Shear stress transfer across concrete-to-concrete interfaces: Experimental evidence and available strength models

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- This paper presents a study of 509 interface shear friction test results collected from literature on previous interface shear experiments conducted between the 1960s and 2017.
- The data collected were analyzed to gauge the reliability of six international code provisions and 14 strength models.
- The analysis results identified the best-performing models for different scenarios, critical knowledge gaps and future research needs, and recommendations for ways current models could be further improved to achieve higher performance.

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In structural concrete, shear force must sometimes be transferred across an interface between two materials. The interface may be between two faces of a crack in monolithic concrete, two concretes cast at different times, or steel and concrete. Such shear transfer is usually modeled as a shear friction phenomenon. This approach, initially proposed in the 1960s by Birkeland and Birkeland,[1] states that the shear strength of a concrete-to-concrete interface comes from the contribution of several resisting mechanisms, namely the cohesion between particles, the friction between concrete parts, and the shear force resisted by the reinforcement crossing the interface. The empirical parameters involved have been calibrated against experimental evidence by numerous investigators.[1–29] Today, the shear friction theory is widely accepted and has been adopted by most design codes, including the *PCI Design Handbook: Precast and Prestressed Concrete*,[30] the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*,[31] and *Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14)*.[32]

However, the various models differ in the ways they account for the controlling parameters and, in some cases, they provide significantly different strength estimates for the same configuration. For instance, ACI 318 advocates the use of a pure friction approach, the AASHTO LRFD specifications recommend a cohesive component and a frictional component, and the *PCI Design Handbook* uses an effective friction model in which the shear strength is nonlinearly related to the amount of reinforcement crossing the interface.

For low steel ratios, the *PCI Design Handbook* and AASHTO LRFD specifications methods tend to give shear strengths significantly higher than the ACI 318 pure friction approach. ACI 318 separately addresses the issues of interface shear transfer and shear friction: shear is transferred across an interface for both, but the latter contains reinforcement across the interface while the former does not. Some of these inconsistencies have been discussed by others (such as Tanner[33]), but at present, there is no general agreement as to which design or analysis approach consistently provides the most accurate predictions and should thus be used.

This paper presents and critically analyzes the results of 509 shear friction push-off and pull-off tests collected from the literature, many generated before recent code updates.

The database analysis used the data to gauge the importance of various structural parameters and to evaluate the performance of six international code provisions and 14 of the main strength models proposed by different authors. The analysis results identified the best-performing models for different scenarios, critical knowledge gaps and future research needs, and recommendations for ways current models could be further improved to achieve higher performance.

Database definition

A survey of the literature showed that experimental data pertaining to concrete-to-concrete interface tests have been conducted using the six test configurations outlined in **Fig. 1**. The database presented in this section addresses only direct

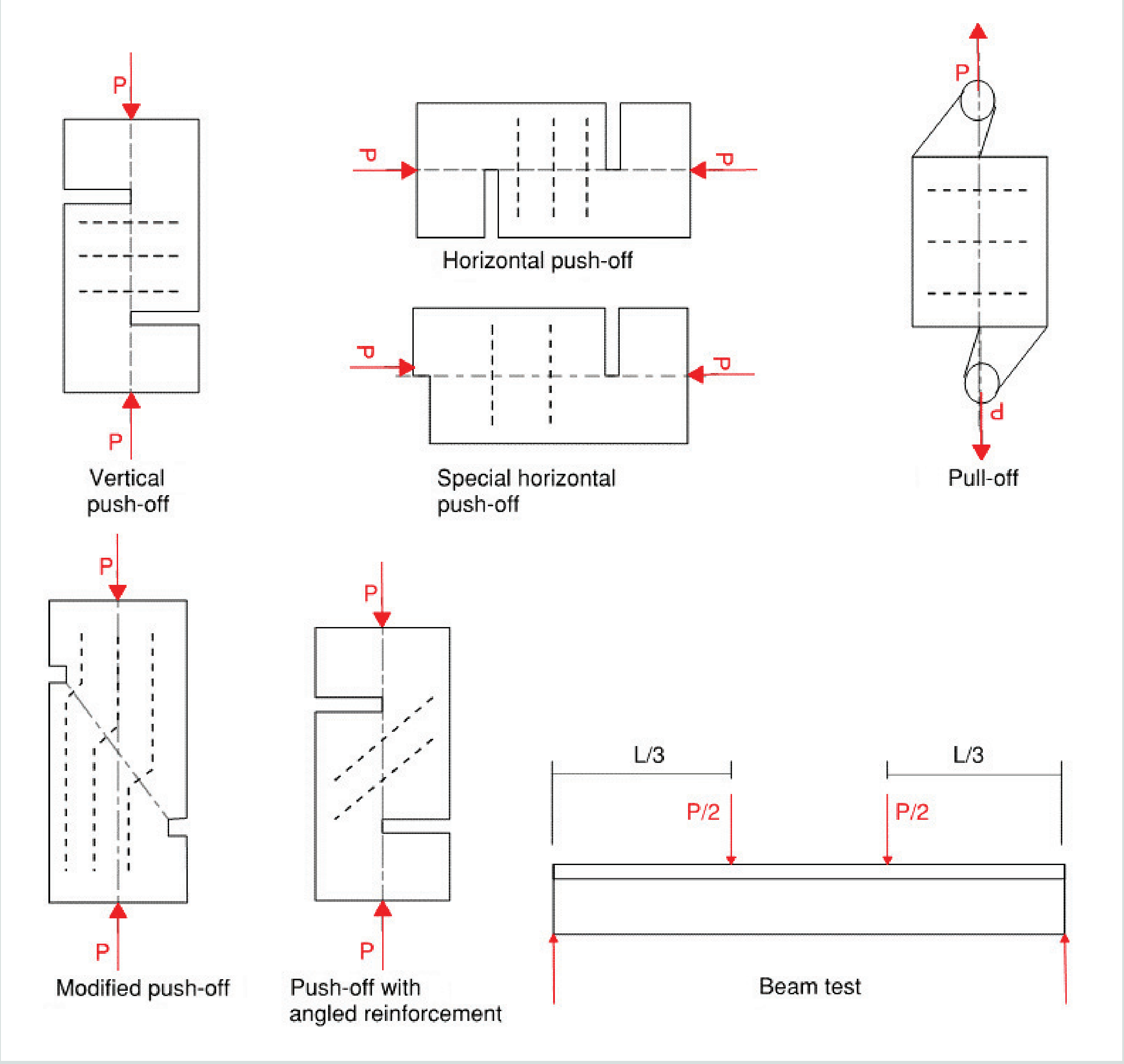

Figure 1. Common test configurations. Note: L = beam length; P = applied load.

push-off and pull-off tests (cases in the top row of Fig. 1), interfaces subject to monotonic pure shear loads, and steel reinforcement normal to the interface. These choices were made in order to focus on the basic shear transfer phenomenon. The main variables considered were the following:

- interface type
- concrete strength and weight
- shear interface area
- aggregate size
- reinforcement ratio, reinforcement strength, and average clamping stress

Many studies have focused on the effects of other more specialized variables on interface transfer, but they were deliberately excluded from this database because of being out of the scope previously mentioned. Examples include the following:

- presence of axial loads (tensile or compressive) acting normal to the shear interface (Mattock and Hawkins,[9] Mattock et al.,[11] Walraven and Reinhardt,[14] Papanicolau and Triantafillou,[21] and Echegaray et al.[27])
- orientation of reinforcement crossing the interface (Vangsirirungrang,[6] Dulacska,[8] Mattock and Hawkins,[9] Hawkins and Kuchma,[23] Nagle and Kuchma,[34] and Mattock[35])
- simultaneous presence of shear forces and bending moments (Mattock et al.[11])
- use of different materials, such as ultra-high-performance concrete, on the two sides of the interface (Crane[36])
- presence of sustained loading prior to loading to failure (Frenay[37])
- application of cyclic loading (Frenay,[37] Valluvan et al.,[18] and Calvi et al.[38,39])

The resulting database of shear tests involved a total of 509 monotonically loaded pull-off and push-off tests, summarized in **Table 1**. The information reported in Table 1 is limited to a chronological list of the studies considered with the variable ranges explored in each. More details on the individual tests are provided in the work of Davaadorj.[40] Table 1 gives details of a subset of the test programs that satisfy the data selection criteria of this research work. The total number of tests conducted in each of the programs listed may be higher than what is reported in the table (that is, all of the studies are included, but not necessarily the total number of tests from each study).

The experimental programs in the database involved specimens made of three types of concrete and four interface types. The concrete types were normalweight, sand lightweight, and all lightweight (referred to in Table 1 as NW, SLW, and ALW, respectively). The interfaces were monolithic uncracked (MO-U), monolithic precracked (MO-P), and cold joints that were intentionally roughened (CJ-R) and not roughened (that is, smooth) (CJ-S).

Additional notation used to identify all other variables in Table 1 included compressive strength of concrete f_c', maximum aggregate size a_g, reinforcement ratio ρ, reinforcement yield stress f_y, and average clamping stress ρf_y.

For cold-joint specimens with different concrete strengths on the two sides of the interface, only the lower compressive strength was reported. No additional restrictions were adopted for all other parameters, hence all values listed in Table 1 are those found in the original publications.

Of the 509 tests included in the database, 354 were normalweight concrete, 103 were sand lightweight concrete, and 52 were all lightweight concrete (70%, 20%, and 10% of the total, respectively).

Figure 2 summarizes the other key variables investigated experimentally over the course of past studies and provides the distributions and cumulative curves of the number of tests as a function of interface types, maximum aggregate size, concrete compressive strength, reinforcement yield stress, reinforcement ratio, and clamping stress.

Additional important parameters considered but not outlined in Fig. 2 were the area of the shear interface ranging from 50 to 160 in.^{2} (32,258 to 103,226 mm^2) and the size of the reinforcement, which did not vary over a wide range. In fact, the bar diameters presented in the database varied between 0.2 and 0.63 in. (5.08 and 16 mm), with the majority of the tests conducted on specimens with a diameter of steel reinforcement $d_s \leq 0.5$ in. (12.7 mm).

The multiparametric distribution depicted in Fig. 2 outlines that a substantial number of tests have been conducted in all of the four major surface types. The majority of tests available (approximately 60% of the total) comprised monolithic specimens, with the most conducted on precracked specimens. Cold-joint specimens represented approximately 40% of the total collected, and the majority were intentionally roughened cold joints.

The concrete strength varied over a wide range, with about 70% of the specimens having a strength between 3000 and 7500 psi (20.7 and 51.7 MPa). Roughly 20% of the specimens tested had a compressive strength between 10,000 and 18,000 psi (68.9 and 124.1 MPa), while relatively few specimens had strengths between 7500 and 10,000 psi—about 10% of the total. Of all these, approximately 80% of the specimens were made of aggregates with a maximum size between 0.375 and 0.75 in. (9.525 and 19.05 mm).

In approximately 90% of the tests, the reinforcement yield stress was lower than 80 ksi (551.6 MPa).

