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Shear Friction and High-Strength Concrete

by Alan H. Mattock

*Design provisions applicable when shear is transferred across a specific plane are set out in ACI 318-99, Section 11.7—Shear Friction. These provisions are used to design the required reinforcement across an existing or potential crack in the design of connections, or across an interface between concretes cast at different times. The design equations set out in Section 11.7.4, and the limitation of 0.2*$f'_c A_c$*, but not more than 800*A_c*, placed upon the shear strength* V_n *in Section 11.7.5, prevent full advantage being taken of the high shear transfer strengths that can be obtained when very-high-strength concretes are used. Shear transfer behavior and the available data from shear transfer tests of initially cracked specimens are re-examined. Simple equations are proposed for shear friction design, which allow the full potential shear transfer strength of all strengths of concrete to be utilized.*

Keywords: precast concrete; reinforced concrete; shear.

INTRODUCTION

Prior to the 1960s, there was no systematic approach to the design of precast concrete connections and bearing details in the U.S. Most textbooks and design handbooks simply showed drawings of typical details, and in some cases, provided rules of thumb for proportioning the reinforcement. A significant breakthrough occurred in the mid-1960s with the publications by Mast[1] and by Birkeland and Birkeland,[2] in which the following philosophy for the design of precast connections and bearing details was proposed:

1. No reliance is to be placed on the tensile strength of concrete; all tensile forces are to be carried by steel reinforcement;

2. It is assumed that due to a variety of unspecified causes such as transportation, handling and erection stresses, and stresses due to shrinkage and temperature effects, cracks occur in the concrete in the most unfavorable locations prior to the application of the design loads; and

3. Reinforcement is to be designed to carry direct tensile forces across the cracks, together with any shear acting along the cracks.

Mast[1] and Birkeland and Birkeland[2] proposed that shear could be transferred across the cracks by what they termed "shear friction" between the rough faces of the cracks. They postulated that as the uneven crack faces slide past one another, the projections on the crack faces ride over one another and force the crack faces apart, stretching any reinforcement crossing the crack sufficiently to cause it to yield. The tensile force so developed in the reinforcement is assumed to compress the crack faces together, which results in frictional resistance to sliding along the crack.

By comparing its predictions of shear strength with available shear transfer strength data from tests, Mast[1] and Birkeland and Birkeland[2] showed that this shear friction hypothesis predicted shear resistance along a crack in a conservative manner. Further experimental studies[3-7] of shear friction were made and were successfully applied in practical design. This led to its codification in the 1977 ACI Building Code, ACI 318-77,[8] and its inclusion in succeeding editions of the Code.

The earlier experimental studies[3-7] of shear friction were made using concretes with compressive strengths in the range of 3000 to 6000 psi. More recent studies[9-12] using test specimens made from concretes with compressive strengths of up to 15,000 psi showed that the simple shear friction theory as codified in Section 11.7 of ACI 318-99[13] does not truly represent the behavior of these very-high-strength concretes, and prevents full advantage being taken of the potential shear friction strength of these concretes.

Walraven[9] carried out shear transfer tests using concretes with compressive strengths up to 9000 psi and made a sophisticated analysis of aggregate interlock effects. Based on this study, he proposed an equation for shear transfer strength that was more accurate than the simple shear friction equation, but which he considered too complicated for design use. He therefore provided a design chart based on the equation.

This paper reports a re-appraisal of available shear friction test data and presents simple, modified shear friction equations suitable for use in design. These equations reflect the influence of concrete strength on shear friction strength. They enable full advantage to be taken of the potentially high-shear-friction strengths of very-high-compressive strength concretes over the whole range of values of the shear friction reinforcement yield strength $A_{vf}f_y$. (A_{vf} = area of shear friction reinforcement; f_y = yield stress of shear friction reinforcement.) This results in a reduction in the amount and the congestion of reinforcement in precast concrete connections and bearing details.

SHEAR FRICTION AND ACI 318-99

The provisions for design using shear friction contained in Section 11.7.3 in the ACI Building Code, ACI 318-99,[13] are based on the assumption that a crack exists in the shear plane before shear is applied along it. This assumption is in keeping with the application of shear friction in the design of connections and other details in precast concrete construction, as first proposed by Mast[1] and Birkeland and Birkeland.[2] In addition, the shear transfer strength and behavior of initially uncracked and initially cracked reinforced concrete differ significantly, as long as slip occurs along the cracked shear plane. Therefore, when developing design provisions for use in the Code, only data obtained from tests of initially cracked specimens (that is, specimens cracked along the shear plane before the application of shear loading) should be considered.

