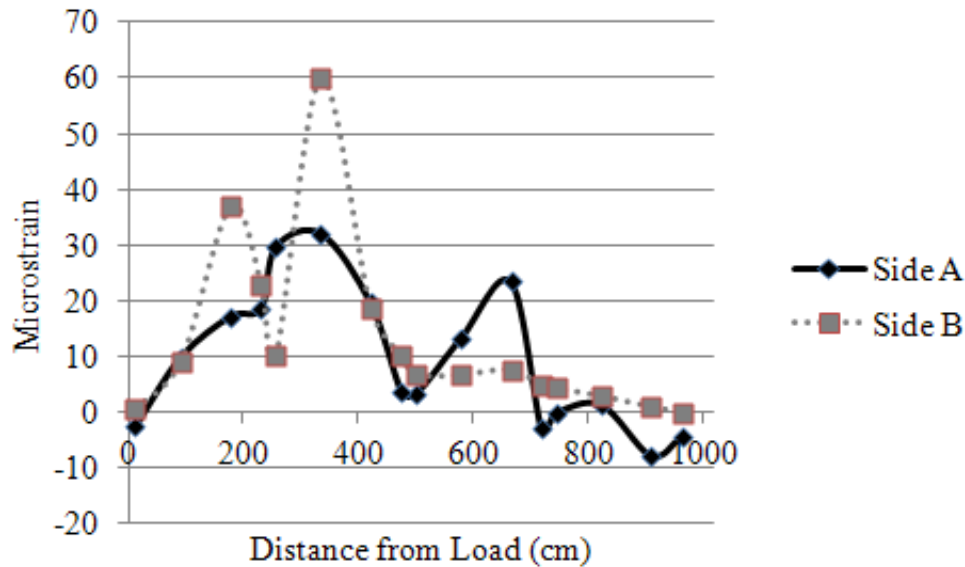


Fig. 20 Post-replacement strain distribution at 133.5 kN

Strain gages installed on the girders were used to locate the post-replacement neutral axis of the specimen at a moment of 308 kN-m (225 kip-ft). Fig. 21 plots the strain in the girder versus the location of the gages. The post-replacement neutral axis was determined by applying a linear trendline to the dataset and finding where it crossed the y-axis. The y-intercept is the neutral axis location. Side A had a neutral axis location of 310 mm from the bottom of the specimen while on Side B it was measured to be 375 mm from the bottom of the section.

The load versus deflection curves for the replaced panel tested were plotted against the reported data from previous research. The load versus deflection curves for the panels were obtained from the loading of the specimens to a negative moment of 2,660 kN-m (1950 kip-ft). Fig. 22 shows the values being compared to the reported values by Wells (2012) for the computer model and laboratory testing.

