Adjacent box girders without internal diaphragms or post-tensioned joints

Kromel Hanna, George Morcous, and Maher K. Tadros

- This paper advocates the elimination of internal box-girder diaphragms and presents the development of two non-post-tensioned transverse continuous connections designed to transfer transverse shear and moment between adjacent boxes in precast, prestressed concrete box-girder bridges.
- Fatigue- and monotonic load testing were performed on the two proposed connections and one commonly used connection to evaluate their fatigue capacity, ultimate capacity, and joint leakage.
- Test results indicate that both proposed connections outperform the current connection in addition to being more economical.

Precast, prestressed concrete box-girder bridges represent approximately one-third of all prestressed concrete bridges built in the United States. The adjacent box-girder system is the most prevalent box-girder system for short- and medium-span bridges, especially on secondary roadways. These bridges consist of multiple precast, prestressed concrete box girders that are butted against each other to form the bridge superstructure and deck. The boxes are laterally connected at their interface using grouted shear keys, tie rods, transverse post-tensioning, or variations thereof. The topping is typically a 2-in.-thick (50 mm) nonstructural wearing surface or a 5 in. to 6 in. (130 mm to 150 mm) structural composite slab. The main advantages of this bridge system include the following:

- ease and speed of construction because of elimination of concrete forming and placement
- shallow superstructure depth that is necessary to maintain the required vertical clearance
- low construction cost compared with I-girder bridges and other competing systems
- improved bridge aesthetics due to the flatness of the soffit and the slenderness of the superstructure

The American Association of State Highway and Transportation Officials' (AASHTO's) *Standard Specifications for Highway Bridges*[1] does not provide adequate guidance for designing and detailing the transverse connection in adjacent box-girder bridges. Although the specifications stipulate that continuous longitudinal shear key and transverse tie reinforcement be provided for using live load distribution factors of multibeam decks, there are no requirements for prestressing the transverse reinforcement. The *AASHTO LRFD Bridge Design Specifications*[2] does not consider the use of transverse mild steel reinforcing rods secured by nuts sufficient to achieve full transverse flexural continuity. The specifications recommend a minimum average effective post-tensioning stress of 250 psi (1700 kPa) between adjacent boxes. However, the specifications do not indicate the contact area over which the required pressure is applied, such as intermediate diaphragms, shear keys, or the entire box side surface.

The *Precast Prestressed Concrete Bridge Design Manual*[3] presents charts for determining the required post-tensioning force to achieve adequate stiffness in the transverse direction for adjacent box girders. El-Remaily et al.[4] developed these charts assuming that post-tensioned transverse diaphragms are the primary mechanism for the distribution of wheel loads across the bridge and the differential deflection limit between adjacent boxes is 0.02 in. (0.5 mm). Hanna et al.[5–7] developed updated design charts to account for the latest AASHTO LRFD specifications' loads and governing parameters, such as bridge width, span-to-depth ratio, and skew angle.

Although the use of transverse post-tensioning to connect adjacent box girders is an effective and practical solution in many cases, it has some disadvantages. Post-tensioned transverse connections require the use of end and intermediate diaphragms to achieve continuity in the transverse direction. Diaphragms in skewed bridges are difficult to construct and may have to be staggered or built in stages. This leads to a significant increase in the construction cost and schedule due to the variation in diaphragm location, large number of post-tensioning operations, and excessive traffic control required in replacement projects. Moreover, post-tensioned diaphragms provide transverse continuity only at discrete locations, which makes the system more susceptible to cracking and leakage.

There are several types of non-post-tensioned transverse connections used in adjacent box girders in the United States. The NASA Road 1 Bridge over Interstate 45 (I-45) between Houston, Tex., and Galveston, Tex., used one of these connections in its construction.[8] This bridge was built in 2002 as one of a pair of parallel bridges carrying four lanes of traffic; the other structure is a traditional steel-girder bridge. The 300-ft-long (90 m) bridge consists of four equal 75 ft (23 m) spans with skew angles ranging from 0˚ to 30˚. Each span has nine adjacent type 4B28 box girders that were formed using the side forms of a standard Texas I-girder spaced approximately 4 ft (1.2 m) apart. A unique transverse connection that eliminated the need for intermediate diaphragms, grouting, and post-tensioning was used between the box girders. The connection consists of a wide and continuous half-depth shear key that was poured monolithically with the reinforced concrete deck. For this connection, a minimum 6-in.-thick (150 mm) composite concrete deck was used to provide adequate stiffness in the transverse direction.

Another common non-post-tensioned transverse connection is the one adopted by the Illinois Department of Transportation (IDOT). This connection consists of a half-depth continuous shear key and transverse ties at the intermediate diaphragm locations. Transverse ties for the system are placed at the skew angle rather than at a right angle. They connect the adjacent box girders in pairs in the transverse direction. Each tie consists of two 1-in.-diameter × 46-in.-long (25 mm × 1200 mm) rods threaded 4 in. (100 mm) at each end and connected with a 5-in.-long (130 mm) coupling nut. A 5 in. (130 mm) noncomposite reinforced concrete topping distributes the loads uniformly across all girders and protects the shear keys from cracking and leakage.

Objective

The objective of this research was to develop non-post-tensioned transverse connections between adjacent box girders that have performance characteristics superior to and more economical than connections currently used. This paper proposes two non-post-tensioned connections that emulate monolithic construction and eliminate the need for diaphragms, post-tensioning, and cast-in-place concrete topping. These connections are termed the wide-joint system and the narrow-joint system. The elimination of end and intermediate diaphragms results in simplified production. It also gives the producer the option of using reusable void forms. Furthermore, elimination of a cast-in-place composite concrete topping would result in greater speed of construction, as well as other savings.

Wide-joint system

The wide-joint system connects adjacent boxes with top and bottom transverse reinforcement located in a wide shear key filled with concrete, instead of the typical concrete composite topping. The wide-joint system differs from the aforementioned NASA Road 1 Bridge connection by eliminating topping and diaphragms. The monolithic joint with top and bottom reinforcement provides a continuous connection that transfers shear and moment between boxes, eliminating the need for intermediate and end diaphragms. The elimination of topping and diaphragms significantly increases the speed of production and construction operations while reducing material, labor, and erection costs.

The shear keys in the wide-joint system are full length and full depth, requiring minimal modifications to the forms of

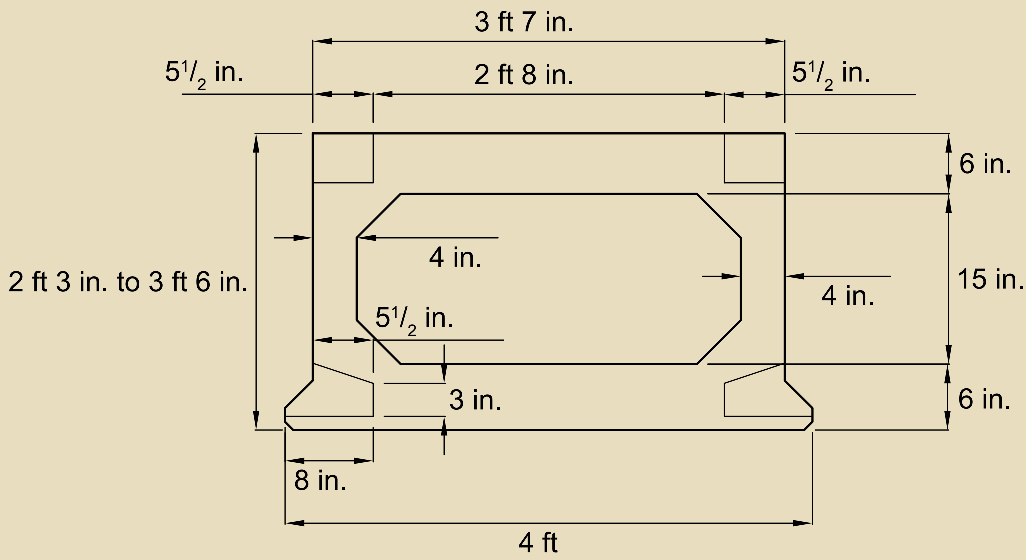

Figure 1. Wide-joint-system box dimensions. Note: 1 in. = 25.4 mm; 1 ft = 0.305 m.