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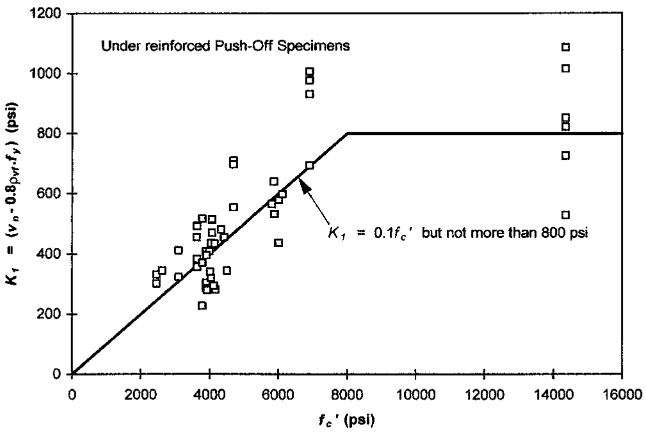

Fig. 1—K_1 *for initially cracked normalweight concrete.*

In the case of initially uncracked reinforced concrete, short diagonal tension cracks occur across the shear plane, forming diagonal concrete struts. These struts act with the reinforcement crossing the shear plane to form a truss, which resists shear along the shear plane. Failure occurs when the struts fail after yield of the reinforcement.[4]

When shear is transferred across a pre-existing crack, slip occurs along the plane of the crack. Shear friction strength is developed by a combination of resistance to the shearing off of projections on the rough crack faces and frictional resistance to slip due to compression between the crack faces. This compression results from tension in the reinforcement caused by separation of the irregular crack faces as they slide over one another. When the crack becomes over-reinforced, the crack faces lock up, that is, they cease to slide over one another. Diagonal tension cracks then form across the original crack, and failure occurs in a manner similar to that which occurs in initially uncracked concrete. At this stage, the shear strength increases with the reinforcement yield strength $A_{vf}f_y$ at a much reduced rate.

The shear friction design method set out in Section 11.7.4 of ACI 318-99[13] assumes that the shear resistance V_n is directly proportional to the yield strength $A_{vf}f_y$ of the shear friction reinforcement crossing the shear plane at right angles. ACI 318-99, Eq. (11-25) is

$$V_n = A_{vf} f_y \mu$$

where $\mu = 1.4$ for a crack in monolithic concrete.

This simplified model of behavior does not reflect the true variation of the shear friction strength of a cracked shear plane with the yield strength of the shear friction reinforcement. Initially, the shear strength increases very rapidly with increase in the yield strength of the shear friction reinforcement; however, the rate of increase gradually decreases to a constant value of approximately 0.8. This rate of increase is maintained until the shear plane becomes over-reinforced at a shear V_n of approximately $0.3f_c'A_c$ in the case of normalweight concrete. The shear resistance subsequently increases at a reduced rate as the shear friction reinforcement is increased. The reinforcement yield strength $A_{vf}f_y$, at which the rate of increase in shear friction strength becomes constant at approximately 0.8, increases with the concrete compressive strength f_c'.

Because the assumed rate of increase μ of shear friction strength, along with an increase in shear reinforcement yield strength $A_{vf}f_y$ in ACI 318-99, Eq. (11-25), is greater than the actual rate of increase of 0.8, this equation eventually becomes unconservative. To prevent this from occurring in design, the ACI Code[13] places an upper limit of $0.2f_c'A_c$, but not more than $800A_c$ lb, on the value of V_n calculated using ACI 318-99, Eq. (11-25). Hence, the value of $800A_c$ controls in design when f_c' is greater than 4000 psi. Therefore, if ACI 318-99, Eq. (11-25) is used in design, it is not possible to take advantage of any further increase in shear friction strength that may occur when high-strength concretes are used.

Alternatives to ACI 318-99 Code, Eq. (11-25)

It is proposed that provisions that reflect the actual increase in shear friction strength that result from the use of high-strength concrete may be developed as follows.

When $\rho_{vf}f_y = A_{vf}f_y/A_c$ is greater than the amount at which the unit shear strength $v_n = V_n/A_c$ commences to increase linearly with the value of $\rho_{vf}f_y$, v_n can be calculated using

$$v_n = K_1 + 0.8\rho_{vf} f_y \text{ (psi)}, \tag{1}$$

but not greater than $K_2 f_c'$ nor K_3 psi

The upper limit corresponds to the point at which the shear plane becomes over-reinforced, that is, the crack locks up and the shear transfer strength increases at a much reduced rate. Test data reviewed later indicate that appropriate values for K_2 and K_3 are 0.3 and 2400 psi for normalweight concrete, and 0.2 and 1200 psi for sand-lightweight concrete and all lightweight concrete.