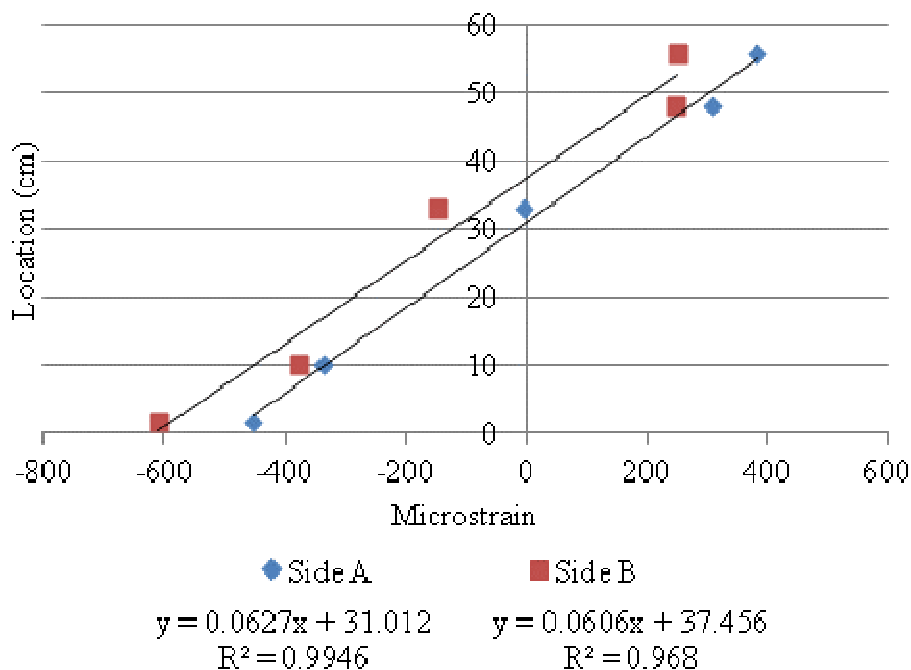


Fig. 21 Post-replacement neutral axis location

Even though Side A showed a reduced stiffness from the control values the behavior for both joints in the specimen, still perform comparable to the previous research. They are slightly lower than the computer model but a little more stiff than the laboratory testing. The increased stiffness as compared to previous research can be attributed to the increased load on the post-tensioning cables. Wells (2012) reported load in the cables to be 125 kN (28 kips), the loads in the cables of the specimen were measured to be 156 kN (35 kips) per cable. During testing visual cracking was observed in all panels between the post-tensioning pockets. The cracks propagated from one side of the panel to the other in zigzag fashion from one pocket to the next as predicted by James (2012) and shown in Fig 23. Cracking was also observed by the corners and above the bearing plates inside of the pockets and underneath the loading areas on the edge of the panels.