the standard box cross section. **Figure 1** shows a modified AASHTO PCI box section that is 33 in. (850 mm) deep and 48 in. (1200 mm) wide. The modifications include a 2.5 in. (65 mm) recess on each box side to create the 5-in.-wide (130 mm) shear key, two 5.5-in.-long (14 mm) × 5.5-in.-wide × 4.5-in.-deep (113 mm) blockouts in the top flange every 4 ft (1.2 m), and two 8-in.-long (200 mm) × 5.5-in.-wide × 4.5-in.-deep blockouts in the bottom flange every 4 ft. **Figure 2** shows the reinforcement for the top- and bottom-flange blockouts. The reinforcing bars protrude from the top and bottom flanges of each box and are lap spliced with short reinforcing bars confined by $^1/_4$-in.-

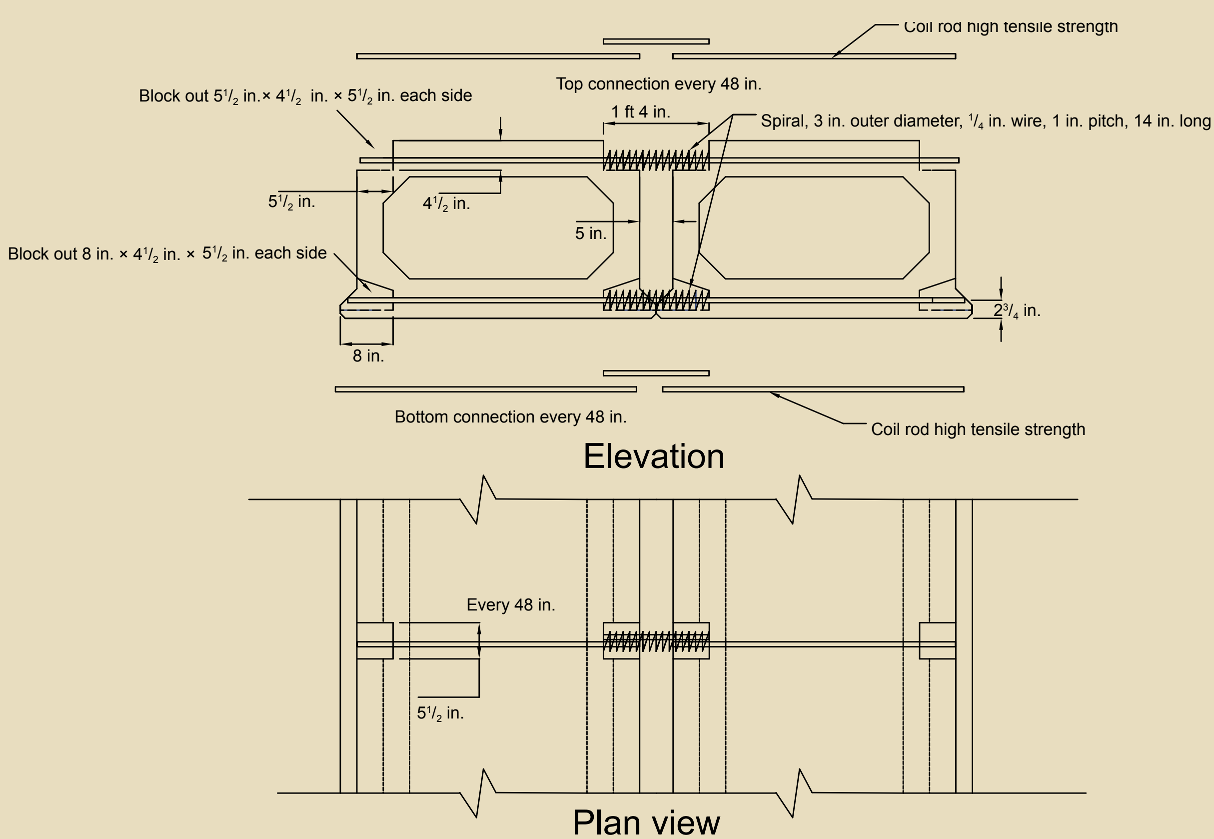

Figure 2. Wide-joint connection detail. Note: 1 in. = 25.4 mm; 1 ft = 0.305 m.

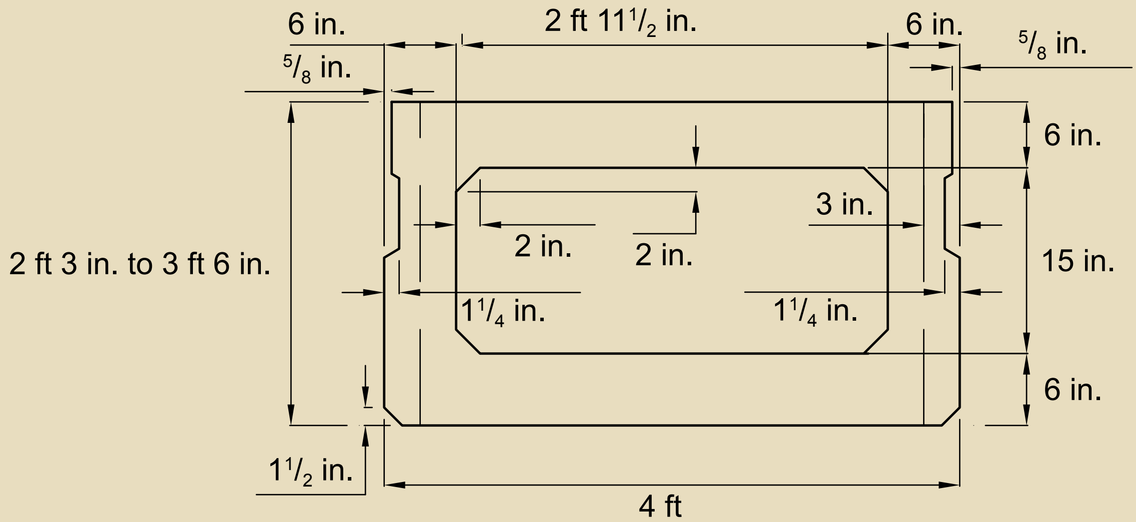

Figure 3. Narrow-joint-system box dimensions. Note: 1 in. = 25.4 mm; 1 ft = 0.305 m.

thick (6 mm) wire coiled to 3-in.-diameter (75 mm) spirals pitched at 1 in. (25 mm). This confinement reinforcement is necessary to provide adequate development length for the short lap splices.

It is recommended to provide an additional 0.5 in. (13 mm) thickness to the top surface of the box and the shear keys when grinding is required to provide a roughened surface for skid resistance. The use of self-consolidating concrete to fill the wide shear key is also recommended to eliminate the need for grouting, which is a costly and time-consuming operation.