To evaluate the effect of concrete compressive strength on the value of K_1, $K_1 = [v_n - 0.8(\rho_{vf}f_y)]$ was plotted against concrete compressive strength f_c' for 47 test results[3,6,7,10,11] from initially cracked pushoff specimens made from normalweight concrete, which failed in an under-reinforced manner. (This was assumed to have occurred if v_n was less than $0.3f_c'$.) This plot is shown in Fig. 1. For simplicity, the following expression is proposed for K_1

$$K_1 = 0.1f_c' \text{ psi, but not more than 800 psi} \tag{2}$$

The upper limit corresponds to the fact that in very-high-strength normalweight concretes, tension cracks pass through much of the aggregate, rather than occurring at the paste-aggregate interface as in lower-strength concretes. This results in a smoother crack face than those that occur in the lower-strength concretes, which limits the shear transfer resistance.

An equation similar to Eq. (1) is found in Section R11.7.3 of ACI 318R-99.[13] In R11.7.3, the value of K_1 is assumed to be 400 psi for normalweight concrete, 200 psi for lightweight concrete, and 250 psi for sand-lightweight concrete, for all concrete compressive strengths.

Review of data from tests without additional normal stress acting across the shear plane indicates that the value of $\rho_{vf}f_y$ at which Eq. (1) becomes valid increases with concrete strength, and may be taken as $K_1/1.45$.

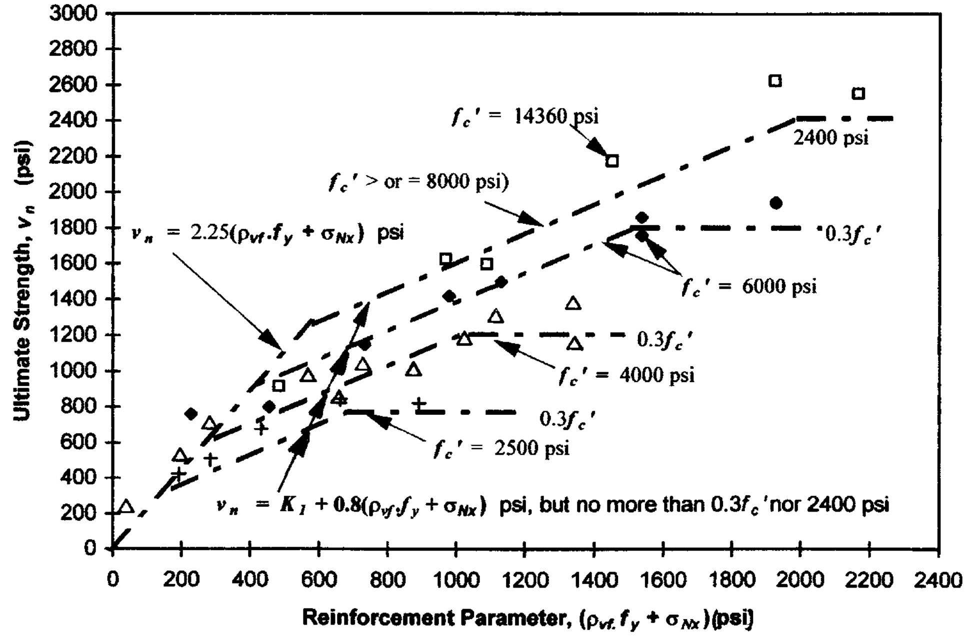

Fig. 2—Shear friction strength of cracked normalweight concrete.

Tests[5] indicate that the effect on shear friction strength of an additional normal stress σ_{Nx} acting across an irregular crack is the same as if the shear friction reinforcement parameter $\rho_{vf} f_y$ was changed by an amount equal to the normal stress (compression positive, tension negative). Equation (1) may therefore be restated as follows

$$v_n = K_1 + 0.8(\rho_{vf}\, f_y + \sigma_{Nx}) \text{ (psi)}, \tag{3}$$

but not greater than $K_2 f_c'$ nor K_3 psi

where $K_1 = 0.1 f_c'$, but not more than 800 psi; $K_2 = 0.3$; $K_3 =$ 2400 psi; and $(\rho_{vf} f_y + \sigma_{Nx})$ is not less than $K_1/1.45$. Equation (3) is equivalent to design Eq. 6 that is proposed in a following section.