Table 1. Summary of interface shear experiments database

Authors	Year	Number of tests	Concrete type	Interface type	Bar size, in.	$\rho \times 10^{-3}$	f_y, ksi	ρf_y, ksi	f'_c, ksi	a_g, in.
Hanson	1960	38	NW	CJ-R, CJ-S	½	0 to 8.18	47.0 to 52.0	0 to 0.4	3.0 to 4.2	¾
Hofbeck et al.	1969	31	NW	MO-U, MO-P	⅜	4 to 26.0	42.4 to 66.1	0.2 to 1.4	2.4 to 4.5	⅞
Mattock and Hawkins	1972	8	NW	MO-U, MO-P	⅜	8 to 18.3	49.5 to 53.7	0.4 to 1.0	3.9 to 5.1	n/a
CTA	1974	2	NW	CJ-S	n.d.	0	n.d.	0	4.0 to 5.5	n/a
Mattock et al.	1975	4	NW	MO-U, MO-P	⅜	10.5 to 16	50.1 to 52.7	0.5 to 0.8	3.8 to 4.2	¾
Mattock et al.	1976	66	NW, ALW, SLW	MO-U, MO-P	⅜	0 to 26.4	47.7 to 53.6	0 to 1.4	2.0 to 6.0	⅜ to ¾
CTA	1976	11	NW	CJ-R, CJ-S	n.d.	0	n.d.	0	3.4 to 4.9	n/a
Walraven and Reinhardt	1981	23	NW	MO-P	⅜ to ⅝	1.4 to 33.5	62.8 to 110	0.15 to 2.2	3.6 to 6.9	⅝ to 10⁄8
Frenay	1985	20	NW	MO-P	⅜	11.2 to 22.4	66.7 to 79.8	0.75 to 1.8	6.7 to 9.9	⅝
Hoff	1993	18	SLW	MO-P	⅜ to ½	5.2 to 9.5	53.6 to 72.1	0.28 to 0.66	8.3 to 11	¾
Walraven and Stroband	1994	6	NW	MO-P	⅜ to ½	5.6 to 25.1	86.3	0.48 to 2.16	14.3	⅝
Pincheira et al.	1998	16	NW	MO-U, CJ-S, CJ-R	n.d.	0	n.d.	0	5.3 to 7.5	¾
Kahn and Mitchell	2002	45	NW	MO-U, MO-P, CJ-S, CJ-R	⅜	4 to 15.0	69.5 to 83	0.28 to 1.25	6.8 to 18.0	¾
Nagle and Kuchma	2007	16	NW	MO-P	⅜ to ⅝	1.4 to 17.8	64.5 to 79.2	0.1 to 1.4	5.8 to 17.5	8⁄10
Mansur et al.	2008	19	NW	MO-P	⅜	4.5 to 26.7	43.5 to 76.9	0.2 to 2.05	5.8 to 15.4	8⁄10
Crane	2010	20	NW	CJ-R, CJ-S	⅜	0 to 7.5	73.5	0 to 0.55	12.2	¾
Scott	2010	27	NW, SLW	CJ-R	½ to ⅝	0 to 4.8	60.0	0 to 0.29	5.7 to 6.2	½ to 1.0
Sagaseta and Vollum	2011	6	NW	MO-P	⅜	4.2 to 8.5	79.8	0.33 to 0.68	4.6 to 7.7	⅜
Harries et al.	2012	16	NW	CJ-R	⅜ to ½	4.0 to 8.0	61.5 to 140	0.27 to 1.05	5.8	¾
Shaw and Sneed	2014	36	NW, ALW, SLW	CJ-R, CJ-S	⅜	13.3	66.2	0.88	4.5 to 7.8	½
Rahal and Khaleefi	2015	15	NW	MO-U	⅕ to ⅓	3.6 to 19.3	37.4 to 59.2	0.13 to 1.15	6.3 to 11.5	¾
Sneed et al.	2016	46	NW, ALW, SLW	MO-U, MO-P, CJ-S, CJ-R	⅝	9.0 to 22.0	72.2	0.65 to 1.2	4.8 to 5.6	⅜
Barbosa et al.	2017	20	NW	CJ-R	½ to ⅝	4.2 to 6.5	67.6 to 93.0	0.3 to 0.56	4.2 to 4.6	⅝
Total	1960 to 2017	509	NW, ALW, SLW	MO-U, MO-P, CJ-R, CJ-S	⅕ to ⅝	0 to 33.5	37.4 to 140	0 to 2.2	2.0 to 18	⅜ to 1.0

Note: a_g = maximum aggregate size; ALW = all lightweight; CJ-R = cold joint intentionally roughened; CJ-S = cold joint not roughened, or smooth; CTA = Concrete Technology Associates; f'_c = concrete compressive strength; f_y = yield stress of reinforcement; LW = lightweight; MO-P = monolithic precracked; MO-U = monolithic uncracked; n/a = not applicable; n.d. = no data; NW = normalweight; SLW = sand lightweight; ρ = reinforcement ratio; ρf_y = clamping stress.

1 in. = 25.4 mm; 1 ksi = 6.895 MPa.

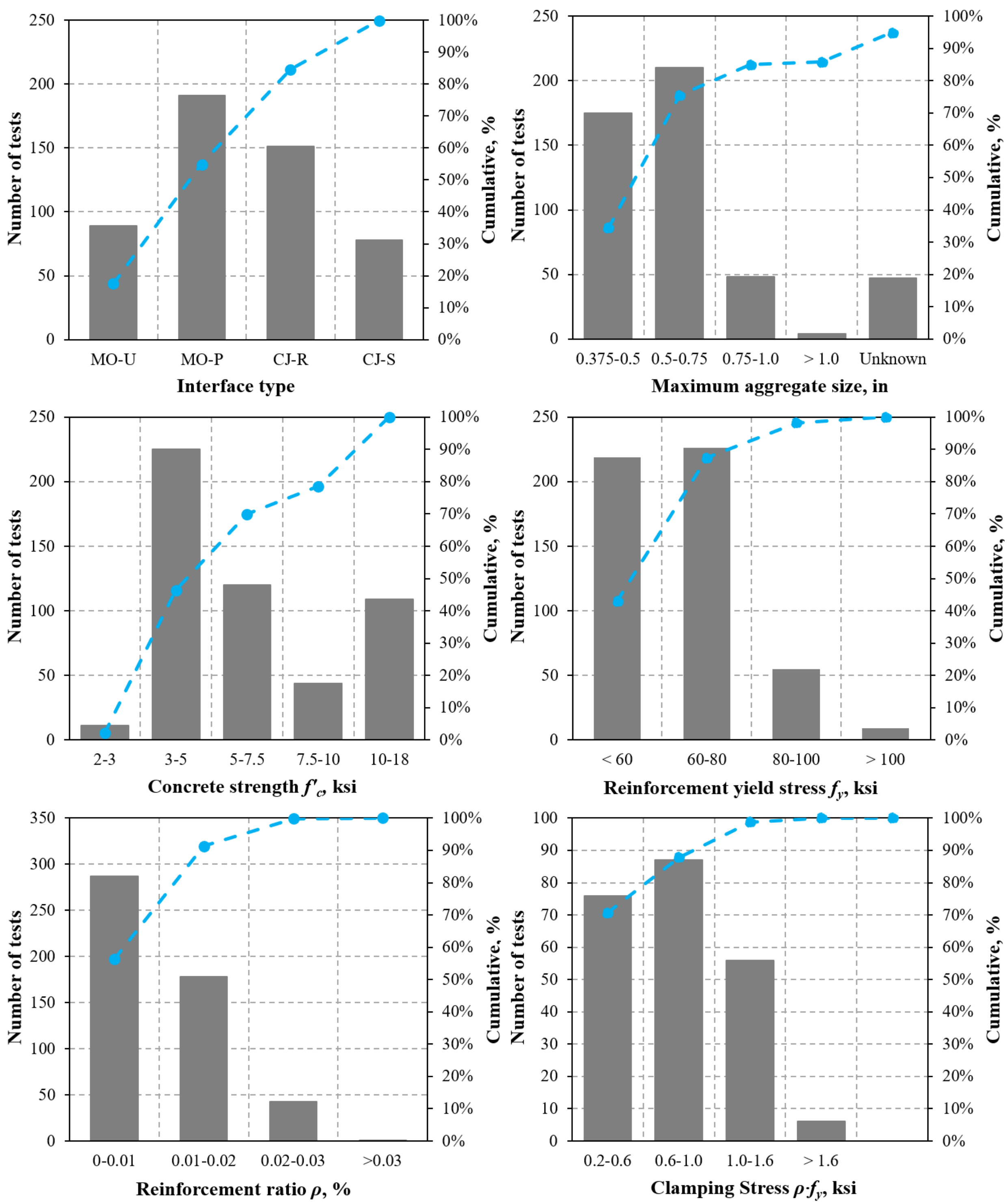

Figure 2. Frequency distribution of tests as a function of main parameters. Note: CJ-R = cold joint intentionally roughened; CJ-S = cold joint not roughened, or smooth; MO-P = monolithic precracked; MO-U = monolithic uncracked. 1 in. = 25.4 mm; 1 ksi = 6.895 MPa.

The tested reinforcement ratios ranged from 0% to 3%, with most of the tests falling in the 0% to 2% range. This, combined with the reinforcement strengths considered, resulted in clamping stresses rarely exceeding 1600 psi (11 MPa). The few tests that exceeded this threshold were for monolithic precracked interfaces.

As shown in Table 1, all experimental programs that constitute the database were carried out between the 1960s and 2017. **Figure 3** and **Table 2** provide a sense of how interface research has evolved over time and show that interface shear has consistently been a topic of interest over the past six decades, with substantial research carried out in the past 10 years for both normalweight and lightweight concrete (approximately 180 tests in total).

Although normalweight concrete has evidently received greater attention, notable studies on lightweight concrete have been conducted both in the 1970s and recently (for example, Sneed et al.[28]).

Alongside the distribution of shear friction tests over time, Fig. 3 provides a summary of the major milestones (listed in Table 2) pertaining to interface shear research, as well as to the applicable shear friction code provisions as they have changed over the years. Figure 3 and Table 2 present the amount of data available at times when models and code provisions were proposed or changes were implemented. For example, the very first ACI 318 shear friction chapter was introduced when only 69 documented direct-shear transfer tests (normalweight concrete only) were available. As more studies became available, codes were updated accordingly, but some design limitations introduced decades ago are still enforced today.

Database analysis

The results of the database experiments are analyzed in this section as a function of the key parameters most likely to affect the interface behavior, with particular attention to the peak shear strength.

Figure 4 shows the experimental peak shear stress recorded for each specimen in the database as a function of the clamping stress. Because all interface types were represented on the same graph, the results showed significant scatter.

