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Shear Friction Tests with High-Strength Concrete

by Lawrence F. Kahn and Andrew D. Mitchell

This experimental study showed that the current ACI shear friction concept can be extended to high-strength concrete. Fifty pushoff specimens were tested with uncracked, precracked, and cold-joint interfaces. These specimens mimicked those used by previous researchers in the development of shear friction. Concrete strengths varied from 6800 to 17,900 psi (46.9 to 123.4 MPa), with transverse reinforcing ratios between 0.37 and 1.47%. An equation is proposed that more accurately predicts the shear friction strength of cold-joint and uncracked interfaces for high-strength concrete. It is recommended that the current upper shear stress limit of $0.2f_c'$ be retained but that the 800 psi (5.5 MPa) shear stress limit be eliminated.

Keywords: high-strength concrete; shear; test.

INTRODUCTION

The purpose of this experimental research was to extend the previous research on shear friction to concretes with strengths greater than 7000 psi (48.3 MPa) and to determine if the current ACI code[1] provisions were applicable for concrete strengths approaching 18,000 psi (124 MPa). The ACI[1] shear friction concept is that shear forces are transferred across a joint by friction between the surfaces. The frictional force is a function of the normal force applied and the coefficient of friction μ between the surfaces. The normal clamping force may result from the transverse shear reinforcement, and the shear provided is given by Eq. (1)

$$V_n = \mu A_v f_y \tag{1}$$

where

V_n = nominal shear strength;
μ = coefficient of friction: 1.4 for a monolithic concrete connection; 1.0 for a cold joint with 1/4 in. roughness amplitude; 0.7 for concrete-steel interface; and 0.6 for a cold joint at a smooth concrete interface;
A_v = area of shear reinforcement across shear plane; and
f_y = yield stress of reinforcement (≤ 60 ksi [413.7 MPa] per Reference 1).

Expressing shear friction in terms of shear stress provides flexibility in data interpretation. This expression is obtained by dividing V_n by the area of the concrete shear interface A_c

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

where

v_n = nominal shear stress V_n/A_c (≤ 800 psi [5.5 MPa]);
ρ_v = shear friction reinforcement ratio A_v/A_c; and
A_c = area of concrete interface.

This fundamental shear friction equation was proposed by Birkeland and Birkeland.[2] The development of shear friction has been documented by others.[3-5] Shear friction tests with cold joints were initially conducted by Anderson[6] and Hanson.[7] The shear friction concept was evaluated and verified extensively with pushoff tests by others[8-10] with respect to reinforcement ratio, surface condition, and concrete strength. Concrete strengths as high as 8000 psi (55.2 MPa) were used. Mattock and Hawkins[10] precracked pushoff specimens using a line load along the shear plane to produce a crack with few irregularities that would produce a worst-case scenario. Further tests by Walraven, Frenay, and Pruijssers[11] with a discussion by Mattock[12] provided an equation that related shear strength to concrete strength, transverse reinforcement ratio, and clamping force. Hoff[13] tested high-strength lightweight concrete pushoff specimens and concluded that the current ACI equations predicted an accurate strength. Walraven and Stroband[14] tested pushoff specimens made with 13,500 psi (93.1 MPa) concrete. In general, however, relatively little work has been done to evaluate shear friction for concrete strengths exceeding 10,000 psi (69 MPa).

RESEARCH SIGNIFICANCE

It is important that the current ACI code[1] provisions be evaluated for their applicability to high-strength concretes. This research shows that the current ACI provisions for shear friction may be applied to high-strength concretes and that those provisions may be modified to provide a more economical design.

