Post-Tensioning Design and Detailing for Transverse Connections in Adjacent Box Girders

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ABSTRACT

Precast prestressed adjacent box girders are regularly considered for short to medium span bridges on secondary roads. Many of these projects have limited funding and/or require rapid construction schedules. Secondary roads are also subject to a certain amount of public criticism, particularly with regard to cost and aesthetic appeal. Research has shown that an "appealing structure" can be defined as having continuously flat soffits and high span-to-depth ratios – characteristics of an adjacent box system. Proper design and detailing of the transverse connection between girders is essential to how much a given system can be considered desirable in terms of initial cost and long-term maintenance required to uphold the structure's aesthetic appeal. Typical connections include diaphragms and cast-in-place decks which directly increase cost and time of construction. In addition, an inadequate amount of post-tensioning is the primary source of reflective cracking leading to leakage of corrosive chemicals, subsequent reinforcement deterioration, excessive girder deflection, and an unacceptable possibility that the system will experience an unexpected failure due to diaphragms blocking inspection of the post-tensioning materials.

This paper presents the results of testing by the University of Nebraska-Lincoln on a post-tensioned transverse system which eliminates the need for diaphragms and a concrete overlay. Another important feature of this system is a "duct within a duct" connection that allows for easier inspection and repair of post-tensioning materials. Results indicate that the system maintains the anticipated compression pressure on the joint under fatigue loading, thus making it an excellent candidate for replacement of current practice where joint cracking is of concern.

Keywords: Adjacent box girder, Transverse connection, Post-tensioning

INTRODUCTION

Precast prestressed concrete adjacent box girders are a series of transversely connected, parallel spaced beams that extend the length of short to medium span bridges (El-Remaily[1] 1996). The connection between boxes is meant to develop continuous behavior across the combined girder width, which acts as the bridge deck perpendicular to traffic. As such, it plays a significant role in the distribution of loads, deflection between beams, and overall system capacity. Inadequate design of this connection can lead to reflective cracking, joint leakage, moisture buildup, corrosion, excessive displacement between girders, and sudden collapse. Improvement to the connection between adjacent box girders is required in order to ensure the safety and continued usage of such bridges.

The majority of failures in adjacent box girders have little to do with negligent design or construction. Instead, the problems stem from inadequate design criteria, poor drainage features, and difficulties associated with inspection. Such issues are easily preventable and separate from the overall system efficiency. This conclusion is made based on members acting independent of the whole when failures occur. Additionally, the majority of these issues can be solved simply by thinking of the system as a composite section rather than focusing on an even distribution of load. It is reasonable to assume that past connection methods involving quarter point diaphragms and structural/non-structural concrete overlays originated based on this simplified concept.

While standard details vary from state to state, most departments of transportation use a combination of diaphragms, structural/non-structural concrete decks, and post-tensioning to achieve continuous behavior between adjacent box girders. Of the three, diaphragms are the main source of frustration, specifically in terms of construction and maintenance. Skewed bridges are especially difficult since diaphragms must be staggered – meaning only a few girders can be connected at each location (Hanna[2] 2010). This use of multiple connection points increases the overall time of construction. When used in combination with structural/ non-structural concrete overlays, the system hardly benefits from the use of precast members.

Another problem with diaphragms is that long-term maintenance becomes difficult due to blocked access to the post-tensioning. Moisture release is also an issue since drainage ducts are often spaced inadequately between diaphragms. This leads to moisture buildup, which can result in corrosion of the hidden post-tensioning and eventually a sudden or unexpected failure. Even if the problem is found prior to collapse, getting to the post-tensioning is still an issue.

As a side note, moisture buildup is a direct result of reflective cracking between joints due to a lack of compressive post-tensioning force. Attempting to fix this issue by adding a non-structural concrete overlay so that cracks will not penetrate deep enough to reach the post-tensioning is both inefficient and uneconomical (Labib[3] 2007). A more effective solution would be to increase the post-tensioning force so that cracking did not occur in the first place.

Despite differing disadvantages for various transverse connections, the basic advantages to each system remain the same. For example, the ease and speed of construction for a given project is going to increase if concrete forming and placing operations are eliminated. In addition, adjacent box girders allow for shallower superstructure depths, which in combination with the hollow void, reduce member weight and provide extra space for utilities such as gas lines, water pipes, telephone ducts, and storm drains. All of this is done without compromising the high torsional stiffness of the section. Furthermore, the cost of construction and aesthetic appeal of a given bridge improves when comparing a boxed section with a continuous flat soffit against I-girders and similarly shaped alternatives (Hanna[4] 2009).

