HORIZONTAL SHEAR STRENGTH OF FULL-SCALE COMPOSITE BOX AND SLAB BRIDGE BEAMS HAVING HORIZONTAL SHEAR REINFORCEMENT WITH LIMITED DEVELOPMENT LENGTH

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ABSTRACT

In order for the composite action between the precast beam and the cast-in-place deck to be effective, sufficient horizontal (interface) shear strength needs to be provided at the interface of the two concrete elements to prevent slip. Horizontal shear strength is provided by three main components: the protrusions on the crack faces (cohesion/aggregate interlock), friction between the faces resulting from the normal compressive stress, and the dowel action of the reinforcing bars. AASHTO LRFD equation assumes that all reinforcement crossing the interface would be fully developed on both sides of the interface at ultimate shear strength.

This study examines if adequate horizontal shear capacity is provided by a very short embedded length (approximately 2 in.) commonly used in composite slab and box bridge beams. Four full-scale composite prestressed bridge beams (two beams with horizontal shear reinforcement and two beams without horizontal shear reinforcement) were tested to evaluate the contribution of dowel action of the reinforcement on horizontal shear capacity. These results were supplemented by tests on push-off and bar pullout specimens. Experimental results show that current AASHTO provisions for horizontal shear in composite concrete beams could over-estimate the horizontal shear contribution from dowel action of the reinforcement and do not represent the true behavior of the horizontal shear resistance mechanism.

Keywords: Precast Concrete, Prestressed Concrete, Composite Beam, Horizontal Shear Strength, Interface.

INTRODUCTION

The shear resistance at the interface between the precast element and the cast-in-place (CIP) element is of paramount importance in order to ensure the successful transfer of stresses. Sufficient shear resistance prevents the relative slip between the two elements thus fostering composite action. A good connection between the two components of the composite system can be achieved by artificially roughening the interface, providing a bonding agent, and/or using shear connectors or ties, mostly in the form of extended stirrups or hooks. When a crack between the surfaces occurs, horizontal shear reinforcement elongates across the crack providing a clamping force at the interface which controls the crack width. In order for the bars to develop their yield strength before pullout or debonding, the bars must be sufficiently anchored on both sides of the interface. It is believed that the clamping action of the reinforcement only comes into play after the crack between the surfaces slightly opens. The horizontal shear stress due to bending is equal in magnitude to the vertical shear stress and can be derived either based on the classical strength of materials approach or an alternative considering the shear force at strength limit state as given by AASHTO Section C5.8.4.2 (AASHTO, 2012)[1].

The horizontal shear at the interface is resisted by a combination of:

(1) Resistance of the protrusions on the crack faces to shearing (i.e. cohesion and/or aggregate interlock) also referred to as "cohesion factor" by AASHTO[1],

(2) Friction between the crack faces, and

(3) Dowel action of the reinforcement.

The AASHTO nominal shear resistance of the interface plane is given by:

$$V_{ni} = cA_{cv} + \mu\left(A_{vf} f_y + P_c\right) \tag{1}$$

The nominal shear resistance should however not be greater than the lesser of:

$$V_{ni} \leq K_1 f_c^{'} A_{cv}, \text{ or} \tag{2}$$

$$V_{ni} \leq K_2 A_{cv} \tag{3}$$

For "a cast-in-place concrete slab on clean concrete girder surfaces, free of laitance with surface roughened to an amplitude of 0.25 in."[1]:

c = cohesion factor = 0.28 ksi;

μ = friction factor = 1.0;

A_{cv} = interface area (in^2);

K_1 = fraction of concrete strength available to resist interface shear = 0.3;

K_2 = limiting interface shear resistance = 1.8 ksi for normal-weight concrete

Equation (2) is a limit preventing shearing or crushing of aggregates whereas equation (3) is due to lack of sufficient experimental data beyond the limit K_2.

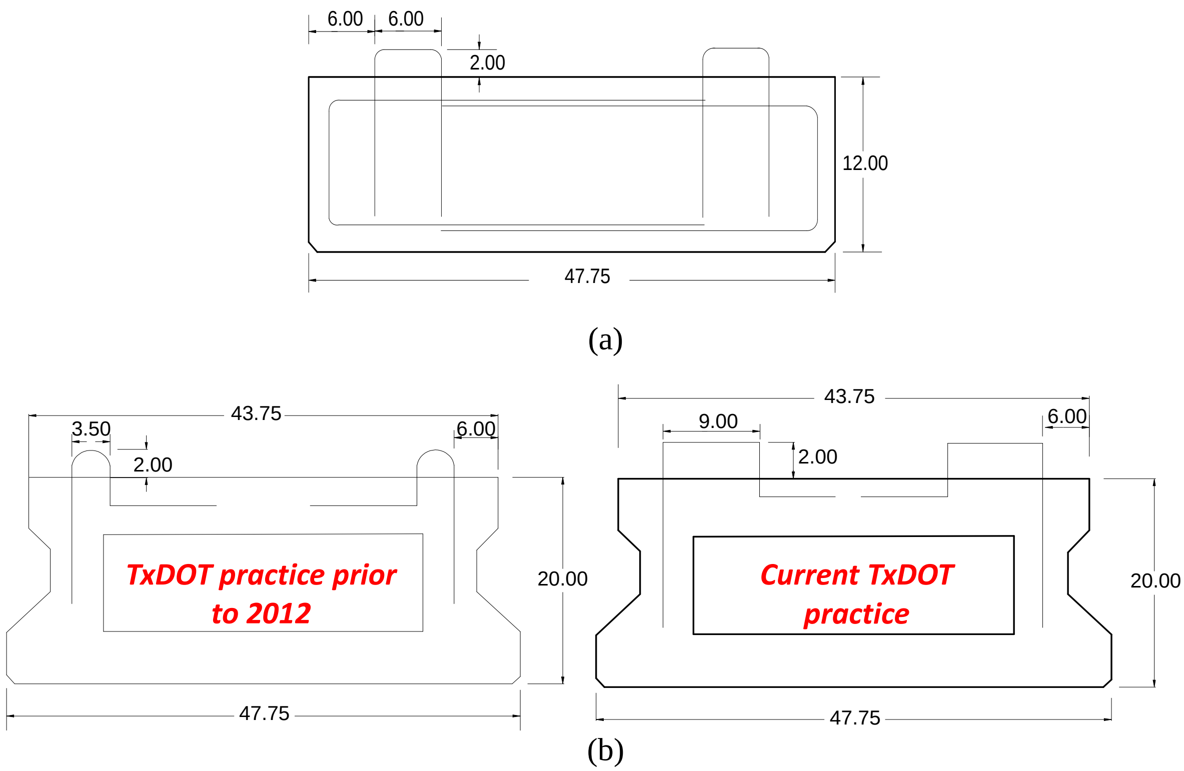
The shear friction concept is used in today's design specifications for horizontal shear transfer, to describe the behavior of a cracked material or an interface between two elements. Before cracking, interface shear is believed to be transferred mainly through concrete and through both concrete and reinforcing bars after cracking. Loss of contact at the interface will result in failure to transmit shearing forces. The loss of contact can occur due to crushing of the interlocking aggregates and cement paste. This occurs when either of these components reaches their compressive capacity which is directly related to concrete strength. When the two sides of a cracked specimen try to shear past each other, friction resists their motion. Friction is assisted by the clamping force provided by the dead load of the CIP slab as well as that provided by the reinforcement bridging the interface.

