

Influence of surface roughness on interface shear strength in composite concrete members

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- This paper presents a study on interface shear strength between precast concrete girders and cast-in-place slabs, focusing on how surface roughness affects cohesion and friction factors used in the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*.
- Push-off tests without interface shear reinforcement compared wood-float, rake, and International Concrete Repair Institute Concrete Surface Profiles (CSPs) 6 through 9 surface finishes. Rougher surfaces generally increased cohesion and friction, but benefits leveled off beyond CSP 7.
- The study also found that concrete compressive strength significantly influences cohesion and overall interface shear capacity, suggesting that interface behavior is governed not only by surface texture but also by the strength of the cement paste and coarse aggregate. Higher-strength concrete specimens exhibited substantially greater cohesion values.
- The study provides roughness-based values for cohesion and friction that may help AASHTO and state transportation agencies develop clearer surface preparation guidance for more consistent interface shear performance.

Composite concrete members are widely used to achieve stiffer and more economical structural sections in building and bridge applications. Effective composite action requires adequate shear resistance at the interface between the precast concrete element and the cast-in-place element to ensure reliable transfer of stresses between the two components. This resistance prevents relative slip between the two concrete elements, fostering effective composite action. A successful connection between two concrete elements is typically achieved by artificially roughening the existing surface, applying a bonding agent, and using shear reinforcing connectors or ties, often in the form of extended stirrups or hooks.¹ Bridge engineers often assume an intentionally roughened surface (surface roughened to an amplitude of 0.25 in. [6.35 mm]) as described in the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*² to calculate interface shear resistance between a cast-in-place concrete slab and a precast concrete girder. However, there is significant variability in achieved surface roughness among precast concrete plants, which suggests that interface shear strength may vary considerably in composite members depending on fabrication practices. This study investigates the influence of surface preparation on the cohesion factor c and friction factor μ used in the shear-friction provisions of the AASHTO LRFD specifications. To quantify interface shear resistance, the AASHTO LRFD specifications adopt a modified shear-friction theory first proposed by Mattock and Hawkins.³ This theory includes a cohesion term and was developed based on monolithically cast (that is, no cold joint)

and precracked push-off specimens. The AASHTO LRFD specifications suggest that the shear at the interface between a precast concrete component and a cast-in-place component is resisted by a combination of the following mechanisms:

- resistance provided by the shearing off of asperities projecting from the faces of the crack³ or by pieces of aggregate bearing on the surfaces of the slip plane and the shearing off of surface protrusions,⁴ commonly referred to as the *cohesion*
- friction between the crack faces that resulted from a net normal clamping force

Based on the definitions provided, both cohesion resistance and friction resistance are mobilized after cracking and interface sliding have developed, and they act concurrently during shear transfer. The shear friction mechanism described in the AASHTO LRFD specifications (**Fig. 1**) suggests that as the upper and lower portions of a rough interface slip across each other, a wedge is formed, causing the crack to dilate perpendicular to the interface. The reinforcement crossing the crack engages as the crack opens, leading to a clamping force represented by $A_{vf}f_y$ where A_{vf} is the area of interface shear reinforcement crossing the shear plane within the interface area and f_y is the yield strength of the interface shear reinforcement. Thus, the clamping action generated by the interface shear reinforcement is passive in nature. In addition to the clamping force, compression can be generated at the interface from loadings on the composite beam, regardless of the crack opening. Hence, the friction coefficient at the interface is engaged not only by the reinforcement bridging the interface but also by the normal force provided by loading, such as the self-weight of the cast-in-place slab and/or other permanent dead loads such as an asphalt wearing surface.

The nominal interface shear resistance V_{ni} , as specified by Eq. (5.7.4.3-3) of the AASHTO LRFD specifications, is as follows:

$$V_{ni} = cA_{cv} + \mu(A_{vf}f_y + P_c) \quad (\text{AASHTO LRFD 5.7.4.3-3})$$

where

c = cohesion factor

A_{cv} = area of concrete considered to be engaged in interface shear transfer

μ = friction factor

A_{vf} = area of interface shear reinforcement crossing the shear plane within the area A_{cv}

f_y = yield strength of the interface shear reinforcement (not to exceed 60 ksi [414 MPa])

P_c = permanent net compressive force normal to the shear plane

However, the nominal shear resistance used for design shall not exceed the lesser of $K_1f'_cA_{cv}$ or K_2A_{cv} , where K_1 and K_2 are modification factors and f'_c is the compressive strength of the weaker concrete on either side of the interface. Here, $K_1f'_cA_{cv}$ represents a limit that prevents shearing or crushing of aggregates, and K_2A_{cv} is an upper limit due to the lack of experimental data beyond the limit of K_2 . In accordance with article 5.7.4.4 of the AASHTO LRFD specifications, for a normalweight cast-in-place concrete slab on clean concrete girder surfaces without laitance and with the surface roughened to an amplitude of 0.25 in. (6.35 mm), the specified values are a cohesion factor c of 0.28 ksi (1.93 MPa), a friction factor μ of 1.0, modification factor K_1 of 0.3, and modification factor K_2 of 1.8 ksi (12.4 MPa). In contrast, for concrete placed against a clean concrete surface without laitance but not intentionally roughened, the corresponding values are reduced to a cohesion factor c of 0.075 ksi (0.52 MPa), a friction factor μ of 0.6, modification factor K_1 of 0.2, and modification factor K_2 of 0.8 ksi (5.5 MPa). These specifications clearly indicate that both the cohesion

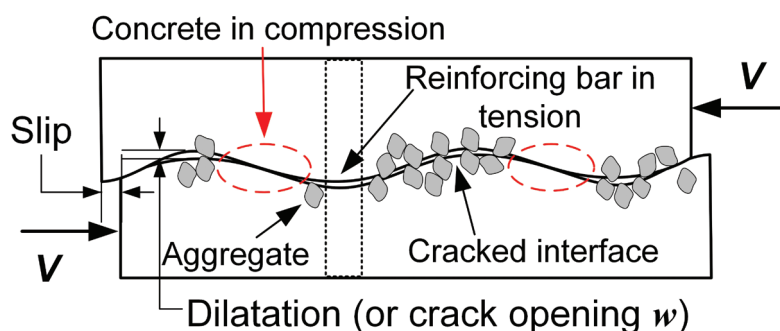


Figure 1. Shear friction mechanism. Note: V = shear force; w = dilatation or crack opening.

parameter c and the friction coefficient μ are significantly affected by the degree of surface roughness.

Notably, after analyzing test results from 509 push-off and pull-off specimens with steel reinforcement perpendicular to the interface and subjected to monotonic pure shear loading, Davaadorj et al.⁵ observed that among all the design specifications considered, the interface shear strength equation in the AASHTO LRFD specifications provides the most accurate strength estimate for intentionally roughened cold joint interfaces; however, the exact surface roughness in the push-off tests summarized by Davaadorj et al. was not reported.

Previous research findings

Hanson⁶ tested 62 push-off specimens and 10 composite T beams to investigate interface shear strength. The beams were designed to attain a high level of interface shear at the interface at a load significantly below that causing flexural failure. Hanson concluded that composite action seems to deteriorate starting at a critical slip value of 0.005 in. (0.127 mm), and he suggested that for a roughened bonded interface with concrete strength ranging from 3000 to 5000 psi (21 to 35 MPa), a maximum shearing stress for composite action should be 500 psi (3.44 MPa).

Seible and Latham⁷ conducted studies on the interface shear transfer of overlaid reinforced concrete bridge decks and investigated the effects of interface preparations and dowels on horizontal load transfer. They concluded that interface dowels are only beneficial in confining the crack after interface separation has occurred.

Loov and Patnaik⁸ conducted a comprehensive study on the interface shear strength of composite concrete beams with a roughened interface, covering a wide range of reinforcing steel ratios. The investigators tested 16 composite beams of varying geometries. The study focused on two primary variables: the clamping stress and the concrete compressive strength. The clamping stress was varied by changing the quantity of steel reinforcement crossing the interface and by adjusting the width of the precast concrete girders. The surface of the interface was maintained in its as-cast condition, with some exposed aggregate protruding. All beams were simply supported and subjected to a point load at the center of the span. The beams were designed such that interface shear failure governed the response. Loov and Patnaik reported that slip was negligible up to an interface shear stress of 220 to 290 psi (1.5 to 2.0 MPa) and increased with stress until reaching a slip ranging from 0.01 to 0.03 in. (0.25 to 0.76 mm). Beyond this point, the interface shear stress decreased as slip continued to increase. The investigators also observed that there was little difference between the interface shear stress at a slip of 0.2 in. (5.08 mm) and the interface shear stress at the peak load. In addition, the results indicated that the stirrups were not engaged until the interface shear stress reached approximately 220 to 290 psi and did not yield until the interface slip reached about 0.02 in. (0.51 mm).

The results indicated that the interface shear strength without reinforcement is a function of $\sqrt{f'_c}$. Loov and Patnaik suggested that protruding aggregates provided sufficient interface shear resistance, thereby eliminating the need for surface finishing. They further concluded that using elastic analysis with cracked transformed section properties is a valid and straightforward method for estimating interface shear stresses in composite concrete beams at failure.

Kovach and Naito⁹ investigated the shear friction behavior of girder-to-deck systems without shear connectors, with the objective of determining whether the allowable interface shear capacity between a precast concrete beam and a cast-in-place slab could be increased without the use of interface shear reinforcement. They suggested that cohesion and adhesion between the girder and deck could provide sufficient shear resistance.

Harries et al.¹⁰ found that the crack opening induced by interface slip, as illustrated in Fig. 1, was insufficient to fully engage the interface shear reinforcement until the shear resistance had already begun to decrease. They suggested that the clamping force provided by the interface shear reinforcement is governed by its stiffness rather than its strength. In addition, they proposed that the concrete compressive strength should be included in the concrete contribution term of AASHTO LRFD Eq. (5.7.4.3-3), expressed in a form such as $\alpha f'_c A_{cv}$, where α is a coefficient for the adjusted cohesion factor accounting for the effect of concrete compressive strength. This suggestion is consistent with the design recommendation by Mattock.¹¹ Harries et al. also stated that the introduction of the cA_{cv} term in the AASHTO LRFD equation may be unconservative and that the friction factor μ is not a true coefficient of friction but rather a surrogate parameter that additionally accounts for the effects of aggregate interlock and cohesion.

Waweru et al.¹² conducted push-off tests to investigate the clamping action generated by the interface shear reinforcement. The test results showed that a 2 in. (51 mm) embedded length into a cast-in-place slab is insufficient to fully develop the interface shear reinforcement to its yield strength f_y . Consequently, the interface shear reinforcement was pulled out of the cast-in-place slab well before yielding. In contrast, pullout failure was suppressed when the interface shear reinforcement was embedded 4 in. (102 mm) into the cast-in-place slab. Waweru and colleagues also observed that the interface shear reinforcement did not yield at the onset of slip; rather, yielding occurred after slip had already taken place.¹² This observation was further confirmed by full-scale composite beam tests also reported by Waweru et al.¹³ Overall, their study demonstrated that the inclusion of interface shear reinforcement increased the interface shear strength 50% or more, regardless of the geometry or embedded length of the interface shear reinforcement.