Past efforts by the University of Nebraska – Lincoln have updated the design criteria for adjacent box girders and analyzed the performance of a connection without diaphragms, concrete decks, and post-tensioning. The purpose of this paper's research is to investigate the effectiveness of post-tensioning, as determined by updated design charts, on the previously developed system. Post-tensioning is intended to keep the shear keys under compression, prevent reflective cracking at the joints, and create an even distribution of load. The experimental program for this system confirms its exceptional performance and applicability to current bridge applications.

LITERATURE REVIEW

Various practices are used in the design of adjacent box girder bridges. Standards differ worldwide, but the majority of the United States relies on publications by the American Association of State Highway and Transportation Officials (AASHTO) and the Precast/Prestressed Concrete Institute (PCI) as the primary reference to state and local bridge design specifications. The most current standards available from these organizations (with regard to adjacent box girders) are the AASHTO Load and Resistant Factor Design (LRFD) Bridge Specifications[5] and the PCI Bridge Design Manual[6].

The AASHTO LRFD Bridge Specifications provide little information on the subject of adjacent members beyond stating that "the use of transverse mild steel rods secured by nuts should not be considered sufficient to achieve full transverse flexural continuity unless demonstrated by test or experience." The manual recommends a minimum average effective post-tensioning pressure of 250 psi; however, the area of this pressure is unspecified and could be either the shear key or the girder contact face (El-Remaily[1] 1996).

El-Remaily et al[1] proposed a standard procedure in 1996 for the design of adjacent box beam bridges, which over time became the PCI Bridge Design Manual Section 8.9 Transverse Design of Adjacent Box Beam Bridges. His procedure is based on the theory that diaphragms are the primary mechanism distributing transverse loads and preventing differential displacement across the width of the bridge deck (PCI[6]). Post-tensioning is mainly used to provide continuous reinforcement along the deck width, as well as compression at the joints to avoid cracking and leakage. His calculations use an effective prestress area equal to the area of the diaphragm, which is conservative considering forces are also taken along the top and bottom girder flanges (El-Remaily[1] 1996).

El-Remaily developed a chart for his procedure that estimates the level of post-tensioning required to fulfill the requirements of AASHTO in 20 to 80 feet wide bridges with zero skew and box depths of 27, 33, 39, and 42 inches. Bridges not meeting this criterion must undergo a more detailed analysis. While El-Remaily's theory is sound, it is also based on AASHTO Standard Bridge Design Specifications – a document published prior to the AASHTO LRFD Bridge Specifications. By comparison, the former uses service loads that are un-conservative in relation to the later.

The University of Nebraska-Lincoln was involved in El-Remaily's research and has continued to investigate methods of improving the connection between adjacent box girders. As stated in the introduction, previous research by the university investigated the behavior of a system that eliminates diaphragms and is able to evenly distribute loads without a deck or post-tensioning. These experiments involved high strength steel rods placed at 4 ft and 8 ft spacing in box girders with wide and narrow joint connections, respectively. The wide joint (WJ) connection involved a shear key that extended the full depth of the box girders and provided top and bottom ties protected by spiral reinforcement. The narrow joint (NJ) connection involved a shorter shear key located just below the top flange and provided top and bottom reinforcement joined by couplers (Hanna[2] 2010).

Results of experiments on the performance of the two connections proved positive in comparison to a current connection taken from an unidentified department of transportation website. The detail eliminates post-tensioning at intermediate and end diaphragms, replacing it with a single mid-depth tie for reinforcement and 5 inches of structural concrete overlay. The current connection was tested with and without this overlay. With the deck, the connection exhibited a lower capacity than the UNL connections. This is primarily due to the reduced amount of reinforcing and tie placement. The system also experienced immediate failure when tested without a concrete deck, thus indicating the superiority of the proposed systems.

Along with the above research, finite element analysis was used to develop charts to estimate the transverse tension force of the two new systems for various bridge widths and span to depth ratios (Fig. 1 and Fig. 2). These charts were used to establish the post-tensioning force required for the newly proposed system. Post-tensioning was incorporated into the former systems in order to provide joint compression, simplify construction operations, and improve inspection of materials.