AASHTO limits the yield stress of interface reinforcement (f_y) to 60 ksi because previous research on pre-cracked specimens had determined that higher values of f_y overestimated the interface shear resistance[1].

RESEARCH SIGNIFICANCE

AASHTO Section 5.8.4.3 specifies c = 0.28 ksi and μ = 1.0 for a surface roughened to an amplitude of 0.25 in. The equation implies that both the cohesion factor c and friction factor μ are affected by the surface roughness. This surface roughness has been implemented in the TxDOT Standards Specifications[2] that states "*Finished, unformed surfaces must not have distortions greater than 0.25 in*." A number of the precast plants in Texas typically use wood float finish on box and slab beams. This is done by sliding a wooden float across the top of the wet concrete resulting in a coarse finish. In light of this fact, it was crucial to investigate the effects of a wood float surface finish on the shear transfer across an interface. It should be mentioned that providing a wood float finish does not guarantee a uniform surface roughness on all beams cast at the same or different precast plants. An effective means to improve the horizontal shear resistance is to specify a rougher finish (i.e. amplitude of roughness greater than 0.25 in.) on top of the beam to improve horizontal shear capacity. An experimental study carried out by Saemann and Washa[3] postulated that increasing the surface roughness leads to an increase in the horizontal shear strength. TxDOT implemented the ICRI[4] (International Concrete Repair Institute) guidelines for concrete surface preparation as a measure of surface roughness on their precast panels and are moving towards implementing it on precast beams as well. The ICRI offers nine distinct surface configurations identified as concrete surface profiles (CSP) ranging from nearly flat (CSP 1) to very rough (CSP 9). The Precast Panel-Fabrication Standard recommends that the top of the panel should be finished to a roughness ranging between a CSP 6 and a CSP 9. These different surface profiles were therefore incorporated into a separate study so as to recommend the roughness that will lead to an improvement in the horizontal shear resistance of the composite beams.

AASHTO LRFD Bridge Design Specifications (2012) section 5.8.4.1 requires that all reinforcement crossing the interface should be fully developed on both sides of the interface by embedment, hooks, or other method to develop the design yield stress. AASHTO Section 5.11.2.4 provides guidelines for determining the development length needed for standard hooks in tension. The equation provided results in an embedded length of 6.7-in. not possible in a 5-in. CIP slab. However, horizontal shear reinforcement do not qualify to be considered as “standard hooks” according to AASHTO and there is no equation suitable for typically horizontal shear reinforcement. Since the shear friction action of the interface shear reinforcement relies on yielding of the bars (Item 3 of Eq. 1), a short embedded length inside the composite slab can lead to localized concrete fracture prior to yielding, thus providing insufficient clamping force. Given that a 5-in. thick composite concrete deck is used on the current TxDOT prestressed slab beams and box beams, the embedded length of interface shear reinforcement is only about 2-in. (Figure 1).



Note: All units in inches

Fig. 1 Standard TxDOT (a) slab and (b) box beam details

This study also gathers information on the current practice used for box and slab beams of the 50 states in the U.S. The study set out to determine other DOTs’ practices especially as is concerned with the width and embedded depth of horizontal shear reinforcements. It was found that there are essentially six types of horizontal shear reinforcement (Figure 2 and Table 1) being used in different states. The embedded length of the horizontal shear reinforcement varied between 2-in. and 6-in. depending on the thickness of the CIP slab used in the respective state. It should be noted that some states do not use either box or slab beam or neither, while some states provide a thin asphaltic concrete as the wearing surface and hence do not provide horizontal shear reinforcement. An embedment length of about 2 in. was observed in nearly 70% of the states having box and slab beams.

The objective of this project is to determine if adequate horizontal shear capacity is provided by the 5-inch concrete deck on slab and box beams, despite lack of reinforcement development.

Table 1. State DOT configuration for horizontal shear reinforcement

Type of horizontal shear reinforcement	***State DOT***
1	*Ohio, North Dakota, West Virginia, Texas, Delaware, Illinois*
2	*Maryland, Michigan*
3	*Colorado, Missouri, Texas*
4	*Missouri, Alabama, Washington, Tennessee*
5	*Indiana, Kentucky, Minnesota, Pennsylvania, Texas**
6	*Maine, Massachusetts, Rhode Island*

* TxDOT standard prior to 2012.