Li et al.¹⁴ investigated the feasibility of partially debonding the girder-to-deck interface to facilitate future deck removal and replacement. Composite prestressed concrete girders with

various interface conditions were tested, including partially debonded/partially troweled and partially debonded/partially roughened interfaces (with a roughness amplitude of 0.25 in. [6.35 mm]). The results showed that both troweled and roughened interfaces developed interface cracking at similar levels of applied force; however, after cracking, the roughened interface was more effective in limiting slip and transferring shear across the interface. In contrast, the troweled interface experienced significantly larger slip and exhibited no residual interface shear strength. Even at peak girder strength, specimens with substantial interfacial slip (up to 0.20 in. [5.08 mm]) retained partial composite action due to dowel action of the reinforcement.

Davaadorj et al.⁵ reported that shear strength of monolithic uncracked specimens increased with concrete compressive strength f'_c , though the rate of increase diminished at high values of $\rho f'_y$ and f'_c , where ρ is the reinforcement ratio, indicating a potential upper bound on shear capacity. Higher concrete strength was found to be particularly beneficial under high clamping stresses. The authors noted that design specifications do not adequately capture these effects and recommended, as suggested by Harries et al.¹⁰ and Mattock,¹¹ that the cohesion term (the first term in AASHTO LRFD Eq. [5.7.4.3-3]), be expressed as a function of both interface conditions and concrete compressive strength rather than interface conditions alone.

Although prior studies have not reached a consensus on the amount of slip required to cause yielding of the interface shear reinforcement, these studies consistently indicate that slip must occur before the interface shear reinforcement becomes engaged in resisting interface shear. To develop the clamping force $A_v f_y$, the interface crack must open sufficiently to mobilize the reinforcement; however, increasing crack width inevitably degrades the cohesion (or aggregate interlock) component of the shear-friction mechanism, resulting in a reduction in cohesion (or aggregate interlock) resistance

at the interface. A key finding is that at the ultimate interface shear capacity, the interface shear force is resisted primarily by cohesion and aggregate interlock (represented by the cohesion factor c in AASHTO LRFD Eq. [5.7.4.3-3]) and by friction (represented by the friction factor μ in AASHTO LRFD Eq. [5.7.4.3-3]) within the concrete, with only a minor contribution from the interface shear reinforcement. The interface shear reinforcement becomes significantly engaged in tension only after slip or separation occurs at the interface.^{10,12,13,15–16}

Variability in concrete surface roughness and its impact on cohesion and friction factors

Increasing surface roughness is generally associated with higher interface shear strength, highlighting the critical role of surface preparation in precast concrete elements,¹⁷ as both cohesion and aggregate interlock are key mechanisms governing shear transfer across the interface. The degree of roughness directly influences the initial cohesion and aggregate interlock resistance before the development of significant slip. A smoother surface may result in premature reliance on interface shear reinforcement, whereas a rougher surface enhances mechanical interlock and delays slip. Consequently, accurate quantification of surface roughness is essential for improved understanding and prediction of interface shear behavior, particularly in evaluating the relative contributions of cohesion/aggregate interlock, and reinforcement engagement.

Although concrete surface roughness can vary depending on fabrication practice and individual judgment, the International Concrete Repair Institute (ICRI) provides standardized guidelines for surface preparation and roughness characterization.¹⁸ Those guidelines define nine concrete surface profiles (CSPs), ranging from CSP 1, which represents a nearly smooth surface, to CSP 9, which represents a very rough surface. CSP 5 through CSP 9 are shown in

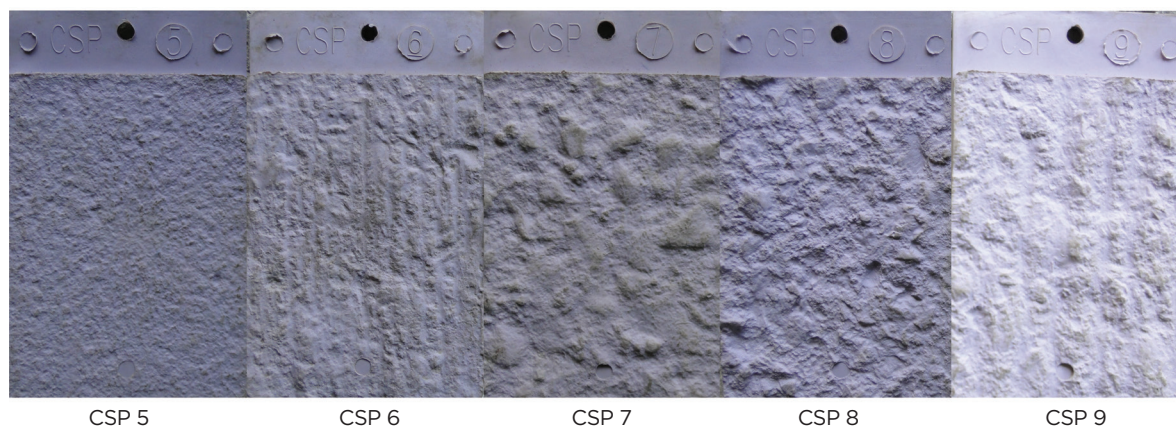
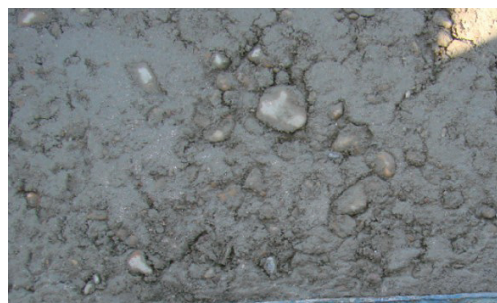


Figure 2. International Concrete Repair Institute concrete surface profiles (CSPs).

Fig. 2. Several quantitative methods are available to assess concrete surface roughness, including the sand patch method (ASTM E965¹⁹) and the two-dimensional laser roughness analyzer (2D-LRA) method.²⁰ The ICRI CSP system offers a rapid and cost-effective alternative that can be readily implemented in precast concrete element fabrication without disrupting production.

Five common types of surface finishes are applied to fresh concrete:²¹

- As-placed roughness: The surface remains untreated, with no effort made to smooth or roughen it after the concrete is placed and vibrated.
- Float finish: After placing and vibrating, a rough wooden float is used to smooth the surface (**Fig. 3**).
- Rake finish: A rake is dragged across the surface perpendicular to the beam length, creating a highly textured finish (**Fig. 3**).
- Broom finish: A stiff broom is drawn transversely across the beam surface to create texture (**Fig. 3**).
- Sheepfoot voids: This mechanical surface finish is created by pressing circular impressions into the surface, with each indentation having a specific depth and spacing to achieve the desired roughness.



Wood-float finish and the finished surface



Rake finish and the finished surface



Broom finish and the finished surface

Figure 3. Examples of concrete surface finishes.

Table 1. Summary of department of transportation surface finish survey

Surface finish	State departments of transportation
Wood float	Colorado, Kentucky, Maine, Michigan, North Dakota, Ohio, Texas, Washington, West Virginia
Rake finish	Alabama, Massachusetts, Rhode Island, Texas
Broom finish	Minnesota, Pennsylvania, Texas

A survey conducted for this study by the authors of representatives of multiple departments of transportation indicates that three surface finishes are predominantly used, as summarized in **Table 1**:

- broom
- rake
- wood float

Among those finishes, the wood-float finish is the most commonly applied method for concrete surface finishing.

For a normalweight cast-in-place slab placed on a clean concrete girder surface without laitance, article 5.7.4.4 of the AASHTO LRFD specifications specifies a cohesion factor c of 0.28 ksi (1.93 MPa) and a friction factor μ of 1.0 for a surface roughened to an amplitude of 0.25 in. (6.35 mm). In addition, item 424 in section 4.2.6 of the Texas Department of Transportation's (TxDOT's) *Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges* states the following:²²

Finishing of Concrete. Finished, unformed surfaces must not have distortions greater than 1/4 in. [6.35 mm]. Screed or rough-float unformed surfaces of members, bringing enough mortar to the surface to embed and cover the coarse aggregate. Provide a uniform rough wood-float finish to the surface of the member unless otherwise shown on the plans. Do not loosen aggregate when roughening the surface using a broom or when providing a tine finish.

TxDOT's *Prestressed Concrete Panel Fabrication Details* further recommends that the top of precast concrete panels be finished to a roughness corresponding to CSP 6 through CSP 9.²³

However, quantifying the roughness produced by typical wood-float finishing is challenging, and it is therefore not known whether typical wood-float finishing produces a surface roughness corresponding to a cohesion factor c of 0.28 ksi (1.93 MPa) and a friction factor μ of 1.0, with an amplitude of 0.25 in. (6.35 mm). During visits to several precast concrete plants in Texas, we observed that the achieved surface roughness can vary significantly among plants (**Fig. 4**), suggesting that interface shear strength may vary considerably in composite members depending on the fabrication practices employed.

This study provides values of the cohesion factor c and friction factor μ obtained from push-off specimens without interface shear reinforcement for different levels of surface roughness, thereby providing information on how surface preparation affects these parameters. Such information may help AASHTO and state transportation agencies develop more specific recommendations for surface preparation methods needed to achieve consistent interface roughness and reliable interface shear strength.

Experimental program

Push-off specimens

The reinforcement layout and specimen dimensions used in this study are presented in **Fig. 5**. The push-off specimen was designed to represent the precast concrete and cast-in-place elements of a composite beam, with geometry and reinforcement layout comparable to those used in previous research.^{3,15,24} The specimens were intentionally constructed without interface shear reinforcement crossing the cold joint. The specimens measured 30 × 14 × 10 in. (762 × 356 × 254 mm) and featured one of the largest interface areas reported in previous push-off tests, providing a shear interface area of 252 in.² (18 × 14 in.) [162,600 mm² (457 × 356 mm)]. Each specimen consisted of a 5 in. (127 mm) thick cast-in-place slab cast atop a 5 in. segment representing the precast concrete beam. Longitudinal and transverse reinforcement was included to prevent flexural failure prior to shear failure along the interface. No. 4 (13M) bars were placed longitudinally at both the top and bottom of the specimen, spaced at 3 in. (76 mm) on center, whereas no. 3 (10M) bars were placed transversely at 5 in. on center.

Notably, in the original shear-friction theory, the contribution of cohesion was neglected because it was assumed that cracking at the interface could occur under the most unfavorable conditions, such as tensile stresses induced by restrained shrinkage, temperature-related deformations, or accidental impact during handling. Consequently, the predicted interface shear strength represents a conservative lower-bound estimate.^{3,25} Testing on monolithically cast specimens with and without precracking indicates that precracked specimens exhibit a lower ultimate interface shear capacity.³ However, as discussed earlier, Mattock and Hawkins,³ who first proposed the modified shear-friction theory based on tests of initially cracked push-off specimens, defined cohesion as the “resistance to shearing off of asperities projecting from the faces of the crack.” This definition implies that cohesion provides a postcracking resistance mechanism. This



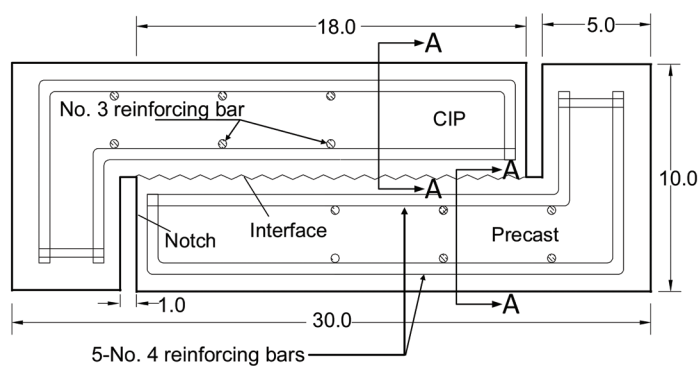
Figure 4. Observed variations in surface roughness produced by Texas precast concrete plants.

behavior can be examined using the results shown in Fig. 5 from a push-off test.