Narrow-joint system

The narrow-joint system differs from the IDOT connection by eliminating diaphragms and replacing the single mid-depth transverse tie with top and bottom transverse ties. The narrow-joint system uses Grade 75 (520 MPa) threaded rods at every 8 ft (2.4 m) to connect each pair of adjacent boxes at the top and bottom flanges. These rods provide a continuous connection that transfers shear and moment between adjacent boxes more efficiently than the mid-depth transverse ties at discrete diaphragm locations.

Figure 3 shows the slight modification made to the standard box cross section to incorporate full-length horizontal and full-depth vertical shear keys. **Figure 4** shows the boxes fabricated with plastic ducts in the top and bottom flanges to create openings for the threaded rods. The bottom duct is inserted between the two layers of prestressing strands, while the top duct is located 3 in. (75 mm) from the top surface to provide adequate concrete cover. Vertical vents are provided at one side of each box to allow the air to escape during the grouting operation.

Analytical investigation

Boxes without internal diaphragms behave fundamentally differently from those with significant reinforced concrete diaphragms. The mathematical model essentially involves a deforming top transverse slab and a similar bottom slab with webs connecting them. This is similar to an I-girder bridge superstructure. Thus, the massive torsional stiffness that has caused reflective cracking in the current applications is substantially reduced. Three-dimensional (3-D) computer models were developed to calculate the load effects on the proposed connections. Each box girder was modeled using shell elements for the flanges and webs of the boxes and frame elements for the connections between the boxes. The assumptions made for developing the computer models include the following:

- Shell and frame elements represent the centerlines of the modeled components.
- The length of shell elements in the direction of traffic is 12 in. (300 mm).
- The thickness of shell elements equals the total concrete thickness of the corresponding component (that is, top flange, bottom flange, web).
- Frame elements are repeated every 4 ft (1.2 m) in the wide-joint model and every 8 ft (2.4 m) in the narrow-joint model along the bridge length.
- The cross section of the connecting frame elements is rectangular. The width of this rectangle equals the spacing between frame elements, while its depth equals the thickness of the corresponding flange.

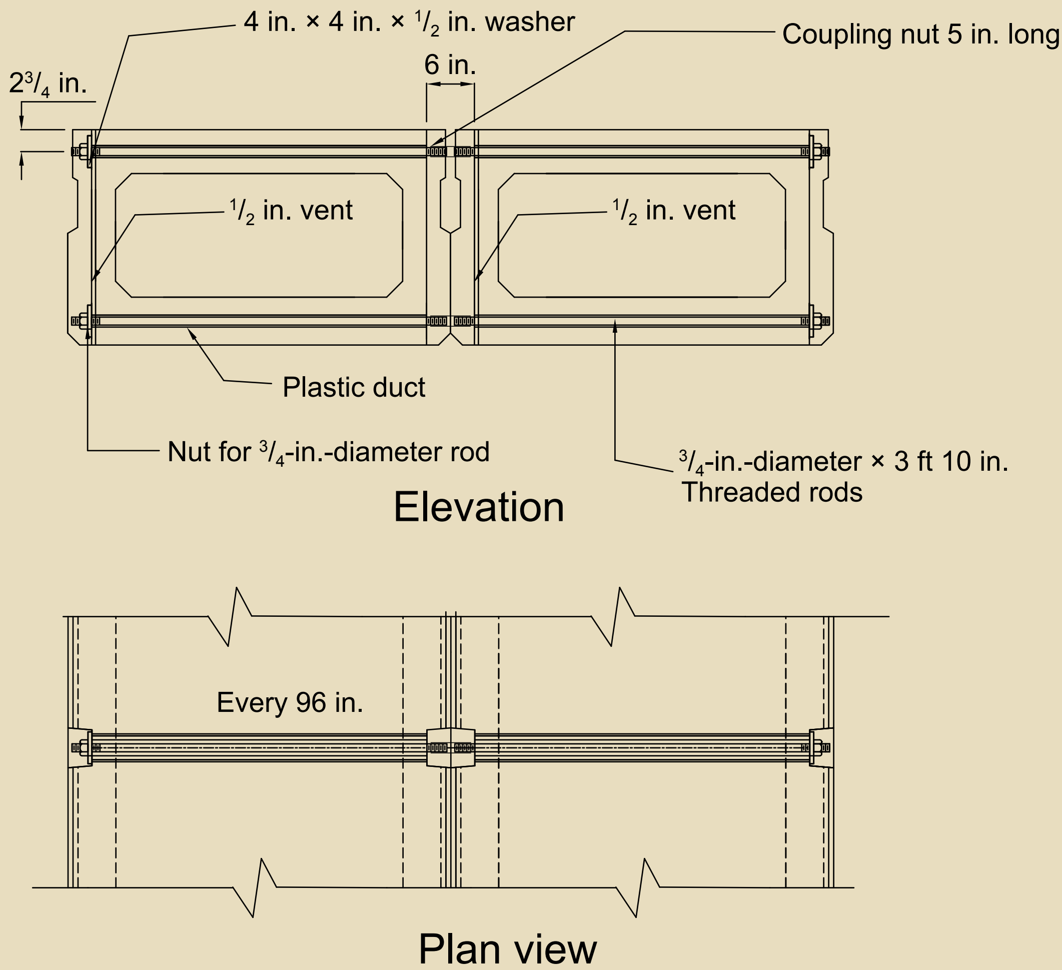

Figure 4. Narrow-joint connection detail. Note: 1 in. = 25.4 mm; 1 ft = 0.305 m.

The loads applied for the analysis include the concrete curb and rail dead load and the HL 93 live load with a dynamic load allowance of 0.33. Single- and multiple-lane loadings were applied to determine the most critical loading case for the design of the transverse connections. The dead load due to the self-weight of box girders and wearing surface was not considered because it is uniformly distributed over the bridge and consequently does not generate load effects in the transverse direction. The weight of the concrete curb and railing was assumed to be 0.48 kip/ft (7.0 kN/m) and was applied to the outside box girders.

The 3-D computer models helped develop the design charts to evaluate the effects of the box-girder depth, span width, and span-to-depth ratio on the two proposed transverse connections. **Figures 5** and **6** show the effect of the box depth and bridge width on the tension force in the transverse connection for the wide-joint and narrow-joint systems. These charts were developed for a 0° skew angle and span-to-depth ratio of 30. The tension force increases as the bridge width increases and decreases as the box depth increases. Also, the effect of the bridge width on the required tension force is greater in narrow bridges (52 ft [15 m] width or less) than in wide bridges (wider than 52 ft). Calculating and plotting the force per unit length, **Fig. 7** compares the required tension force for the two proposed connections. It indicates that the force required for the wide-joint system is greater than that for the narrow-joint system for any given bridge width and box depth. These required forces are a result of the factored dead load DL and live load LL (1.25DL + 1.75LL) and can be divided by the yield strength of the reinforcing steel to determine the required area of reinforcement.

Figures 8 and **9** show the effect of the span-to-depth ratio and box depth on the transverse tension force in the wide-joint and narrow-joint connections, respectively. These charts were developed for a bridge width of 52 ft (15 m) at a 0° skew angle. Both charts show that the required transverse tension force increases as the span-to-depth ratio increases. This effect is more pronounced in the narrow-joint system than in the wide-joint system and in shallower boxes than in deep boxes.