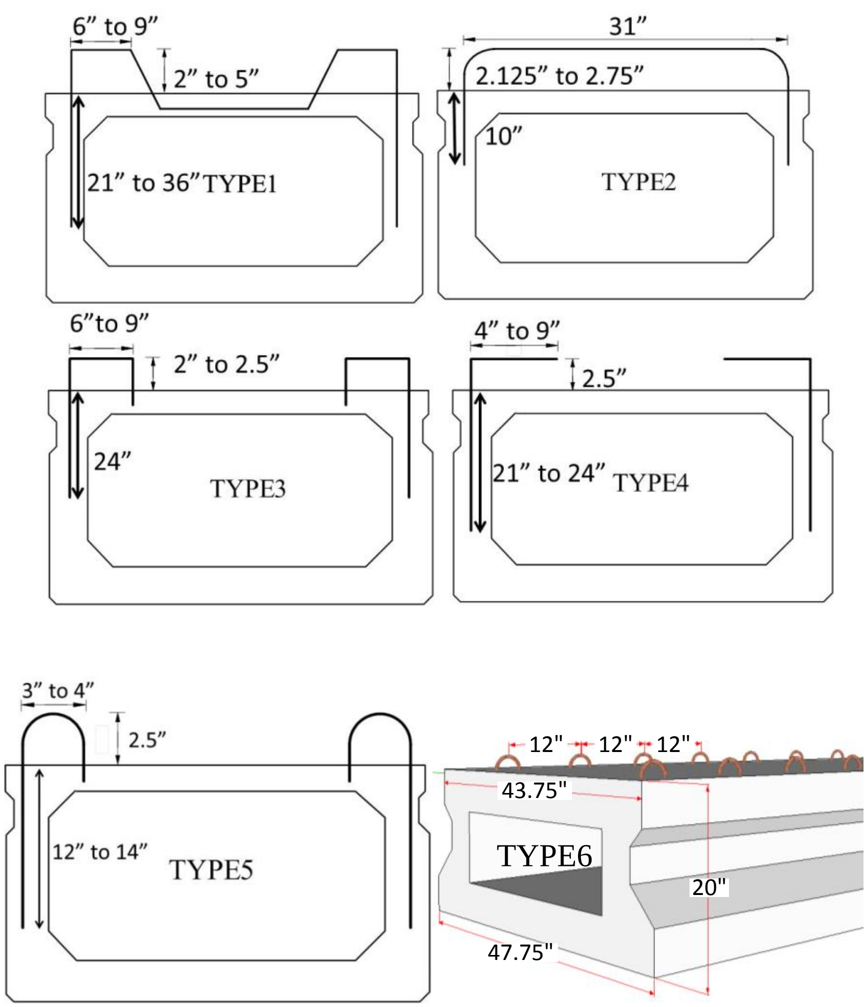


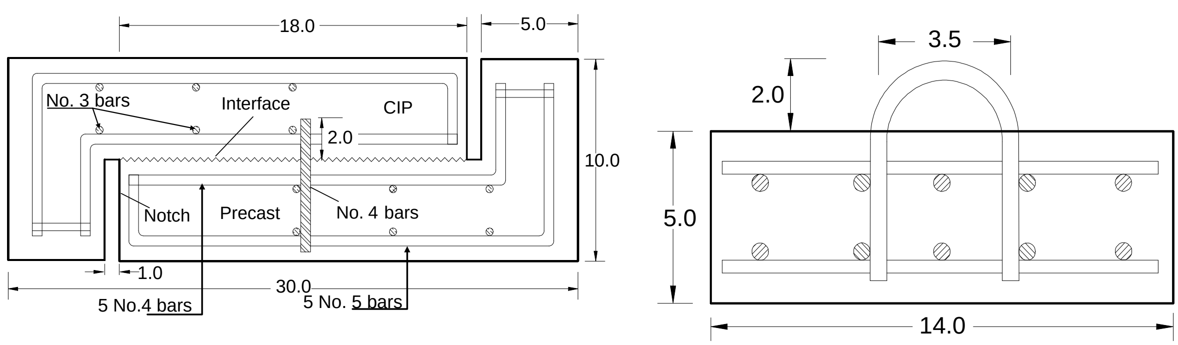
Fig. 2 Types of Standard box beams for DOTs in the US

EXPERIMENTAL PROGRAM

TEST SPECIMENS

Geometry of Push-off Specimens

Each test specimen measured $30 \times 14 \times 10$ in. ($762 \times 355.6 \times 254$ mm), this gives a shear interface area of 252 in^2 (18×14 in.). This interface area size is close to the largest interface areas used by prior push-off specimens. The specimen consisted of a 5-inch (127 mm) thick cast-in-place part (which is consistent with the cast-in-place slab on box and slab beams) on top of a 5-inch (127 mm) precast part. Minimum longitudinal and transverse reinforcement was provided to prevent premature flexural failure from occurring. Similar push-off specimen geometry and reinforcement layouts have been used by other researchers (Mattock and Hawkins (1972)[5] Khan and Mitchell (2002)[6]; and Hofbeck et al., (1969)[7]). It should be noted that this reinforcement layout provides a higher reinforcement ratio as compared to that used in actual girder/deck (Figure 3).



(a) (b)

Fig. 3 Reinforcement layout for push-off specimen (a) elevation view and (b) section view

Strain gauges were installed on the horizontal shear reinforcement on both sides so as to obtain strain information. The strain gauges were located 0.5-in. away from the interface to ensure they were not damaged once a crack occurred. The horizontal shear reinforcement was then tied to the reinforcement caging at approximately the center of the interface area (Figure 3a).

It was noted from studies done on other DOT practices that some states (Maine DOT, Rhode Island DOT and Massachusetts DOT) place the interface shear reinforcement perpendicular to the cross section. It was therefore decided that specimens having the interface shear reinforcement perpendicular to the cross section will also be tested to ascertain if there are any advantages in using this configuration.

Geometry of Pullout Specimens

Different pullout specimen geometries were proposed depending on the width of the horizontal shear reinforcement. The four different widths of horizontal shear reinforcement tested were 3.5-in. (88.9 mm), 6-in. (152.4 mm), 9-in. (228.6 mm), and 12-in. (304.8 mm). The 3.5 in. width reinforcements tested were of both a 180° and a 90° bend. The 6 in., 9 in., and 12 in. width specimens had two configurations; one having a continuous bar with a 90° bend and the other having two bars spliced at the center with a 90° bend angle. According to the TxDOT CIP slab standard drawings, the transverse reinforcement is spaced at 6-in. (152.4 mm) maximum whereas the longitudinal reinforcement is spaced at 12-in. (304.8 mm) maximum. Hence transverse reinforcement in all the specimens were provided at 6-in. (139.7 mm) and longitudinal reinforcement at 12-in. (304.8 mm) spacing with a 2.5-in. (63.5 mm) clear cover as specified in the standards. The TxDOT standard drawings also show that the horizontal shear reinforcement is located at 6 inches from the end of the beam. This dimension was maintained constant for all of the different bar widths, therefore the total length of each specimen was the bar width plus 6 inches on both sides, to represent the shortest dimension in the standard drawings that will result in more realistic confinement of the bar by the surrounding concrete as used in the actual decks. This resulted in four different specimen geometries; 15.5×12×5 in. (393.7×304.8×127 mm) for the 3.5-in. reinforcement, 18×12×5 in. (457.2×304.8×127 mm) for the 6-in. reinforcement, 21×12×5 in. (533.4×304.8×127 mm) for the 9-in. reinforcement, and 24×12×5 in. (609.6×304.8×127 mm) for the 12-in. reinforcement, as shown in Figure 4.

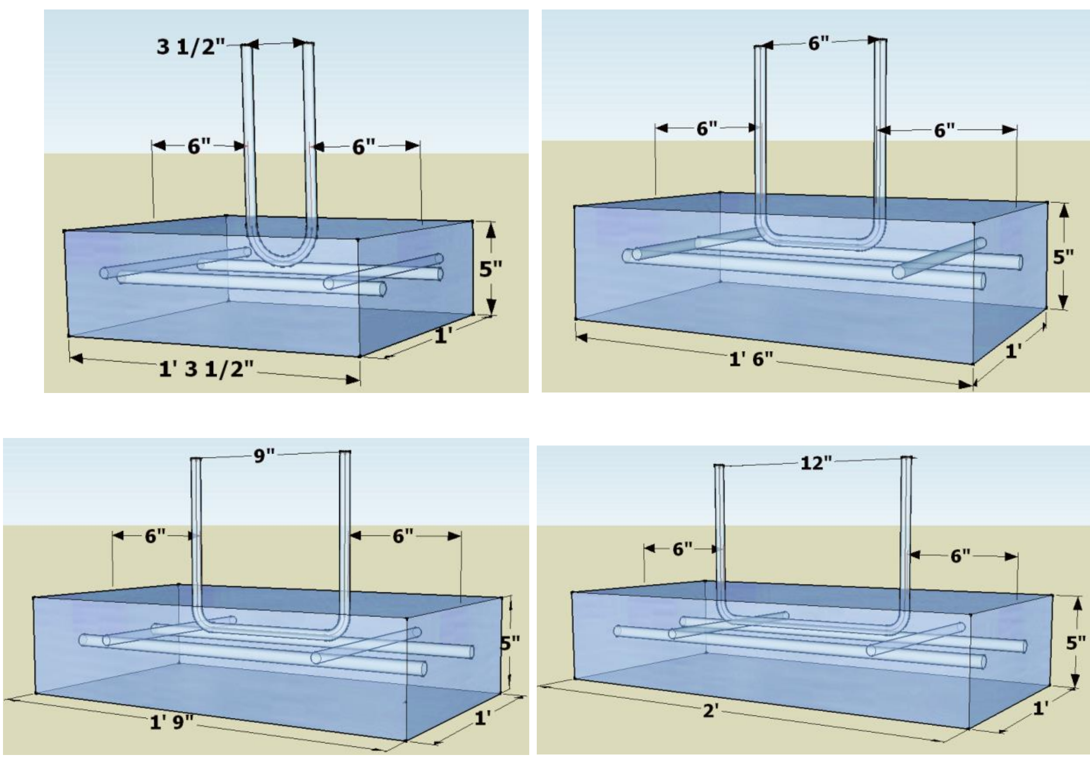
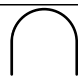
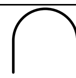
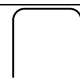
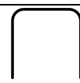
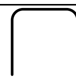