EXPERIMENTAL PROGRAM

The pushoff specimens were designed to be nearly identical to those used by Hofbeck, Ibrahim, and Mattock[8] and by Anderson[6] so that the results of the current experiments could be compared directly to previous tests. The 50 pushoff specimens had concrete design strengths of 4, 7, 10, and 14 ksi (27.6, 48.3, 69.0, and 96.5 MPa). Transverse reinforcement was either one, two, three, or four two-leg No. 3 stirrups, as illustrated in Fig. 1 and photographed in Fig. 2. The shear plane was 5 in. wide x 12 in. long (127 x 305 mm); the transverse reinforcing ratio ρ_v varied from 0.37 to 1.47%. Table 1 lists the 50 specimens with their 28-day and date-tested concrete strengths. The latter were used in all subsequent evaluations. The first character of the specimen number is the design concrete strength in ksi units, the second character denotes the number of two-leg No. 3 stirrups crossing the shear plane, and the third character indicates the joint surface condition: U = uncracked, C = precracked, and CJ = cold joint. An "a" or "b" distinguishes two specimens constructed identically.

All concrete mixtures contained No. 67 crushed granite coarse aggregate and natural sand. Concretes with design

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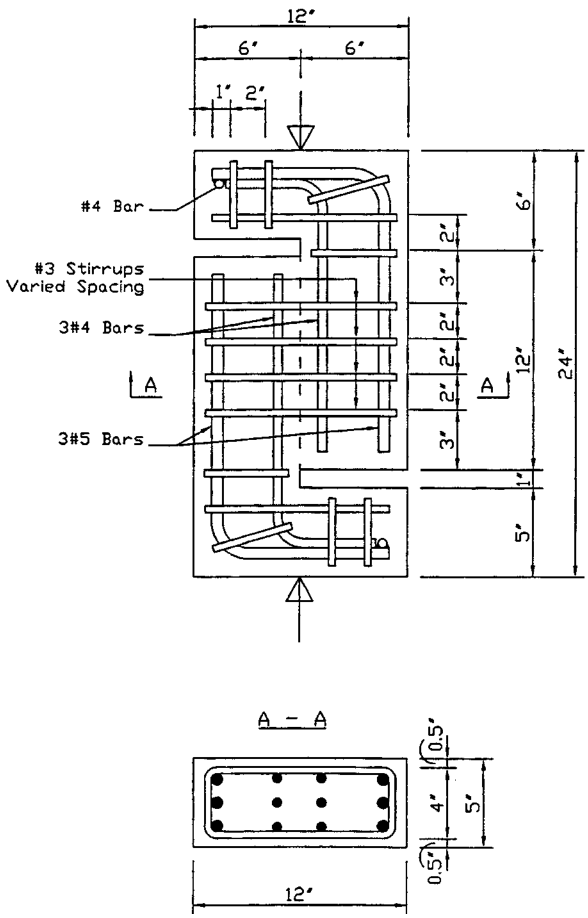


Fig. 1—Typical design of pushoff specimens.

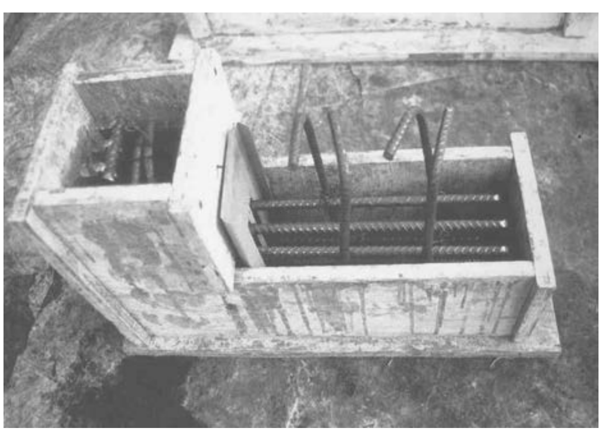


Fig. 2—Cold-joint specimen prior to casting.

strengths of 7000 psi (48.3 MPa) and greater used fly ash (13% replacement) and various amounts of silica fume. The Grade 60 reinforcement did not have a flat-top yield curve; the 0.2% offset yield point averaged 83.0 ksi (572.3 MPa) for 7, 10, and 14 specimens, and it averaged 69.5 ksi (479.2 MPa) for the 4 specimens. Details of specimen construction and testing are given in Reference 4.