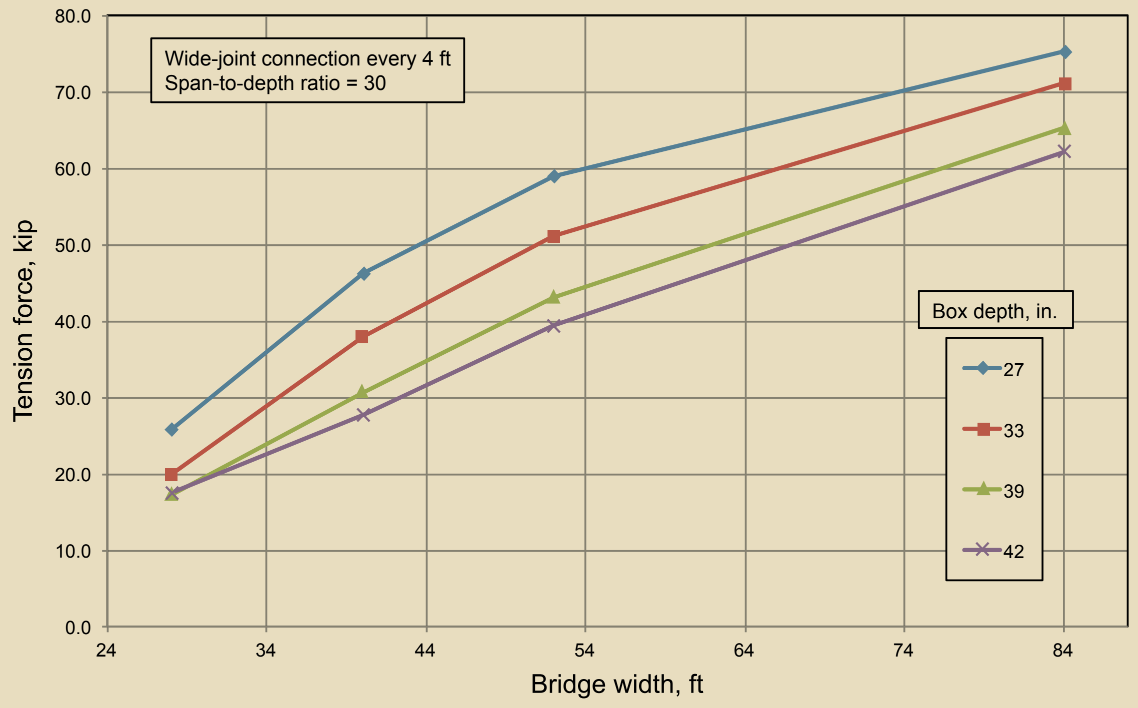

Figure 5. Effect of box depth and bridge width on the required transverse tension force for the wide-joint system. Note: 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN.

Experimental investigation

Three transverse connections were experimentally investigated to determine and compare their structural performance under static and cyclic loading conditions. The three systems were the connection currently used by IDOT, the wide-joint connection, and the narrow-joint connection. Three specimens were fabricated. Each specimen consisted of two boxes that were 8 ft × 4 ft × 27 in. (2.4 m × 1.2 m × 690 mm).

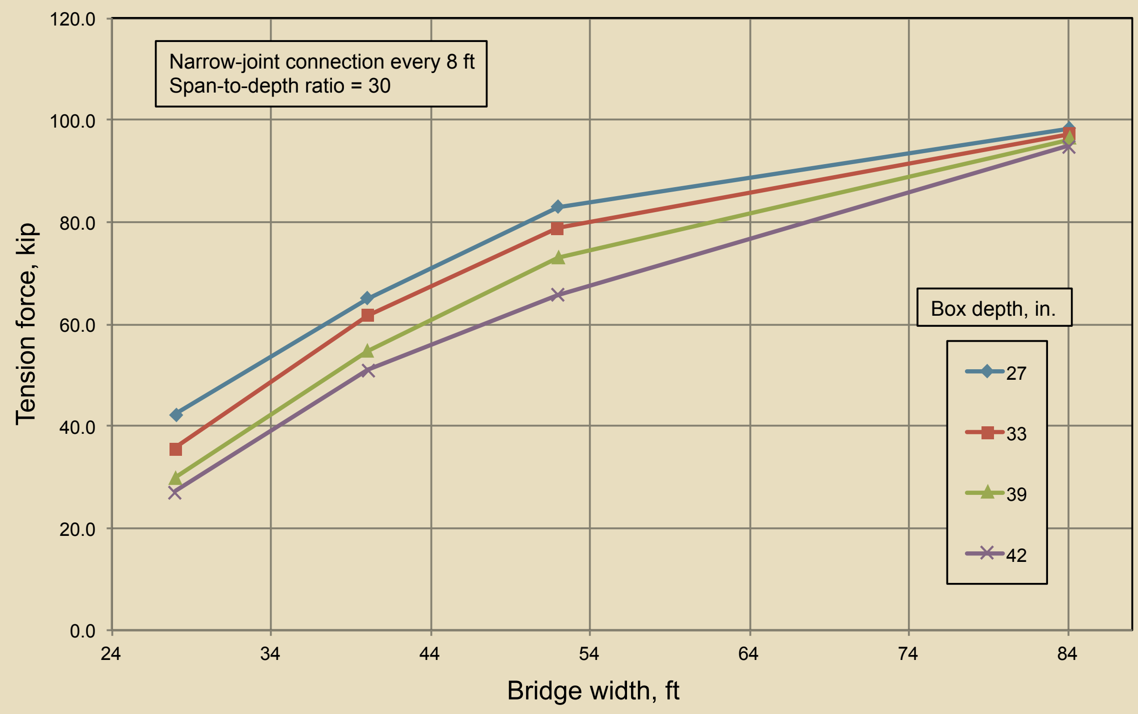

Figure 6. Effect of box depth and bridge width on the required transverse tension force for the narrow-joint system. Note: 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN.

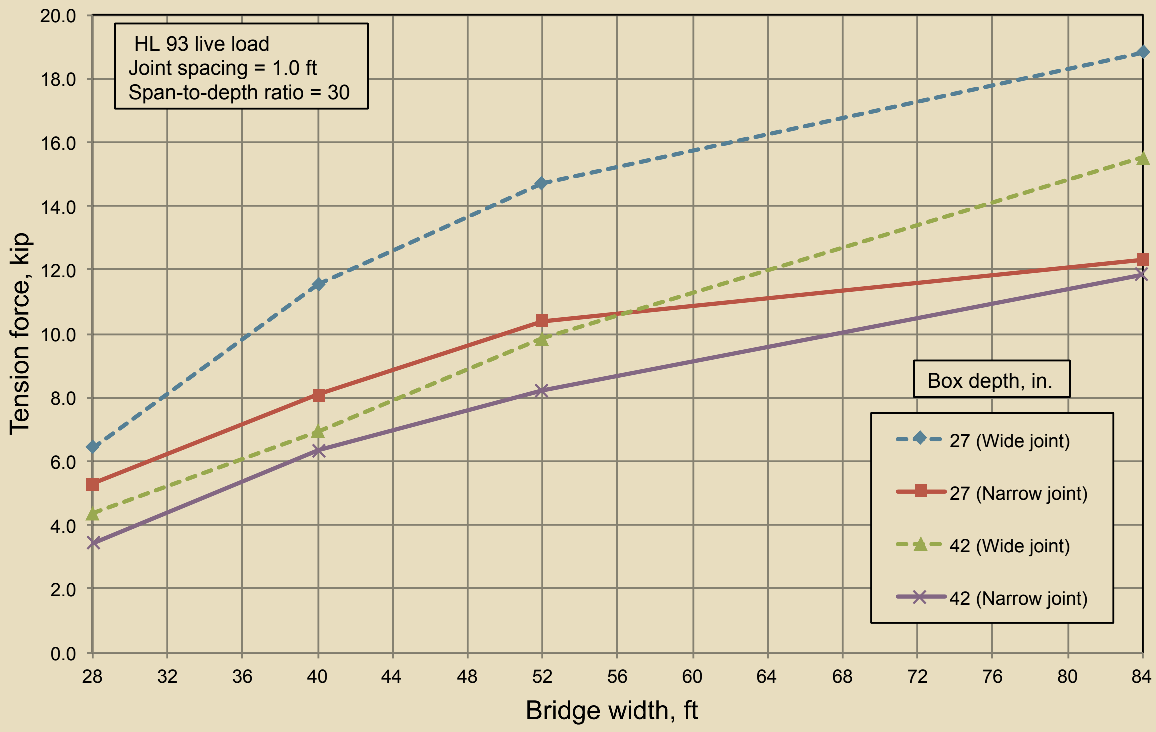

Figure 7. Tension force required for wide-joint and narrow-joint systems. Note: 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN.