Fig. 4 Pullout specimen configuration

The test matrix for the component tests is as shown in Table 2.

Table 2. Push-off and pullout specimens test matrix

Bent-bar Configuration	Push-off	Pullout
3.5-in. width (180^o curvature)* 	3	3
3.5-in. width (180^o curvature, longitudinally placed) 	3	-
3.5-in. width and 90^o curvature (90^o curvature) 	-	3
6-in. width (90^o curvature) £ 	3	3
6-in. width (90^o curvature, 4 in. embedment) 	3	-
9-in. width (90^o curvature) § 	3	3
9-in. width (90^o curvature, 4 in. embedment) 	3	-
12-in. width (90^o curvature) 	-	3
6-in. width and 90^o curvature long tail 	-	3
9-in. width and 90^o curvature long tail 	-	3
12-in. width and 90^o curvature long ta 	-	3
Total	**18**	**24**

Note: *TxDOT practice before 2012 for box beams; £ Details as shown in TxDOT standard drawings for slab beams; §Current TxDOT practice for box beams; Width measured from center-to-center; #4 bar used in all configurations.

Full-Scale Beam Specimen Configuration

Two types of full-scale composite beams: TxDOT box beam (4B20) and TxDOT slab beam (4SB12) (see Table 3) were tested to determine if adequate horizontal shear capacity is provided by the 5-in. concrete deck in current TxDOT slab and box beams, despite lack of reinforcement development. "Adequate horizontal shear capacity" means that horizontal shear failure will not occur before reaching the beam's ultimate flexural strength. The precast beams with a 5-in. composite deck were constructed and instrumented at a fabricator in Houston and then delivered to UT Arlington Civil Engineering Laboratory Building (CELB) for testing. The slab beam measured 360×47.75×12in. whereas the box beam measured 360×47.75×20 in. One of the box (4B20#1) and slab beam (4SB12#1) was designed using the current reinforcement detail according to TxDOT[2] specifications so as to represent a typical beam used in practice (Figure 5). Strain gauges were mounted on the horizontal shear reinforcement to provide useful information to check the calculations.

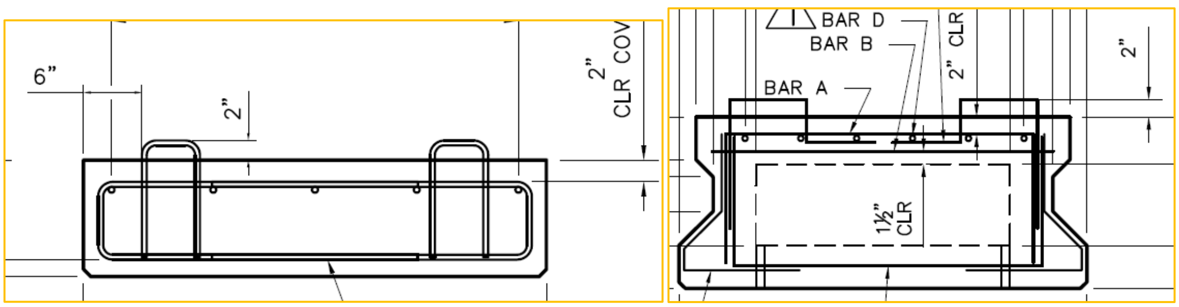
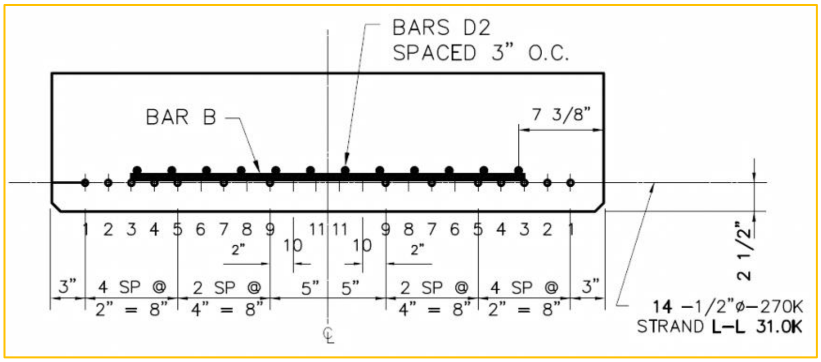


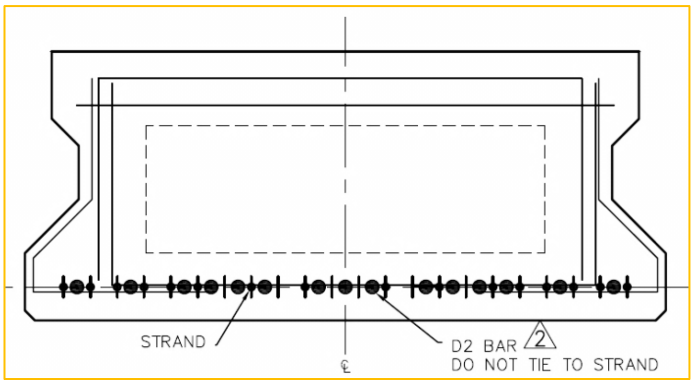
Fig. 5 Typical (a) slab and (b) box beam

The same detailing was maintained for the second box (4B20#2) and slab beam (4SB12#2) with the addition of flexural reinforcement so as to force a horizontal shear failure and evaluate the horizontal shear strength of the beam as a whole. A total of fifteen #8 rebars were added as flexural reinforcement in the box beam while twelve #8 rebars were added in the slab beams. These beams were designed to result in a shear demand higher than that in the strongest TxDOT box beams and slab beams typically used in practice. The flexural reinforcements were placed in the most convenient spacing to avoid obstruction of voids and strands (Figure 6). The beams were designed to ensure tension-controlled behavior and no premature shear failure even with the additional longitudinal reinforcement. Results from push-off and pullout tests had showed that horizontal shear reinforcement provided very minor contribution to the interface shear strength, hence they were completely eliminated in these beams. Strain gauges were mounted on the flexural reinforcements to measure strain on the bars during the test.