In addition, the results were not separated as a function of any of the test variables. Trends observed were that concrete strength appeared to consistently increase with increasing

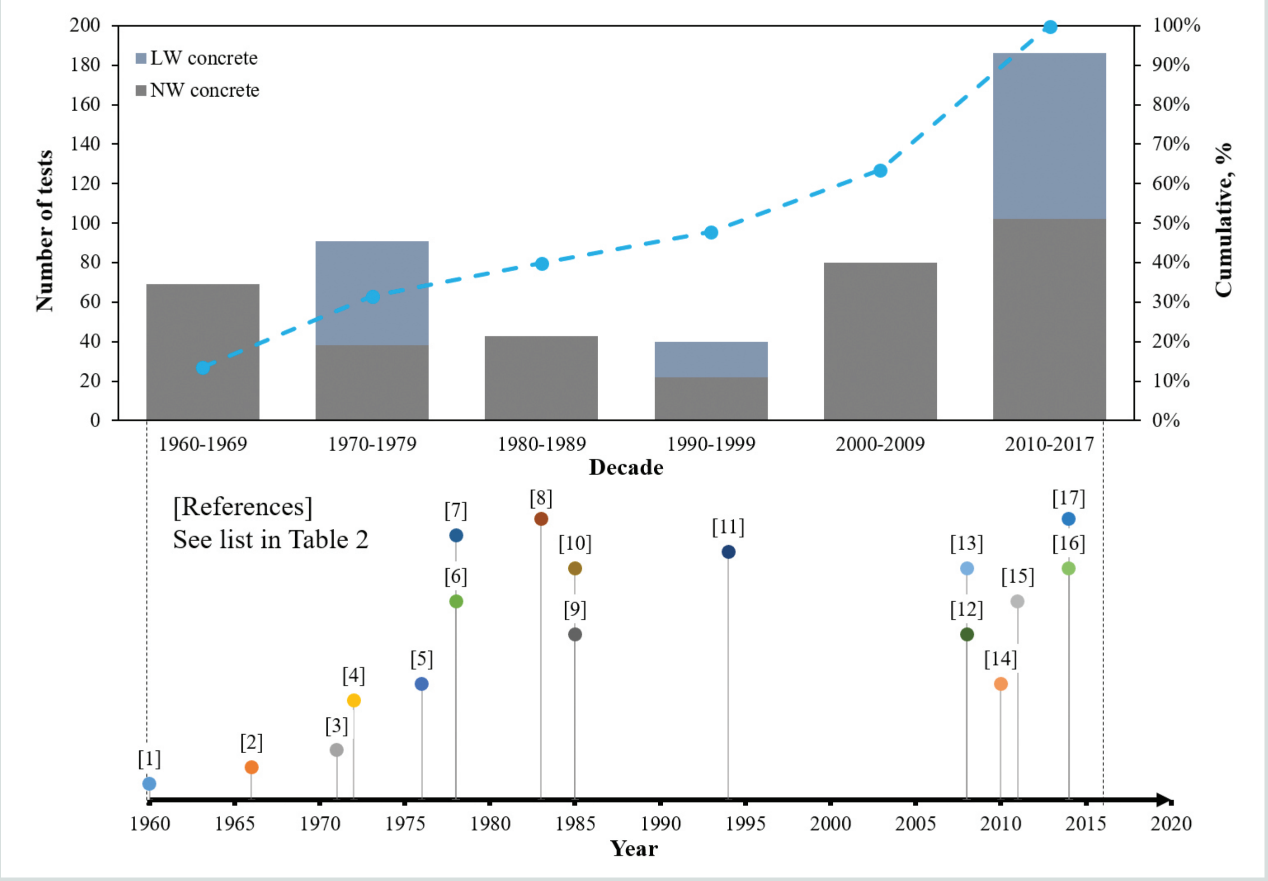

Figure 3. Distribution of shear friction tests in time. Note: LW = lightweight; NW = normalweight.

Table 2. Shear friction research major milestones

Reference number in Fig. 3	Year	Event description
1	1960	Hanson (1960) test program takes place.
2	1966	First introduction of shear friction theory: Birkeland and Birkeland (1966) followed by Mast (1968) and Hofbeck et al. (1969).
3	1971	ACI 318's first shear friction chapter (ACI 318-71).
4	1972	Mattock and Hawkins (1972), followed by Mattock et al. (1975), study effects of external normal force.
5	1976	Mattock et al. (1976) first study influence of lightweight aggregate.
6	1978	Shaikh's (1978) effective shear friction concept is introduced, relying on the works of Mattock (1974), Birkeland and Birkeland (1966), and Raths (1977).
7	1978	First appearance of effective shear friction in *PCI Design Handbook*, second edition. Shear friction coefficients of cold joints that aren't roughened and steel interfaces are 0.4 and 0.6, respectively. Corresponding upper limits are smaller than in current *PCI Design Handbook*.
8	1983	ACI 318 adopts constant cohesion terms (based on aggregate type) in addition to the friction component. Minimum clamping force is set to 200 psi.
9	1985	Studies on effects of high-compressive-strength concrete begin.
10	1985	*PCI Design Handbook* adopts higher upper limits. Shear friction coefficients for cold joints that aren't roughened and steel interfaces are raised. Upper limits on effective shear friction coefficients are introduced.
11	1994	AASHTO LRFD specifications, first edition: Shear friction method that utilizes cohesion and external normal force. To take concrete/aggregate type into account, λ factor is used.
12	2008	Tanner (2008) reviews *PCI Design Handbook* fourth, fifth, and sixth editions and addresses mathematical flaws.
13	2008	ACI 318 adopts various shear strength upper limits based on interface type.
14	2010	Significant lightweight concrete studies are conducted: Scott (2010), Sneed and Shaw (2014), Sneed et al. (2016), and others.
15	2011	PCI's effective shear method becomes applicable only to monolithic and intentionally roughened cold joint interfaces. λ^2 term in equation of μ_{eff} becomes λ.
16	2014	AASHTO LRFD specifications change to tabulated cohesion, μ, and upper limits for each interface and concrete type.
17	2014	ACI 318 equation form drops the cohesion term and becomes only dependent on friction term.

Note: λ = concrete weight reduction factor; μ = coefficient of friction; μ_e = effective coefficient of friction. 1 psi = 6.895 kPa.

clamping stress, and the raw data suggested that lightweight concrete specimens may be somewhat weaker than their normalweight concrete counterparts.

The same experimental results, separated as a function of the interface type, are shown in Fig. 4. Looking at the various interface types separately allowed a much more rational comparison of the results. The trends were more apparent, but the scatter was still significant in all cases.

The amount of test data pertaining to monolithic interfaces (280 tests in total, with 89 uncracked tests and 191 precracked tests) surpassed that of cold-joint interfaces (229 tests, with 151 tests of intentionally roughened interfaces and 78 tests of interfaces that were not). However, enough test results were available for all interface types to allow the observation of the main trends. In particular, many tests have been conducted with clamping stress ρf_y values ranging from 0 to 1500 psi (0 to 10.3 MPa), with only a few outside this range. Two trends

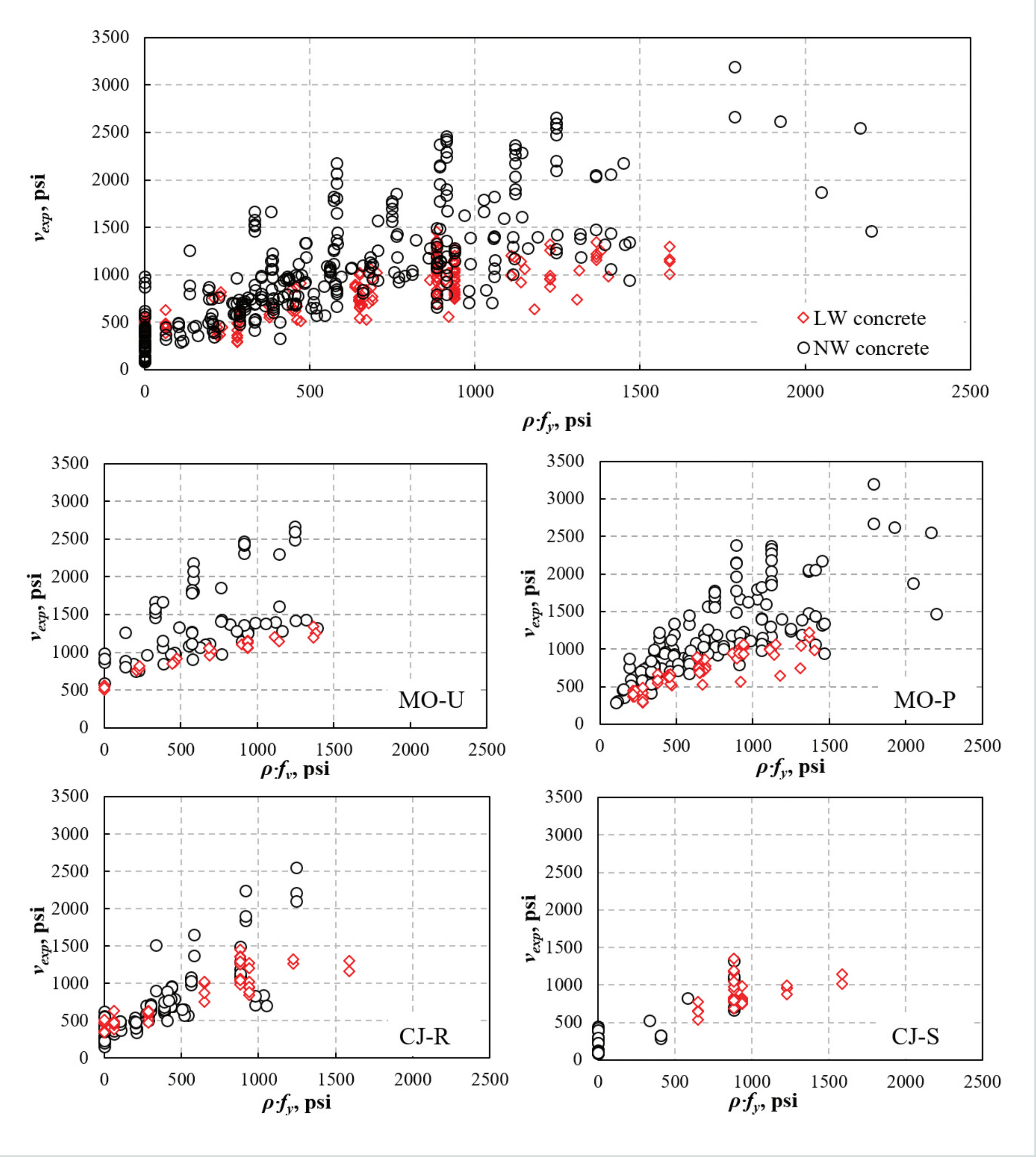

Figure 4. Interface shear strength as a function of clamping stress. Note: CJ-R = cold joint intentionally roughened; CJ-S = cold joint not roughened, or smooth; f_y = yield stress of reinforcement; LW = lightweight; MO-P = monolithic precracked; MO-U = monolithic uncracked; NW = normalweight; v_{exp} = shear strength measured experimentally (stress); ρ = reinforcement ratio; ρf_y = clamping stress. 1 psi = 6.895 kPa.

were detected from the whole data set: the interface strength appeared to increase with increasing clamping stress, and the lightweight concrete specimens appeared to fail at lower shear stresses than their normalweight counterparts. As discussed later, this may not have been due to the concrete type but rather other properties, such as the concrete strength. In addition, the specimens' strength appeared to be affected by the interface type, though at this stage it was difficult to quantify the extent to which that was true given that many other variables (such as concrete strength and steel strength) may have been the reason for a higher or lower observed response.

One important observation was that the experimental data for not roughened cold joint interfaces was somewhat sparse, with normalweight data only associated with clamping stresses less than 600 psi (4.1 MPa) and lightweight data mainly associated with clamping stresses greater than 600 psi. It was difficult to draw definitive conclusions regarding the influence of lightweight concrete on the shear strength of not roughened cold joint interfaces. The not roughened cold joint results were affected by significant scatter, with some of the normalweight tests failing at surprisingly low stress. The problem was further exacerbated by the poor definition of a not roughened cold joint interface, namely, any cold joint that did not satisfy the ¼ in. (6.35 mm) roughness requirement. This led to a variety of different surface conditions, all of which were treated as being in the same smooth joint category.

In contrast, the data pertaining to monolithic and intentionally roughened cold joint specimens were extensive for both lightweight and normalweight concrete at all clamping stress values lower than 1500 psi (10.3 MPa). This allowed for a more thorough and rational analysis of the results, which will be discussed later. At this stage it was evident that, when broken out by concrete and interface type, the results shown in Fig. 4 may be misleading, given that the strength of the various specimens was affected by a number of additional hidden variables, such as concrete and steel strength. These aspects are discussed in the next two sections of the paper.

Effects of concrete strength

The analysis of the available results, as well as the conclusions set forth by the authors of previous studies, suggested that although the clamping stress is the main parameter affecting the shear strength of a given interface, the concrete compressive strength played an important role as well. The influence of this parameter on the shear strength of different interface types is examined in this section.