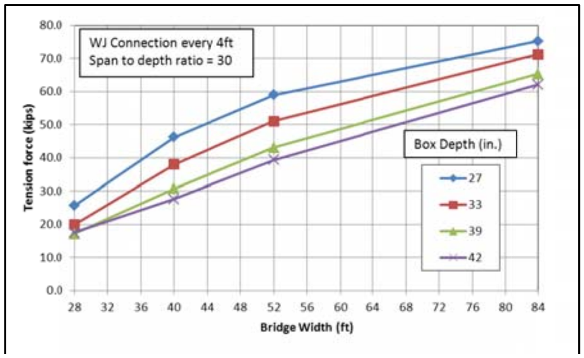

Fig. 1 Required transverse tension force for WJ connection

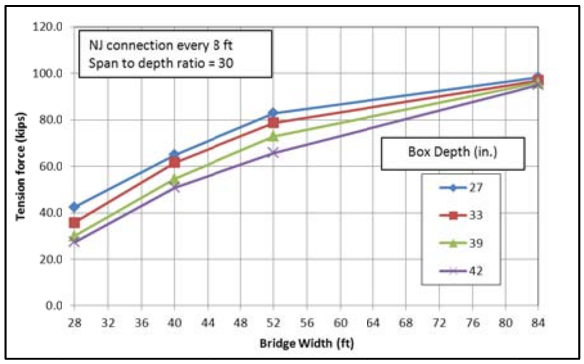

Fig. 2 Required transverse tension force for NJ connection

EXPERIMENTAL PROGRAM

The box girder connection for the experimental program is based on modifications to the narrow joint system described in the Literature Review and developed by Hanna et al[4]. The modified system still eliminates the use of typical diaphragms and a concrete overlay for load distribution; however, post-tensioning was added for investigation. Reinforcement remains at an 8 ft spacing and section dimensions are the same as an AASHTO 27 inch deep box girder (
Fig. 3). The modified system was tested based on the fatigue loading of a 52 ft x 64 ft x 27 in. bridge, which was scaled down to a 8 ft x 8 ft x 27 in. specimen.

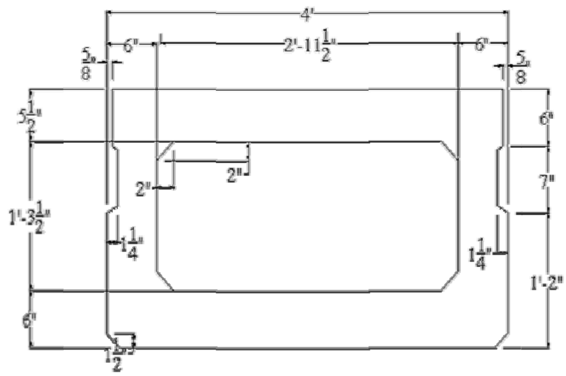

Fig. 3 Box Girder Dimensions

MATERIALS AND CASTING PROCEDURES

Reinforcement is shown in Fig. 4. Strands were replaced with 16-#4 bars for testing. The main detail to note is the “duct within a duct” system used to install post-tensioning. The exterior duct is meant to maintain a void in the concrete during casting of the individual girders. The interior duct keeps the reinforcement from bonding to the girders during shear key placement. Fig. 5 is a close-up of this detail and Fig. 6 is the complete specimen for testing.