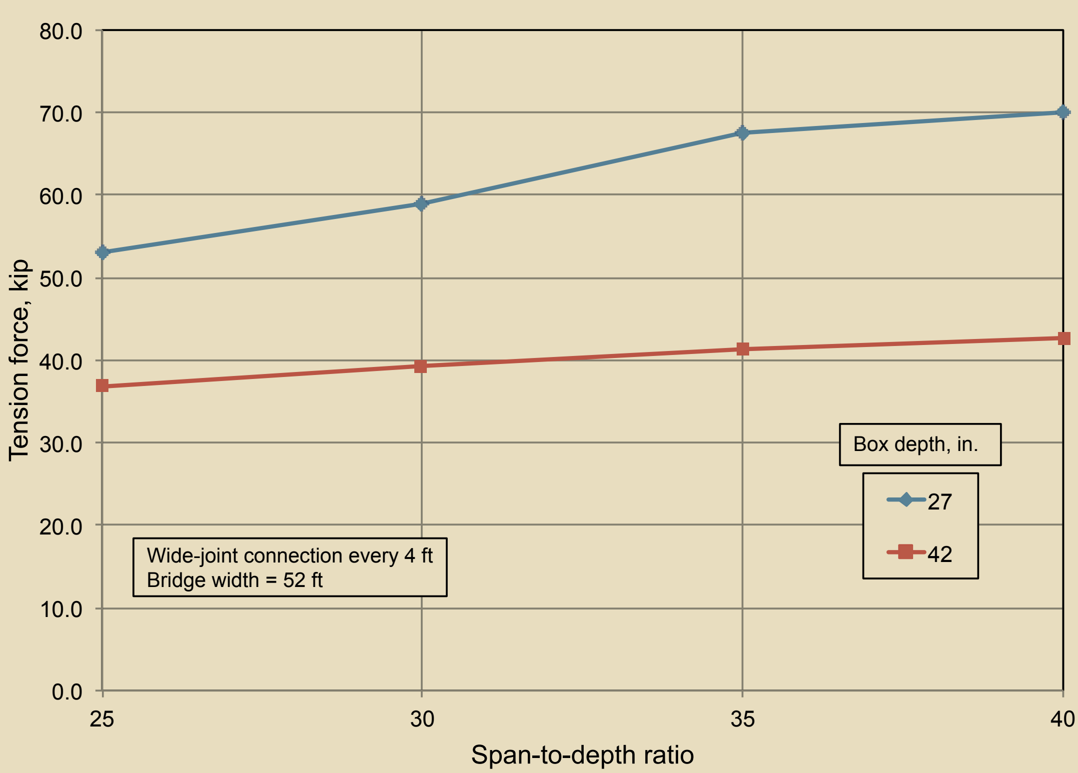

Figure 8. Effect of span-to-depth ratio on required transverse tension force for wide-joint system. Note: 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN.

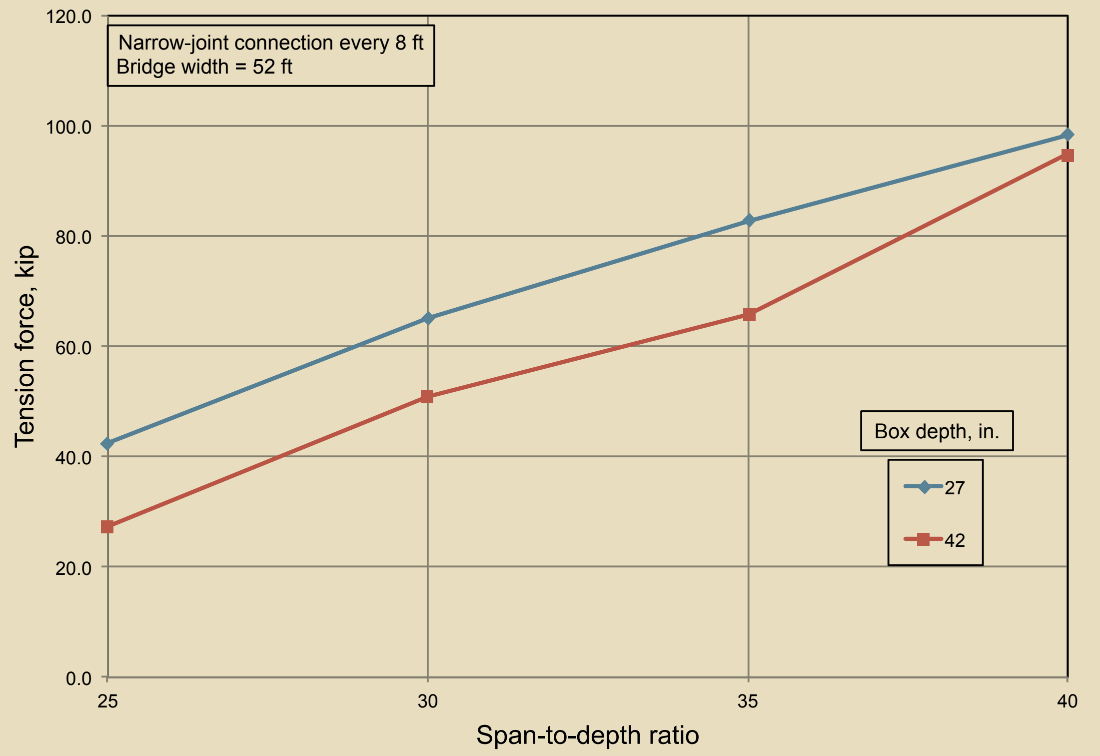

Figure 9. Effect of span-to-depth ratio on required factored transverse tension force for narrow-joint system. Note: 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN.

The developed finite element models and design charts presented in the previous section were used to calculate the required transverse reinforcement for both wide-joint and narrow-joint connections. The example bridge was 48 ft × 28 ft × 27 in. (15 m × 8.5 m × 700 mm). The required transverse reinforcement was $^{3}/_{4}$-in.-diameter (20 mm), Grade 75 (520 MPa) coil rods for the narrow-joint system and no. 6 (19M), Grade 60 (410 MPa) bars for the wide-joint system. The IDOT connection requires two 1-in.-diameter (25 mm) threaded rods connected with a coupling nut regardless of the bridge width, depth, or span.

Number 4 (13M), Grade 60 (410 MPa) reinforcing bars were used to replace the longitudinal prestressing strands due to the short length of the specimens and the fact that specimens are tested only in the transverse direction.