To simultaneously illustrate the influence of clamping stress and concrete strength, the experimental data presented two-dimensionally in the previous subsection were examined in three dimensions. **Figures 5** and **6** show the experimental shear strength of all four interface types as a function of f'_c and ρf_y. Three-dimensional surfaces (left side of the figure) were also

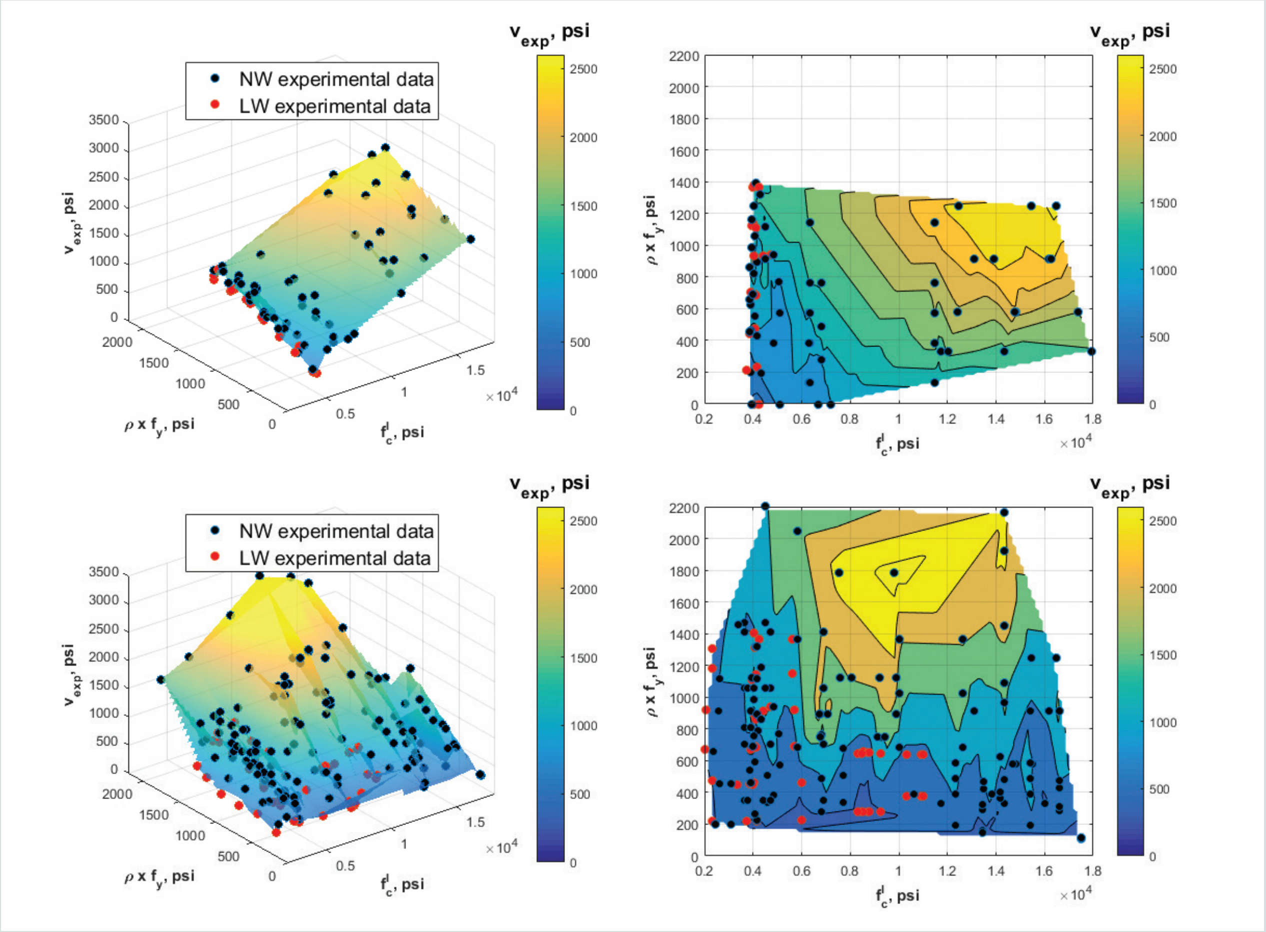

Figure 5. Monolithic specimens' shear strength versus clamping stress and concrete compressive strength. Note: f'_c = concrete compressive strength; f_y = yield stress of reinforcement; LW = lightweight; MO-P = monolithic precracked; MO-U = monolithic uncracked; NW = normalweight; v_{exp} = shear strength measured experimentally (stress); ρ = reinforcement ratio; ρf_y = clamping stress. 1 psi = 6.895 kPa.

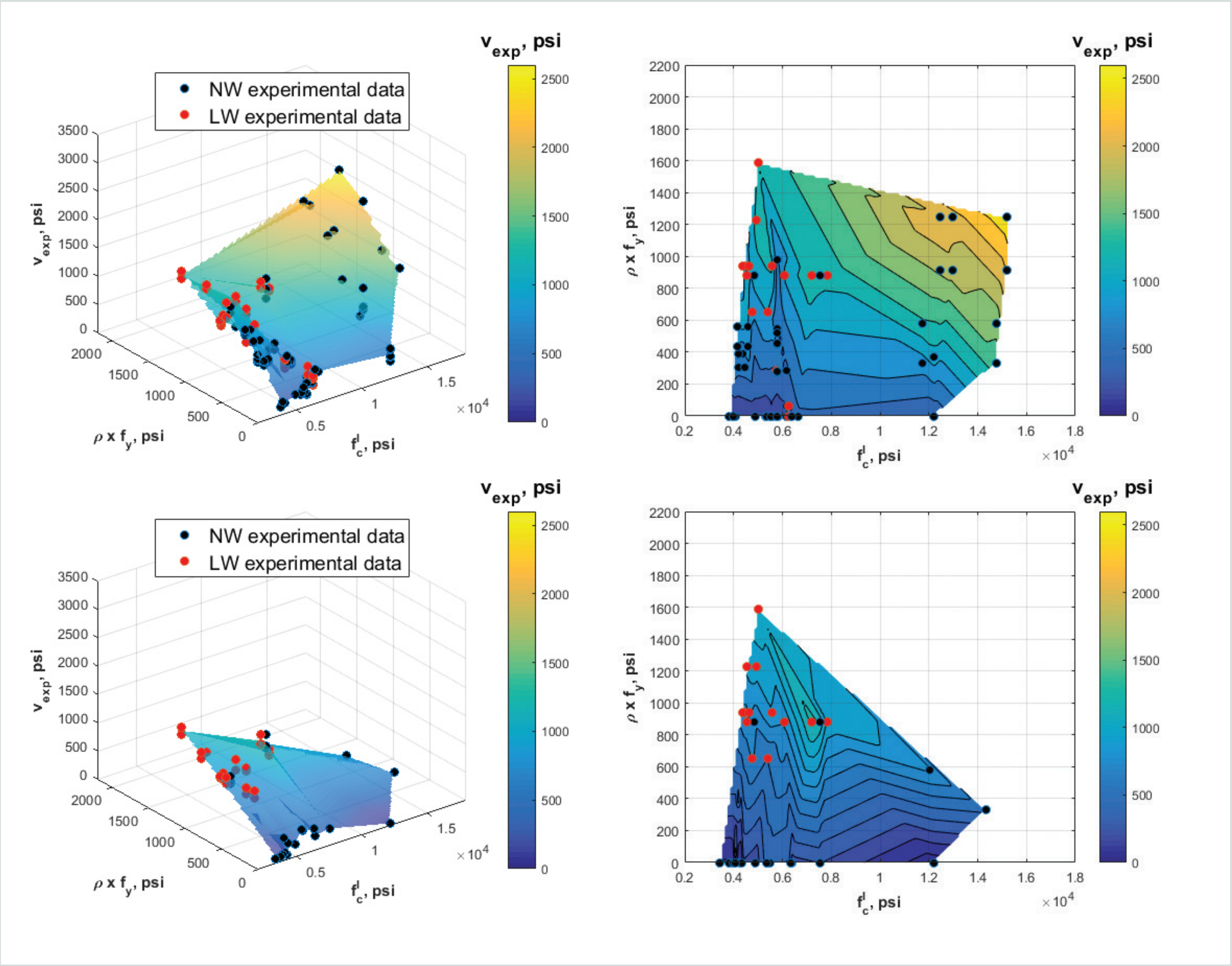

Figure 6. Cold-joint specimens' shear strength versus clamping stress and concrete compressive strength. Note: CJ-R = cold joint intentionally roughened; CJ-S = cold joint not roughened, or smooth; f'_c = concrete compressive strength; f_y = yield stress of reinforcement; LW = lightweight; NW = normalweight; v_{exp} = shear strength measured experimentally (stress); ρ = reinforcement ratio; ρf_y = clamping stress. 1 psi = 6.895 kPa.

presented to illustrate the main trends. These surfaces were based on extrapolation when no data points are available. They were intended to provide qualitative interpretations of the experimental results and may not necessarily provide accurate strength values for regions of the graphs with scarce experimental evidence. To provide a more complete overview of the results, the data were also shown in the form of contour plots (right side of the figure).

Figure 5 outlines the results pertaining to monolithic uncracked and monolithic precracked interfaces, and the following major trends were observed:

- The shear strength v_{exp} generally increased with increasing ρf_y for both monolithic uncracked and precracked specimens. In the context of shear friction, this trend indicated a positive friction coefficient.

- For monolithic uncracked specimens, the shear strength was not zero when $\rho f_y = 0$, suggesting that some cohesive component of shear strength existed. In contrast, while no monolithic precracked specimen tests with $\rho f_y = 0$ were available, the data suggested that very small to no cohesion could be expected for monolithic precracked specimens. This was consistent with the notion that cohesion will not exist across an open crack.

- For monolithic uncracked specimens, the shear strength generally increased with increasing f'_c. Interestingly, the rate at which the specimens gained strength appeared to decrease at high values of ρf_y and f'_c. This suggests that there may have been an absolute upper bound on shear strength, or an upper bound on the usable value of f'_c. However, the data were too sparse to be conclusive.

- The concrete strength f'_c did not appear to affect the shear strength of monolithic precracked specimens, particularly for clamping stress values lower than 1500 psi (10.3 MPa). For higher clamping stresses, increases in

f_c' appeared to provide some benefit, but the tests in that region were few, trends were not clear, and only a handful of tests were available in that region of the graph.

- For both interface types, lightweight and normalweight specimens with similar f_c' values had approximately the same peak strength.

Figure 6 outlines the results pertaining to intentionally roughened and not roughened cold joint interfaces. Overall, the cold joint data were less voluminous than the monolithic data. The surface conditions for cold-joint interfaces were generally not well defined in test programs beyond the distinction of being intentionally roughened or not. Both joint features implied greater scatter and less reliability. This was particularly true for the data on cold joints that were not roughened, which were overall deemed insufficient to draw any reliable conclusions.

For cold joint interfaces, the following major trends were observed:

- The shear strength v_{exp} generally increased with increasing ρf_y for both interface types.

- For intentionally roughened cold joint specimens, the shear strength was not zero when $\rho f_y = 0$. This was consistent with what was observed for monolithic uncracked interfaces and suggested that some cohesive component of shear strength existed. However, the cohesion appeared to be lower than that observed in the monolithic uncracked case and did not increase with increasing f_c'. In contrast, very low to no cohesion was observed for specimens that did not have roughened cold joints, with very scattered results, which was probably due to the inherent variability in the roughness conditions.