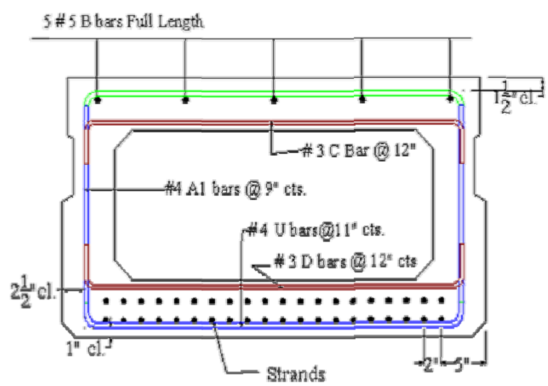

Fig. 4 Box Girder Reinforcement

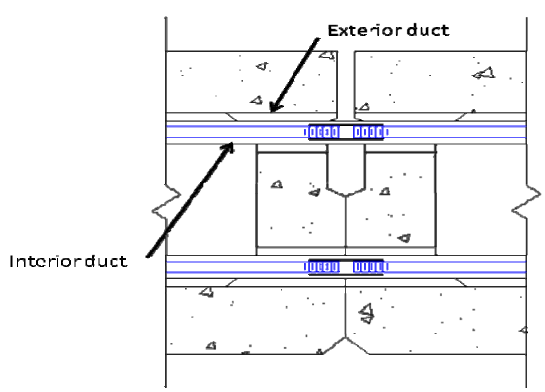

Fig. 5 Detail of Duct within Duct Setup

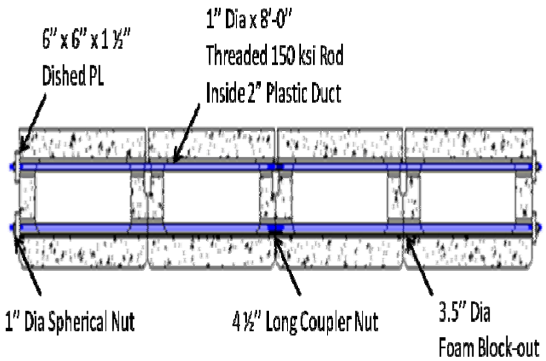

Fig. 6 Test Specimen

Fig. 7 and Fig. 8 represent the girders that were formed and cast at the UNL lab on the Omaha campus. An unforeseen issue with bearing on the end girders required a second form setup to grout the exterior shear keys. Grout was required to have at least 8 ksi strength and ended up reaching a 28 day strength of 10 ksi. This was important so that the girders did not fail at the bearing locations. It should be noted that with time the grout at the bearing location experienced excessive cracking, however the actual specimen had only a few cracks due to shrinkage. The grout cracked because it was confined to a thickness of less than 1 in. and was unable to be kept moist for curing due to forming constraints.

Fig. 7 Side View of Box Girder

Fig. 8 Fully Cast Box Girder

The final step in forming the specimen was to position the box girders adjacent to each other so that preparation could begin for pouring the interior shear keys. A continuous PVC pipe was inserted into the duct to keep the hole open for post-tensioning. Post-tensioning was tightened to 5 kips before grouting. This was done as a way to get the boxes as close together as possible before grout was poured. Post-tensioning materials include a 1.5” x 6” x 6” dished plate, spherical nuts, couplers, and 8 ft – 1 in diameter threaded rods with 150 ksi ultimate strength. Grout is a semi-fluid, non-shrink mix that does not contain corrosive chemicals which could pose a risk to the post-tensioning.

Fig. 9 shows the complete specimen immediately after grouting. Once the grout reached 8 ksi, the post tensioning was jacked to 0.7fpuAps or 90 kips according to AASHTO specifications. Jacking started at the top and was done in cycles so that the difference of force between ties was no greater than 30 kips per cycle.

Fig. 9: Specimen after Grouting Shear Keys

TEST LOAD DETERMINATION

The loads for testing were found by developing 3D models of the box girder system using SAP2000. The first model was used to determine the axial force produced by ultimate and fatigue loads in the transverse direction of a 52 ft x 64 ft x 27 in bridge (Fig. 10). Bridge loads include an assumed curb and rail load equal to 0.48 k/ft and 12 foot lanes with standard

HL-93 design vehicular and fatigue truck loading. The weight of the structure is uniform and does not affect the transverse direction, thus it was not included in the design analysis. Curb and rail loads are applied to the exterior edge of the exterior box girders. Ultimate loading conditions include single and multiple lanes, while fatigue conditions consider a single lane only. Single lanes are placed at the center of the bridge width to create max tension in the bottom flange of the box girders and at the edge to create max tension in the top flange.

LRFD specifications include a 33% dynamic load factor for ultimate analysis and a 15% dynamic load factor for fatigue analysis. Long term affects are accounted for with an infinite life factor of 2 for fatigue. This translates to LRFD load combinations of 1.25D + 1.75L for ultimate and 1.25D + (0.75)(2)L for fatigue.