As indicated in Fig. 2, the cold-joint specimens were cast so that the shear plane was horizontal. The surface was left as-cast; it was not floated or intentionally roughened. Generally, the

Table 1—Pushoff specimen concrete compressive strengths

Specimen identification no.	f_c' (28-day), psi	f_c' (test day), psi
SF-4-1-U	5361	6805
SF-4-1-C	5361	6805
SF-4-2-U	5361	6805
SF4-2-C	5361	6805
SF-4-3-U	5361	6805
SF-4-3-C	5361	6805
SF-7-1-U	9347	11,734
SF-7-1-C	9347	11,734
SF-7-1-CJ	9347	11,734
SF-7-2-U	9957	12,410
SF-7-2-C	9957	12,410
SF-7-2-CJ	9347	11,734
SF-7-3-U	10,692	13,103
SF-7-3-C	10,692	13,103
SF-7-3-CJ	10,259	12,471
SF-7-4-U	10,259	12,471
SF-7-4-C	10,259	12,471
SF-7-4-CJ	10,259	12,471
SF-10-1-U-a	9515	12,053
SF-10-1-U-b	11,117	14,326
SF-10-1-C-a	9515	12,053
SF-10-1-C-b	11,117	14,326
SF-10-1-CJ	11,117	14,326
SF-10-2-U-a	12,158	14,676
SF-10-2-U-b	11,775	14,804
SF-10-2-C-a	12,158	14,676
SF-10-2-C-b	11,775	14,804
SF-10-2-CJ	9515	12,053
SF-10-3-U-a	12,710	16,170
SF-10-3-U-b	11,333	13,934
SF-10-3-C-a	12,710	16,170
SF-10-3-C-b	11,333	13,934
SF-10-3-CJ	9485	12,953
SF-10-4-U-a	12,429	15,468
SF-10-4-U-b	12,256	16,476
SF-10-4-C-a	12,429	15,468
SF-10-4-C-b	12,256	16,476
SF-10-4-CJ	9485	12,953
SF-14-1-U	14,948	17,957
SF-14-1-C	14,095	16,015
SF-14-1-CJ	12,764	14,756
SF-14-2-U	13,735	17,362
SF-14-2-C	13,408	15,496
SF-14-2-CJ	12,764	14,756
SF-14-3-U	13,185	16,255
SF-14-3-C	12,910	15,392
SF-14-3-CJ	12,506	15,218
SF-14-4-U	13,767	16,059
SF-14-4-C	13,881	15,982
SF-14-4-CJ	12,506	15,218

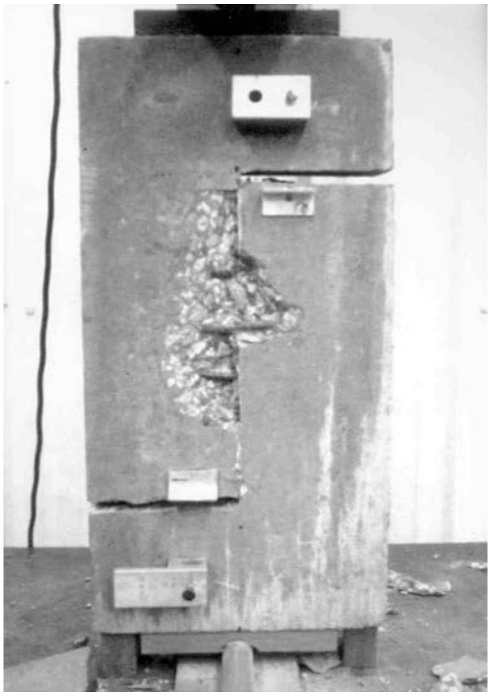


Fig. 3—Typical loading and failure of precracked specimen.

cold-joint surface was rough, approximately 1/4 in. (6 mm) amplitude; Specimens 10-1-CJ and 10-2-CJ had a very high slump mixture in which the as-cast surface was smooth. All other specimens were cast monolithically, so the shear plane was vertical.