- Similar to what was observed for monolithic uncracked specimens, the shear strength tended to increase with increasing f_c'. The highest shear strength was recorded for specimens that simultaneously used high-strength concrete and high-strength steel reinforcement.

- No consistent trends were observed with respect to the influence of the concrete strength f_c' on the shear strength of cold joints that were not roughened. Overall, data for cold joints that were not roughened were too sparse and scattered to allow any definitive conclusions to be drawn. Normalweight data were available only for clamping stress values lower than 600 psi (4.1 MPa), whereas lightweight data were only available for higher clamping stresses. In addition, the results for the few high-strength normalweight concrete specimens failed at surprisingly low shear-stress values. The scatter and inconsistency of the results was attributed to the variability of the interface roughness (simply referred to as "not intentionally roughened"), which may have been drastically different in the different studies conducted.

- For the monolithic interface types, the use of lightweight concrete did not seem to make a difference with respect to the peak strength for intentionally roughened cold joint interfaces. Again, the shear strength of lightweight specimens was consistent with that of normalweight specimens with similar concrete strength. Trends were not observed for specimens with cold joints that had not been roughened.

Effects of reinforcement yield strength f_y

Reinforcement congestion is often a problem in reinforced concrete structures, particularly at joint locations, connections between slabs and shear walls, and the like. Thus, a reduction in the amount of reinforcement necessary to ensure an effective transfer of forces between elements is highly desirable.

One way of achieving this is to use high-strength steel shear reinforcement in place of regular-strength bars. The use of high-strength reinforcement in interface shear-transfer application has received increasing research attention in recent years (for example, Barbosa et al.[41]). However, an upper limit on the yield strength permitted for calculating shear strength is still enforced in the applicable code provisions, and reinforcement with f_y > 60 ksi (413.7 MPa) is still not permitted.

An analysis of the assembled database revealed that only a handful of tests that used reinforcement with f_y >100 ksi (689.5 MPa) were performed in past studies, hence nothing conclusive can be said on the implications of using them in practice. The great majority of the available test data was from specimens with f_y < 80 ksi (551.6 MPa), while about 10% of the specimens (roughly 50 in total) had steel reinforcement with yield strengths between 80 and 100 ksi.

To isolate the shear-strength contribution of the steel reinforcement strength from that of the concrete strength (which was shown to play an important role, particularly pertaining to certain interface types), the results summarized in **Fig. 7** are normalized with respect to the square root of the concrete compressive strength f_c', typically associated with the concrete tensile strength. This normalization process resulted in a much lower scatter for all interface types.

As shown in Fig. 7, for a given clamping stress, concrete strength, and interface condition, the reinforcement yield stress f_y did not seem to have any particular effect (beneficial or detrimental) with respect to the interface shear capacity. This suggests that using steel reinforcement with yield strength beyond 60 ksi (413.7 MPa) (and particularly Grade 80 steel [551.6 MPa]) should potentially be allowed. This observation was in agreement with recent findings of Barbosa et al.,[41] who observed that an increase in f_y (Grade 80 versus Grade 60) resulted in equal or slightly higher interface shear capacity. (The authors' conclusions were limited to no. 4 [13M] and no. 5 [16M] reinforcing bars.)

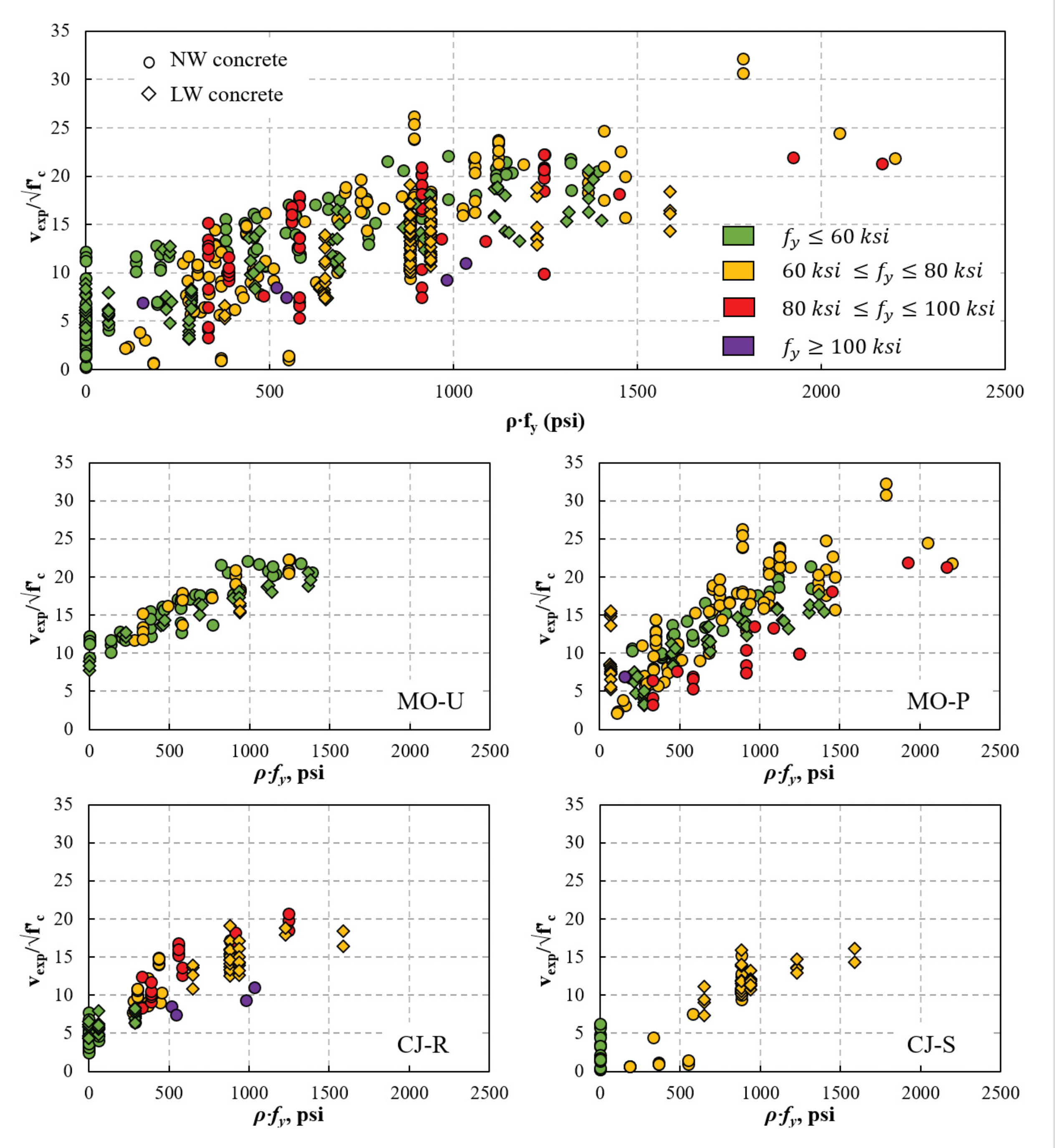

Figure 7. Normalized interface shear strength as a function of clamping stress. Note: CJ-R = cold joint intentionally roughened; CJ-S = cold joint not roughened, or smooth; f'_c = concrete compressive strength; f_y = yield stress of reinforcement; LW = lightweight; MO-P = monolithic precracked; MO-U = monolithic uncracked; NW = normalweight; v_{exp} = shear strength measured experimentally (stress); ρ = reinforcement ratio; ρf_y = clamping stress. 1 ksi = 6.895 MPa; 1 psi = 6.895 kPa.

Effects of other parameters

In line with the conclusions of previous research studies, the analysis of the database assembled in this research program revealed that the most critical parameters influencing the shear strength of an interface were the clamping stress, the concrete compressive strength, and the interface type. In contrast with current shear friction code provisions but consistent with recent research findings (such as Sneed et al.[28]), using lightweight concrete did not seem to have any effects on the interface strength.

A number of other parameters may influence the interface shear strength but were not considered in this study. Examples include the size of the reinforcement crossing the interface, orientation of the reinforcement, aggregate

Table 3. Performance of tested design and strength models

Interface	Monolithic uncracked		Monolithic precracked		Intentionally roughened cold joint		Not roughened (smooth) cold joints	
Model	Mean-V_{pred}/V_{exp}	COV,%	Mean V_{pred}/V_{exp}	COV, %	Mean V_{pred}/V_{exp}	COV, %	Mean V_{pred}/V_{exp}	COV, %
CSA (2014)	0.71	35	0.93	19	0.59	36	0.53	41
PCI (2004)	0.63	39	0.83	22	0.60	53	0.60	102
PCI (2010)	0.63	39	0.83	22	0.60	53	0.36	82
ACI 318 (2014)	0.53	44	0.71	21	0.47	55	0.41	80
AASHTO (2007)	0.85	18	0.98	18	0.84	20	0.74	35
CEB (1990)	0.92	40	1.36	42	1.06	53	1.27	93
CEN (2004)	n/a	n/a	n/a	n/a	0.73	31	0.74	44
Mast (1968)	0.79	62	1.05	37	0.54	62	0.46	77
Birkeland and Birkland (1966)	0.89	89	0.90	23	0.74	51	0.95	89
Mattock (1974, 1976)	0.73	23	0.94	21	1.00	33	1.82	66
Raths (1977)	0.76	41	1.02	23	0.83	51	1.08	89
Loov (1978)	0.70	38	n/a	n/a	n/a	n/a	n/a	n/a
Shaikh (1978)	0.75	22	0.95	19	0.86	32	1.23	64
Walraven et al. (1987)	0.86	37	1.25	22	0.96	51	1.25	81
Hsu, Mau and Chen (1987)	0.92	38	1.29	17	n/a	n/a	n/a	n/a
Tsoukantas and Tassios (1989)	n/a	n/a	n/a	n/a	1.10	50	0.31	77
Lin and Chen (1989)	0.85	18	1.03	17	1.03	17	1.63	39
Kono et al. (2003)	0.62	17	0.76	21	0.78	17	1.25	36
Mattock (2001)	0.78	19	1.10	19	1.01	32	0.41	80
Mansur et al. (2008)	n/a	n/a	0.99	14	n/a	n/a	n/a	n/a
Harries et al. (2012)	0.74	17	0.63	30	0.73	28	1.26	41

Note: COV = coefficient of variation; n/a = not applicable; PCI = PCI Industry Handbook Committee; V_{exp} = shear strength measured experimentally (force); V_{pred} = predicted shear strength (force).

size, loading protocol, and loading conditions.[1–33] In many instances, the evidence collected was not sufficient to draw definitive conclusions. For instance, bar sizes ranging only from no. 3 to 5 (10M to 16M) were used in previous studies, and this range was too small to identify clear trends. Analogously, while it is known that cyclic loads tend to lower the interface strength, the experimental programs that considered cyclic loading conditions as a variable were substantially fewer than the studies that looked at monotonic loads. As a result, there is still debate as to how the cyclic nature of the loads should be incorporated in rational interface behavior models.

Loads applied normal to the interface contribute to increase (if compressive) or decrease (if tensile) the interface strength, and inclined reinforcement is generally believed to strengthen the interface in shear, though there is debate as to whether this conclusion holds true in the presence of reversed cyclic loads. Both phenomena are included in the equations provided by ACI 318.[32]

Further research is necessary to evaluate or confirm the effects of these and other parameters and to establish how they should be incorporated into a rational interface behavior model and simplified code-oriented strength models.

Performance of available strength models

In this section, the capabilities of a number of interface strength models are evaluated against the experimental results collected in the database described in the previous sections.