Ultimate axial and fatigue demands were 80.3 kips and 15.3 kips, respectively. The ultimate demand is comparable to the estimated effective post-tensioning force of 82 kips required by the previously developed design charts. The 82 kips is based on a span to depth ratio of 30, while the SAP2000 model represents a span to depth ratio equal to 28.4 (Ex: 64 ft / 27 in = 28.4). The post-tensioning system for the experiment is a threaded rod with yield strength of 120 ksi. AASHTO estimates the effective force per duct after losses to be 0.80fpyAsp = 0.80 x 120 ksi x 0.85 in^2 = 81.6 kips. This is enough to handle the anticipated loads.

The second model developed was used to determine the loads required to reach fatigue and ultimate capacity of the experimental 16 ft x 8 ft x 27 in system. Modeling is different from the large span bridge in several ways. First it should be noted that the supports are along the transverse direction, thus making the deadweight of the structure act in the transverse direction. Bottom tension is created by a simple span setup with load at the center connection between middle girders, while top tension results from a simple span with cantilever setup where the load is applied at the connection of the two cantilevered end girders (Fig. 11). Supports are assumed at 7 in from the edge of each exterior girder for bottom tension and 7 in from the end with supports at the connection between the two middle girders for top tension. An additional support is placed at the center of the top flange near the end support in order to represent the actual lab conditions.

Analysis showed that the axial force due to dead weight for the bottom and top tension setups were 4.9 kips and 21.4 kips. A load equal to 17.4 k brings the axial force to fatigue conditions for the bottom tension setup; however, the fatigue load is already surpassed by the top tension setup. Fatigue for this condition will instead be represented by a fatigue truck wheel load equal to 18.4 kips. This creates a tension in the top flange equal to 51.6 kips. This is acceptable since the value is less than the ultimate axial force.

Model behavior for both systems is based on the following:

- Elements correspond to the centerline of the actual structure.
- Shell elements act together as a composite box section. The shells are defined with a thickness specific to the top flange (5.5 inches), bottom flange (6 inches), and web (6 inches).

- Shells are 12 inches in the direction of traffic. Flanges are divided into 6 sections and webs into 4 sections.
- Frame elements connect the boxes at the center of the top and bottom flanges. These elements are the same depth as the corresponding flange and have a width equal to the full span of the girders (8 feet).
- Concrete strength is 8 ksi.

Ultimate capacity of the 16 ft x 8 ft x 27 in specimen was determined by strain compatibility and found to be 280 kip-ft. A single point load causing positive moment on a simply supported beam is equal to 4x the moment divided by the span length (i.e. M = PL/4). Span length, assuming a 4 inch bearing on both sides, is 15ft 4 inches. This relationship indicates that a total load of 73.0 kips will induce failure. Part of this load is due to the weight of the box (approximately 1.5 klf). The corresponding moment due to weight would be 48.5 kip-ft. As such, the actual load to cause failure would produce 231.5 kip-ft of moment. Solving for the load gives a 60 kip applied force to break the joint between adjacent boxes.

The loads to apply for testing in order to represent design conditions were determined by iteration of an applied load to the 16 ft x 8 ft x 27 in model until the resultant moment in the critical section of the model was equal to the moment produced by the large scale model. A summary of results is provided in Table 1.

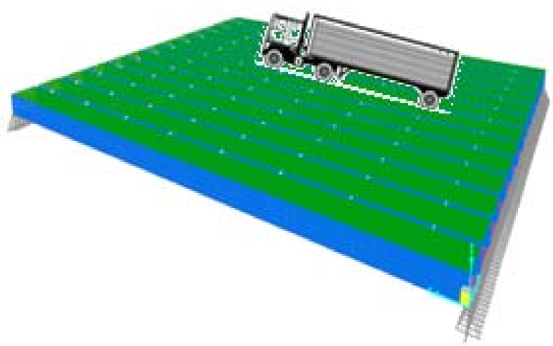

Fig. 10 52’x64’x27” Adjacent Box Girder SAP2000 3D Model

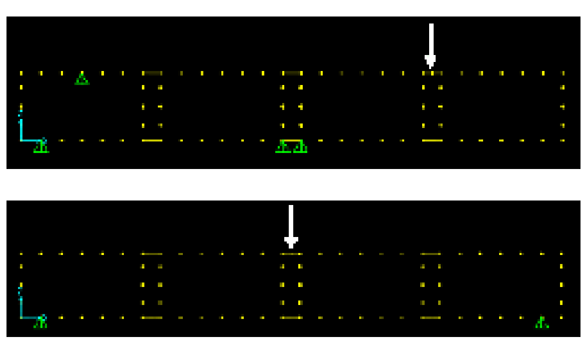

Fig. 11 SAP2000 3D Model for Top & Bottom Tension Support Setups of Specimen

Table 1 Results for (a) Full Scale Behavior, (b) and (c) Applied Loads for Specimen Testing