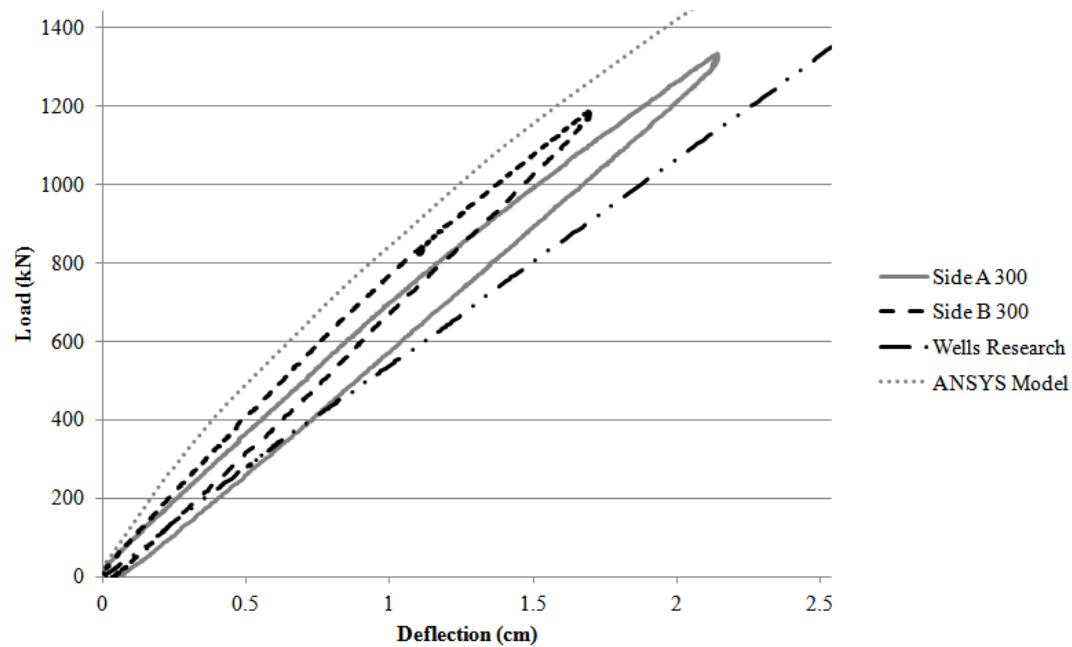


Fig. 22 Maximum moment comparison

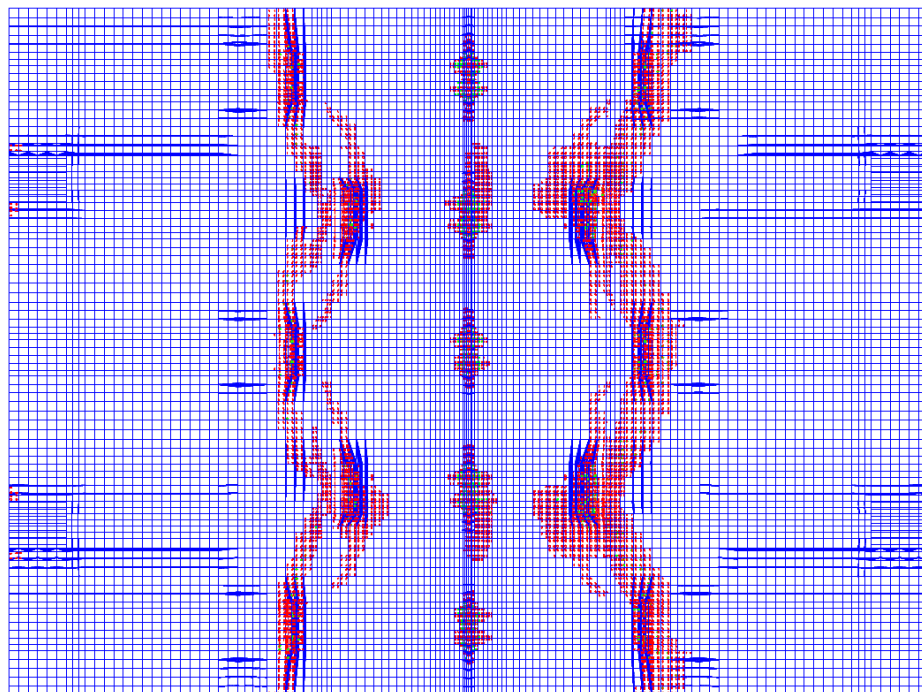


Fig. 23 Computer model crack propagation

Comparison to Previous Research and Theoretical Values

Calculations were performed to determine the plastic neutral axis of the section using the transformed section method. The plastic neutral axis was determined when the concrete had cracked and therefore the section had a reduced moment of inertia. This would move the neutral axis down into the web of the girders closer to the neutral axis of the girders themselves. The reported neutral axis location in Fig. 20 shows the neutral axis for Side A and Side B to be 310 mm and 375 mm, respectively. The theoretical calculations found the neutral axis to be 400 mm from the bottom of the section. Side B reported to be the stiffer joint therefore would resist a higher load before failure. This is proven because the neutral axis was higher in the section than that of side A. The neutral axis for the girders themselves is 270 mm from the bottom of the section. A strain distribution for the maximum moment was also plotted to identify locations of high strain and compare the measured strains between Side A and Side B. Fig. 23 plots the strains at the moment 2,660 kN-m (1950 kip-ft).

Both sides exhibited similar behavior to the induced moment. The locations of high strain were located at the post-tensioning pockets in the panels. The reported strains for Side A are significantly higher than the strains for Side B. This is attributed to the increased strain in the panel because of the stressing of the cables during rehabilitation. As mentioned, the adjacent panels were resisting the applied force of the cables through the shear pockets. Thus as the cables were stressed the panel was being pulled in both directions between the two sets of pockets resulting in a higher strain in the reinforcing steel. During testing cracking in panel 2 was more defined than the cracking in panel 3 due to the increased stress in the panel as shown in the Fig. 24.