For the CIP part of the specimens having horizontal shear reinforcement, the horizontal shear reinforcement was instrumented with strain gauges at midspan and near the ends of the beams prior to casting. The reinforcement for the CIP was consistent with that used on TxDOT bridges. The surfaces of the precast beams were cleaned to remove dust and dirt particles before casting.



(a)



(b)

Fig. 6 Second set of specimens with additional longitudinal mild steel reinforcement (a) slab and (b) box beam

CASTING

Component specimens

The push-off specimens were cast in two parts with the precast part of the specimens cast with TxDOT Class "H" concrete (5000 psi). A wood float finish was provided at the interface on all the specimens as this is the finish used on box and slab beams by most of the precast plants in Texas. The CIP part of the specimens was then cast two weeks later with Class "S" concrete (4000 psi). The surfaces of the precast part were air blown to remove dust and dirt particles before casting of the CIP part.

The pullout specimens were cast with concrete Class "S" because they represent the slab portion of the superstructure. It has been observed in previous research including Rehm's [1969][8], that the direction of casting affects the bar slip (top-bar effect) hence the bars were cast in the direction in which they are normally cast on site. Concrete used to make the component specimens was supplied by a local commercial ready-mix concrete supplier.

Full-Scale Beam Specimens

The beams were prepared by a fabricator in Houston. The prestressing bed was first cleaned and sprayed with water because of the high temperatures on the day of casting. The prestressing strands were then drawn and stressed at a jacking force of 31 kips per strand. The longitudinal reinforcement was then placed followed by the stirrups. Strain gauges were installed on the longitudinal bars and on stirrups so as to monitor the strains on the bars. Cylinders were also made to test the compressive strength of the mix after 28 days. Since the fabricator typically provides a rake finish on all slab and box beams, a rake finish was provided on all the specimens as shown in Figure 7. The CIP part of the specimens was cast a few weeks later with concrete Class “S” as shown in Figure 8.

Fig.7 Rake finish on beam specimens

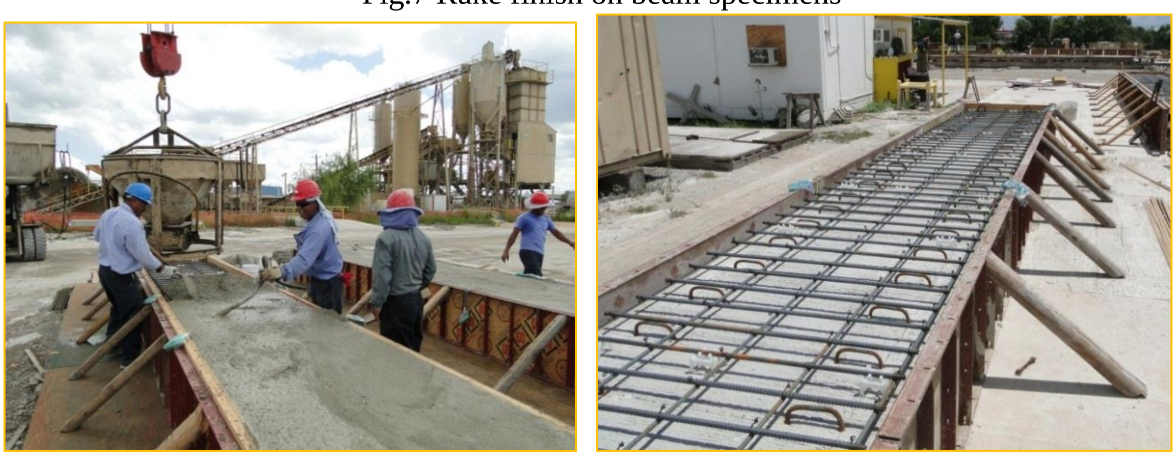


Fig. 8 CIP casting

TEST SETUPS AND PROCEDURES

Push-off Specimens

The test setup for the horizontal push-off test with horizontal shear reinforcement is shown in Figure 9. It consisted of a hydraulic cylinder that applies the horizontal force on the interface, a load cell to record the load being applied, and a W8×24 loading beam. A 14×1×0.5 inch loading strip was used to ensure minimal eccentricity of the applied loads. The specimen was instrumented with two Linear Variable Differential Transformers (LVDT's) placed on both the CIP and precast parts to measure the slip during testing. For specimens that had a 4-in. shear reinforcement embedded length, a vertical LVDT was placed to measure the crack opening at the interface.

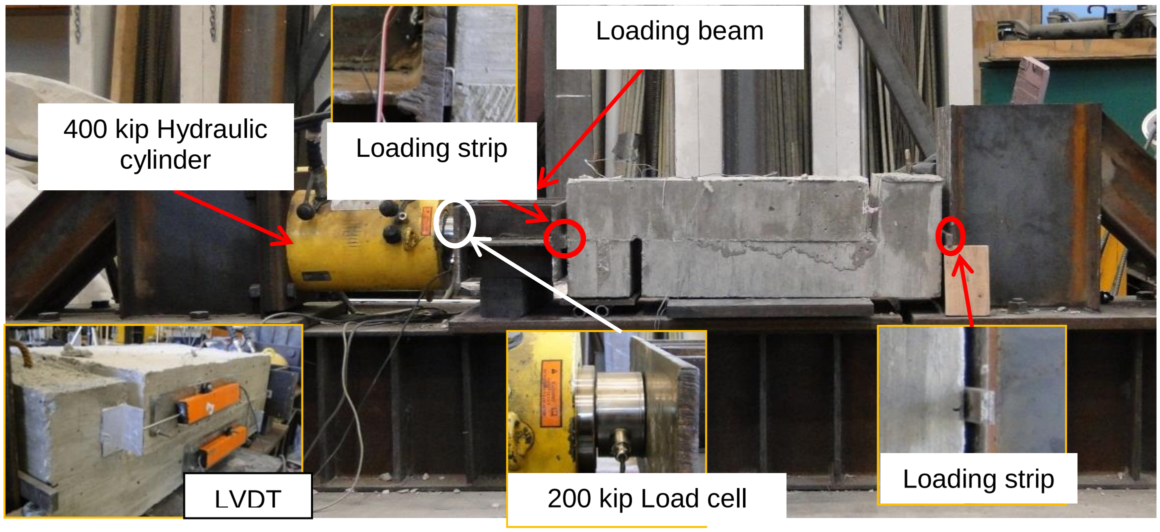


Fig. 9 Push-off test setup

Pullout Specimens

The test setup for the bar pullout test is as shown on Figure 10. It consists of a 100-kip servo-controlled closed-loop MTS machine, a top plate, and a bottom plate. The specimen is placed on top of the bottom plate and the two threaded bars are passed through the holes in the top plate which have been pre-drilled for each respective bar width. The specimen is restrained by two restraining blocks on the bottom block which have slotted holes to aid in adjusting the side plates to fit the specimens. The bars are fastened with a terminator onto the top block so as to be held in place as the MTS machine applies a tensile load. Linear variable differential transformers (LVDTs) were also provided to measure bar slip and strain gauges mounted on the bar to record bar strains.