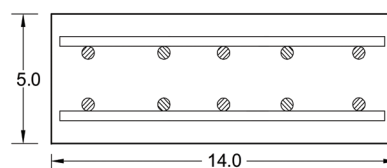
To measure crack widths (dilatation) along the interface, a digital image correlation (DIC) noncontact deformation measurement system with a strain-measurement accuracy of 0.01% (in./in. [mm/mm]) was used to capture full-field strains and displacements. The specimen was tested in a horizontal orientation because the loading frame obstructed the DIC field of view when the specimen was positioned vertically. The friction generated by the compressive stress resulting from the self-weight of the concrete above the interface (approximately 180 lb [800 N]) was negligible. Therefore, the measured interface shear resistance can be attributed primarily to cohesion. The crack-width distribution along the interface prior to peak load is shown in Fig. 5. As observed, cracking had already

developed along the interface before the specimen reached its peak load. This observation supports the interpretation that cohesion is a postcracking resistance mechanism and that significant cohesion-related shear resistance can be mobilized after cracking has occurred along the interface.

The results in Fig. 5 show that dilatation occurs as slip develops along the interface. Because no interface shear reinforcement crossed the interface in the test specimens, crack width was able to develop without restraint, which led to a reduction in shear strength due to diminished aggregate interlock.²⁶ In contrast, a composite bridge girder always contains at least the minimum required interface shear reinforcement crossing the interface. Consequently, any crack opening associated with slip would be restrained, which helps preserve aggregate interlock across the interface and thus contributes to shear



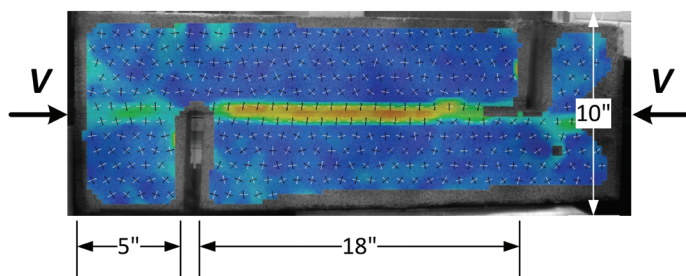
Elevation view



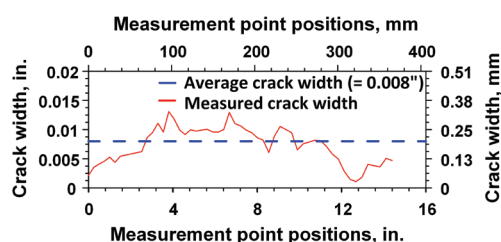
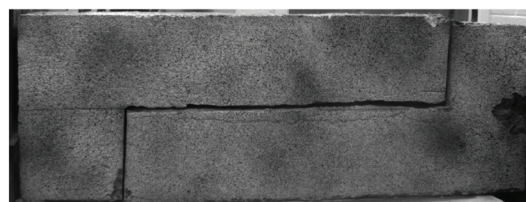
Section A-A

Section view

Immediately before reaching the peak load and failure



After failure



Typical crack width (dilation) distribution along the interface of a push-off specimen, from a digital image correlation noncontact deformation measurement system, before and after reaching the peak load (primarily governed by cohesion).

Figure 5. Reinforcement layout of the push-off specimen and displacement results. Note: All dimensions are in inches. CIP = cast-in-place; V = shear force. No. 3 = 10M; no. 4 = 13M; 1" = 1 in. = 25.4 mm.

strength. Conversely, the test specimens were not precracked, which could result in higher measured interface shear strength than would occur at a precracked interface. However, shrinkage- or temperature-induced cracking in an actual composite bridge girder is likely to be localized rather than continuous along the entire interface over the span. Therefore, the uncracked test condition is considered a reasonable representation of the overall interface condition in an actual composite bridge girder.

Although the test results are compared with values of the cohesion factor c and the friction factor μ specified in the AASHTO LRFD specifications, the experimental conditions may not fully represent the actual interface behavior in a composite bridge girder. Therefore, the results are not intended to justify modifications to the values of the cohesion factor c and the friction factor μ in the AASHTO LRFD

specifications. Instead, they are used to evaluate the influence of surface roughness on interface shear strength under controlled experimental conditions.

Concrete surface profile preparation

To approximate ICRI CSPs on fresh concrete, we developed interface preparation methods that use common finishing tools, such as concrete rakes and brooms, to expose aggregate along the interface. **Figure 6** shows the tools used, and **Table 2** summarizes the corresponding procedures. Using these methods, surface profiles ranging from CSP 6 to CSP 9 were produced for the push-off test specimens. Five concrete surface roughness levels—wood-float finish, CSP 6, CSP 7, CSP 8, and CSP 9—were prepared at the Civil Engineering Laboratory at the University of Texas at Arlington to investigate the relationships between interface roughness and both

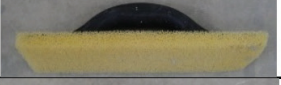
Wood float	
Coarse sponge float	
Rubber float	
Concrete rake	
Concrete broom	

Figure 6. Tools used for surface roughness preparation.

Table 2. Concrete surface profile surface preparation methods*

Surface finish	Tools	Technique
CSP 6	Concrete broom	One brush in the same direction
CSP 7	Sponge float	Slide three times in the same direction
CSP 8	Concrete broom	One brush back and forth
CSP 9	Concrete rake, concrete broom, sponge float	Slide once with rake, once with broom, once with sponge, in that order, all in the same direction

* These techniques are applied after the concrete has been leveled using a trowel or wood float immediately following concrete placement. Approximately 30 minutes later, a sponge float is lightly passed over the surface to remove any excess bleed water, after which each specific technique is performed.



Figure 7. Wood-float finish, showing the preparation process and the resulting surface.



Figure 8. Concrete surface profile (CSP) 6 finish, showing the preparation process and the resulting surface.



Figure 9. Concrete surface profile (CSP) 7 finish, showing the preparation process and the resulting surface.



Figure 10. Concrete surface profile (CSP) 8 finish, showing the preparation process and the resulting surface.



Figure 11. Concrete surface profile (CSP) 9 finish, showing the preparation process and the resulting surface.

the cohesion factor c and the friction factor μ .

Figures 7 through 11 illustrate the preparation processes for the wood-float finish and CSP 6 through CSP 9 surfaces. **Figure 12** compares the ICRI CSP templates with the

concrete surface roughness generated using the procedures summarized in Table 2. As shown in Fig. 12, the methods described in Table 2 can reasonably reproduce the target surface profiles. The CSP classifications were determined primarily through visual comparison with the standard ICRI CSP profile

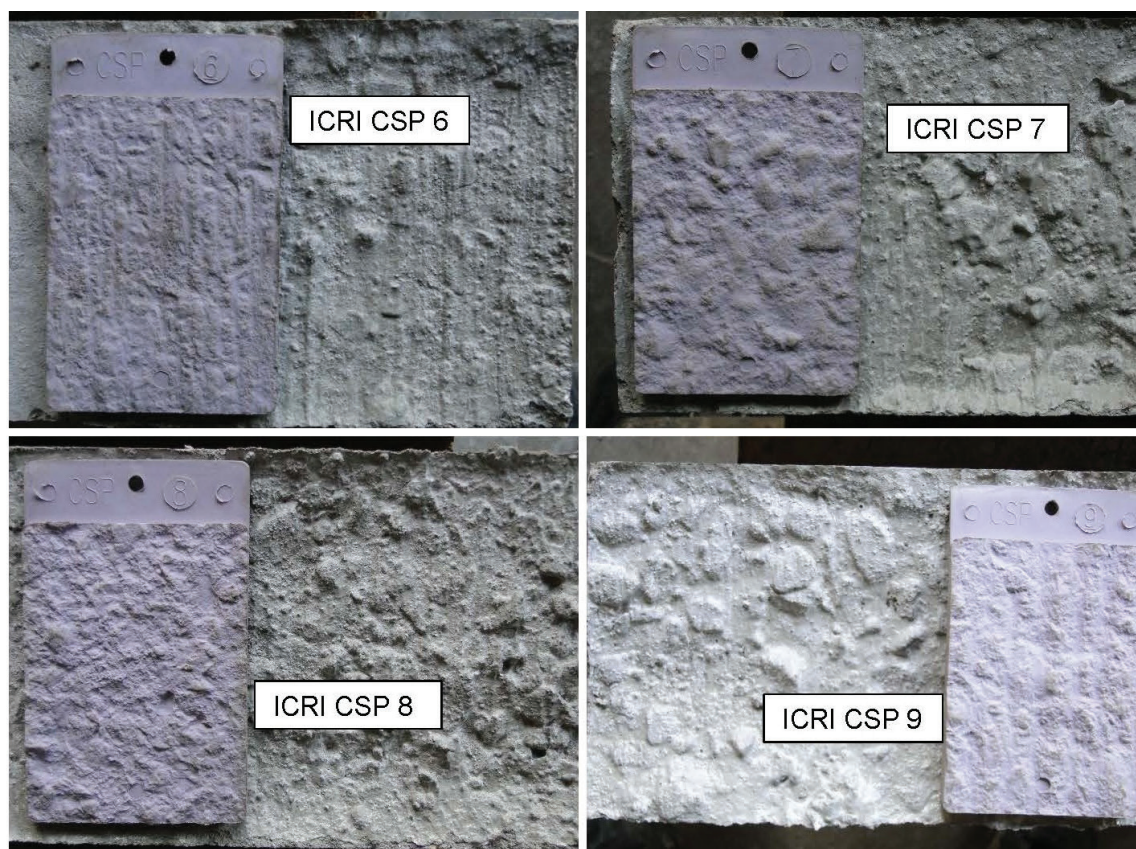


Figure 12. Comparison of International Concrete Repair Institute (ICRI) concrete surface profile (CSP) templates and concrete surfaces produced using the developed surface preparation methods.

templates and were, therefore, based on engineering judgment. Currently, no universally adopted code-based method exists for quantitatively measuring concrete surface profile roughness instrumentally.

Specimen construction and material properties

For each surface roughness level, six specimens—three to determine the cohesion factor *c* and three to determine the friction factor *μ*—were fabricated (Table 3). The notches in the specimens were formed with 1 in. (25 mm) thick closed-

cell extruded polystyrene foam pieces (Fig. 13). Ready-mixed concrete with a specified compressive strength of 5000 psi (35 MPa) was used to cast the precast concrete portion of each push-off specimen (Fig. 13). After casting, the surfaces of the precast concrete portions were finished using the methods shown in Fig. 7 through 11. Both the specimens and the accompanying concrete cylinders were then covered and cured under similar conditions.