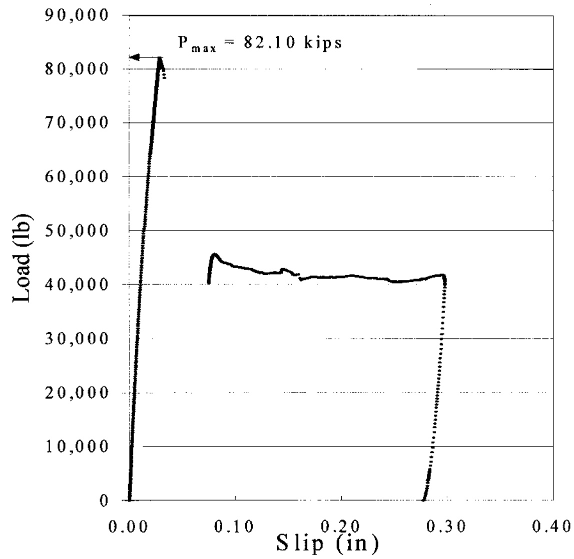
Approximately 4 months after casting, the specimens were tested as shown in Fig. 3. Prior to testing, the C specimens were precracked using the procedure described by Hofbeck, Ibrahim, and Mattock[8] by loading a knife-edge plate along the 12 in. (305 mm) shear plane. All cracks were smooth and straight. The slip across the shear interface was measured with two linear variable displacement transducers (LVDTs) mounted across the 1 in. (25 mm) openings. The compression load was applied through the pins at the top and bottom of the specimens. Load-slip data were recorded until a slip of 1/4 in. (6 mm) was achieved.

RESULTS AND DISCUSSION

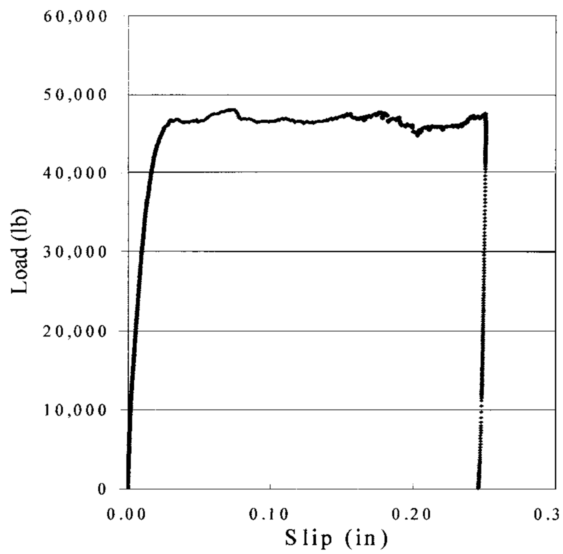
Observed behavior

In both the uncracked and cold-joint specimens, initial cracks were observed at loads between 50 and 75% of the peak ultimate capacity. The cracks were 1 to 3 in. long and were oriented diagonally between 15 and 45 degrees to the shear plane, similar to what was observed by Mattock, Li, and Wang.[15] The initial joint failure emanated from a vertical crack connecting the diagonal ones near the shear plane. Large amounts of spalling and cracks were observed at this fracture. Beyond the initial joint fracture, the load decreased to a nearly constant residual capacity. Both the cold-joint and uncracked specimens showed similar load-slip curves, as illustrated in Fig. 4(a).

In the precracked specimens, slip between the two faces began immediately upon the application of load. Cracking was not observed away from the shear plane until after significant yielding. Figure 4(b) illustrates the load-slip behavior of precracked specimens. The ultimate load was defined as the maximum load before or at a slip of 0.2 in. (5 mm). The residual shear capacity V_{ur} was the load recorded at a slip of 0.2 in. (5 mm). The residual shear stress was used to compare the postyield capacity of uncracked, cold-joint, and precracked specimens.