Full-Scale Beams

The test setup for the full-scale beams (Figure 11) is composed of a reaction frame with a hydraulic cylinder attached to apply the load. Two W12×72 wide flange sections were used as the loading beam (stacked one on top of the other) so as to apply the load uniformly along the width of the beam. A load cell was placed between the hydraulic cylinder and the loading beam to accurately record the load being applied. The specimen was instrumented with three LVDT's placed at the interface to measure the relative slip between the precast and CIP parts during testing. Two LVDT's were placed under the midpoint of the beam to measure the displacement during loading. To reduce any load eccentricities, the actual position of the loading beam and load cell were marked on the specimen before testing.

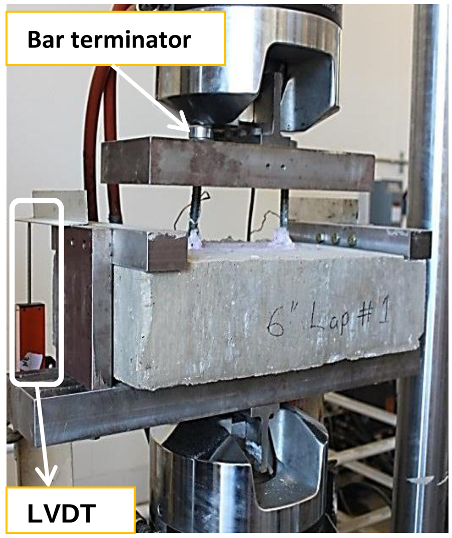


Fig. 10 Pullout test setup

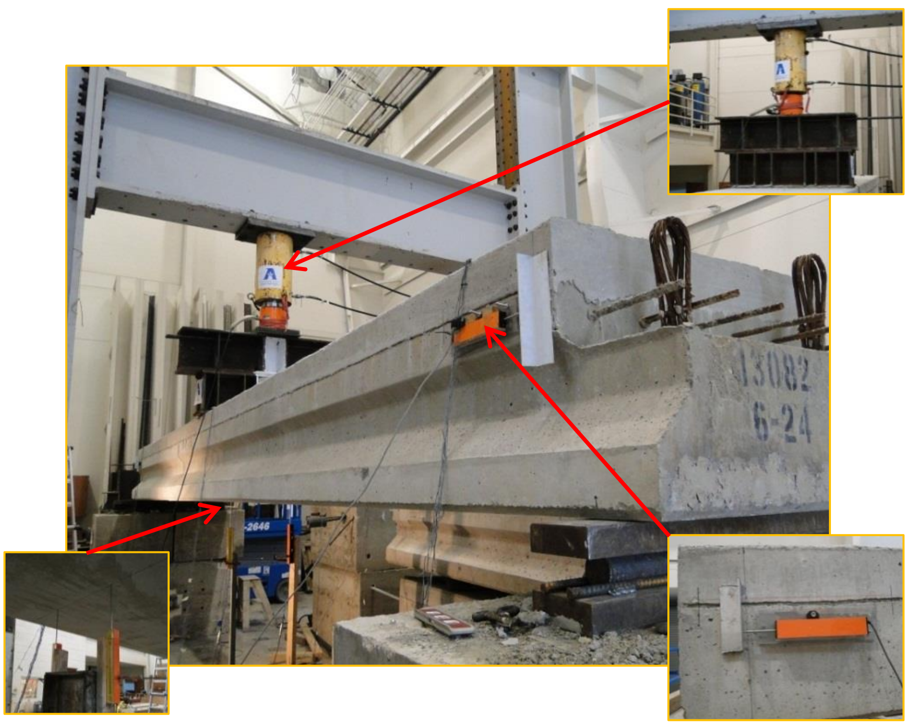


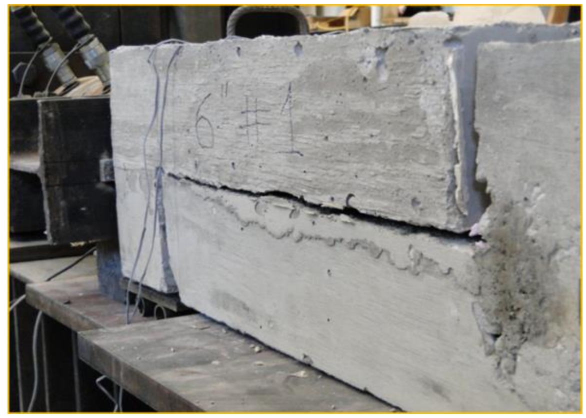
Fig. 11 Full-scale beam test setup

TEST RESULTS

GENERAL BEHAVIOR OF PUSH-OFF SPECIMENS

Results from specimens having horizontal shear reinforcement with a width of 3.5 inches placed in both the longitudinal and transverse location (used by Maine DOT, Rhode Island DOT and Massachusetts DOT), showed no significant change in the failure load. The bars did not yield and the main mechanism of failure was by bar pullout (Figure 12a). These results are consistent with the results from bar pullout tests, which indicated that the short embedded length (2-in.) could not provide sufficient bond to allow bars to yield before pullout. A bar stress of not more than 24 ksi was recorded for these specimens. Specimens having a 6-in. width horizontal shear reinforcement with a 90^{o} curvature, failed at the interface with the bar pulling out from the CIP part. An average peak load of 63.5 kips (282.46 kN) with larger fractured volume of the concrete on the CIP part was observed compared to the 3.5-in. horizontal shear reinforcement. The highest bar stress recorded was 18 ksi. On the other hand, specimens that had the 6-in. horizontal shear reinforcement embedded 4-in. into the CIP showed higher peak loads. Bars in all the specimens in this case were very close to nominal yield strength at failure with yielding occurring at a slip of approximately 0.1-in. No pullout of the bar was observed (Figure 12b) hence it was not possible to examine the failure plane after the test. The specimens having a 9-in. width bar with a 90^{o} curvature showed an increase in the average peak load to specimens 79.3 kips (352.74 kN). The failure was by bar pullout with no yielding experienced in all the specimens and a maximum bar stress of 19 ksi recorded. Severe fracture of the concrete in the CIP part of the specimen was observed in most of the specimens. This is consistent with the large fracture volume noticed in the bar pullout test with the same configuration. For the specimens having a 4-in. embedded length, yielding of the bar occurred at a slip of less than 0.1-in. An average peak load of 76.4 kips (339.84 kN) was recorded for these specimens. Strain gauge information shows very small strain on the bars before the maximum horizontal shear strength was reached in the specimens having a 2-in. embedded length. This indicates that the dowel action of the bar does not contribute to the maximum shear strength. Although the 4-in. embedded length specimens registered higher strains on the horizontal shear reinforcement, a majority of the specimens did not reach yielding at maximum peak load. The strain on the reinforcement markedly increases once a crack had occurred at the interface. This indicates that using the AASHTO equation could over-estimate the dowel action which assumes that the horizontal reinforcement can significantly contribute to the interface shear strength by yielding the reinforcement. Previous tests had been conducted on push-off specimens without horizontal shear reinforcement and having a woodfloat finish. Comparing those results to these, it was observed that there is a 50% increase in horizontal shear strength regardless of the geometry and embedment length. It was observed that although dowel action did not contribute to horizontal shear strength, the presence of horizontal shear reinforcement provided an overall increase in horizontal shear strength. The maximum horizontal shear strength recorded from these specimens was lower than the horizontal shear strength predicted by AASHTO equation (Figure 13). The results of the push-off test are tabulated on Table 3.