The cast-in-place portions of the specimens were cast one week later. Before the cast-in-place concrete was placed, the surfaces of the precast concrete sections were air blown to remove dust,

Table 3. Experimental matrix for push-off specimens

Push-off specimens	Number of shear resistance components tested		Number of tests
	cA_{cv}	μP_c	
	Cohesion	Friction	
Wood-float finish*	3	3	6
Wood-float finish†	2	0	2
Rake finish†	2	0	2
ICRI surface roughness CSP 6*	3	3	6
ICRI surface roughness CSP 7*	3	3	6
ICRI surface roughness CSP 8*	3	3	6
ICRI surface roughness CSP 9*	3	3	6
Total			34

Note: A_{cv} = area of concrete considered to be engaged in interface shear transfer, per unit length of interface; *c* = cohesion factor; CSP = concrete surface profile; ICRI = International Concrete Repair Institute; P_c = permanent net compressive force per unit length normal to the shear plane; *μ* = friction factor.

*Specimen cast in the laboratory. The average 28-day compressive strength was 5.0 ksi (35 MPa) for the precast concrete segment and 4.5 ksi (31 MPa) for the cast-in-place segment.

†Specimen cast at the precast concrete plant. The average 28-day compressive strength was 11 ksi (76 MPa) for both the precast concrete and cast-in-place segments.

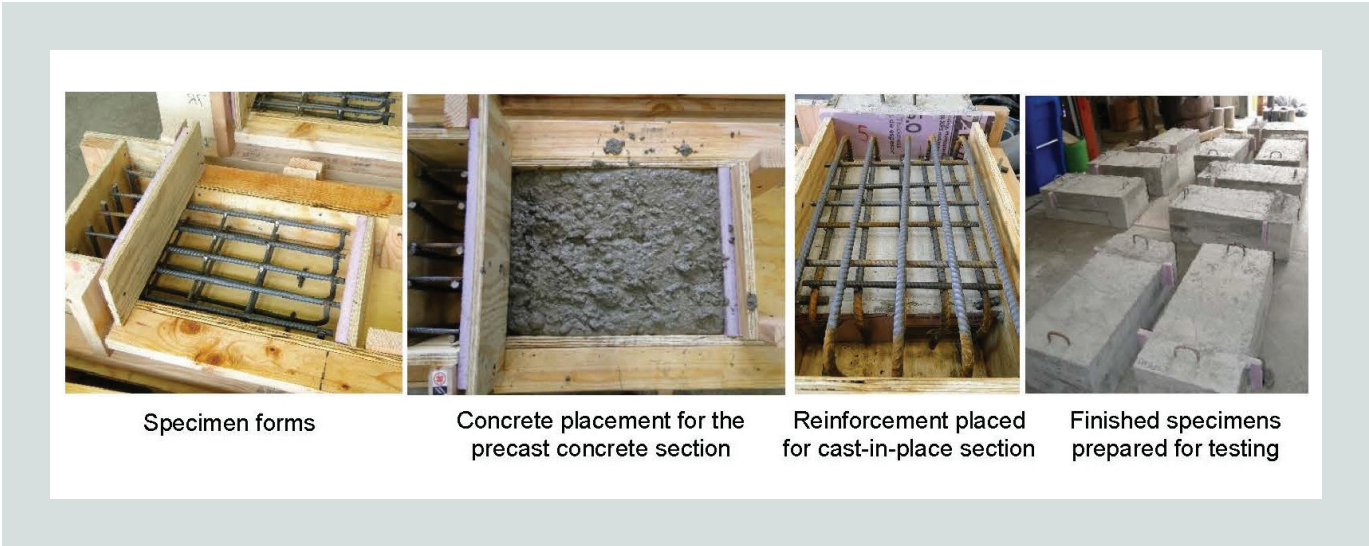


Figure 13. Casting of push-off specimens.

loose particles, and laitance and then the reinforcement cage was positioned (Fig. 13). The cast-in-place concrete had a specified compressive strength of 4000 psi (28 MPa). Following casting, the specimens were covered with plastic sheets and cured for 28 days. After curing, the specimens were unmolded and prepared for testing (Fig. 13). Standard tests (ASTM C31²⁷ and ASTM C39²⁸) were conducted to evaluate the concrete compressive strength. At 28 days, the measured compressive strength reached 4500 psi (31 MPa) for the cast-in-place portion and 5000 psi (35 MPa) for the precast concrete portion.

As shown in Table 3, two wood-float-finished push-off specimens and two rake-finished specimens were also fabricated at a precast concrete plant that produces precast, prestressed concrete girders for TxDOT. The measured 28-day compressive strength of the concrete was 11 ksi (76 MPa) for both the precast concrete and cast-in-place portions.

Test setup and instrumentation

We conducted push-off tests with methodologies similar to those reported by Hofbeck et al.,¹⁵ Mattock and Hawkins,³ Kahn and Mitchell,²⁴ and Harries et al.¹⁰ Two test configurations were used to independently evaluate the cohesion and friction components of the nominal interface shear equation from the AASHTO LRFD specifications.

The first test configuration was designed to isolate the cohesion contribution cA_{cv} by placing the specimen in a vertical orientation. (Most of the prior studies also tested specimens in a vertical configuration.) No reinforcement crossed the interface in the push-off specimens, and the vertical placement effectively eliminated the normal force P_c . Consequently, the second and third terms in AASHTO LRFD Eq. (5.7.4.3-3), representing the friction contribution ($A_v f_y + P_c$) were eliminated from

the shear-friction equation. As a result, the measured shear resistance reflected the cohesion contribution in AASHTO LRFD Eq. (5.7.4.3-3). The test setup for the vertical push-off test (Fig. 14) consisted of a 400 kip (1780 kN) universal testing machine, a load cell, a W12 × 79 (W310 × 118) steel section used as the loading beam, and two steel loading strips. The load cell is narrower than the specimen width, which could have resulted in a nonuniform load distribution along the interface if the wide-flange loading beam had not been used. Therefore, wide-flange sections were selected to provide a more uniform load distribution, allowing the load to transfer through the web to the loading strip. One steel bar measuring 16 × 1 × 0.5 in. (406 × 25 × 13 mm) was welded to the underside of the wide-flange section to serve as a loading strip, while the second strip of the same dimensions was placed beneath the specimen at midpoint. These steel strips minimized load eccentricity, which could otherwise induce undesirable flexural stresses in the specimens if the wide-flange section were in direct contact with the specimen. Lateral bracing was installed on both sides to restrain the concrete sections in the event of interface separation at failure. The load cell was secured to the machine to prevent displacement during sudden failure. Each specimen was instrumented with two linear variable differential transformers (LVDTs) on each segment to measure slip between the precast concrete and cast-in-place portions during testing. The vertical load was then applied at a rate of approximately 40 to 60 lb/sec (0.18 to 0.27 kN/sec) until failure occurred.

The second test employed a horizontal push-off setup to evaluate the friction component of the nominal interface shear equation in the AASHTO LRFD specifications. This configuration was chosen because placing the specimen in a horizontal position facilitates the application of a larger normal force by the universal testing machine to develop frictional resistance at the interface. As shown in Fig. 15, the specimens

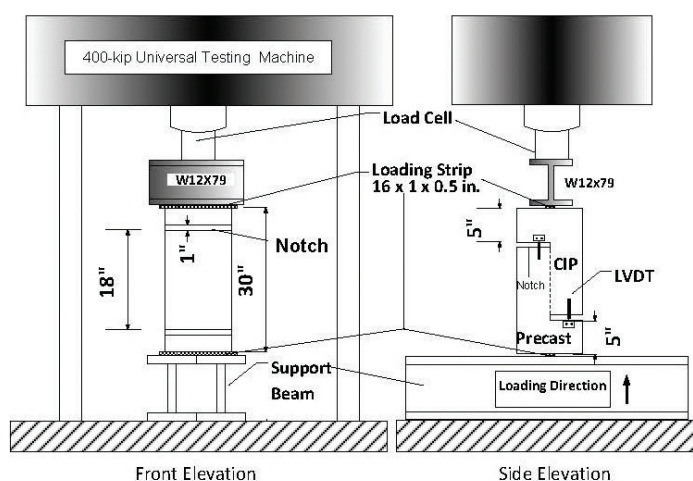
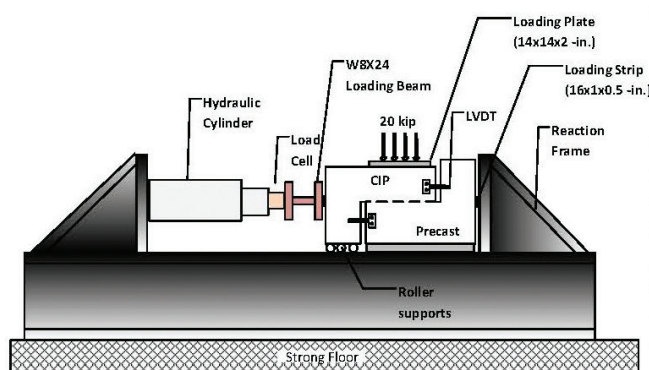


Figure 14. Test setup for push-off tests evaluating the cohesion factor. Note: CIP = cast-in-place concrete; LVDT = linear variable differential transformer. W12 × 79 = W310 × 118; 1" = 1 in. = 25.4 mm; 1 kip = 4.448 kN.

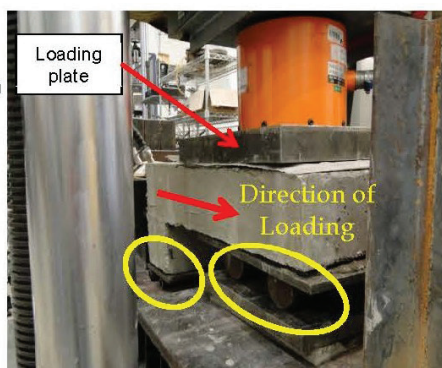
were positioned horizontally to engage the contribution of the normal force. Because the specimens contained no interface reinforcement, the shear resistance attributed to the friction component μP_c was determined by subtracting the results of the first test, which represented the cohesion and aggregate interlock contributions. The horizontal push-off test setup had a 400 kip (1780 kN) universal testing machine to apply a constant normal force, a load cell to monitor the applied load, a hydraulic cylinder to impose the horizontal force at the interface, and a W8 $\times$ 24 (W200 $\times$ 36) loading beam. To ensure direct load transfer, 16 $\times$ 1 $\times$ 0.5 in. (406 $\times$ 25 $\times$ 13 mm) steel strips were positioned on both sides of the specimen. Each specimen was instrumented with two LVDTs on both the cast-

in-place and precast concrete portions to measure slip during testing.