Static loading was performed to determine the ultimate flexural capacity of each connection. The cyclic loading simulated the effects of live load and impact by generating positive and negative bending moments and shear at the transverse connection. Each pair of boxes was connected and supported using 12-in.-wide (300 mm) steel beams parallel to the transverse connection to restrain the vertical movement of the boxes. The distance between the centerline of supports was 7 ft (2.1 m), which was the loading span. The loading frame was attached to the floor using threaded rods to provide adequate stability under cyclic loading. The load was applied using a 14 in. × 14 in. (360 mm × 360 mm) loading plate located 5 in. (130 mm) from the centerline of the connection.

Testing of current IDOT connection

To prepare the specimen for testing, two threaded rods and a coupling nut connected the two boxes, and the horizontal and vertical shear keys between the boxes were filled with non-shrink, multipurpose grout. This grout had an average compressive strength of 3200 psi (22 MPa) after 24 hours, which exceeds the 3000 psi (21 MPa) minimum strength specified by IDOT. A water dam was also built in the top of the box girder and filled with water to measure the water leakage through the joint during testing.

The calculated applied load was a 16 kip (71 kN) wheel load of HL93 multiplied by the 1.15 dynamic load factor for fatigue, according to AASHTO LRFD specifications. This resulted in a total of 18.4 kip (81.8 kN) applied vertically to the specimen in both the downward and upward directions for 10,000 cycles. The joint was carefully monitored for cracks and water leakage during the test. After 10,000 cycles, the test was stopped because of excessive cracking and water leakage at the joint. The poor performance of the joint (**Fig. 10**) was primarily due to the inadequate negative moment capacity of the current IDOT connection, which caused cracks to extend from the top surface of the specimen through the depth of the shear key.

Figure 10. Failure of the current connection without topping.

Another fatigue test was performed after adding a 5 in. (130 mm) noncomposite concrete topping to the specimen, which is a common IDOT practice. The concrete topping was reinforced with one layer of no. 5 (16M) bars at 12 in. (300 mm) in both directions. The reinforcement was placed at the mid-depth of the topping. After the topping achieved the specified concrete strength of 5000 psi (34 MPa), a static load was applied to the specimen. The static load increased from 0 kip to 18.4 kip (0 kN to 81.8 kN), decreased from 18.4 kip to -18.4 kip, and then returned to 0 kip. This test was performed to evaluate the stiffness of the connection before applying fatigue loads. The cyclic load was applied for 2 million cycles, and the total movement of the specimen was recorded for each cycle. The movement of 0.09 in. (2.3 mm) remained relatively constant, which indicated that the connection maintained its stiffness through the 2 million cycles.

The calculated moment capacity of the IDOT system with a 5-in.-thick (130 mm) noncomposite concrete topping was 145.4 kip-ft (197.1 kN-m) using strain compatibility. Using simple beam analysis, this moment is equivalent to a point load of 112.5 kip (500.4 kN) applied at the midspan of the test specimen. The actual failure load was 138.7 kip (616.9 kN), 23% higher than the theoretical load. The tie assembly of this system is repeated every 24 ft (7.3 m), so the flexural capacity for a 24-ft-long segment is equivalent to a uniform load of 5.8 kip/ft (85 kN/m) along the bridge length. **Figure 11** shows the failure under ultimate load, which was primarily delamination at the interface between the box and the noncomposite concrete topping due to inadequate horizontal shear resistance.

Figure 11. Failure of the current system.

Testing of wide-joint connection

To prepare the wide-joint connection specimen for testing, the surface of the shear key was roughened. The first box girder was placed on top of the support from one side, while a temporary support was provided at the other side. Splicing bars with the spiral reinforcement were attached to the first box at the two connecting locations. The second box was placed against the first box, and the splicing bar and spiral were attached. The shear key was then cleaned and filled with self-consolidating concrete.

After the joint concrete cured and reached a compressive strength of 6000 psi (41 MPa), a static load was applied. The static load increased from 0 kip to 6.14 kip (0 kN to 27.3 kN), reversed from 6.14 kip to -6.14 kip, and returned to 0 kip. The calculated applied load was $^{1}/_{3}$ of the IDOT connection applied load of 18.4 kip (81.8 kN), representing the equivalent load per two connections spaced at 4 ft (1.2 m). The cyclic load was then applied, ranging from 6.14 kip to -6.14 kip for 2 million cycles at a rate of 2 cycles/sec. The total movement recorded over 2 million cycles remained constant at approximately 0.02 in. (0.5 mm). The dam on the top surface of the specimen was filled with water and monitored for cracks and water leakage during the fatigue test. Another static load was applied after 2 million cycles to determine whether there was any change to the connection stiffness as a result of connection fatigue.

Before increasing the magnitude of the cyclic load, a higher static load was applied to the specimen. The static load increased from 0 kip to 18.4 kip (0 kN to 81.8 kN), reversed to -18.4 kip, and returned to 0 kip. Then the cyclic load range of ±18.4 kip was applied for 3 million cycles at a rate of 2 cycles/sec. The joint performed well under the cyclic loads, and there were no visible cracks or water leakage. The total movement recorded over the 3 million cycles remained constant at approximately 0.07 in. (1.8 mm).

The calculated moment capacity of the wide-joint system using strain compatibility was 125 kip-ft (169 kN-m), which corresponds to an ultimate point load of 161 kip (716 kN) applied at the midspan of the specimen. The actual failure load was 162 kip (721 kN), which is equivalent to a distributed load of 20.3 kip/ft (296 kN/m) along the 8-ft-long (2.4 m) bridge segment.

Figure 12. Failure of the wide-joint connection.

Figure 12 shows the failure conditions of the wide-joint specimen. The top cracks were located between the web and the top flange of the box girder. This indicated that the elimination of intermediate diaphragms allowed the top flange to work as a slab and transfer the load continuously. The failure at the bottom of the box occurred at the end of the splicing bar.

Testing of narrow-joint connection

The narrow-joint connection specimen was built by placing the two box girders against each other and inserting threaded rods at the top and bottom openings. Couplers were used to connect the threaded rods at the joint, and a $^1/_2$-in.-thick (13 mm) steel plate and nuts were used at the outside face of the box girder. The plastic ducts of the threaded rods were damaged during concrete placement at both top and bottom locations. As a result, the opening was not sufficient for placement of the $^3/_4$-in.-diameter (20 mm) threaded rods. The opening was increased to 1 in. (25 mm) diameter using a concrete drill to allow for the threaded rod placement. To avoid this complication in the future, a smooth reinforcing bar can be placed inside the plastic duct before concrete placement and removed after release of the prestressing force. The horizontal and vertical shear keys were cleaned and filled with a non-shrink, nonmetallic grout.

After the joint cured and the grout compressive strength reached 6000 psi (41 MPa), the loading was applied to the specimen. The static- and cyclic load applications followed the same sequence as described in the testing of the wide-joint connection.

The joint performed well under the cyclic loads, and there were no visible cracks or water leakage. The total movement recorded in the first 2 million cycles was constant at 0.02 in. (0.5 mm). The movement increased to approximately 0.07 in. (1.8 mm) when the load increased from 6.14 kip (27.3 kN) to 18.4 kip (81.8 kN). The movement of 0.07 in. remained constant in the following 3 million cycles.

Figure 13. Failure of the narrow-joint connection.