(a)



(b)

Fig. 12 Typical failure mode of push-off specimen failure with (a) 2-in. embedment; bar pulled out from the CIP slab (the concrete shown in the figure is the precast part); (b) 4-in.embedment; no pullout of the bar was observed

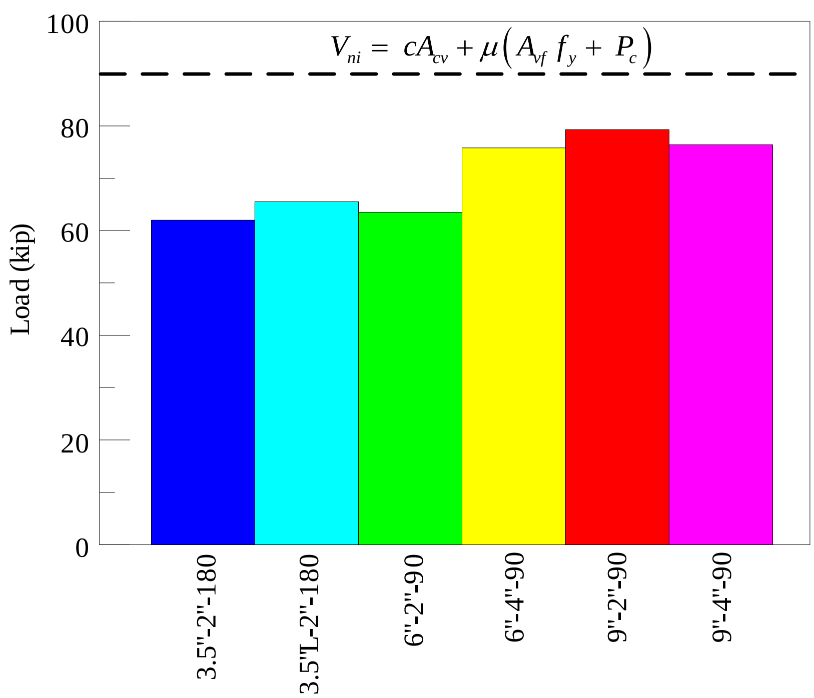


Fig. 13 Horizontal shear strength comparison.

Table 3. Push-off test results

Specimen A - B - C	*Failure Load (kip)*	*Average shear strength (kip)*	*Strain ε_{su} on bar at failure ($\mu\varepsilon$)*	*Stress σ_{su} on bar at failure (ksi)*
3.5”- 2”-180°	*65.7*	*62*	*143*	*4.1*
	64		*347*	*10*
	56.3		*826*	*24*
3.5”L- 2”-180°	*60.3*	*65.5*	*157*	*4.6*
	‡		*-*	*-*
	70.6		*323*	*9.4*
6”- 2”-90°	*61.5*	*63.5*	*294*	*8.5*
	63		*230*	*6.7*
	66.1		*617*	*17.9*
6”- 4”-90°	*80*	*75.8*	*1834*	*53.2*
	71		*1084*	*31.4*
	76.5		*2271*	*65.9*
9”- 2”-90°	*66.5*	*79.3*	*449*	*13.02*
	91.1		*655*	*19*
	80.2		*574*	*16.6*
9”- 4”-90°	*75.3*	*76.4*	*806*	*23.4*
	77.4		*2030*	*59*
	‡		*-*	*-*

**Specimen notation: (A) bar width, (B) Embedment length, (C) bend angle, L is reinforcement placed in the longitudinal direction. ‡Failure not at the interface (value neglected)*

GENERAL BEHAVIOR OF PULLOUT SPECIMENS

All the test specimens exhibited similar modes of failure as seen in Figure 14. First cracking of the concrete occurred on the front face and back face of the specimen radiating from the bar leading to a strength drop. As the load increased, cracks started forming around the bar. Once the cracks propagated around, the bar was gradually pulled out. The 12-in. specimens showed a more explosive failure after the concrete on the inside of the tail portion cracked. Observation of the specimens after failure also implies that the bar tends to pull away from the concrete on the outside of the bend.

The change in degree of bend and for the specimens having 3.5-in. bar width did not show a significant increase in strength. It was also observed that a 9-in. width horizontal shear reinforcement with a 90° bend (9-90) led to a marginal increase in the pullout strength compared to specimens having a 3.5-in. width reinforcement and a 180° bend (3.5-180). This means that the latest modification in TxDOT details will not lead to any improvement in the contribution to the horizontal shear resistance resulting from the horizontal shear reinforcement. However, the failure region did increase for a 90° bend compared to an 180° bend. The specimens having lapped horizontal shear reinforcements did not show any effect in the pullout strength or mode of failure.

Table 4. Pullout test results

*Specimen** *A-B*	*Block size (in.)*	*Bend Angle (deg.)*	*Failure Load (kip)*	*Strain ε_{su} on bar (με)*	*Stress σ_{su} on bar (ksi)*
3.5-180	*15.5 × 12 × 5*	*180*	*5.6*	*885*	*26*
			6.3	*522*	*15*
			5.3	*1011*	*29*
3.5-90	*15.5 × 12 × 5*	*90*	*5.3*	*1252*	*36*
			6.2	*927*	*27*
			6.8	*1055*	*31*
6-90	*18 × 12 × 5*	*90*	*6.3*	*1424*	*41*
			4.6	*1815*	*53*
			5.5	*978*	*28*
6-90L	*18 × 12 × 5*	*90*	*7.7*	*1387*	*40*
			6.3	*863*	*25*
			5.0	*1158*	*36*
9-90	*21 × 12 × 5*	*90*	*6.2*	*1241*	*36*
			6.5	*1348*	*39*
			6.0	*785*	*23*
9-90L	*21 × 12 × 5*	*90*	*4.1*	*509*	*15*
			6.1	*875*	*25*
			4.0	*957*	*28*
12-90	*24 × 12 × 5*	*90*	*5.7*	*1271*	*37*
			5.3	*995*	*29*
			8.1	*1230*	*36*
12-90L	*24 × 12 × 5*	*90*	*6.9*	*1477*	*43*
			6.9	*910*	*26*
			7.8	*1637*	*47*