For $(\rho_{vf} f_y + \sigma_{Nx})$ less than $K_1/1.45$ psi, insufficient data exists to define closely the relationship between v_n and $(\rho_{vf} f_y + \sigma_{Nx})$. Because the actual relationship corresponds to a convex upward curve from the origin to the point at which Eq. (3) becomes valid, use of a linear relationship is conservative. This is represented by the following equation

$$v_n = 2.25(\rho_{vf}\, f_y + \sigma_{Nx}) \text{ psi} \tag{4}$$

This corresponds to Eq. 7, which is proposed in a following section. Specimen 3.1 of Hofbeck, Ibrahim, and Mattock[3] had $\rho_{vf} f_y = 49$ psi and v_u(test) of 240 psi. Equation (4) yields a value of v_u(calc) of 110 psi, which is conservative. Specimen 210204 of Walraven, Frénay, and Pruijssers[11] had $\rho_{vf} f_y = 154$ psi and a v_u(test) of 467 psi. Equation (4) yields a value of v_n(calc) of 347 psi, which is also conservative. Equation (4) therefore appears to be reasonable and is also less conservative than ACI 318-99, Eq. (11-25).

In Fig. 2, Eq. (3) and (4) are compared with test data[3,7,10] from four series of pushoff specimens with concrete compressive strengths of 2500, 4000, 6000, and 14,360 psi, respectively. It can be seen that the equations reflect the trends in the experimental data very closely.

In Table 1, Eq. (3) and (4) are used to calculate the shear strength v_n(calc) of 82 pushoff specimens[3-6,10,11] subject to monotonically increasing load to failure, for which the concrete compressive strength f_c' varied between 2453 and 14,358 psi. Nine of the specimens had tension or compression acting across the shear plane in addition to shear acting along the shear plane. The average value of the ratio of the test strength v_n to the calculated strength v_n(calc), is 1.066, and the standard deviation is 0.120. Sixteen of the test specimens had ultimate shear strengths v_n greater than $0.3 f_c'$, and are considered to have experienced over-reinforced failures. The variation of v_n/v_n(calc) with the concrete compressive strength f_c' is shown in Fig. 3.

In Table 2, Eq. (3) is used to calculate the shear strength of 16 pushoff specimens[11] that had been subjected to cyclic loading from zero to between 46 and 66% of their static ultimate strength before being loaded monotonically to failure. The concrete compressive strengths varied between 5560 and 9288 psi. The average ratio of test to calculated shear strength for this series is 1.115, and the standard deviation is 0.115.

In Table 3, Eq. (3) is used to calculate the shear strength of 16 pushoff specimens[11] that had been subjected to sustained shear loads of between 40 and 82% of their static ultimate strength before being loaded monotonically to failure. The concrete compressive strengths varied between 6724 and 9902 psi. The average ratio of test to calculated shear strength for this series is 1.295, and the standard deviation is 0.148.

In Table 4 and Fig. 4, data are shown from tests[6,12] of 34 initially cracked pushoff specimens made from sand-lightweight concrete with compressive strengths ranging from 2230 to 10,745 psi. These are compared with the strengths calculated using Eq. (3) and (4), using values of 250 psi for K_1, 0.2 for K_2, and 1200 psi for K_3. It can be seen that it is appropriate to use the single value of 250 psi for K_1 regardless of the compressive strength of the concrete. This is due to the fact that in lightweight concretes of all strengths, cracks pass through the aggregate, producing a relatively

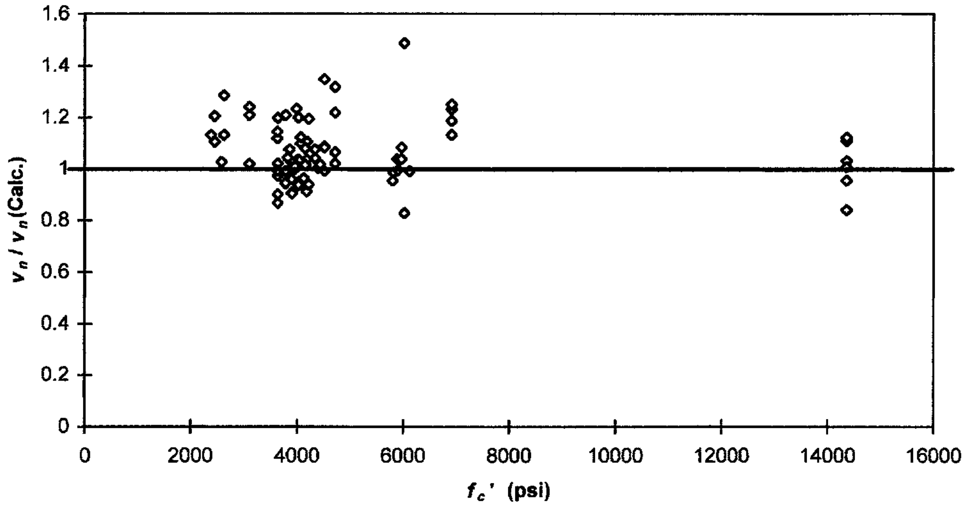

Fig. 3—Pushoff tests of initially cracked normalweight concrete (short-term loading).