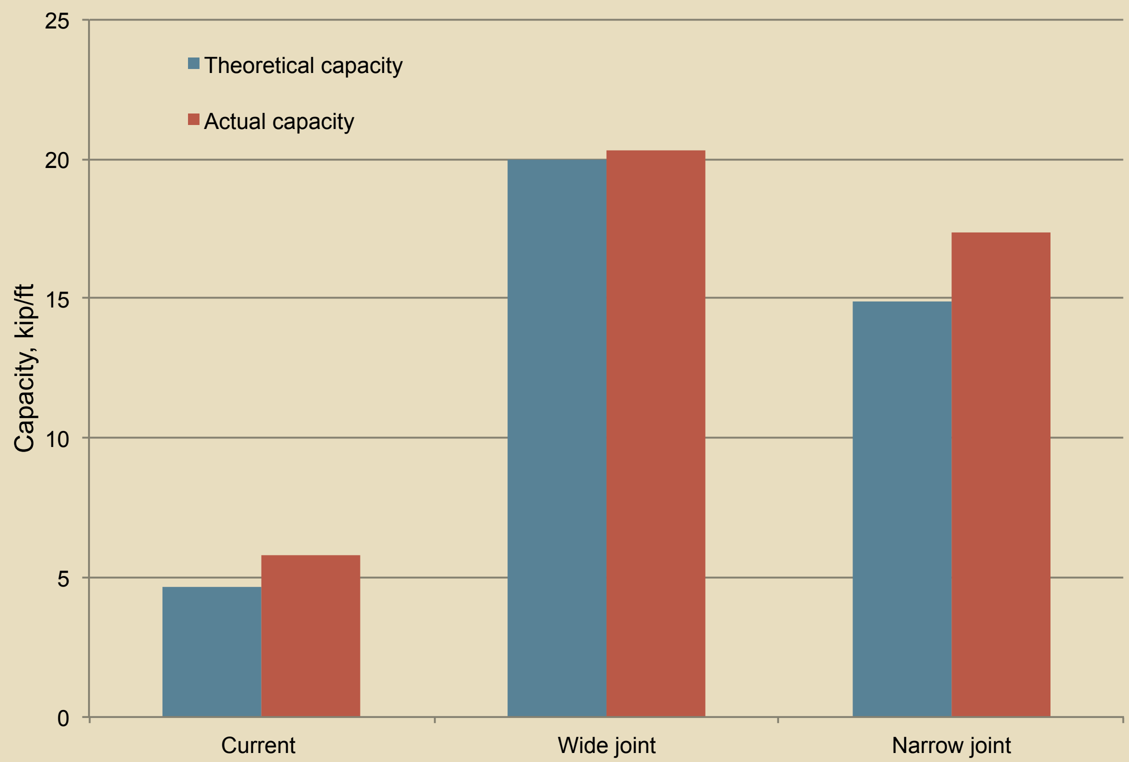

Figure 14. Theoretical capacity and testing capacity of the three systems. Note: 1 ft = 0.305 m; 1 kip = 4.448 kN.

The calculated moment capacity of the narrow-joint system was 83 kip-ft (113 kN-m) using strain compatibility. This corresponds to a point load of 107 kip (476 kN) applied at the midspan of the specimen. The actual failure load was 156 kip (694 kN), significantly higher than the theoretical load. Distributing the failure load uniformly along the 8-ft-long (4.2 m) segment results in an equivalent uniform load of 19.5 kip/ft (285 kN/m). The failure of the narrow-joint specimen in **Fig. 13** shows the grout split around the coupling nut due to stress concentration at the bottom of the box connection.

Comparison between connection systems

Figure 14 and **Table 1** compare the theoretical flexure capacities and actual testing capacities of the three systems. The wide-joint and narrow-joint systems have significantly higher theoretical capacities and testing capacities than the IDOT detail with the 5-in.-thick (130 mm) noncomposite concrete topping has.

Conclusion

This paper presents a new proposal consisting of two main elements: the elimination of all intermediate diaphragms in adjacent box girder decks and the use of non-post-tensioned concrete to resist interface shear, torsion, and flexure. Two new non-post-tensioned transverse connections in adjacent box girders were presented.

The proposed narrow-joint and wide-joint connections were developed to be structurally superior to existing connections while being more practical and economical. Both connections were designed to eliminate the need for post-tensioning, cross and end diaphragms, and concrete topping. This will significantly simplify box production, increase the speed of bridge construction, and reduce the to-

Table 1. Comparing the flexure and shear capacities of the three systems

Connection type	Theoretical capacity		Actual capacity		Difference, %
	Moment, kip-ft	Load, kip	Moment, kip-ft	Load, kip	
Current connection	145	112.5	179	139	19.0
Wide-joint connection	125	161	126	162	0.8
Narrow-joint connection	83	107	119	156	30.3

Note: 1 ft = 0.305 m; 1 kip = 4.448 kN.

tal cost. The elimination of diaphragms facilitates inspection of voids, provides better drainage of moisture, results in a lighter section weight for handling and shipping, and allows for continuous rather than discrete (quarter-point) connections. The proposed narrow-joint and wide-joint connections have horizontal and vertical shear keys for shear transfer with top- and bottom-flange reinforcement to provide an adequate moment transfer mechanism between the boxes.

Finite element models were developed to determine the effect of design parameters on the design of the proposed connections, such as span length, bridge width, and girder depth. The latest AASHTO LRFD specifications' live load and dynamic load factors were applied to determine the required reinforcement in the two connections. Design charts were developed for each system to facilitate connection design in various bridge widths ranging from 28 ft to 84 ft (8.5 m to 26 m); span-to-depth ratios of 25, 30, and 35; and four standard depths of 27 in., 33 in., 39 in., and 42 in. (690 mm, 840 mm, 990 mm, and 1070 mm). Based on the analysis results, the following conclusions can be made for the wide-joint and narrow-joint connections:

- The required connection reinforcement increases as the bridge width increases.
- The required connection reinforcement decreases as the bridge depth increases.
- The required connection reinforcement increases as the span-to-depth ratio increases.

Based on test results, it can be concluded that non-post-tensioned transverse connections can be designed and detailed to have comparable performance to post-tensioned connections and be more economical and practical. A significant improvement in production economy is caused by the ability to remove all diaphragms and to use a reusable steel void-forming system. The proposed non-post-tensioned wide-joint and narrow-joint connections performed well under static and cyclic loads; therefore, both connections are recommended for practical applications in bridge construction.

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References

1. American Association of State Highway and Transportation Officials (AASHTO). 1996. *Standard Specifications for Highway Bridges*. 16th ed. Washington, DC: AASTHO.

2. AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications, Fourth Edition—2008 Interim Revisions*. Washington, DC: AASHTO.

3. PCI Bridge Design Manual Steering Committee. 2003. *Precast Prestressed Concrete Bridge Design Manual*. MNL-133. 2nd ed. Chicago, IL: PCI.

4. El-Remaily, A., M. K. Tadros, T. Yamane, and G. Krause. 1996. Transverse Design of Adjacent Precast Prestressed Concrete Box Girder Bridges. *PCI Journal*, V. 41, No. 4 (July–August): pp. 96–113.

5. Hanna, K. E., G. Morcous, and M. K. Tadros. 2007. Transverse Design and Detailing of Adjacent Box Beam Bridges. In *The PCI National Bridge Conference: Proceedings, October 22–24, 2007, Phoenix, Arizona*. CD-ROM.

6. Hanna, K. E. 2008. Behavior of Adjacent Precast Prestressed Concrete Box Girder Bridges. PhD diss. University of Nebraska, Lincoln, NE.

7. Hanna, K. E., G. Morcous, and M. K. Tadros. 2009. Transverse Post-tensioning Design and Detailing of Precast Prestressed Concrete Adjacent Box Girder Bridges. *PCI Journal*, V. 54, No. 4 (Fall): pp. 160–174.

8. PCI Bridges Committee. Forthcoming. The State of the Art of Practice of Precast/Prestressed Adjacent Box Beam Bridges. Chicago, IL: PCI.