(a)

Full Scale Behavior (52' x 64' x 27'' Bridge)	**Fatigue**	**Ultimate**
Max Axial Forces over Bridge Lifetime	15.3 k	80.3 k

(b)

Applied Fatigue Load (16' x 8' x 27'' Specimen)	**Axial Force due to Weight**	**Applied Load**	**Total Axial Force**
Top Tension Setup	21.4 k	18.4 k	51.6 k
Bottom Tension Setup	4.9 k	15.3 k	15.3 k

(c)

Applied Capacity Load (16' x 8' x 27'' Specimen)	**Weight**	**Applied**	**Capacity**
Moment	44.1 kip-ft	235.9 kip-ft	280.0 kip-ft
Load	23.0 k	61.5 k	84.5 k

FATIGUE AND ULTIMATE TESTING

The final stage before proceeding with the experiment was to setup the actual test frame. Both top and bottom tension were tested for the specimen. This requires two different setups of the test frame. Top tension is the critical case since it results in cracking at the top of the member, which is the primary concern for exposure to enviromental corrosion. Cracks were monitored by leakage from a dam built arround the middle joint, as well as white latex paint to aid with visability.

Beam supports for both cases are placed at stiff locations – or in other words – at webs and joints. For top tension, the supports go at the middle joint and edge of the specimen so that a load can be applied at the center of the connection between the two cantilevered box girders. Bottom tension is the result of supports at either end with a load at the center. A neoprene bearing pad was placed between the specimen and the beam to provide additional flexability to the system.

The specimen was able to handle its own weight as it was lifted to the intial top tension test frame. No visible damage took place. After placement, a beam had to be set on top of the specimen in order to prevent it from lifting off its support under loading. This beam was placed as close to the end as lab conditions would allow. Furthermore, no visible damage took place when the specimen finished the top tension test and was lifted again in order to rearrange the supports for the bottom tension setup.

An actuator was used to apply 5 million cycles of loading to a 15 in x 15 in bearing plate on the specimen. Five strain gages were placed to monitor changes in strain near the load and middle joint. Pictures of the test frame setups can be seen in Fig. 12 through Fig. 14.

Fig. 12 Initial Specimen Lifting

Fig. 13 Actuator at Exterior Joint for Top Tension Setup

Fig. 14 Actuator at Interior Joint for Bottom Tension Setup

After fatigue testing the specimen was moved onto concrete blocks so that it could be tested for ultimate capacity. Bearing was provided by 4 in by 8 in planks at each end of the specimen. The specimen did not experience any cracking during lifting and placement. A loading cell was placed directly over the center joint to produce tension once more in the bottom flange (Fig. 15). A deflection gage was placed beneath the specimen to take measurements along with the five strain gages on top.

Fig. 15 Actuator at Interior Joint for Ulimate Capacity Setup

RESULTS & DISCUSSION

TESTING RESULTS AND ANALYSIS

The box girder experienced a deflection of 0.10 to 0.12 inches during the top tension fatigue test. No cracking occurred in the joint; however, cracking was noticed in the longitudinal direction of the box girder top flanges. These cracks occurred before loading and are due to shrinkage where wood 2x4s were used to brace the sides of the specimen during casting. Testing continued since the focus of the test is on the joint and not the flange. It should also be noted that the dam began to leak over the joint near the end of the test. This formed a water stain that makes the specimen joint appear failed when it is not. Once the leaked water had dried and the dam was refilled, no other water formed along the specimen joint to indicate the connection had developed a crack.

Strain gage data taken from 3 locations on either side of the joint (center and edges) suggests that the stress on the box girder is the same before and after top tension loading. Two of the gages had overlapping initial and final strains. These occur under the load and on the North-West side of the joint. The strain gage on the South-East side of the joint initially read nearly half the average value of the other strain gages (36 $\mu\varepsilon$). At the end of the test; however, this strain gage had increased to the 36 $\mu\varepsilon$ average. The North-East strain gage also doubled, suggesting that the load was being unevenly distributed to the East. The last strain gage broke due to water submersion and accurate data was unable to be obtained from the specimen.