The models considered include the provisions contained in six major international codes—namely, the *PCI Design Handbook* (sixth and seventh editions),[29,30] the AASHTO LRFD specifications,[31] ACI 318,[32] the Canadian Standards Association's *Design of Concrete Structures*,[42] *CEB-FIP Model Code 1990*,[43] and Eurocode.[44]

A total of 14 additional strength models proposed by various authors between the 1960s and 2018 were also considered. (See the list reported in **Table 3**.)

For the sake of brevity, only the details pertaining to the three U.S. codes and design manuals are reported in this section. For details pertaining to all of the other models, the reader is invited to refer to the original publications or to the work of Davaadorj,[40] which provides a comprehensive summary.

ACI 318-14

According to chapter 22.9 of ACI 318, the shear stress that can be transferred across an interface subjected to pure shear and crossed by perpendicular steel reinforcement should be calculated as

$$v_n = \rho f_y \mu$$

where

ρ = interface reinforcement ratio

f_y = reinforcement yield stress (limited in design to 60 ksi [413.7 MPa])

μ = interface friction coefficient given as a function of the interface type (as reported in **Table 4**) and the concrete weight reduction factor λ (equal to 1.0 for normalweight, 0.85 for sand lightweight, and 0.75 for lightweight concrete)

Table 4 also reports the various strength limits.

PCI Design Handbook, seventh edition

The seventh edition of the *PCI Design Handbook*[30] provides two equations to compute the shear stress that can be transferred across an interface subjected to pure shear and crossed by perpendicular steel reinforcement:

$$v_n = \rho f_y \mu_e \tag{1}$$

and

$$v_n = \rho f_y \mu \tag{2}$$

where μ_e in Eq. (1) is the effective coefficient of friction, and is given by

$$\mu_e = \frac{\phi 1000 \lambda \mu}{v_u}$$

where

ϕ = strength reduction factor

v_u = factored shear stress demand, psi

1000 = units of psi

Limiting values for μ_e are reported in **Table 5** alongside the μ values for the various interface conditions. Table 5 also reports the various strength limits.

The basic coefficient of friction μ already includes the lightweight concrete factor λ. This approach, in which the effective coefficient of friction (a component of the capacity) is a function of the demand V_u, can be rearranged to give

$$v_n = \sqrt{1000 \rho f_y \lambda \mu}$$

This makes the shear strength v_n proportional to the $\sqrt{\rho f_y}$ rather than ρf_y, which has the effect of increasing shear strengths at low clamping stress values and therefore has some resemblance to the addition of a cohesion term.

Table 4. ACI 318 friction coefficients and upper limits

Case	Interface type	μ	$v_{n,max}$
1	Concrete to concrete, cast monolithically	1.4λ	For normalweight concrete (monolithic or roughened): least of $\begin{Bmatrix} 0.2f'_c \\ (480+0.08f'_c) \\ 1600 \end{Bmatrix}$
2	Concrete to hardened concrete with roughened interface	1.0λ	
3	Concrete placed against hardened concrete that is not intentionally roughened	0.6λ	For all other cases: lesser of $\begin{Bmatrix} 0.2f'_c \\ 800 \end{Bmatrix}$
4	Concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars	0.7λ	

Note: f'_c = concrete compressive strength; $v_{n,max}$ = nominal shear stress; λ = concrete weight reduction factor; μ = coefficient of friction.

Equation (1) is recommended for use with monolithically cast interfaces and intentionally roughened cold joints, while Eq. (2) should be used for nonroughened cold joints and steel-to-concrete interfaces.

AASHTO LRFD Bridge Design Specifications, seventh edition

The AASHTO LRFD specifications recommend that the shear stress that can be transferred across an interface subjected to pure shear and crossed by perpendicular steel reinforcement should be computed as a function of a friction and cohesion term:

$$v_n = c + \mu\rho f_y \leq \min(K_1 f_c', K_2)$$

where

c = cohesion stress

K_1 = f_c' coefficient for shear stress upper limit

K_2 = shear stress upper limit

Table 5. *PCI Design Handbook* friction coefficients and upper limits

Interface	μ	Maximum μ_e	$v_{n,max}$, psi
Monolithic	1.4λ	3.4	$0.2\lambda f_c' < 1000\lambda$
Cold joint (roughened)	1.0λ	2.9	$0.2\lambda f_c' < 1000\lambda$
Cold joint (smooth)	0.6λ	n/a	$0.2\lambda f_c' < 800\lambda$
Concrete to steel	0.7λ	n/a	$0.2\lambda f_c' < 800\lambda$

Note: f_c' = concrete compressive strength; n/a = not applicable; $v_{n,max}$ = nominal shear strength (force);
λ = concrete weight reduction factor. μ = coefficient of friction; μ_e = effective coefficient of friction.

The coefficient c represents the cohesive strength and depends on the interface conditions. Cohesion values are reported in **Table 6** alongside the μ values for the various interface conditions. Table 6 also reports the various strength limits. In addition to limiting the yield stress of the reinforcement to 60 ksi (413.7 MPa), the clamping stress f_y should be greater than 0.05 ksi (0.34 MPa).

Discussion of the results

The performance of seven code-based models and 15 other models for interface strength were evaluated against the test data and the results obtained are summarized in Table 3. They were evaluated primarily by looking at the predicted-to-measured strength ratios. Not all models were applicable to all interface and concrete types. In addition, in some cases the input parameters (such as concrete type, minimum reinforcement ratio, and geometrical details) necessary to employ some of the models were missing. In these cases, not applicable (n/a) was reported when summarizing the results (Table 3).

Table 3 summarizes the results (separated as a function of the interface type) in terms of mean strength ratios and associated coefficients of variation (COVs). The results are not separated by concrete type, but appropriate equations are used for normalweight, sand lightweight, or all-lightweight concrete specimens.

A model that applied to a wide range of data with good accuracy could be considered superior to a model with better accuracy over a more limited range. Models with strict applicability restrictions could be checked against a smaller number of test results from the database. Furthermore, even in the case of poor mean predictions, models that result in a small COV had the potential to perform well after a minor modification of the parameters.

The results reported in Table 3 clearly indicate that no single design provision or strength model can be considered general in nature while consistently showing high performance for all interfaces. In order to provide a single number to char-

Table 6. AASHTO LRFD specifications friction coefficients, cohesion coefficients, and upper limits

Concrete type and interface type	c, ksi	μ	K_1	K_2, ksi
Cast-in-place slab on girder (normalweight concrete, roughened)	0.28	1.0	0.3	1.8
Cast-in-place slab on girder (lightweight concrete, roughened)	0.28	1.0	0.3	1.3
Monolithic (normalweight concrete)	0.40	1.4	0.25	1.5
Monolithic (lightweight concrete), cold joint (lightweight, roughened)	0.24	1.0	0.25	1.0
Cold joint (normalweight concrete, roughened)	0.24	1.0	0.25	1.5
Cold joint (smooth)	0.075	0.6	0.2	0.8
Concrete to steel	0.025	0.7	0.2	0.8

Source: Data from AASHTO (2007).
Note: c = cohesion stress; K_1 = f_c' coefficient for shear stress upper limit; K_2 = shear stress upper limit; μ = coefficient of friction. 1 ksi = 6.895 MPa.

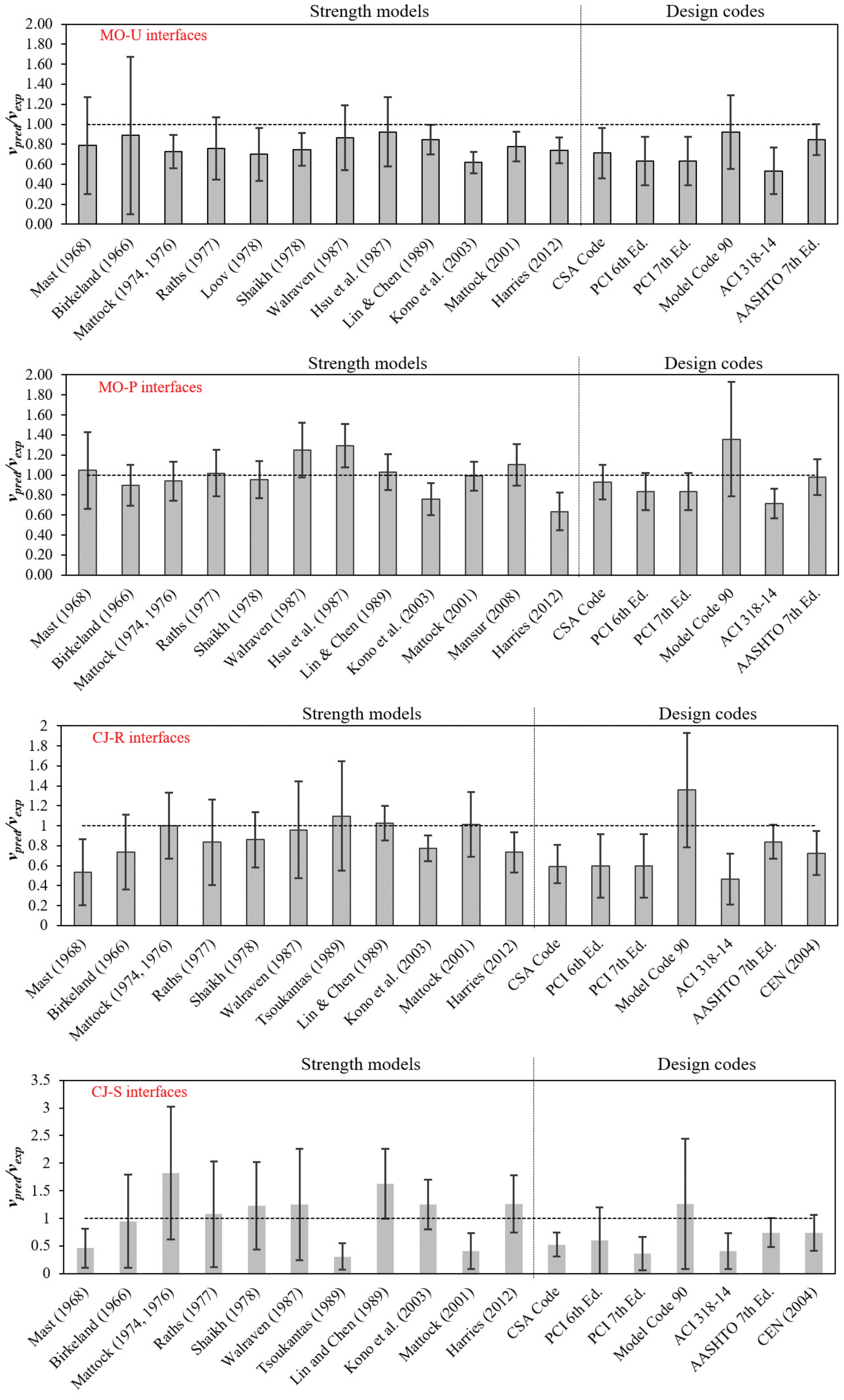

Figure 8. Accuracy of available models and design codes. Note: CJ-R = cold joint intentionally roughened; CJ-S = cold joint not roughened, or smooth; MO-P = monolithic precracked; MO-U = monolithic uncracked; v_{exp} = shear strength measured experimentally (stress); v_{pred} = predicted shear strength (stress).

acterize the model performance, a model error value e_m was defined as follows:

$$e_m = \sum_{i=1}^{4} (1-\mu_i) COV_i$$

The AASHTO LRFD specifications were found to give the lowest error of all models and the lowest of the code-based models. The lowest performance, with a mean v_{pred}/v_{exp} of 0.74 and a COV of 35%, was associated with cold joint interfaces that were not roughened. This outcome was common to all models and was to be expected given the sparsity of data pertaining to cold joint interfaces that were not roughened and the uncertainties associated with their surface roughness conditions.