(a)



(b)

Fig. 4—(a) Typical load-slip curve for uncracked and cold-joint specimens; and (b) typical load-slip curve for precracked specimens.

The ultimate strengths and behaviors of the cold-joint and uncracked specimens were similar. The residual capacities of the cold-joint and uncracked specimens were about the same as those of the precracked specimens. Residual strength was first observed in the tests performed by Karagozian.[9] Mattock, Li, and Wang[15] made the observation that the residual capacity of an uncracked specimen was approximately equal to the capacity of a precracked specimen with a similar reinforcement arrangement. The data from this study confirmed that observation.

Table 2 presents the experimental results. The ultimate shear carried by the specimen is V_u. The ultimate shear was divided by the area of the shear interface A_c (60 in.^{2} [387 cm^2]) to obtain the average shear stress v_u. The clamping force $A_v f_y$ was divided by A_c to give the average clamping stress $\rho_v f_y$. The clamping force was calculated using both the actual

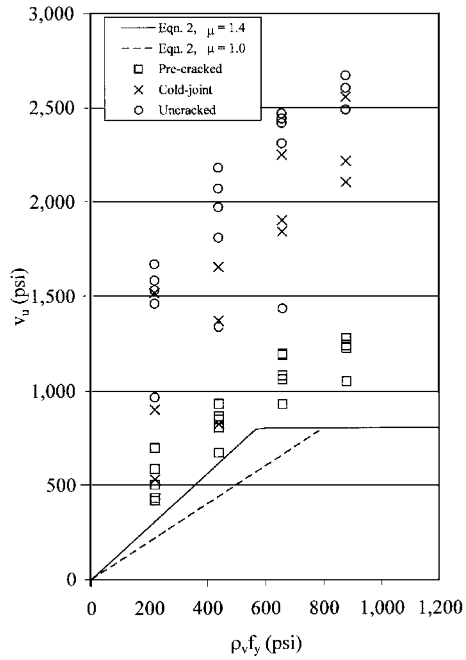


Fig. 5—Comparison of all specimens to Eq. (2) with f_y = *60 ksi (413.7 MPa).*

yield stress of the reinforcement and using f_y = 60 ksi (413.7 MPa) as listed in Table 2.

Analysis

In comparing the experimental data to the ACI Eq. (2) and to other relations, the capacity reduction factor was taken as 1 (ϕ = 1.0). Further, results were found to be much more consistent when the yield stress f_y was taken as 60 ksi (413.7 MPa) rather than using the measured yield stress, as discussed in detail by Mitchell.[4] When the actual yield stress was used, there was greater scatter in friction results when comparing normal-strength concretes tested by others and high-strength concretes of this study. The high-strength concretes appeared to give lower friction values, although μ was always greater than 1 for all interface conditions based on Eq. (2). When a maximum yield stress of 60 ksi (413.7 MPa) was used, the results of normal- and high-strength concretes showed less scatter and gave strengths greater than calculated using predictive equations.

It was observed that the final interface crack in the cold-joint and initially uncracked specimens was as smooth as the crack in the precracked specimens. The crack went through all aggregate; there was no roughness. Therefore, the strain in the transverse reinforcement needed to allow slip between these smooth surfaces was small. In the following figures, the clamping force was computed using the 60 ksi (413.7 MPa) value for yield stress so that the transverse strain was limited, the results for all concrete strengths were consistent, and all experimental shear-friction results were greater than given by Eq. (2) and (3).

Figure 5 relates the ultimate shear stress v_u to the clamping stress $\rho_v f_y$. The horizontal line represents the 800 psi (5.5 MPa) limit imposed by the ACI code.[1] All shear stresses are greater than the stress given by Eq. (2) using a friction coefficient μ of 1.4. Even for the precracked specimens, a shear stress value two standard deviations below the mean is still greater than that predicted by Eq. (2). All cold-joint specimens, even those with smooth, as-cast surfaces, had strengths significantly greater than predicted using a μ of 1.4.