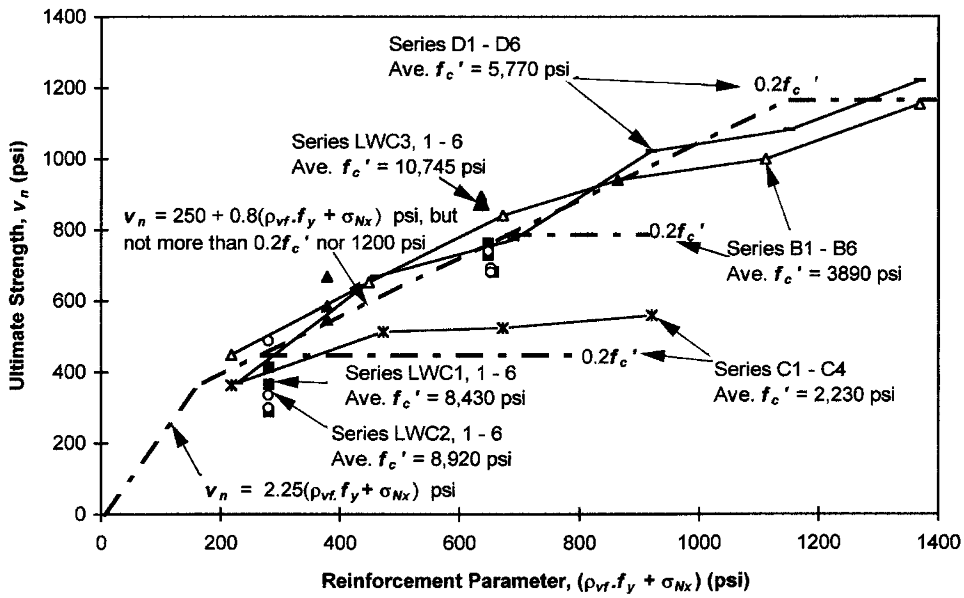

Fig. 4—Shear friction strength of cracked sand-lightweight concrete.

smooth crack surface. The roughness of this surface is independent of concrete compressive strength.

The reason for the adoption of an upper limit for the shear transfer strength of sand-lightweight concrete of $0.2f_c'$, but not more than 1200 psi, can readily be seen in Fig. 4.

The overall average of the ratio of the test strength to the calculated strength is 1.014, with a standard deviation of 0.186. The reason for the low values of strength of the specimens of Series LWC1 and LWC2 with reinforcement parameters $\rho_{vf}f_y$ of 280 psi is not known. The strengths of these specimens made of 8500 psi concrete is exceeded by that of specimens made of much lower-strength concrete.

In Table 5 and Fig. 5, test data[6] are shown from 14 initially cracked pushoff specimens made from all lightweight concrete. These data are compared with the shear friction strength calculated using Eq. (3) and (4), and the proposed values of K_1 = 200 psi; K_2 = 0.2; and K_3 = 1200 psi. The average value of the test strength to the calculated strength is 1.104, and the standard deviation is 0.106.

Test data[7] from 11 initially cracked, normalweight concrete composite specimens with the interface deliberately roughened in accordance with Section 11.7.9 of ACI 318-99[13] are compared in Table 6 and Fig. 6. The shear friction strength is calculated using Eq. (3) and (4), and the proposed values of K_1 = 400 psi; K_2 = 0.3; and K_3 = 2400 psi. It can be seen that when concretes of two different strengths are joined at the interface, it is appropriate to use the compressive strength of the lower-strength concrete when calculating K_3f_c'. The average value of test strength to calculated strength is 1.07, and the standard deviation is 0.084.