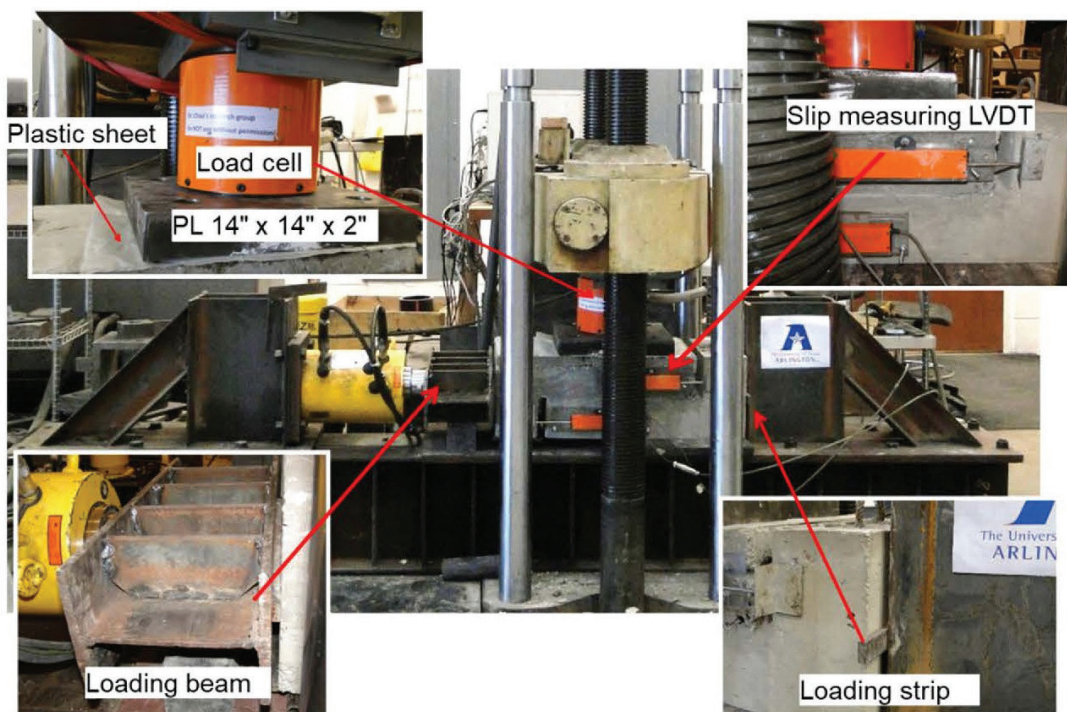
The compression load was applied to the specimen through a 14 $\times$ 14 $\times$ 2 in. (356 $\times$ 356 $\times$ 51 mm) loading plate (Fig. 15). The top surface of the push-off specimen was grouted, and a 2 in. thick plate was placed between the specimen and the load cell. Grout was evenly applied between the loading plate and the specimen to ensure uniform load transfer, and a plastic sheet was placed on top of the grout prior to positioning the loading plate to minimize friction. To minimize load eccentricities, the loading beam and load cell were carefully aligned and marked at the specimen center. The compressive force was applied ver-



Horizontal test configuration



Friction testing between compression loading plate, plastic sheeting, and specimen



Testing details

Figure 15. Horizontal push-off test setup for evaluating the friction factor. Note: CIP = cast-in-place concrete; LVDT = linear variable differential transformer; PL = plate. W8 $\times$ 24 = W200 $\times$ 36; 1" = 1 in. = 25.4 mm; 1 kip = 4.448 kN.

tically by the 400 kip (1780 kN) machine and maintained at a constant 20 kip (89 kN) before the horizontal load was applied. The horizontal load was then applied at a rate of approximately 100 lb/sec (0.44 kN/sec) until failure occurred. The load cell used for the vertical specimens had a maximum capacity of 450 kip (2000 kN), whereas the load cell used for the horizontal specimens had a maximum capacity of 200 kip (890 kN). Both load cells had an accuracy on the order of $\pm 0.5\%$ of full scale. Therefore, the measurement accuracy would be approximately ± 2.25 kip (± 10 kN) for the load cell used for the vertical specimens and ± 1.0 kip (± 4.45 kN) for the load cell used for the horizontal specimens.

Because some friction was expected between the compression loading plate and the plastic sheet placed on top of the specimen, a separate test was conducted to quantify the contribution of that friction to the total measured shear (Fig. 15). This setup used a

segment from a failed push-off specimen to evaluate the friction effect. Steel rollers were placed beneath the specimen to prevent shear transfer between the specimen and the plate resting on the rollers. A compressive load of 20 kip (89 kN) was applied and maintained while the horizontal load was gradually increased. The specimen moved almost immediately upon application of the horizontal force. The force required to initiate movement, attributed to friction between the loading plate and the push-off specimen, was 0.5 kip (2.2 kN), indicating that the plastic sheet was effective in reducing friction to a negligible level.

Experimental results

Cohesion factor test

Table 4 summarizes the failure loads, shear stresses, and cohesion factors obtained from the push-off test results. In

Table 4. Results of push-off tests for cohesion factor c

Specimen identification ^a	Surface finish type	Failure load, kip	Shear stress, ksi [†]	Average shear stress c , ksi	28-day f'_c , ksi [‡]
L-WF-1	Wood float	39	0.15	0.14	4.5
L-WF-2	Wood float	24	0.09		4.5
L-WF-3	Wood float	46	0.18		4.5
L-C6-1	CSP 6	57	0.23	0.20	4.5
L-C6-2	CSP 6	54	0.21		4.5
L-C6-3	CSP 6	42	0.17		4.5
L-C7-1	CSP 7	50	0.20	0.23	4.5
L-C7-2	CSP 7	71	0.28		4.5
L-C7-3	CSP 7	53	0.21		4.5
L-C8-1	CSP 8	57	0.23	0.21	4.5
L-C8-2	CSP 8	61	0.24		4.5
L-C8-3	CSP 8	42	0.17		4.5
L-C9-1	CSP 9	48	0.19	0.21	4.5
L-C9-2	CSP 9	57.5	0.23		4.5
L-C9-3	CSP 9	55	0.22		4.5
P-WF-1	Wood float	76	0.30	0.39	11
P-WF-2	Wood float	121	0.48		11
P-RF-1	Rake	112	0.45	0.46	11
P-RF-2	Rake	119	0.47		11

Note: 1 kip = 4.448 kN; 1 ksi = 6.895 MPa.

^aEach specimen is identified by the specimen identification listed in the first column. The three components of the identification denote the casting location (L = laboratory or P = precast concrete plant), the surface finish type (WF = wood float or RF = rake finish), concrete surface profile (CSP) (C6 = CSP 6, C7 = CSP 7, C8 = CSP 8, or C9 = CSP 9), and the specimen test number (1, 2, or 3).

[†]The shear stress was calculated by dividing the measured failure load by the interface area of 252 in.² (18 × 14 in.) [162,600 mm² (457 × 356 mm)].

[‡] f'_c is the measured 28-day compressive strength of the weaker concrete on either side of the interface (cast-in-place or precast concrete).

accordance with the nominal interface shear equation of the AASHTO LRFD specifications, the cohesion factor c was calculated by dividing the measured peak load by the contact area. Each specimen is identified by the specimen identification listed in the first column of Table 4, which denotes the casting location (L = laboratory or P = precast concrete plant), surface finish (WF = wood float or RF = rake finish, or concrete surface profile [C6 = CSP 6, C7 = CSP 7, C8 = CSP 8, or C9 = CSP 9]), and specimen test number (1, 2, or 3). **Figure 16** summarizes the average cohesion factors for the various surface roughness levels at two different 28-day compressive strengths, together with a cohesion factor c of 0.28 ksi (1.93 MPa) specified in article 5.7.4.4 of the AASHTO LRFD specifications for a normalweight cast-in-

place concrete slab placed on a clean concrete girder surface without laitance and roughened to an amplitude of 0.25 in. (6.35 mm). As shown in Fig. 16, all surface finishes evaluated in this study for the 4.5 ksi (31 MPa) concrete exhibited cohesion factors lower than the value specified in the AASHTO LRFD specifications.

Test specimens with a compressive strength of 4.5 ksi (31 MPa) Among all laboratory-produced specimens with a 28-day compressive strength f'_c of 4.5 ksi (31 MPa), the wood-float-finished specimens exhibited the lowest interface shear resistance, with an average cohesion factor of 0.14 ksi (0.97 MPa). This value is approximately 50% lower than the cohesion factor c of 0.28 ksi (1.93 MPa) specified in article

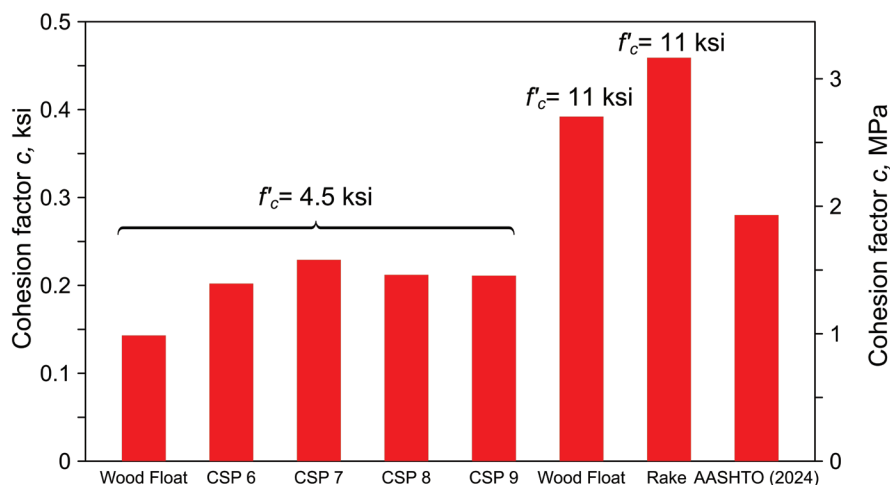


Figure 16. Comparison of cohesion factor values. Note: AASHTO (2024) = cohesion factor c of 0.28 ksi specified in article 5.7.4.4 of the AASHTO LRFD specifications for a normalweight cast-in-place concrete slab placed on a clean concrete girder surface without laitance and roughened to an amplitude of 0.25 in.; CSP = concrete surface profile; f'_c = measured 28-day compressive strength of the weaker concrete on either side of the interface (cast-in-place or precast concrete). 1 in. = 25.4 mm; 1 ksi = 6.895 MPa.

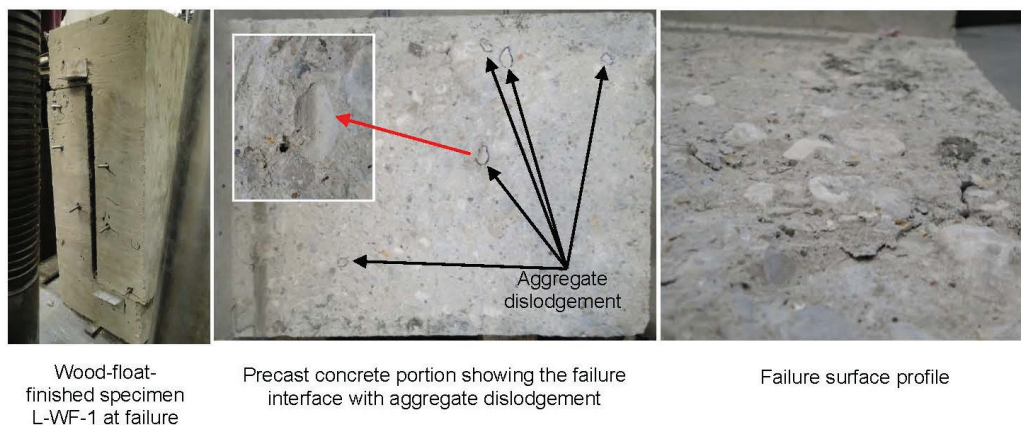


Figure 17. Wood-float-finished specimen L-WF-1 at failure.

5.7.4.4 of the AASHTO LRFD specifications. For all specimens, failure occurred at the interface and no flexural cracks were observed (**Fig. 17**). The use of a wood float to finish the surface may partially cover exposed aggregate with cement paste, resulting in a weaker bond at the interface between the old and new concrete. That likely explains why the wood-float-finished specimens exhibited lower cohesion factors than specimens with rougher surface finishes.