Figure 6 shows that the residual shear stresses of cold-joint, uncracked, and precracked specimens were similar and that the residual capacities were greater than predicted by

Table 2—Pushoff specimen test results

Specimen identification no.	$A_v f_y$, kips	$\rho_v f_y$, psi	$\rho_v f_y$,* psi	V_u, kips	v_u, psi	v_{ur}, psi
SF-4-1-C	15.3	255	220	35.00	583	383
SF-4-1-U	15.3	255	220	57.88	965	461
SF-4-2-C	30.6	510	440	55.69	928	757
SF4-2-U	30.6	510	440	80.08	1335	845
SF-4-3-C	45.9	765	660	71.13	1186	935
SF-4-3-U	45.9	765	660	85.83	1431	1064
SF-7-1-C	18.3	304	220	41.68	695	341
SF-7-1-CJ	18.3	304	220	54.00	900	317
SF-7-1-U	18.3	304	220	87.55	1459	428
SF-7-2-C	36.5	609	440	51.73	862	862
SF-7-2-CJ	36.5	609	440	82.10	1368	690
SF-7-2-U	36.5	609	440	118.11	1969	628
SF-7-3-C	54.8	913	660	71.51	1192	876
SF-7-3-CJ	54.8	913	660	110.30	1838	668
SF-7-3-U	54.8	913	660	138.43	2307	774
SF-7-4-C	73.0	1217	880	62.73	1046	1046
SF-7-4-CJ	73.0	1217	880	132.68	2211	1303
SF-7-4-U	73.0	1217	880	149.09	2485	1005
SF-10-1-C-a	18.3	304	220	25.78	430	430
SF-10-1-C-b	18.3	304	220	29.97	500	429
SF-10-1-CJ	18.3	304	220	31.73	529	302
SF-10-1-U-a	18.3	304	220	100.09	1668	391
SF-10-1-U-b	18.3	304	220	91.88	1531	386
SF-10-2-C-a	36.5	609	440	50.78	846	811
SF-10-2-C-b	36.5	609	440	48.11	802	755
SF-10-2-CJ	36.5	609	440	49.29	822	504
SF-10-2-U-a	36.5	609	440	130.65	2178	739
SF-10-2-U-b	36.5	609	440	124.05	2068	758
SF-10-3-C-a	54.8	913	660	64.65	1078	1016
SF-10-3-C-b	54.8	913	660	63.36	1056	997
SF-10-3-CJ	54.8	913	660	113.91	1899	983
SF-10-3-U-a	54.8	913	660	144.82	2414	N/R
SF-10-3-U-b	54.8	913	660	147.90	2465	N/R
SF-10-4-C-a	73.0	1217	880	74.16	1236	1188
SF-10-4-C-b	73.0	1217	880	76.28	1271	999
SF-10-4-CJ	73.0	1217	880	126.04	2101	1308
SF-10-4-U-a	73.0	1217	880	156.03	2601	1199
SF-10-4-U-b	73.0	1217	880	160.04	2667	N/R
SF-14-1-C	18.3	304	220	24.88	415	410
SF-14-1-CJ	18.3	304	220	90.91	1515	469
SF-14-1-U	18.3	304	220	94.95	1583	452
SF-14-2-C	36.5	609	440	40.18	670	637
SF-14-2-CJ	36.5	609	440	99.19	1653	795
SF-14-2-U	36.5	609	440	108.46	1808	543
SF-14-3-C	54.8	913	660	55.50	925	907
SF-14-3-CJ	54.8	913	660	134.71	2245	931
SF-14-3-U	54.8	913	660	146.23	2437	1050
SF-14-4-C	73.0	1217	880	73.27	1221	1183
SF-14-4-CJ	73.0	1217	880	153.12	2552	1193
SF-14-4-U	73.0	1217	880	155.97	2600	1267

*f_y = 60 ksi (413.7 MPa).