In Fig. 7, data are shown for 12 composite pushoff specimens[7] made of normalweight concrete that had a smooth interface. In the six specimens of Series C, trouble was taken to obtain good bond between the two concretes, then, before testing, the shear plane was cracked so as to produce a crack width of 0.01 in. In the six specimens of Series H, a bond breaker was used to prevent bond between the two concretes, but no crack was formed in the shear plane before testing. It can be seen that for a smooth interface that is initially cracked, or over which bond is broken, the shear resistance along the shear plane is equal to the shear yield strength of the shear friction reinforcement crossing the shear plane at right angles. No true shear friction action can be developed if there is no roughness of the

Table 1—Shear friction in initially cracked normalweight concrete

Researcher	Specimen	$\rho_{vf}f_y$, psi	σ_{Nx}, psi	f_c', psi	v_n, psi	$0.3f_c'$, psi	K_1, psi	v_n(calc), psi	Test/calc
Mattock[7]	A1	227	0	6020	760	1806	602	511	1.49
	A2	454	0	6020	800	1806	602	965	0.83
	A3	732	0	5820	1150	1746	582	1168	0.98
	A4	976	0	5880	1420	1764	588	1369	1.04
	A5	1128	0	6125	1500	1838	613	1515	0.99
	A6	1536	0	5900	1760	1770	590	1770	0.99
	A6A	1536	0	5970	1860	1791	597	1791	1.04
	A7	1928	0	5970	1940	1791	597	1791	1.08
Hofbeck, Ibrahim, and Mattock[3]	2.1	223	0	3100	590	930	310	488	1.21
	2.2	446	0	3100	680	930	310	667	1.02
	2.3	670	0	3900	840	1170	390	926	0.91
	2.4	893	0	3900	1000	1170	390	1104	0.91
	2.5	1120	0	4180	1300	1254	418	1254	1.04
	2.6	1340	0	4180	1385	1254	418	1254	1.10
	3.2	223	0	4010	520	1203	401	502	1.04
	3.3	446	0	3100	680	930	310	667	1.02
	3.4	740	0	4040	1028	1212	404	996	1.03
	3.5	1040	0	4040	1152	1212	404	1212	0.95
	4.1	293	0	4070	704	1221	407	641	1.10
	4.2	583	0	4070	980	1221	407	873	1.12
	4.3	874	0	4340	1180	1302	434	1133	1.04
	4.4	1165	0	4340	1400	1302	434	1302	1.08
	4.5	1455	0	4390	1320	1317	439	1317	1.00
	5.1	223	0	2450	510	735	245	423	1.20
	5.2	446	0	2620	700	786	262	619	1.13
	5.3	670	0	2385	810	716	239	716	1.13
	5.4	893	0	2580	795	774	258	774	1.03
	5.5	1120	0	2620	1010	786	262	786	1.28
Mattock, Li, and Wang[6]	N1	224	0	4180	460	1254	418	504	0.91
	N2	464	0	3900	780	1170	390	761	1.02
	N3	690	0	3995	960	1199	400	952	1.01
	N4	896	0	4150	1150	1245	415	1132	1.02
	N5	1120	0	3935	1175	1181	394	1181	1.00
	N6	1120	0	4120	1190	1236	412	1236	0.96
Walraven and Stroband[10]	10	483	0	14,358	914	4307	800	1087	0.84
	11	967	0	14,358	1624	4307	800	1574	1.03
	12	1450	0	14,358	2175	4307	800	1960	1.11
	13	1924	0	14,358	2625	4307	800	2339	1.12
	14	1087	0	14,358	1595	4307	800	1670	0.96
	15	2166	0	14,358	2553	4307	800	2533	1.01
Walraven, Frenay, and Pruijssers[11]	110208t	352	0	4426	737	1328	443	724	1.02
	110208	352	0	3785	798	1136	379	660	1.21
	110208g	352	0	3624	737	1087	362	644	1.14
	110408	705	0	3785	934	1136	379	943	0.99
	110608	1057	0	3785	1072	1136	379	1136	0.94
	110808h	1410	0	3624	1217	1087	362	1087	1.12
	110808h	1410	0	3624	1244	1087	362	1087	1.14
	110706	809	0	3908	1043	1172	391	1038	1.00
	210204	154	0	4512	467	1354	451	347	1.35
	210608	1057	0	4512	1410	1354	451	1297	1.09
	210216	1468	0	4512	1342	1354	451	1354	0.99
	210316	2200	0	4512	1466	1354	451	1354	1.08
	210808h	1410	0	3107	1156	932	311	932	1.24
	120208	352	0	3637	773	1091	364	645	1.20
	120408	705	0	3637	947	1091	364	928	1.02
	120608	1057	0	3637	983	1091	364	1091	0.90