Production of box girders

Box girders are more challenging to produce than open-section girders, such as I-girders and U-girders. Traditional production methods include the use of expanded polystyrene (EPS) for void forming and the two-stage box production using the U-girder and a top slab. Although the use of EPS for void forming is practical and economical for many applications, steel forms can be significantly more cost-effective with their standardized shapes and repeated usage. More important, EPS forms do not provide precise dimensions and do not allow void inspection. The two-stage production of box girders results in slower production, higher cost, and lower quality than single-cast box girders.

The main advantages of the proposed concepts are as follows:

- The use of easily removable steel forms provides precise dimensions and forms can have multiple reuses while maintaining high production quality.

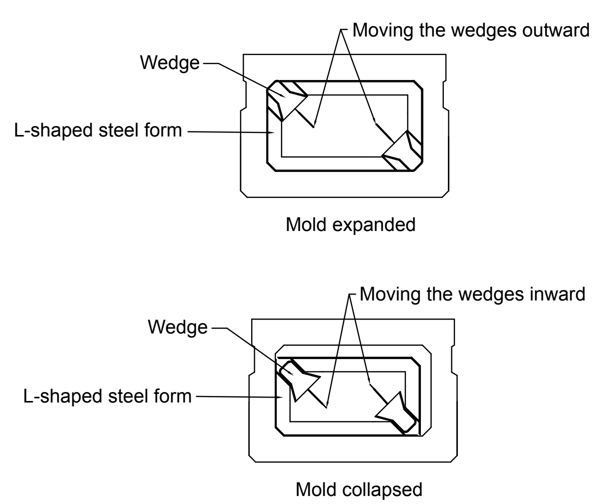

Figure A1. Form system adapted from current industry system.

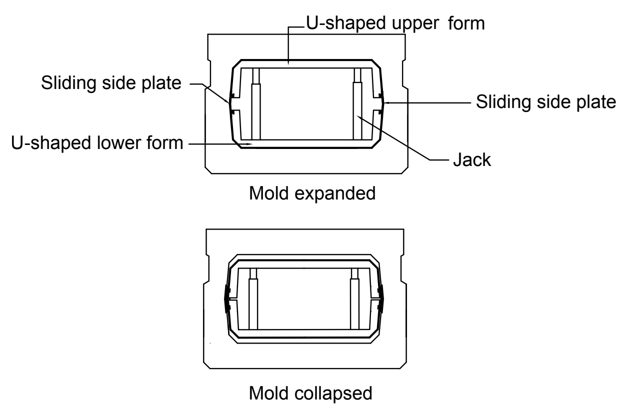

Figure A3. Proposed void forming system.

- The use of continuous forms eliminates intermediate and end diaphragms and allows for void inspection.

- The adoption of single-cast forms permits placing the concrete for the entire box in one stage and eliminates construction joints. This can be easily achieved using self-consolidating concrete that can be placed through one stem until it rises up the other stem without the need for vibration.

Several steel void form systems already exist in the U.S. market and abroad. For example, precast concrete manufacturers use voided box-girder systems for the railroad industry. However, the main impediments of these systems for use in the production of highway box girders are the longer spans (over 120 ft [37 m]) and greater camber values (up to 3 in. [75 mm]) of highway bridge girders. Therefore, the concepts proposed in this paper include two systems adopted by two private companies with slight modifications in addition to one new system proposed by the authors.

Figure A1 shows the system adopted by a private company, which consists of two L-shaped steel forms and two corner wedges. By moving the wedges outward, the form expands

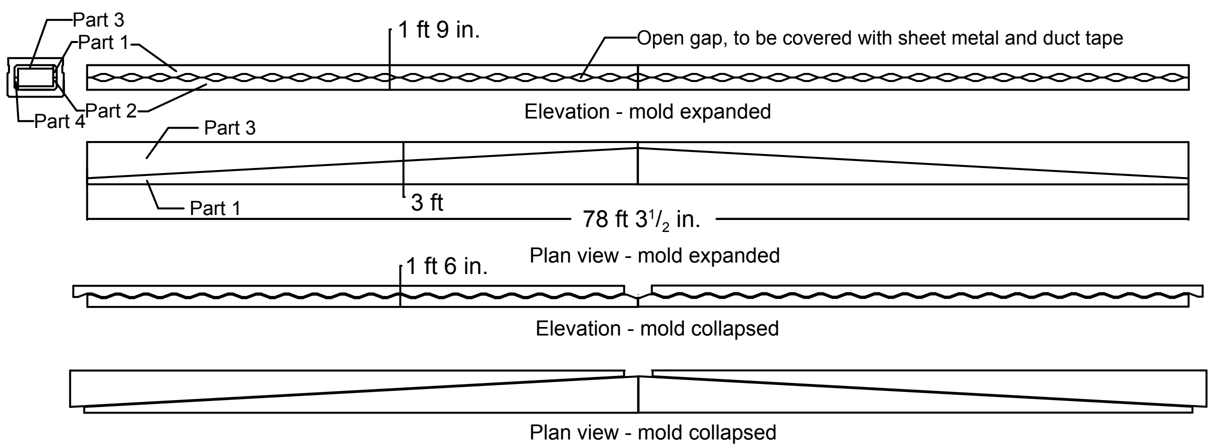

Figure A2. Form system adapted from current industry system. Note: 1 in. = 25.4 mm; 1 ft = 0.305 m.

to the required dimension. By moving the wedges inward, the form collapses to the position that allows the initial camber to take place at release before form removal.

Figure A2 shows the system adopted by a different private company, which consists of four tapered parts with wave-shaped webs. The top and bottom halves of the form can be drawn toward each other using external jacks and released from the inside face of the form before the release of prestressing to allow for initial camber.

Figure A3 shows the system proposed by the authors, which consists of one U-shaped lower form and one inverted U-shaped upper form. The forms can expand and collapse using multiple vertical jacks. Two additional sliding side plates are needed to cover the longitudinal joint when the two forms are expanded. This form can accommodate various box depths, span lengths, and initial cambers.

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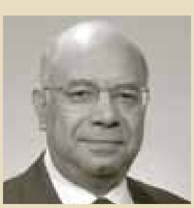

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Abstract

Precast, prestressed concrete adjacent box-girder bridges are widely used in short- and medium-span bridges. Rapid construction and low cost are the main advantages of this system. Although the use of transverse post-tensioning to connect adjacent box girders can be effective and practical, end and intermediate diaphragms are necessary for continuity. Construction of diaphragms in skewed bridges is difficult and may significantly increase in the construction cost and schedule. Moreover, post-tensioned transverse diaphragms result in continuity at discrete locations, making the system more susceptible to cracking and leakage.

This paper presents the development of two non-post-tensioned transverse connections for possible simplification of this already efficient bridge system. The proposed continuous connections transfer shear and moment between adjacent boxes without the need for diaphragms. The connections are based on monolithic emulation of a multicell cast-in-place box-girder superstructure, similar to a system commonly used in California. The wide-joint connection consists of a wide full-depth shear key filled with flowable concrete and top and bottom reinforcement. The narrow-joint connection consists of top and bottom couplers and full-depth grouted shear keys. Finite element models were used to develop design charts for different bridge configurations. Fatigue and static load testing were performed on the proposed connections and one commonly used connection to evaluate their fatigue capacity, ultimate capacity, and joint leakage. Test results indicate that the proposed connections outperform the current connection in addition to being more economical. Innovative concepts for the production of precast concrete box girders for highway bridges are also presented.

Keywords

Box girders, shear key, post-tensioning, transverse connections, bridge superstructure.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process.

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