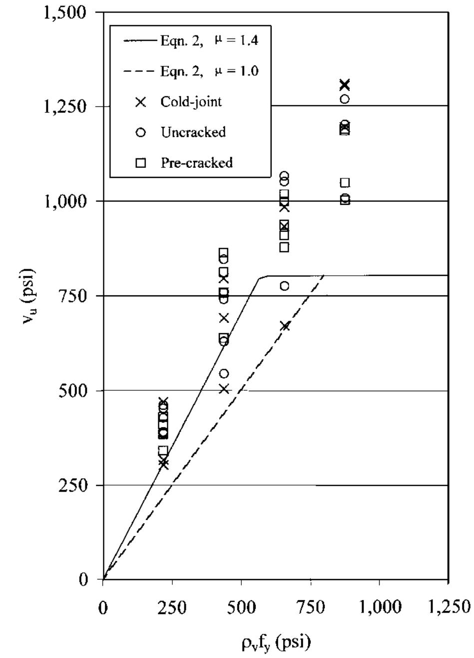


Fig. 6—Comparison of residual strengths f_y *= 60 ksi (413.7 MPa).*

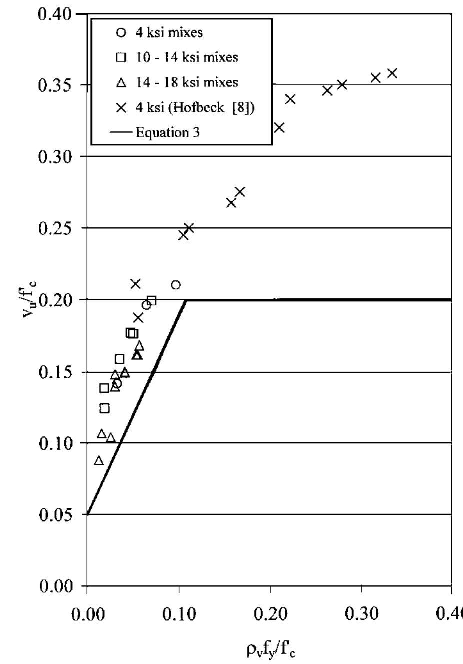


Fig. 8—Comparison of uncracked specimens with Eq. (3), f_y *= 60 ksi (413.7 MPa).*

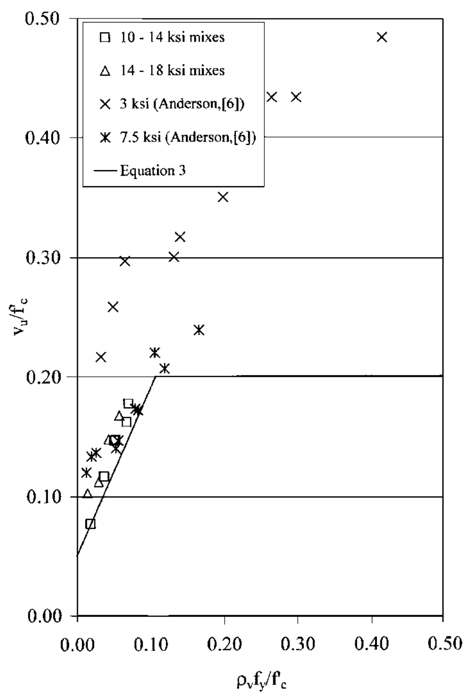


Fig. 7—Comparison of cold-joint specimens with Eq. (3), f_y *= 60 ksi (413.7 MPa).*

Eq. (2) when a μ of 1.0 was used. This result implies that the precracking method simulates an initial joint fracture more than just an accidental crack, as discussed by Mast[3] and by Hofbeck, Ibrahim, and Mattock.[8]