**Specimen notation: (A) bar width, (B) bend angle, (L) spliced bars.*

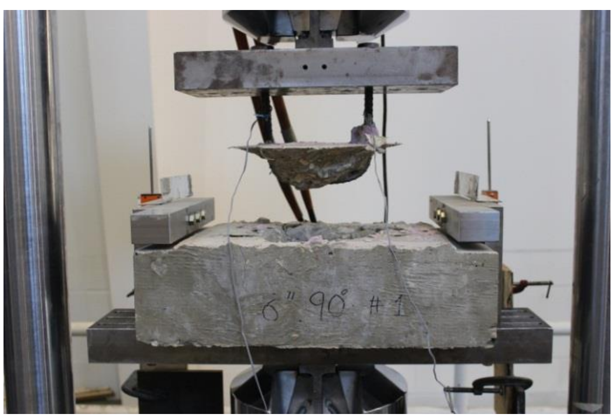


Fig. 14 Pullout test; modes of failure

GENERAL BEHAVIOR OF FULL-SCALE BEAMS

4SB12#1 (slab beam)

The load was first applied at 5-10 kip interval until the first flexural crack was observed at 55 kips, near the mid-span of the beam. With an applied load to 82 kips, the cracks continued to propagate eventually reaching the interface between the CIP slab and the precast beam. At 90 kips, the cracks propagated into the 5-in. CIP slab but not along the interface. The beam eventually failed in flexure at a peak load of 102 kips corresponding to a displacement of 7 in. Crushing of concrete occurred under the loading point and the cracks significantly widened to more than 6 mm. No cracks at the interface were observed and no slips were recorded. No significant strain was measured from the strain gauge data of the horizontal shear reinforcement, thus indicating that there was very minor contribution from the horizontal shear reinforcement.

4SB#2 (slab beam)

In the second slab beam which had no horizontal shear reinforcement, the first flexural crack was also observed at a load of 55 kips. The cracks had propagated into the CIP slab at a load of 180 kips. However, no cracks propagating along the interface were observed. The beam also failed by flexure at a load of 197 kips with crushing of the concrete at the loading point. The load then dropped to 177 kips after failure. With continued load application, an explosive failure occurred with a decrease in load of more than 100 kips and the formation of a horizontal crack 2-in. below the interface. The cracks widened to 5 mm and the concrete at the loading point was further crushed. The interface remained intact without any cracks forming across it. Although some hairline cracks propagated along the interface at failure on the south-face side, it did not extend further and was not observed on the north-face side of the beam.

4B20#1 (box beam)

The first flexural crack in the box beam specimen was observed at 110 kips. At a load of 170 kips, flexural cracks had progressed reaching the interface of the CIP slab and the precast beam. Cracks of 1.0 mm in width were recorded with some spalling being observed. At 189 kips, the cracks progressed into the CIP slab with crack widths as wide as 1.5 mm been recorded. It should be noted that the flexural cracks did not propagate along the interface but instead passed through the interface into the CIP slab. The beam failed in flexure due to concrete crushing in the compression zone within the CIP slab. No cracking was observed within the interface of the prestressed beam and the CIP slab.

4B20#2 (box beam)

Similar results were observed for the box beam without horizontal shear reinforcement. At 180 kips, cracks had progressed further up the beam with shear cracks being observed. At a load of 350 kips, the cracks had progressed up to the interface of the CIP and precast sections. Initial crushing of the concrete under the loading point was observed at 400 kips. With increase in load, the concrete crushed and the cracking propagated into the interface leading to a sudden horizontal shear failure. However, the horizontal shear crack did not propagate the entire length of the beam. Therefore no slip was recorded by the LVDTs at the ends.

Summary of all the test results is presented in Table 5.

Fig. 15 Full-scale beam specimens at failure

Table 5. Full-scale beam test results

Specimen	*Horizontal Shear Reinforcement*	*Design load (kips)*	*Failure Load (kips)*	*Failure Mode*	*Strain ε_{su} on horizontal shear reinforcement at failure (με)*
4SB12 #1	*Yes*	*82*	*101*	*Flexure*	*100*
4SB12 - modified	*No reinforcement*	*160*	*195*	*Flexure*	*-*
4B20 #1	*Yes*	*160*	*193*	*Flexure*	*250*
4B20 - modified	*No reinforcement*	*318*	*407*	*Flexure / Horizontal Shear**	*-*

* Horizontal shear failure was a secondary failure after flexural failure had occurred.

CONCLUSIONS

1. The 2-in. embedded length currently used in TxDOT practice for box and slab beams does not yield at ultimate capacity as revealed in the push-off test and supported by the bar pullout test as well as the full-scale beam test. All bars were pulled out at a lower bars stress before yielding. The stress of the horizontal shear reinforcement according to the measured strains was about 40% of the yield strength predicted by AASHTO equation (which assumes the bar would yield).

2. It is also observed that there is no significant difference in interface shear strength when the horizontal shear reinforcement was placed longitudinally or transversely to the beam length.

3. Increase in the width of the horizontal shear reinforcement did not provide any significant increase in the horizontal shear strength as demonstrated by the push-off tests.

4. Specimens having a 4-in. embedded length showed much higher strains at failure, suggesting that the bar was providing restraint from slipping and crack opening. Strain measurements also showed that the bars in all specimens with 4-in. embedded length had yielded by a slip of 0.1-in.

5. Results from the pullout tests indicate that there is no significant difference in strength when different widths of horizontal shear reinforcement and degree of bend are used. In all of the specimens tested in the bar pullout test, failure was induced due to fracture of concrete and there was no yielding in the horizontal shear reinforcement.

6. It is evident from the results of the full-scale tests that pure horizontal shear failure did not occur even with a very high shear demand. Throughout the testing of box and slab beams with horizontal shear reinforcement, only very small stresses between 3-7 ksi were noticed in the horizontal shear reinforcements. In essence all the beams failed due to flexure, with or without horizontal shear reinforcement. A horizontal shear crack was observed in the box beam reinforced with large amount of additional reinforcement; however the horizontal shear crack did not occur until after the flexural failure occurred. It also did not extend the full length of the beam.

7. The data suggests that the horizontal shear strength is predominantly controlled by the concrete behavior and not the shear reinforcement. Therefore roughening the surface is sufficient to avoid horizontal shear failure for typical TxDOT slab and box beams (and most of these types of beams used by other states). However, some minimum horizontal shear reinforcement may be useful in restraining separation of the interface near ultimate loading.

8. The tests conducted in this research have revealed that the AASHTO equation used to design for horizontal shear in composite concrete beams can overestimate the

contribution of the horizontal shear reinforcement and does not represent the true behavior of horizontal shear reinforcement with short embedded length.

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