The three wood-float-finished specimens failed at loads of 39, 24, and 46 kip (173, 107, and 205 kN), indicating noticeable variability in the measured shear stresses. Postfailure observations of the precast concrete portion, where the surface finish was applied, indicated coarse aggregate dislodgement. Specimens L-WF-1 and L-WF-3, which exhibited more extensive aggregate dislodgement, developed higher shear stresses than specimen L-WF-2, which showed less aggregate dislodgement. This behavior suggests that effective aggregate interlock is achieved when sufficient contact exists between the exposed aggregate of the precast concrete segment and the concrete of the cast-in-place segment. The failure surface photos in **Fig. 17** illustrate the aggregate dislodgement and failure surface profile in the precast concrete portion of specimen L-WF-1, and **Fig. 18** shows the failure interface of specimen L-WF-2.

An increase in shear stress was observed for the CSP 6 specimens, which exhibited an average cohesion factor of 0.20 ksi (1.38 MPa). The precast concrete surfaces of the CSP 6 specimens showed greater aggregate exposure than those of the wood-float-finished specimens (**Fig. 19**). As surface roughness increased from CSP 6 to CSP 7, the average shear stress increased by approximately 0.03 ksi (0.20 MPa). **Figure 19** shows representative precast concrete surfaces of the CSP 7 specimens. Those specimens exhibited an average cohesion factor of 0.23 ksi (1.58 MPa), with measured shear stresses ranging from 0.20 to 0.28 ksi (1.93 MPa).

The CSP 8 specimens (**Fig. 19**) exhibited an average cohesion factor of 0.21 ksi (1.45 MPa); however, considerable variability among the three specimens was observed, with

interface shear stresses as low as 0.17 ksi (1.17 MPa) recorded for specimen L-C8-3, which exhibited localized failure near the notch (CSP 8 specimen L-C8-3 at failure) (**Fig. 19**). The CSP 9 specimens (**Fig. 19**) demonstrated more consistent failure loads, with an average interface shear stress comparable to that of the CSP 8 specimens. Significant coarse aggregate dislodgement and aggregate fracture were observed in both the CSP 8 and CSP 9 specimens. Posttest inspection showed rougher failure surfaces with numerous protrusions and exposed aggregate; however, the measured cohesion factors remained well below the 0.28 ksi (1.93 MPa) value recommended in the AASHTO LRFD specifications.

Interestingly, the cohesion strength of 0.20 ksi (1.38 MPa) originally suggested by Mattock and Hawkins,³ based on initially cracked push-off specimens with concrete compressive strengths ranging from 2.5 to 5.8 ksi (17.2 to 40 MPa), is similar to the cohesion strengths measured in the present study.

Test specimens with a compressive strength of 11 ksi (76 MPa) The specimens fabricated at the precast concrete plant achieved a 28-day compressive strength of 11 ksi (76 MPa), which was more than twice that of the specimens constructed in the laboratory. The strength differences were due to differences in mixture proportions and curing methods. As shown in **Table 4**, the average cohesion factor for the wood-float-finished specimens cast at the precast concrete plant was 0.39 ksi (2.7 MPa); however, some variability was observed, consistent with the variability previously reported for wood-float-finished specimens cast in the laboratory. The same wood-float finishing technique was used for specimens produced in the laboratory and those produced in the precast concrete plant; therefore, the substantially higher cohesion factors measured for the precast concrete specimens cannot be attributed to surface preparation. Instead, the results indicate that the cohesion factor is significantly influenced by the concrete compressive strength and is approximately proportional to it. This observation is consistent with the design recommendations proposed by Mattock¹¹ and Harries et al.¹⁰ and suggests a direct relationship between cohesion factor and concrete compressive



Figure 18. Wood-float-finished specimen L-WF-2 at failure.

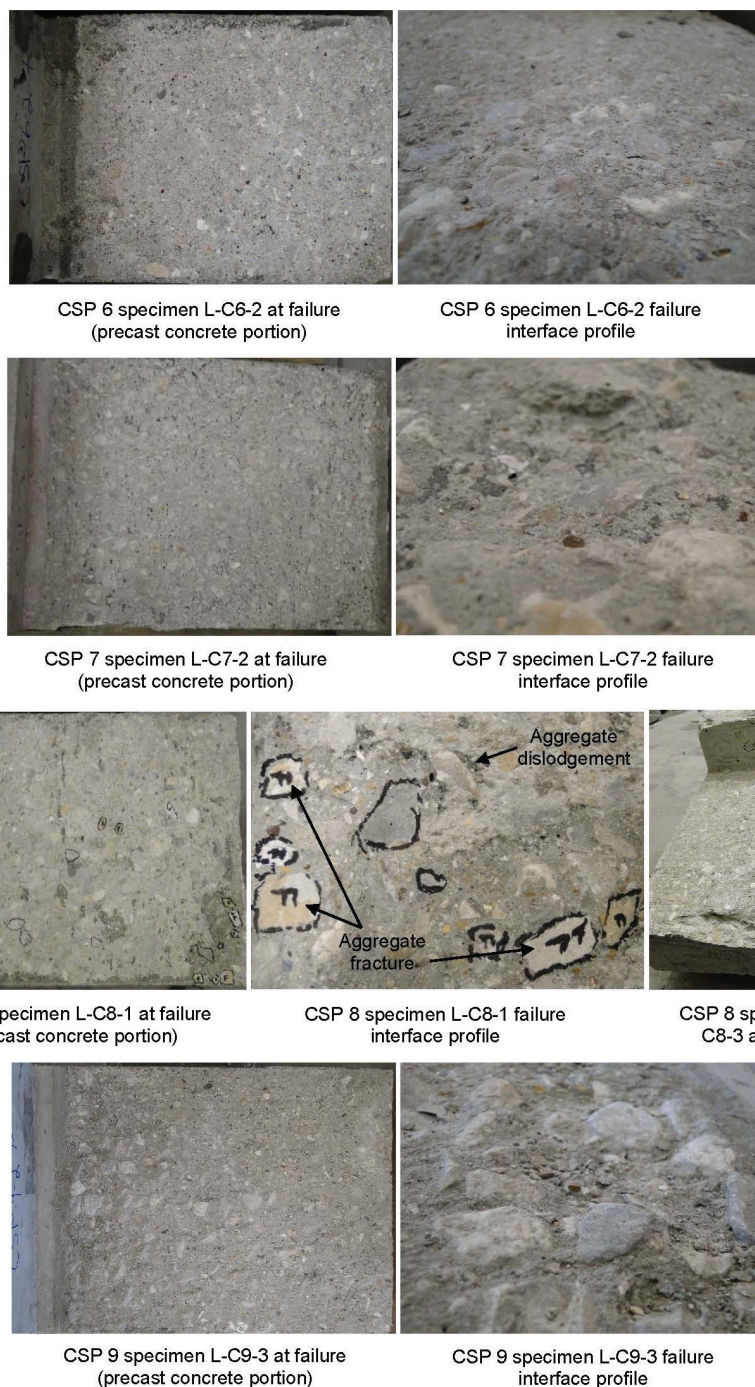


Figure 19. Failure conditions and interface profiles of test specimens with various concrete surface profiles (CSPs).

strength that is not explicitly accounted for in the equation from the AASHTO LRFD specifications.

The results for specimens prepared with a rake finish were more uniform than those with the wood-float finish. Compared with the wood-float specimens, the rake-finished specimens exhibited rougher surfaces, resulting in consistently higher cohesion factors. The average cohesion factor

for the rake-finished specimens was 0.46 ksi (3.17 MPa), approximately 17% higher than that of the wood-float-finished specimens.

Summary and discussion of cohesion factor push-off test results According to article 5.7.4.4 of the AASHTO LRFD specifications, a cohesion factor c of 0.28 ksi (1.93 MPa) is recommended for a normalweight cast-in-

place concrete slab placed on a clean concrete girder surface without laitance and roughened to an amplitude of 0.25 in. (6.35 mm). In comparison, the TxDOT specifications limit surface distortions of finished, unformed concrete surfaces to a maximum of 0.25 in. TxDOT explains that excessively rough interface conditions may promote bridging of coarse aggregates in the cast-in-place concrete and the formation of interfacial voids, which can adversely affect interface shear transfer and lead to reduced shear strength. Results from the cohesion factor push-off specimens indicate that increasing the surface roughness from CSP 7 to CSP 9 provides limited improvement in interface shear strength, whereas wood-float finishes yield the lowest shear stresses. These findings suggest that surface roughness exceeding CSP 7 may be unnecessary and may even be detrimental, thereby supporting TxDOT's requirement that surface distortions not exceed 0.25 in.

In addition, in this study, failure occurred either through fracture of the coarse aggregates or within the cement paste surrounding the aggregates, indicating that both coarse aggregate strength and cement paste strength may be dominant factors governing interface strength. The test results suggest that the cohesion factor depends not only on the level of surface roughness but also on the fracture strength of the coarse aggregates and the strength of the cement paste, including the ability of the paste to bond to exposed aggregates before aggregates are dislodged at the surface of the precast concrete segment. In other words, beyond a certain surface roughness threshold, interface shear resistance does not continue to increase and is governed primarily by the strength of the cement paste and the coarse aggregates.

Friction factor μ test

Table 5 summarizes the results of the friction factor push-off tests, including the friction factor μ calculated from the previously obtained cohesion factors c (Table 4). All specimens had a 28-day f'_c of 4.5 ksi (31 MPa) in the cast-

in-place portion. The friction factor μ was determined using AASHTO LRFD Eq. (5.7.4.3-3), combining the average cohesion factors c from the cohesion factor push-off tests, the failure load from the friction factor push-off tests, and the applied normal force P_c of 20 kip (89 kN). **Figure 20** shows typical failure cracking at the interface.

Figure 20 also shows typical failure interface profiles for various surface finishes. The wood-float-finished specimens failed at an average load of 53 kip (236 kN), corresponding to an average friction factor μ of 0.89, below the friction factor μ of 1.0 specified in the AASHTO LRFD specifications. A progressive increase in the friction factor was observed from CSP 6 to CSP 8, whereas CSP 8 and CSP 9 exhibited similar friction factors. The friction factor increased with surface roughness only up to a threshold level, indicating that additional increases in surface roughness beyond this threshold may provide limited enhancement of frictional resistance. CSP 8 and CSP 9 recorded the highest friction factor, approximately 1.8, which is 80% higher than the 1.0 value recommended in the AASHTO LRFD specifications for cast-in-place concrete slabs on clean concrete girder surfaces without laitance with the surface roughened to an amplitude of 0.25 in. (6.35 mm).

Table 5 also shows that if the cohesion factor is assumed to be 0.28 ksi (1.93 MPa), as specified by AASHTO, the resulting friction factor μ would range from approximately -0.88 for the wood-float finish to +0.87 for the CSP 9 finish. The negative value is not physically meaningful, and the other resulting values are also much smaller than those specified in the AASHTO LRFD specifications.