Table 1 (continued)—Shear friction in initially cracked normalweight concrete

Researcher	Specimen	$\rho_{vf} f_y$, psi	σ_{Nx}, psi	f_c', psi	v_n, psi	$0.3f_c'$, psi	K_1, psi	v_n(calc), psi	Test/calc
	120808	1410	0	3637	1060	1091	364	1091	0.97
	120706	809	0	3637	1004	1091	364	1011	0.99
	120216	1468	0	3637	947	1091	364	1091	0.87
	230208	352	0	6916	975	2075	692	792	1.23
	230408	706	0	6916	1571	2075	692	1256	1.25
	230608	1057	0	6916	1822	2075	692	1537	1.19
Walraven, Frenay, and Pruijssers[11]	230808	1410	0	6916	2058	2075	692	1820	1.13
	240208	357	0	2453	587	736	245	531	1.11
	250208	352	0	4709	991	1413	471	753	1.32
	250408	705	0	4709	1260	1413	471	1035	1.22
	250608	1057	0	4709	1400	1413	471	1317	1.06
	250408	705	0	4709	1260	1413	471	1035	1.22
	250608	1057	0	4709	1400	1413	471	1317	1.06
	250808	1410	0	4709	1442	1413	471	1413	1.02
	E1C	543	0	3855	881	1157	386	820	1.07
	E2C	546	-100	4220	929	1266	422	779	1.19
	E3C	552	-163	3960	714	1188	396	707	1.01
	E4C	529	-200	3820	673	1146	382	645	1.04
Mattock, Johal, and Chow[5]	E5C	548	-300	4020	527	1206	402	558	0.94
	E6C	533	-400	3985	369	1196	399	299	1.23
	F1C	787	0	4220	988	1266	422	1052	0.94
	F4C	806	-200	3890	839	1167	389	874	0.96
	F6C	812	-400	4150	804	1245	415	745	1.08
	10.7	962	387	4020	1445	1206	402	1206	1.20
Mattock and Hawkins[4]	10.8	985	0	4020	1115	1206	402	1190	0.94
	10.10	312	813	5800	1410	1740	580	1480	0.95
Average test/calc			1.066	Standard deviation					0.120

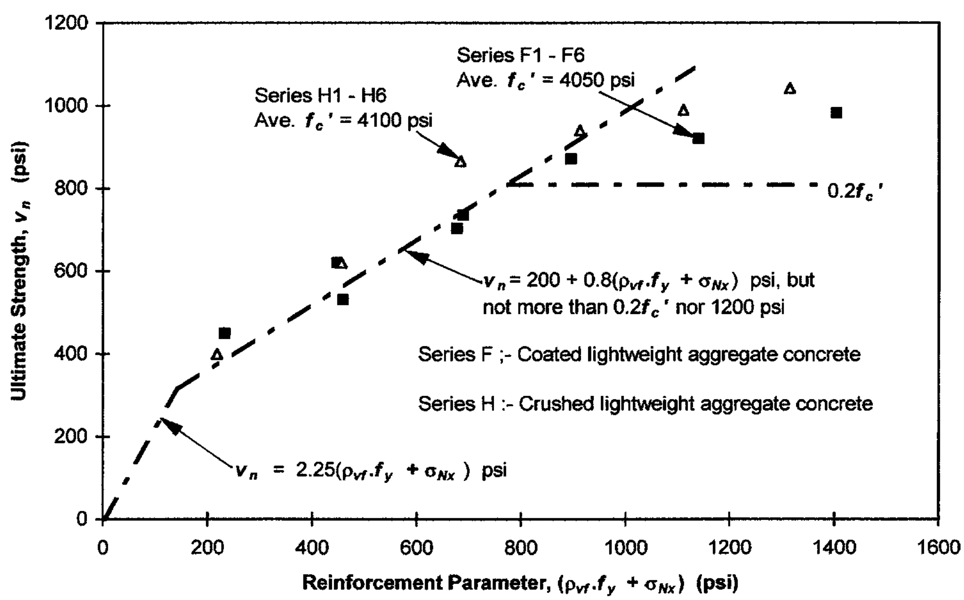

Fig. 5—Shear friction strength of cracked all lightweight concrete.

crack faces. In this case, additional normal stress acting across the shear plane will not increase the shear transfer strength.