Similarly, all strength models performed poorly overall for cold joint interfaces that were not roughened. Performances similar to those achieved using the AASHTO LRFD specifications for cold joint interfaces that were not roughened could be achieved using the models of Kono et al.[59] and Harries et al.[25]

Models that performed satisfactorily (to different extents) for interfaces other than cold joints that were not roughened were those of Lin and Chen,[58] Kono et al.,[59] Mattock,[60] and Harries et al.[25] In general, these models showed acceptable estimates of the mean strength ratios with relatively low COV values.

An alternative view of the results summarized in Table 3 is provided in **Fig. 8** for the four interface types present in the database.

Accuracy of U.S. code provisions

This section presents a more detailed analysis of the performance of the interface code provisions currently in effect in the United States, namely, ACI 318, the *PCI Design Handbook*, and the AASHTO LRFD specifications. In particular, a closer look was taken at how the models perform at different clamping stress ρf_y ranges for the four interface types. The ranges were 50 to 400 psi, 401 to 800 psi, 801 to 1200 psi, and 1201to 1600 psi.

Figure 9 summarizes the results of the U.S. code accuracy. A minimum clamping stress value greater than 50 psi (0.34 MPa) was used so that AASHTO LRFD specifications provisions would be applicable. This was done to avoid excessively misleading representations of the results given that both the *PCI Design Handbook* and the ACI 318 shear friction equations predict zero strength for unreinforced interfaces.

With reference to Fig. 9, the following trends were observed for the individual interfaces:

- Monolithic uncracked: All three codes provided conservative estimates of the interface strength. Consistent with the previous section, the AASHTO LRFD specifications equations tended to provide a better overall estimate of the measured strength. The *PCI Design Handbook* and ACI 318 strength estimates tended to be lower than the recorded experimental values, particularly at low clamping stresses. The *PCI Design Handbook* predictions tended to be more accurate than those of ACI 318.

- Monolithic precracked: Both the *PCI Design Handbook* and ACI 318 predictions were conservative at all clamping stress values, with the *PCI Design Handbook* strength estimates somewhat more accurate than the ACI 318 estimates. Although the AASHTO LRFD specifications are again on average more accurate than both the *PCI Design Handbook* and ACI 318, on occasion (for example, at low clamping stress values), the AASHTO LRFD specifications strength estimates were unconservative. This outcome had not emerged earlier when looking at the overall performance of the models. There were only four data points for clamping stress that were larger than 1600 psi (11 MPa), hence the predictions of all codes were not particularly meaningful in that range.

- Intentionally roughened cold joint: The AASHTO LRFD specifications provided the most accurate predictions for intentionally roughened cold joint interfaces. The predictions of the *PCI Design Handbook* and ACI 318 tended to be more conservative, with the *PCI Design Handbook* consistently providing more accurate strength estimates than ACI 318 for all clamping stress values. This suggested that both the cohesion plus friction approach of the AASHTO LRFD specifications and the effective friction approach of the *PCI Design Handbook* were more effective than the pure friction approach of ACI 318 at estimating the correct strength, particularly at small ρf_y values.

- Cold joint that is not roughened: ACI 318 and the *PCI Design Handbook* provided identical strength predictions for cold joint interfaces that were not roughened, given that the effective friction approach is not allowed for this interface type. The strength predictions were conservative at all clamping stress values, but the scatter was much larger than for the other interface types. The AASHTO LRFD specifications tended to be more accurate on average but provided unconservative strength estimates for low clamping stresses (with high scatter). Smooth interfaces were sensitive to the interface conditions, which could have varied greatly between different experimental programs, as well as within the same experimental program. This was particularly true at low clamping stresses, when cohesion presumably played a greater role, because cohesive failure was likely to be brittle and therefore sensitive to the fine details of the loading equipment and testing procedure. In addition, fewer data points were available (and only for certain clamping stress ranges) compared with all other interface types. For these reasons, poor performance and high scatter were almost inevitable for any strength model.

Figure 10 provides another overview of the performance of the U.S. codes. The graphs show the shear strength measured experimentally (stress) v_{exp} as a function of the

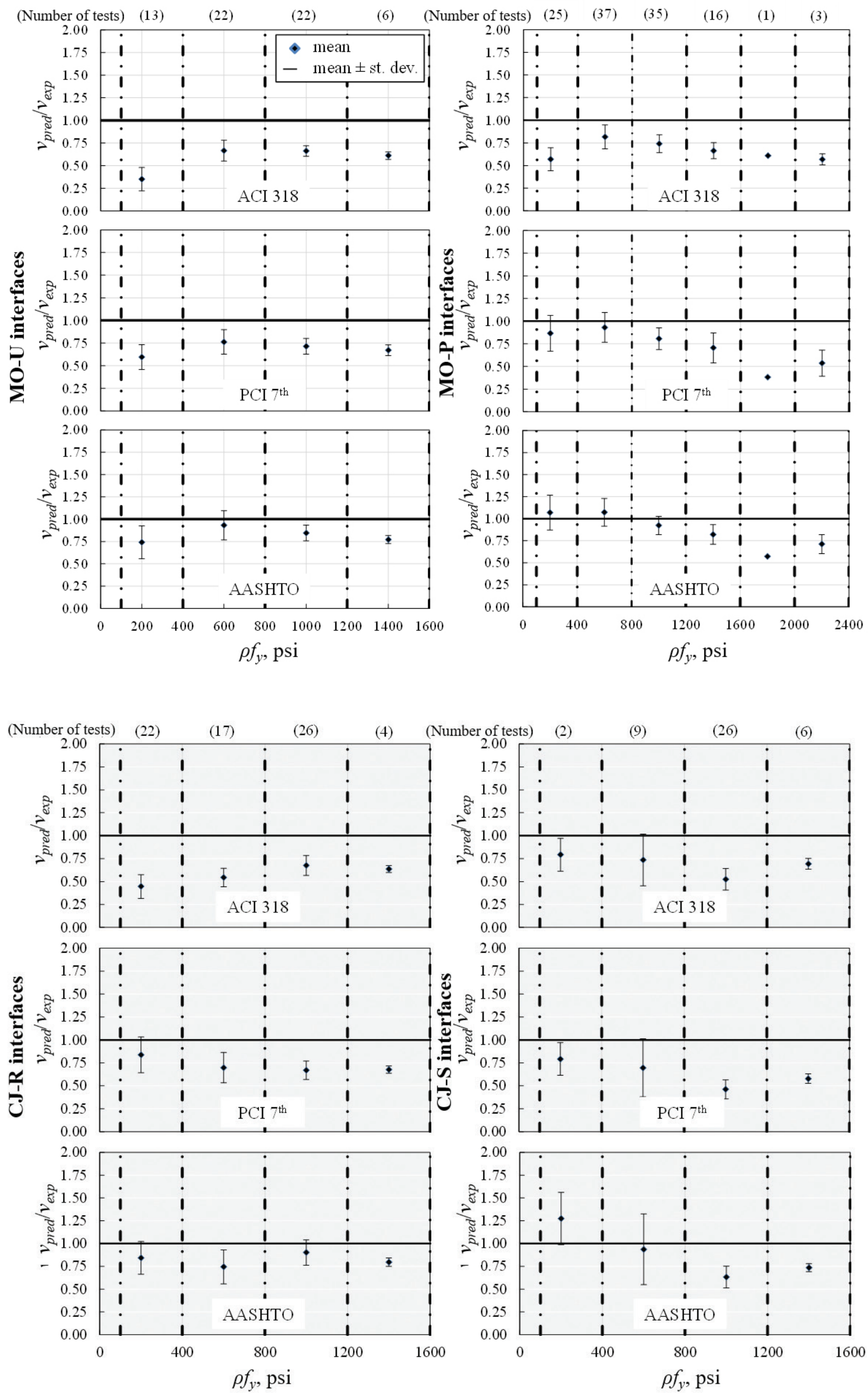

Figure 9. Accuracy of U.S. design codes as a function of clamping stress. Note: AASHTO = American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*, fourth edition; ACI 318 = American Concrete Institute's *Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318-14R)*; CJ-R = cold joint intentionally roughened; CJ-S = cold joint not roughened, or smooth; f_y = yield stress of reinforcement; MO-P = monolithic precracked; MO-U = monolithic uncracked; PCI 7th = *PCI Design Handbook: Precast and Prestressed Concrete*, seventh edition; v_{exp} = shear strength measured experimentally (stress); v_{pred} = predicted shear strength (stress); ρ = reinforcement ratio; ρf_y = clamping stress. 1 psi = 6.895 kPa.

clamping stress ρf_y. The results pertaining to normalweight concrete specimens are shown on the left, and the lightweight concrete specimens are grouped on the right. The results are separated by interface type. Because current code provisions do not differentiate between monolithic uncracked and precracked, the experimental results pertaining to these two cases are shown in the same chart and referred to simply as MO. The ACI 318, *PCI Design Handbook*, and AASHTO LRFD specifications strength predictions were shown alongside the experimental results. The predicted interface shear strengths were estimated without employing any safety factors and were shown using separate lines for the minimum and maximum concrete strength f_c' reported in the database for that particular category.

The three codes considered provided interface strength estimates that differed remarkably from one another pertaining to monolithic and intentionally roughened cold joint interfaces, whereas they predicted essentially identical strengths for cold joint specimens that were not roughened.