As mentioned earlier, leakage was noticed along the flange of the box girder. This leakage occurred mostly at a crack on the North-East side and parallel to the joint. Smaller leaks were due to cracks located North-West and South-West of the joint. These cracks occurred where the top boards for bracing had been placed during the initial casting of the individual girder specimens (perpendicular to the joint). Since the measurement on the west side did not show any strain change, it is reasonable to conclude that parallel cracking is the cause for strain change on the East side. It can also be inferred that the crack caused some strain change in the specimen that otherwise would not have occurred. Strain gage data is shown in Fig. 16. Fig. 17 and Fig. 18 show the North-East crack and leakage.

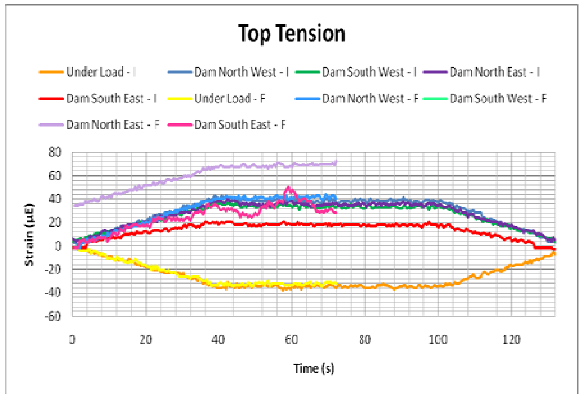

Fig. 16 Initial and Final Strains on Specimen (Top Tension)

Fig. 17 Interior View of Cracking at Flange along North-East Side of Joint

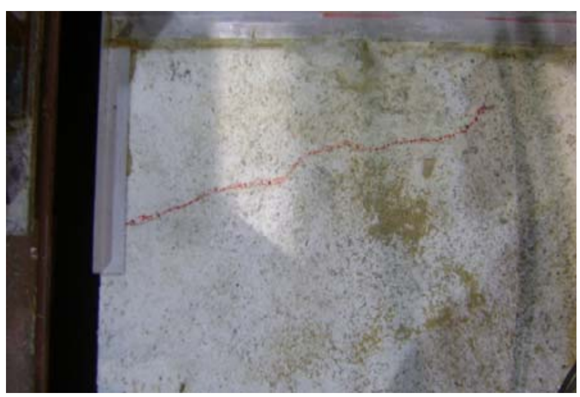

Fig. 18 Exterior View of Cracking at Flange along North-East Side of Joint

The box girder experienced a deflection of 0.08 to 0.11 inches during the bottom tension fatigue test. No cracking occurred in the joint and leakage decreased since the top flange was under compression.

Again, strain gage results suggest zero change of strain. Two of the gages had overlapping initial and final strains. These occur under the load and on the South-East side of the joint. The strain gage on the North-West side of the joint decreased approximately 5 με, while the strain gage on the North-East side increased approximately 5 με. The South-West strain gage appears to overlap, however the initial readings show a change of strain when the load is constant. This is probably because the strain gage should have been replaced after the top tension test. The adhesive for the strain gages did not hold up well after being submerged in water a second time. Both North strain gages and the South-West strain gage had to be replaced before obtaining their final readings. Once more, results suggest that the parallel crack caused the changes in strain. It makes sense that the North-East gage would experience an increased compression strain because it is on the compression side of the crack, while the North-West gage would lose compression strain as a result of the increasing tension on the other side of the crack. Strain gage data is shown in Fig. 19.