Shear force versus slip responses

Figure 21 shows the shear force compared with slip responses for specimens with surface roughness levels of CSP 6 and CSP 7. Not all CSP 6 and CSP 7 specimens are included because displacement measurements from some of the LVDTs

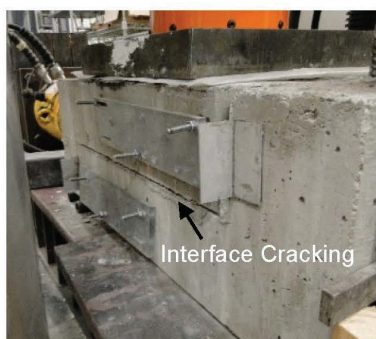
Table 5. Results of push-off tests for friction factor μ

Surface type	Average failure load V_{nf} , kip ^a	Average shear stress c , ksi	Average friction factor $\mu^{\dagger}$	Friction factor $\mu^{\dagger}$ if c is taken as 0.28 ksi per AASHTO
Wood float	53	0.14	0.89	-0.88
CSP 6	76	0.20	1.28	0.27
CSP 7	86	0.23	1.40	0.77
CSP 8	89	0.21	1.80	0.92
CSP 9	88	0.21	1.75	0.87
AASHTO (2024)	n/a	n/a	1.00	1.00

Note: AASHTO = American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*; CSP = concrete surface profile; n/a = not applicable. 1 kip = 4.448 kN; 1 ksi = 6.895 MPa.

^aThe measured 0.5 kip (2.22 kN) friction force between the loading plate and the specimen has been subtracted from the applied load.

[†]According to AASHTO LRFD Eq. (5.7.4.3-3), $\mu = (V_{nf} - cA_{cv}) / (A_{vf}f_v + P_c)$, where $A_{vf}f_v = 0$ and $P_c = 20$ kip (89 kN). $A_{cv} = 252$ in.² (162,600 mm²). The average value of c was obtained from Table 4.



Typical interface cracking at failure of push-off specimens tested in the horizontal position



Typical wood-float-finished specimen failure interface profile



Typical CSP 6 specimen failure interface profile



Typical CSP 7 specimen failure interface profile



Typical CSP 8 specimen failure interface profile



Typical CSP 9 specimen failure interface profile

Figure 20. Representative failure interface profiles of test specimens with various concrete surface profiles (CSPs).

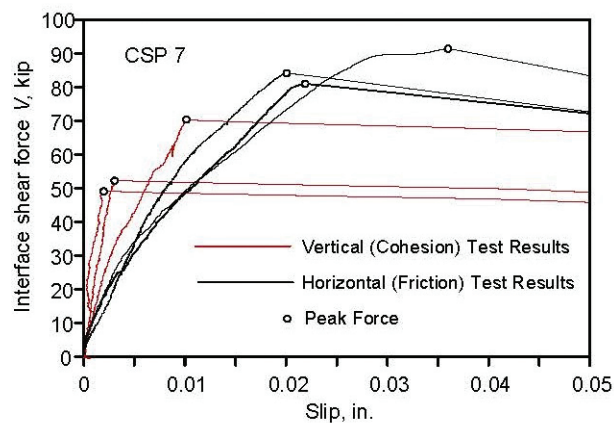
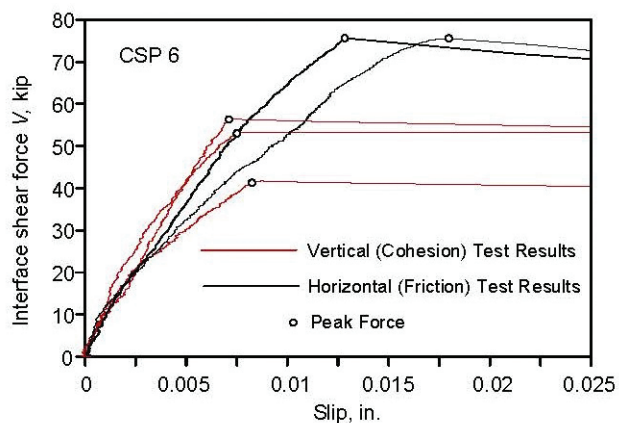


Figure 21. Interface shear force compared with slip responses for specimens under cohesion and friction tests. Note: CSP = concrete surface profile. 1 in. = 25.4 mm; 1 kip = 4.448 kN.

were erroneous. The nearly straight descending branches after the peak force should not be interpreted as indicating a gradual degradation in shear resistance. Rather, the specimens failed explosively upon reaching the peak load, and the force measurements recorded after peak load are not considered reliable. The peak force in the friction tests occurred at significantly larger slip values than those observed in the cohesion tests. The results indicate that the application of a normal compressive stress across the interface delays crack opening and dilatation, thereby allowing the interface to sustain higher shear resistance at larger slip values.

Evaluation of interface shear strength predictions using experimental results

Figure 22 presents the predicted interface shear strength per unit length of interface (AASHTO LRFD Eq. [5.7.4.3-3]) V_{ni} using the values specified in the AASHTO LRFD specifications together with the experimental results, considering two interface areas and reinforcement configurations. The permanent net compressive force normal to the shear plane is typically small and is assumed to be zero. The following observations can be made from Fig. 22:

- Surface roughness levels beyond CSP 7 provide limited additional increase in interface shear strength, regardless of spacing or the amount of interface shear reinforcement.
- Wood-float-finished surfaces with a concrete compressive strength of 4.5 ksi (31 MPa) exhibit the lowest interface

shear strength; however, increasing the compressive strength to 11 ksi (76 MPa) results in a significant increase in interface shear strength, particularly for larger interface areas.

- Increasing surface roughness is more effective in regions with closely spaced shear reinforcement, such as near the girder ends, whereas its benefit becomes less pronounced in regions with larger reinforcement spacing, such as near the midspan of a girder.

Conclusion

This study investigates the interface shear strength of cast-in-place concrete slabs on precast concrete girder surfaces with a particular focus on the effects of surface roughness on the cohesion and friction factors used in the shear-friction equation in the AASHTO LRFD specifications. Interface shear resistance is critical for achieving effective composite action because it prevents relative slip and ensures reliable stress transfer between the precast concrete and cast-in-place components. Observations from several precast concrete plants in Texas indicate significant variability in achieved surface roughness, emphasizing the need for AASHTO and state departments of transportation to develop more specific recommendations for surface preparation methods that ensure consistent interface roughness and reliable interface shear strength. To evaluate these effects, push-off tests were conducted on specimens with a cold-joint interface and no interface shear reinforcement crossing the interface. The

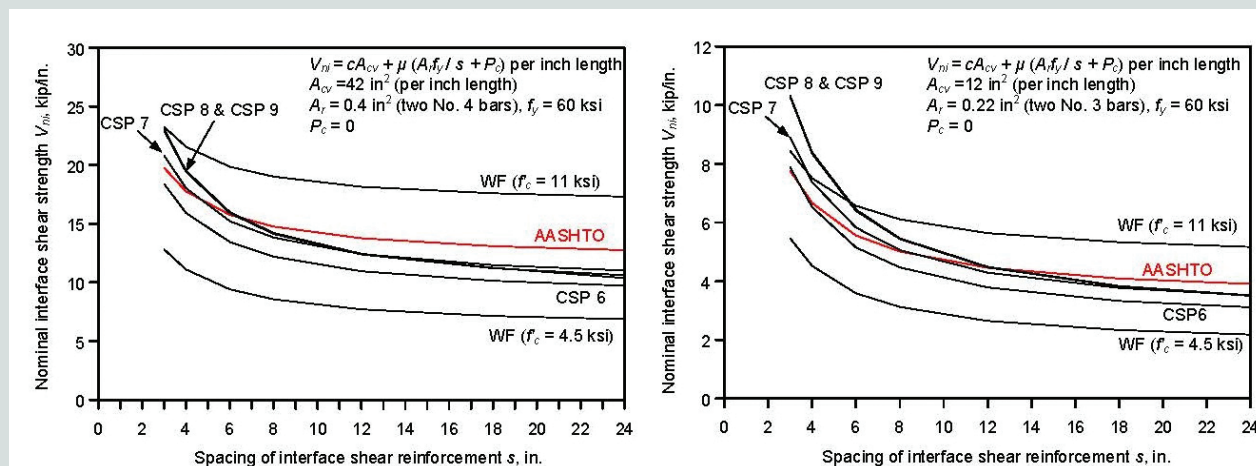


Figure 22. Comparison of the predicted interface shear strength per unit interface length for two cases with different interface areas and interface reinforcement areas using cohesion and friction factor values specified in the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications* ($c = 0.28$ ksi, $\mu = 1.0$) for composite members and the average cohesion c and friction μ factors obtained from the experimental tests. Note: The μ factor is assumed to be the same for wood-float finishes with different concrete compressive strengths of the cast-in-place portion. AASHTO = calculated values using the AASHTO LRFD specifications; CSP = concrete surface profile; A_{cv} = area of concrete considered to be engaged in interface shear transfer, per unit length of interface; $A_r = A_{cv} \times s$ = total area of interface shear reinforcement crossing the shear plane within reinforcement spacing s ; f'_c = measured 28-day compressive strength of the weaker concrete on either side of the interface (cast-in-place or precast concrete); f_y = yield strength of the interface shear reinforcement, not to exceed 60 ksi; P_c = permanent net compressive force per unit length normal to the shear plane; WF = wood float. no. 3 = 10M; no. 4 = 13M; 1 in. = 25.4 mm; 1 in.² = 645.2 mm²; 1 kip = 4.448 kN; 1 ksi = 6.895 MPa.

specimens incorporated a range of surface finishes, including wood-float, rake-finished, and ICRI concrete surface profiles (CSP 6 through CSP 9). The tests were designed to isolate and quantify the individual contributions of cohesion and friction at the interface.

The following conclusions and recommendations are drawn from this study:

- Cohesion is influenced not only by surface roughness but also by the fracture strength of coarse aggregates and the strength of the cement paste, including its ability to bond to exposed aggregates before aggregates are dislodged at the surface of the precast concrete segment. Observations of failure occurring either through fracture of the coarse aggregates or within the surrounding cement paste highlight that beyond a certain surface roughness threshold, interface shear resistance is governed primarily by the properties of the concrete matrix rather than further increases in roughness.
- Among common surface finishes, the wood-float finish is one of the most widely used. However, the test results indicate that typical wood-float finishing may not produce a surface roughness that meets the AASHTO LRFD specifications for a cohesion factor c of 0.28 ksi (1.93 MPa) and a friction factor μ of 1.0, which are intended for intentionally roughened surfaces with an amplitude of 0.25 in. (6.35 mm).
- The friction factor generally increases with surface roughness, eventually reaching a plateau limited by the concrete strength. The results also indicate that excessively rough surfaces beyond CSP 7 provide limited additional benefit and may even be detrimental due to potential interfacial voids or aggregate bridging, thereby supporting TxDOT's requirement to limit surface distortions to a maximum amplitude of 0.25 in. (6.35 mm). Wood-float finishes consistently produced the lowest interface shear strengths. This behavior may be attributed to the tendency of the wood float to partially cover exposed aggregate with cement paste, thereby reducing aggregate interlock across the interface between the old and new concrete. Consequently, the wood-float-finished specimens exhibited lower cohesion factors than specimens with rougher surface finishes.
- Although the test results are compared with values of c and μ from the AASHTO LRFD specifications, the experimental conditions may not fully represent the actual interface behavior in a composite bridge girder. Accordingly, the results are not intended to justify revising the AASHTO-specified values of c and μ . Rather, they are used to evaluate the influence of surface roughness on interface shear strength under controlled experimental conditions.
- If the experimental results from this study are used to derive interface shear design parameters, a simplified set of

design-oriented values may be considered. Because surface roughness is rarely measured and is difficult to quantify during construction, a simplified classification of surface conditions is adopted. For smoother surfaces, such as those produced by typical wood-float finishes, a cohesion factor c of 0.14 ksi (0.97 MPa) and a friction coefficient μ of 1.0 are suggested. For rougher surfaces, defined as CSP 6 and above (as required by TxDOT for the top surface of precast concrete panels) or rake-finished surfaces, a cohesion factor c of 0.21 ksi (1.45 MPa) and a friction coefficient μ of 1.4 are recommended. Those cohesion values are based on measured 28-day compressive strength f'_c of 4.5 ksi (31 MPa). By combining the concrete strengths investigated in this study with recommendations by Mattock¹¹ and Harries et al.,¹⁰ we propose the revised cohesion factor c of $\alpha f'_c$, where α is 0.031 for smoother surfaces, such as those produced by typical wood-float finishes, and α is 0.047 for rougher surfaces, defined as CSP 6 and higher or rake-finished surfaces.