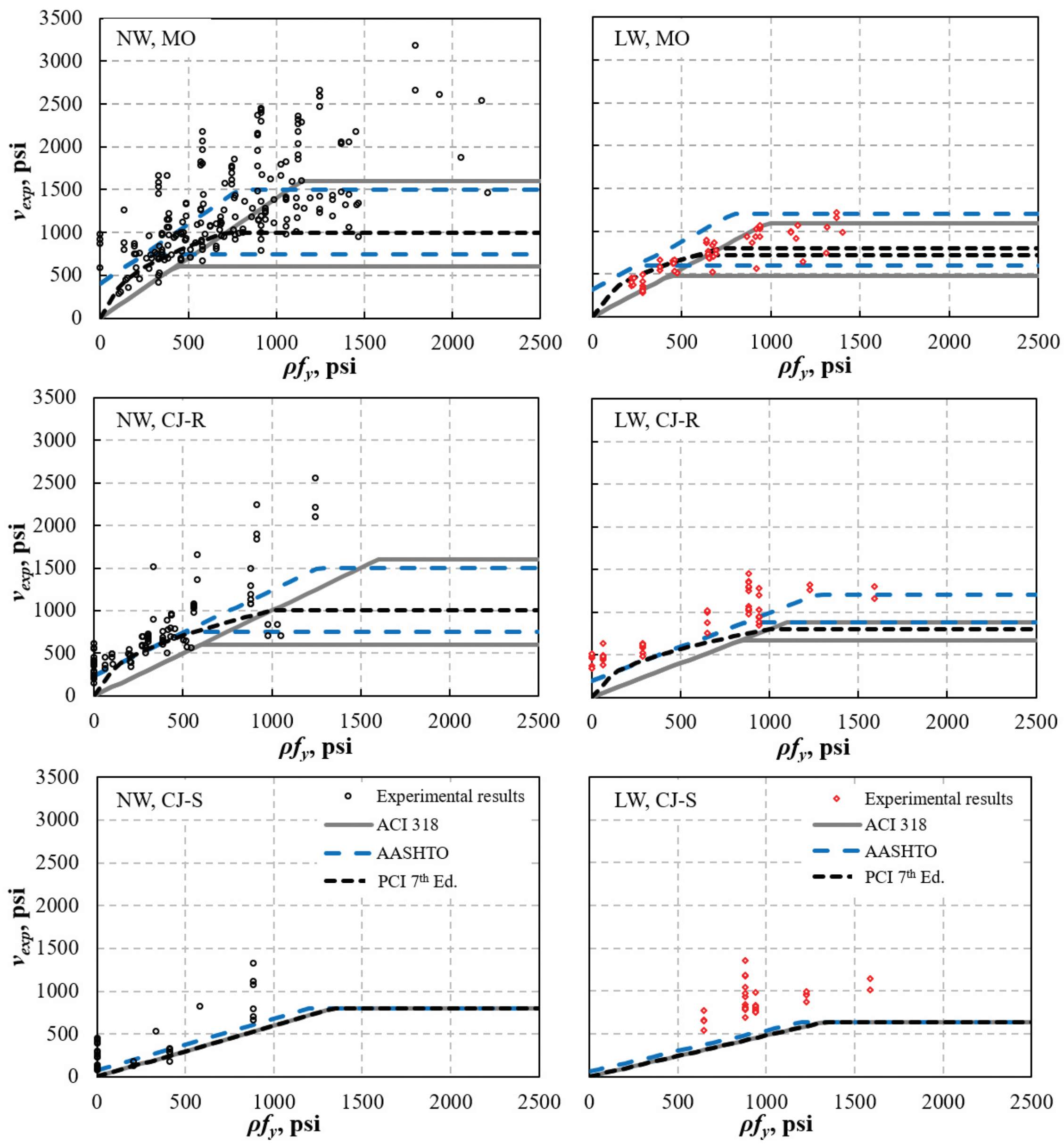

Figure 10. Interface strength predictions of U.S. design codes. Note: Upper- and lower-bound curves correspond to the extremes of concrete strength in the relevant part of the database. AASHTO = American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*, fourth edition; ACI 318 = American Concrete Institute *Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318-14R)*; f_y = yield stress of reinforcement; PCI 7th = *PCI Design Handbook: Precast and Prestressed Concrete*, seventh edition; v_{exp} = shear strength measured experimentally (stress); ρ = reinforcement ratio; ρf_y = clamping stress. 1 psi = 6.895 kPa.

The following observations were made for the normalweight specimens:

- The AASHTO LRFD specifications provided the most accurate strength predictions for monolithic and intentionally roughened cold joint interfaces at all clamping stress levels. The *PCI Design Handbook* appeared to be more effective than ACI 318 at predicting the strength at low clamping stress values, but the 1000 psi (6.9 MPa) upper limit that is currently enforced seemed to be excessively strict and the interface strength tended to be underpredicted at medium to high clamping stress values, particularly in the case of monolithic specimens. The AASHTO LRFD specifications and ACI 318 upper limits appeared to be more consistent with the experimental measurements.

- The strength predictions obtained for cold joint interfaces that were not roughened were reasonable overall, and all three codes tended to be conservative. However, there were a few specimens that failed at surprisingly low stresses (in the 250 to 500 psi [1.7 to 3.4 MPa] clamping stress range) and all provisions were unconservative. The scatter was very large at zero clamping stress, which made it difficult to estimate a reasonable cohesion value. In general, there were too few data points and the surface roughness conditions were too poorly defined to draw definitive conclusions about the reliability of current code provisions.

Similar observations were made for the lightweight specimens:

- The AASHTO LRFD specifications appeared to overestimate the strength of monolithic interfaces, possibly because of an excessively large cohesion term. In contrast, the *PCI Design Handbook* provided very accurate strength estimates and the ACI 318 predictions were reasonable overall, but more conservative.

- The AASHTO LRFD specifications provided the most accurate strength predictions for intentionally roughened cold joint interfaces. The *PCI Design Handbook* was somewhat more accurate than ACI 318, which tended to underpredict the interface strength at low clamping stress values. The AASHTO LRFD specifications upper limits also appeared to be more in line with those observed experimentally.

- All codes provided virtually identical strength predictions for cold joint interfaces that were not roughened. Although all estimates were conservative, there were too few experimental data points to identify clear trends and to gauge whether the codes captured those trends adequately.

Two additional observations were noted. The first, associated with practical implementation, concerned the monolithic uncracked and monolithic precracked interfaces. The experimental data clearly showed that monolithic uncracked interfaces were stronger than monolithic precracked interfaces. Researchers have also been careful to differentiate between the two. However, codes do not differentiate because it is almost impossible to guarantee that a particular shear plane will remain uncracked and therefore qualify for the higher μ value that such a surface would rightfully attract. Even in a prestressed member, the locations where shear friction is likely to be invoked tend to be disturbed regions, such as the dapped end of a double tee, and where cracking is likely to occur. A designer would face the same dilemma of needing certainty that the potential sliding plane would remain uncracked. It therefore seems appropriate that code requirements for monolithic concrete should be based exclusively on the monolithic precracked data.

The second observation concerned the concrete weight reduction factor λ and concrete strength. A cursory glance at the data, for example in Fig. 4, gives the impression that lightweight concrete had lower shear friction capacity because the lightweight data were low on the graph. However, three-dimensional figures such as Fig. 5 show that almost all of the lightweight specimens that have been tested have concrete strengths in the lower end of the range, but they have essentially the same shear friction strength as normalweight specimens of the same concrete strength.

Furthermore, there was a clear trend that higher concrete strength led to higher shear friction strength, particularly under high clamping stresses. However, almost all codes penalize concrete on the basis of unit weight (lightweight versus normalweight) when that distinction is not supported by the data and fail to account for the beneficial effects of concrete strength, which is supported. Sneed[28] has come to a similar conclusion about the lightweight factor. The same λ factor appears in other code provisions—for example, those pertaining to the development length of reinforcement. Although that behavior lies outside the scope of this study, it would be of interest to determine whether the λ factor is justified. It is worth noting that, in ACI 318, development lengths of bar reinforcement do depend on concrete strength, while those for strands do not.

Conclusion

This paper presents a database of 509 interface shear test results collected from the literature. The database was analyzed critically to identify major trends and current research gaps. The data collected were also used to gauge the reliability of six code provisions and 14 strength models. The findings from this research support the following main conclusions:

- The interface shear strength was mostly (but to different extents) affected by the interface conditions (that is, roughness), the clamping stress, and the concrete strength. The roughness level and the clamping stress played an important role for all interfaces, whereas the concrete strength mostly affected the interface response of monolithic uncracked and intentionally roughened cold joint interfaces.

- A cohesion component existed in all interfaces, but the experimental data analyzed suggested that it could be significant for intentionally roughened cold joint and monolithic uncracked interfaces, while it could be neglected (unless supported by new experimental evidence) for monolithic precracked and not roughened cold joint interfaces.

- The data did not support a dependence of shear strength on steel yield strength for a given clamping stress. However, the data were sparse for $f_y > 100$ ksi (689.5 MPa) and more research is needed in this area.

- The interface shear strength of lightweight concrete interfaces was, on average, no less than that of normalweight concrete interfaces for the same concrete strength. The widely used λ factor appeared not to be justified.

- The compressive strength of the concrete played a role, but primarily for high ρf_y values. In that region, the data were too sparse to permit development of a suitable relationship.

Of all the design codes considered, the AASHTO LRFD specifications provided the most accurate strength estimates across all interfaces. The *PCI Design Handbook* was generally more accurate than ACI 318, which tended to consistently underestimate the interface strength for all interface types and at all clamping stress levels. Among the researchers' strength models analyzed, those of Lin and Chen,[58] Kono et al.,[59] Mattock,[60] and Harries et al.[25] were the most accurate overall, but they were still less accurate overall than the AASHTO LRFD specifications. In general, all codes and models fell short when attempting to predict the strength of not roughened cold joint interfaces. This was due to the sparsity of data and to the poorly defined roughness conditions associated with this interface type, which therefore led to a wide variety of different surface conditions in tests.

Recommendations

- Based on the experimental observations, it is recommended that the current limit on the reinforcement yield stress be raised from 60 to 80 ksi (413.7 to 551.6 MPa). Other interface strength upper limits should also be revised. In particular, the *PCI Design Handbook* upper limits could be raised to be consistent with the AASHTO LRFD specifications and ACI 318 upper bounds.

- In the current shear friction code provisions, lightweight concrete usage is permitted with the use of concrete modification factor λ. However, the experimental results analyzed suggest that there is no inherent weakness associated with using lightweight concrete. It is recommended that the concrete modification factor be removed from the equations. This recommendation is consistent with that of Sneed et al.[28]

- If reliable models for cold-joint interfaces are desired, there is a need for more experiments that better define interface roughness or a roughening procedure. Such a definition should enter the design practice to guarantee that certain standards and consistency are achieved. More specifically, even if current ACI 318 standards are not met, a common definition of roughness should still be available to avoid the implementation of interfaces with drastically different roughness and, in turn, strength.

- Based on the analysis of the available experimental data and on the observation of the best strength predictions obtained using design codes (that is, the AASHTO LRFD specifications) and strength models (such as Mattock[60] and Harries et al.[25]) that contain both a cohesion and a friction term, it is recommended that this form be adopted universally in the future.

- It is further recommended that the cohesion term in the model be a function of both the interface conditions and the concrete strength (as suggested by both Mattock[60] and Harries et al.[25]) rather than considering only the interface conditions (as currently done in the AASHTO LRFD specifications).

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Notation

a_g = maximum aggregate size

A_c = area of shear interface

A_{cr} = shear interface area

c = cohesion stress

d_s = diameter of steel reinforcement

e_m = model error

f_c' = concrete compressive strength

f_y = yield stress of reinforcement

K_1 = f_c' coefficient for shear stress upper limit

K_2 = shear stress upper limit

L = beam length

P = applied load

v_{exp} = shear strength measured experimentally (stress)

v_n = nominal shear strength (stress)

v_{pred} = predicted shear strength (stress)

V_{exp} = shear strength measured experimentally (force)

$V_{n,max}$ = nominal shear strength (force)

V_{pred} = predicted shear strength (force)

V_u = shear demand

λ = concrete weight reduction factor

μ = interface friction coefficient

μ_e = effective coefficient of friction

ρ = reinforcement ratio

ρf_y = clamping stress

ϕ = strength reduction factor

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Abstract

This paper presents results of a database of interface shear experiments collected from the literature and analyzed to identify the main parameters affecting the strength of concrete-to-concrete interfaces subjected to shear loads and to gauge the reliability of major international codes and interface strength models. The database included 509 push-off and pull-off specimens, with steel reinforcement normal to the interface and subjected to monotonic pure shear loading. The experimental data were analyzed mainly in terms of interface type, clamping stress, concrete compressive strength, concrete unit weight, and steel strength.

The analysis of the database revealed that clamping stress and interface type were the main parameters influencing the interface strength but that the concrete strength could play an important role as well. In contrast, it was found that the concrete unit weight appeared to have no effect on the interface strength. Of the code provisions considered, it was found that the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications* shear friction equations were the most accurate at predicting the interface strength for all interface types and conditions, while the *PCI Design Handbook*: *Precast and Prestressed Concrete* and American Concrete Institute's *Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14)* equations tended to underestimate the strength, with the *PCI Design Handbook* being generally more accurate than ACI 318-14.

Keywords

Concrete joints, concrete-to-concrete interface, interface shear, shear friction, shear strength.

Review policy

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

Reader comments

Please address any reader comments to *PCI Journal* editor-in-chief Tom Klemens at tklemens@pci.org or Precast/Prestressed Concrete Institute, c/o *PCI Journal*, 8770 W. Bryn Mawr Ave., Suite 1150, Chicago, IL 60631.