The experimental data were compared with other theoretical relations[4,10,14,16,17] along with previous data based on lower-strength concretes. The other relations did not accurately predict the shear friction capacities of high-strength concrete. Equation (3) given as follows was developed to give a simple, accurate, and conservative prediction of the shear capacity of a cold-joint or an uncracked joint for a wide range of concrete strengths. Equation (3) has a component in the form of the original equation proposed by Birkeland and Birkeland.[2] The equation, however, is also in agreement with the rationale of Basler and Witta,[18] Mattock and Hawkins,[10] and Mattock.[19] The equation incorporates a frictional component (μ = 1.4) and a component for bond and asperity shear ($0.05 f_c'$). By taking the component for bond and asperity shear as a percentage of the concrete compressive strength, the equation better predicted results for both normal- and high-strength concretes as compared with a constant value

$$v_u = 0.05 f_c' + 1.4 \rho_v f_y \leq 0.2 f_c' \text{ [psi]} \quad (3)$$

An upper limit for the shear strength was set at 20% of the compressive strength of the concrete because that agreed with the test results and because it was the same as ACI code[1] provisions. A constant upper limit like the ACI code[1] value of 800 psi (5.5 MPa) was not required for the concrete strengths considered (3 to 18 ksi [20.7 to 124.1 MPa]).

Figure 7 presents rough cold-joint data from Anderson[6] and from the current study compared with Eq. (3) with the yield stress limited to 60 ksi (413.7 MPa). The ultimate shear stresses and the clamping forces were divided by the concrete strength for ease of comparison. In all cases, the actual ultimate shear stress was greater than that predicted. Figure 8 similarly compares uncracked specimen data from Hofbeck, Ibrahim, and Mattock[8] and from the current study to Eq. (3). Again, the actual ultimate shear stresses were greater than predicted.

CONCLUSIONS AND RECOMMENDATIONS

Tests of 50 pushoff specimens with concrete strengths between 6800 and 17,900 psi (46.7 and 123.4 MPa) showed that current ACI[1] shear friction provisions gave a conservative estimate for interface shear strength for high-strength concrete. The current pushoff tests extended the studies initiated by Anderson[6] and by Hofbeck, Ibrahim, and Mattock[8] to high-strength concrete. For precracked, cold-joint, and uncracked specimens, shear friction strengths were stronger than predicted by Eq. (2) using a coefficient of friction μ equal to 1.4 when f_y was limited to 60 ksi (413.7 MPa). Birkeland and Birkeland[2] and Mast[3] suggested that a coefficient of friction equal to 1.4 should be used for designing a rough cold-joint interface. This was later supported by cold-joint tests by Karagozian and Case[9] for concrete strengths up to 8000 psi (55.2 MPa). The data from the current research support this conclusion for high-strength concretes up to 18,000 psi (124.1 MPa). The proposed Eq. (3) provides a good prediction for the strength of cold-joint and uncracked joint interfaces. It is recommended that the yield stress in the transverse reinforcement be limited to 60 ksi (413.7 MPa) to limit the slip along the smooth cracks in high-strength concrete and to give a uniform friction coefficient for normal- and high-strength concrete. Further, it is proposed that the upper limit of shear stress be 20% of the concrete strength and that it not be limited to 800 psi (5.5 MPa).

The method of precracking pushoff specimens utilized in this experiment was developed by Hofbeck, Ibrahim, and Mattock[8] to consider the accidental crack first proposed by Mast.[3] The strength of a line load precracked interface was shown to be similar to the residual strength of both a cold-joint and an uncracked interface after initial joint failure had occurred. Such residual postyielding strength was conservatively predicted using the current ACI code[1] shear friction relation with a μ equal to 1.0.

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NOTATION

A_c = area of concrete interface
A_v = area of shear reinforcement across shear plane
f_c' = concrete compressive strength
f_y = yield stress of reinforcement
V_n = nominal shear strength
v_n = nominal shear stress V_n/A_c
V_u = ultimate experimental shear strength
v_u = ultimate experimental shear stress V_u/A_c
V_{ur} = residual experimental shear strength (shear force at a slip = 0.2 in. [5 mm])
μ = coefficient of friction
ρ_v = shear friction reinforcement ratio A_v/A_c
ϕ = capacity reduction factor, taken as 1.0

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