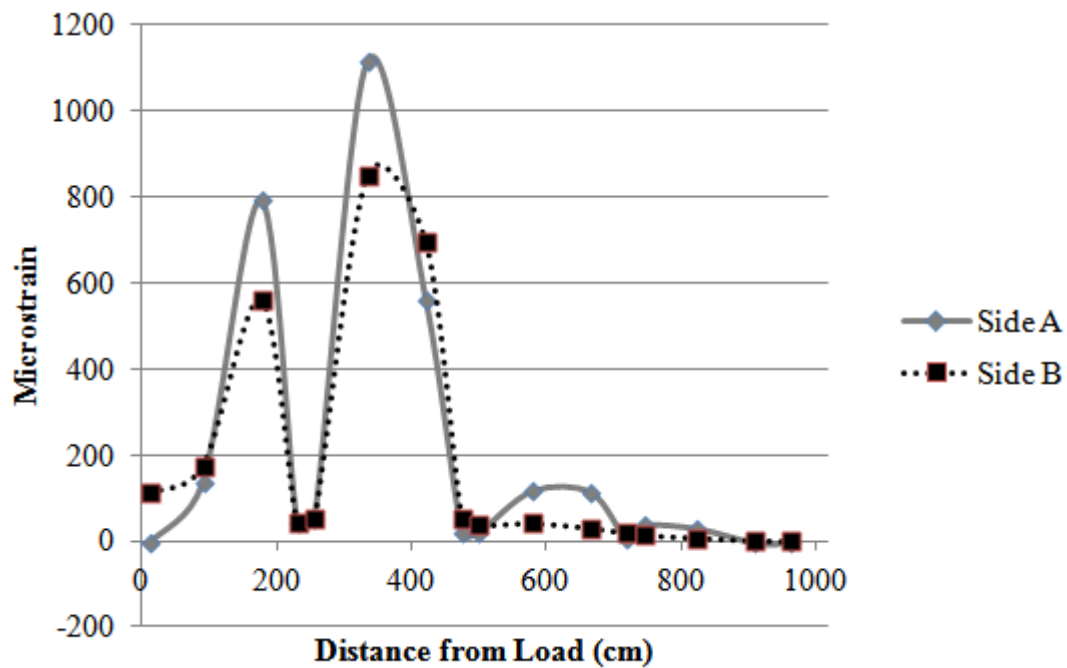


Fig. 24 Maximum moment strain distribution

The specimen was found to perform adequately compared to previous research and current post-tension systems used by UDOT. The deck system was stiffer than the research reported by Wells (2012) this is due to the increased pressure along the joints because of the load in the cables.

CHAPTER 4

CONCLUSION

The purpose of this research was to investigate the effects of single panel replacement on the entire bridge deck system. Due to data collected with comparisons to previous research and theoretical calculations the following conclusions have been made:

1. Conventional post-tensioning methodologies do not apply to the curved cable system. Current specifications indicate that stressing a cable to 80% of the yield strength will give an assumed 10 % due to instantaneous losses. This would leave a residual force in the cable of 70% of the yield strength. The laboratory tested showed that the curved cable connection has more rotation about the chuck and due to the shorter length in the cable the short-term losses are reduce the load in the cable to 66% of the original stressing load.
2. Single panel replacement increases stress in the concrete panels. The initial construction process of this research the stressing of the post-tensioning cables was performed before the shear pockets were grouted. This allowed the panels to move slightly and provided almost no resistance to lateral loading. After the single panel was replaced the other panels had the shear pockets grouted in. When the post-tensioning cables were stressed the adjacent panels were unable to move because of the lateral resistance provided by the shear pockets. Therefore, cables induced additional stress in the concrete panel that was evident by the 29%

increase in the strain in the steel reinforcing and the more defined cracking at similar loads as the initially constructed panel.

3. Single panel replacement is a viable solution and the bridge deck system will perform adequately as compared to other systems currently in use. The load vs. deflection curves when compared with previous research show that the replaced joint performed comparable to the reported values. The bridge had a higher stiffness and would fail at approximately the same load as the previously reported values. Therefore, even though single panel replacement does effect the joint capacity and the stress in the concrete, it will perform satisfactorily.

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