- Surface roughness levels beyond CSP 7 provide limited additional increase in interface shear strength V_{ni} , regardless of reinforcement spacing or the amount of interface shear reinforcement. Wood-float-finished surfaces with a concrete compressive strength of 4.5 ksi (31 MPa) exhibit the lowest interface shear strength; however, increasing the compressive strength to 11 ksi (76 MPa) results in a significant increase in shear capacity, particularly for sections with larger interface areas. Overall, increasing surface roughness is more effective in regions with closely spaced shear reinforcement, such as near girder ends, while its influence becomes less significant in regions with larger reinforcement spacing, such as near midspan.

Acknowledgments

This research was supported by TxDOT. The contents of this paper reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the TxDOT. The authors would like to thank the reviewers for their valuable and constructive comments and suggestions.

References

1. Naaman, A. E., and S. H. Chao. 2022. *Prestressed Concrete Analysis and Design—Fundamentals*. 4th ed. Ann Arbor, MI: Techno Press 3000.
2. AASHTO (American Association of State Highway and Transportation Officials). 2024. *AASHTO LRFD Bridge Design Specifications*. 10th ed. Washington, DC: AASHTO.
3. Mattock, A. H., and N. M. Hawkins. 1972. "Shear Transfer in Reinforced Concrete—Recent Research." *PCI Journal* 17 (2): 55–75. <https://doi.org/10.15554/pci.j.03011972.55.75>.

4. Wight, J. K. 2022. *Reinforced Concrete: Mechanics and Design*. 8th ed. New York, NY: Pearson.
5. Davaadorj, O., P. M. Calvi, and J. F. Stanton. 2020. "Shear Stress Transfer Across Concrete-to-Concrete Interfaces: Experimental Evidence and Available Strength Models." *PCI Journal* 65 (4): 87–111. <https://doi.org/10.15554/pcij65.4-04>.
6. Hanson, N. W. 1960. "Precast-Prestressed Concrete Bridges—2. Interface Shear Connections." *Journal of PCA Research and Development Laboratories* 2 (2): 38–58.
7. Seible, F., and C. T. Latham. 1990. "Horizontal Load Transfer in Structural Concrete Bridge Deck Overlays." *Journal of Structural Engineering* 116 (10): 2691–2710. [https://doi.org/10.1061/\(ASCE\)0733-9445\(1990\)116:10\(2691\)](https://doi.org/10.1061/(ASCE)0733-9445(1990)116:10(2691)).
8. Loov, R. E., and A. K. Patnaik. 1994. "Horizontal Shear Strength of Composite Concrete Beams with a Rough Interface." *PCI Journal* 39 (1): 48–69. <https://doi.org/10.15554/pcij.01011994.48.69>.
9. Kovach, J. D., and C. Naito. 2008. *Interface Shear Capacity of Composite Concrete Beams Without Interface Ties*. ATLSS rep. 08-05. Bethlehem, PA: ATLSS Center, Lehigh University. <https://www.lehigh.edu/~cjin3/ATLSS-08-05.pdf>.
10. Harries, K. A., G. Zeno, and B. Shahrooz. 2012. "Toward an Improved Understanding of Shear-Friction Behavior." *ACI Structural Journal* 109 (6): 835–843. <https://doi.org/10.14359/51684127>.
11. Mattock, A. H. 2001. "Shear-Friction and High-Strength Concrete." *ACI Structural Journal* 98 (1): 50–59. <https://doi.org/10.14359/10146>.
12. Waweru, R., G. Palacios, and S. H. Chao. 2018. "Strength of Interface Shear Reinforcement with Limited Development Length." *ACI Structural Journal* 115 (4): 983–996. <https://doi.org/10.14359/51702061>.
13. Waweru, R., G. Palacios, and S. H. Chao. 2016. "Toward a More Realistic Interface Shear Strength Equation for Composite Members." *Proceedings, PCI Convention and National Bridge Conference*, Chicago, IL. <https://doi.org/10.15554/pro.2016PCI/NCBC-Sinterface-hear-Strength-Equation-Composite-Members>.
14. Li, C., R. D. Lequesne, and A. Matamoros. 2019. "Girder–Deck Interface: Partial Debonding, Deck Replacement, and Composite Action." *Journal of Bridge Engineering* 24 (1): 04018099. [https://doi.org/10.1061/\(ASCE\)BE.1943-5592.0001311](https://doi.org/10.1061/(ASCE)BE.1943-5592.0001311).
15. Hofbeck, J. A., I. O. Ibrahim, and A. H. Mattock. 1969. "Shear Transfer in Reinforced Concrete." *ACI Journal*, *Proceedings* 66 (2): 119–128. <https://doi.org/10.14359/7349>.
16. Seible, F., and C. T. Latham. 1990. "Horizontal Load Transfer in Structural Concrete Bridge Deck Overlays." *Journal of Structural Engineering*, 116 (10): 2691–2710. [https://doi.org/10.1061/\(ASCE\)0733-9445\(1990\)116:10\(2691\)](https://doi.org/10.1061/(ASCE)0733-9445(1990)116:10(2691)).
17. Santos, P. M. D., and E. N. B. S. Júlio. 2012. "A State-of-the-Art Review on Shear-Friction." *Engineering Structures* 45: 435–448. <https://doi.org/10.1016/j.engstruct.2012.06.036>.
18. ICRI (International Concrete Repair Institute). 2013. *Selecting and Specifying Concrete Surface Preparation for Sealers, Coatings, Polymer Overlays, and Concrete Repair*. ICRI Guideline 310.2R-2013. Des Plaines, IL: International Concrete Repair Institute.
19. ASTM International. 2015, reapproved 2024. *Standard Test Method for Measuring Pavement Macrotexture Depth Using a Volumetric Technique*. ASTM E965-15(2024). West Conshohocken, PA: ASTM International.
20. Santos, P. M. D., and E. N. B. S. Júlio. 2014. "Interface Shear Transfer on Composite Concrete Members." *ACI Structural Journal* 111 (1): 113–122. <https://doi.org/10.14359/51686543>.
21. Deschenes, D., and C. Naito. 2006. "Horizontal Shear Capacity of Composite Concrete Beams without Ties." *Proceedings, PCI National Bridge Conference*, Chicago, IL. <https://doi.org/10.15554/pro.2006PCI/NCBC-Shear-Capacity-Composite-Beams-Without-Ties>.
22. TxDOT (Texas Department of Transportation). 2024. *Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges*. Austin: TxDOT. <https://www.txdot.gov/business/resources/txdot-specifications.html>.
23. TxDOT. 2024. *Prestressed Concrete Panel Fabrication Details*. MS-PCP-FAB-24. Austin: TxDOT. <https://ftp.dot.state.tx.us/pub/txdot-info/cmd/cserve/standard/bridge/MS-PCP-FAB-24.pdf>.
24. Kahn, L. F., and A. D. Mitchell. 2002. "Shear Friction Tests with High-Strength Concrete." *ACI Structural Journal* 99 (1): 98–103. <https://doi.org/10.14359/11040>.
25. Mast, R. F. 1968. "Auxiliary Reinforcement in Concrete Connections." *Journal of the Structural Division* 94 (6): 1485–1504. <https://doi.org/10.1061/JSDEAG.0001977>.
26. Reinhardt, H. W., and J. C. Walraven. 1982. "Cracks in Concrete Subject to Shear." *Journal of the Structural*

27. ASTM International. 2024. *Standard Practice for Making and Curing Concrete Test Specimens in the Field*. ASTM C31/C31M-24. West Conshohocken, PA: ASTM International.
28. ASTM International. 2024. *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens*. ASTM C39/C39M-24. West Conshohocken, PA: ASTM International.

Notation

A_{cv}	= area of concrete considered to be engaged in interface shear transfer, per unit length of interface
A_r	= total area of interface shear reinforcement crossing the shear plane within reinforcement spacing s
A_{vf}	= area of interface shear reinforcement crossing the shear plane within the area A_{cv}
c	= cohesion factor = $\alpha f'_c$
f'_c	= measured 28-day compressive strength of the weaker concrete on either side of the interface (cast-in-place or precast concrete)
f_y	= yield strength of the interface shear reinforcement
K_1	= fraction of concrete strength available to resist interface shear
K_2	= limiting interface shear resistance
P_c	= permanent net compressive force per unit length normal to the shear plane
s	= spacing of interface reinforcement
V	= shear force
V_{ni}	= nominal interface shear resistance
w	= dilatation or crack opening
α	= coefficient for the adjusted cohesion factor c , accounting for the effect of concrete compressive strength
μ	= friction factor
ρ	= reinforcement ratio

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Abstract

This study investigates the interface shear strength between precast concrete girders and cast-in-place concrete slabs, with particular emphasis on the influence of surface roughness on the cohesion factor c and friction factor μ used in the shear-friction provisions of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*. Push-off tests were conducted on specimens with cold joints and without interface shear reinforcement, using surface finishes including wood-float and rake finishes, as well as International Concrete Repair Institute concrete surface profiles (CSPs) ranging from CSP 6 to CSP 9. The results indicate that both cohesion and friction factors generally increase with increasing surface roughness, though the cohesion and friction values are ultimately governed by the strength of the cement paste and coarse aggregate. Wood-float finishes exhibited the lowest cohesion and friction values, whereas excessively rough surfaces beyond CSP 7 provided limited additional benefits. Increasing concrete compressive strength leads to an increase in interface shear capacity, particularly for sections with larger interface areas. Overall, increasing surface roughness is more effective in regions with closely spaced shear reinforcement, such as near girder

ends, while its influence becomes less significant in regions with larger reinforcement spacing, such as near midspan. This study provides values of c and μ for different levels of surface roughness. This information is important because there is significant variability in achieved surface roughness among precast concrete plants, which suggests that interface shear strength may vary considerably in composite members depending on fabrication practices. The results of this study help clarify the influence of surface preparation on these parameters and may support AASHTO and state departments of transportation in developing more specific recommendations to achieve consistent surface roughness and reliable interface shear strength.

Keywords

Cohesion, composite beam, concrete surface profile, friction, interface shear, push-off test, shear friction.

Review policy

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Publishing details

This paper appears in *PCI Journal* (ISSN 0887-9672) V. 71, No. 6, November–December 2026, and can be found at <https://doi.org/10.15554/pci.71.6-01>. *PCI Journal* is published bimonthly by the Precast/Prestressed Concrete Institute, 8770 W. Bryn Mawr Ave., Suite 1150, Chicago, IL 60631. Copyright © 2026, Precast/Prestressed Concrete Institute.

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