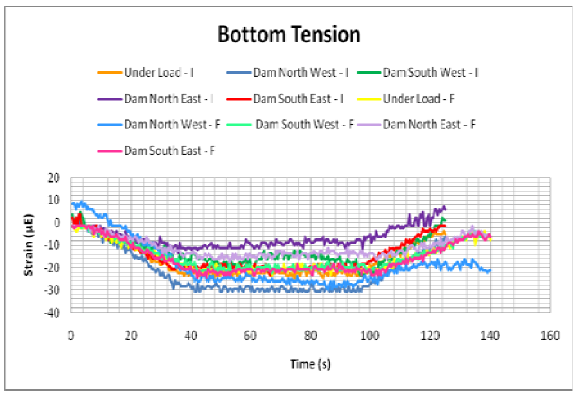

Fig. 19 Initial and Final Strains on Specimen (Bottom Tension)

The ultimate load taken by the specimen was 67 kips applied and 90 kips total. This is greater than the expected 61.5 kips applied and 84.5 kips total loading from hand calculations. The joint cracked evenly across the bottom and extended up to where the web indents and forms the shear key that runs the entire length of the specimen (Fig. 20). Deflection stayed evenly below 0.1 inch for approximately 60 kips before increasing exponentially to 3.31 inches between 40 and 67 kips (Fig. 21).

Fig. 20 Side (Bottom Left) and Underside (Bottom Right) Cracking at Interior Joint

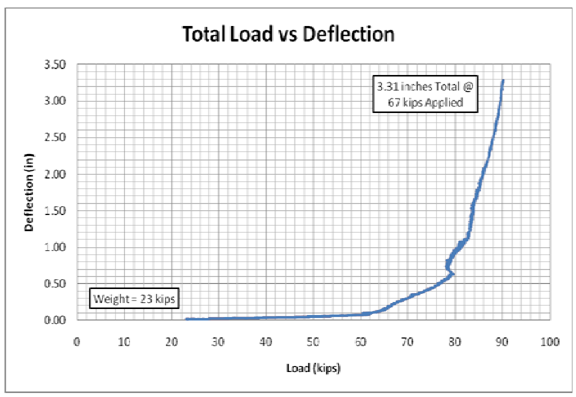

Fig. 21 Total Load versus Deflection of Specimen

SUMMARY AND CONCLUSION

The purpose of this research was to evaluate post-tensioning in a previously developed non-post-tensioned adjacent box girder system. The objective of the original research was to eliminate diaphragms, a concrete overlay, and post-tensioning in order to simplify the system and reduce costs. System benefits include significantly simplifying box production, improving the rate of construction, and making it easier to inspect voids. Post-tensioning is meant to increase the capacity of the section by placing the joints under compression and preventing reflective cracking and leakage.

Along with the original study, design charts were updated to meet AASHTO LRFD specifications for the amount of transverse post-tensioning required to achieve fully composite behavior between adjacent box girders. This was done with the help of finite element analysis. These charts were used to verify new models developed for the current research experiment and also to determine the loading that would simulate fatigue of a 52 ft x 64 ft x 27 in bridge in a 16 ft x 8 ft x 27 in specimen.

The experimental program consisted of applying an 18.4 kip load for 5 million cycles at an exterior joint of the 4 box specimen with supports at the center and opposite edge. This produced tension at the more critical top flange where the system is vulnerable to environmental chemicals and corrosives. Cracking was monitored by a dam over the joint and 5 strain gages placed next to the load and on either side of the center joint. No cracking or strain loss occurred during the course of the experiment. Supports were then rearranged to both ends of the member and the load repositioned to the center joint for tension in the bottom flange. A 17.4 kip load was applied for 5 million cycles and again no cracking or strain loss occurred.

Ultimate capacity was calculated as 280 kip-ft using strain compatibility and found to be slightly higher during testing. An applied load of 67 kips in addition to the self-weight of the specimen reached 300 kip-ft before failure. Cracking did not propagate until after reaching a 60 kip total load and then deflection increased exponentially from 0.1 inch to 3.31 inches over the next 30 kips applied.

Based on test results, it can be concluded that a post-tensioned transverse connection without diaphragms or a concrete overlay can be designed and detailed to have comparable performance to typical connections while being more economical and practical. This comparison is made based the development of a connection that adequately transfers moment and shear in the transverse direction, while the statement about being more economical and practical is based on system benefits such as improved construction, reduced structural weight, and ease of inspection. Grout specified for the transverse joint should be a semi-fluid, non-shrink mix for best results. Details provided in Fig. 5 on page 7 indicate how a duct within a duct system can be used to keep the post-tensioning unbounded for ease of inspection and maintenance. Additional details typical for an AASHTO box girder are provided in Fig. 4. The specimen tested in the experimental program confirms the system's excellent performance under both static and cyclic loads and, therefore, is recommended for implementation in relevant bridge applications.

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