The trend of the data at high values of $\rho_{vf} f_y$ indicate that 800 psi is a reasonable absolute upper limit for the shear transfer strength in this case, because the concretes used in the specimens had strengths of about 6000 psi. For lower-concrete strengths, $0.2f_c'$ would be a reasonable upper limit for shear strength. It can be seen in Fig. 7 that the unit shear strength v_n may be expressed as

$$v_n = 0.6\rho_{vf}\ f_y \text{ but not more than } 0.2\,f_c' \text{ nor } 800 \text{ psi} \quad (5)$$

In Table 7, a comparison is made between the test strengths v_n and the strengths v_n(calc) calculated using Eq. (5) for these

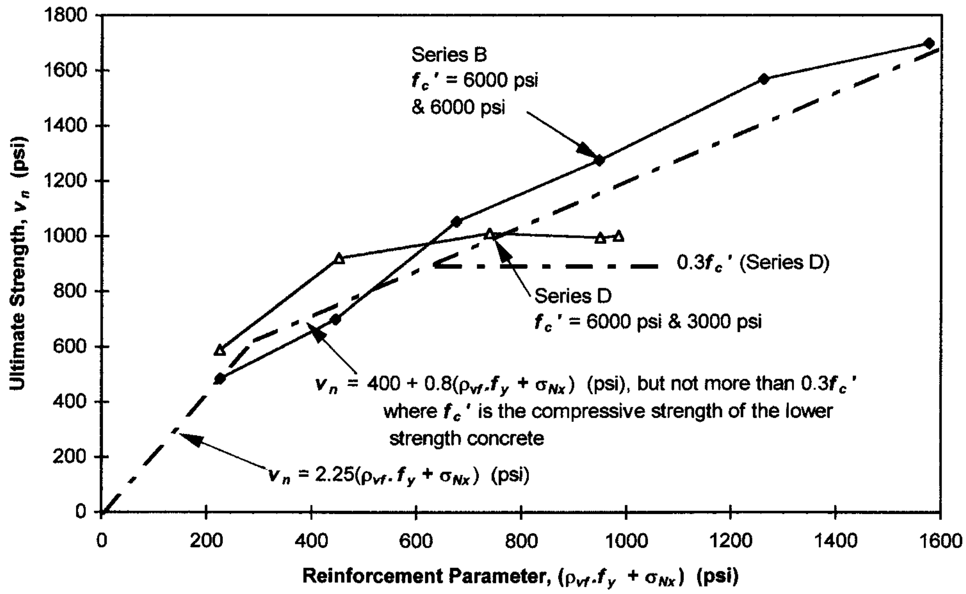

Fig. 6—Normalweight concrete, composite specimens, 1 (concrete placed against intentionally roughened surface).

Table 2—Initially cracked normalweight concrete specimens subject to cyclic load before static load test to failure

Researcher	Specimen	$\rho_{vf}f_y$, psi	σ_{Nx}, psi	f_c', psi	v_n, psi	$0.3f_c'$, psi	K_1, psi	v_n(calc), psi	Test/calc
Pruijssers and Ling[11]	15	1121	0	6420	1584	1926	642	1539	1.03
	16	1121	0	6430	1585	1929	643	1540	1.03
	48	1340	0	5560	1479	1668	556	1628	0.91
	23	747	0	6716	1475	2015	672	1269	1.16
	33	747	0	6189	1465	1857	619	1217	1.20
	51	893	0	6694	1707	2008	669	1384	1.23
	71	893	0	6286	1804	1886	629	1343	1.34
	25	1121	0	6333	1755	1900	633	1530	1.15
	24	1121	0	6213	1626	1864	621	1518	1.07
	18	1121	0	6324	1784	1897	632	1529	1.17
	42	1121	0	7064	1810	2119	706	1603	1.13
	29	747	0	8563	1537	2569	800	1398	1.10
	62	893	0	9288	1836	2786	800	1514	1.21
	26	1121	0	8716	1958	2615	800	1697	1.15
	41	1121	0	8837	2070	2651	800	1697	1.22
	40	1121	0	9258	2319	2777	800	1697	1.37

Note: Average test calculation = 1.155; standard deviation = 0.115.

specimens with a smooth interface. The average value of v_n/v_n(calc) is 1.130, and the standard deviation is 0.190.

No additional data is available for the case of concrete anchored to clean, unpainted, as-rolled structural steel by headed studs or by reinforcing bars, so it is proposed that the present Code[13] provisions be continued for this case. In this case also, an additional normal stress acting across the shear plane will not increase the shear transfer strength.

DESIGN PROPOSALS

Current design procedures for shear friction in ACI 318-99[13] tend to be conservative for high-strength concrete. As a simple way to take advantage of the improved shear friction strength in high-strength concrete, it is proposed that Eq. (3), (4), and (5) be used in design. These equations are expressed as follows in terms of force rather than stress to be consistent with ACI 318-99.[13]

1. For shear transfer across cracks in monolithic concrete and across the interface when concrete is placed against hardened concrete with its surface intentionally roughened as specified in 11.7.9

a. When $(A_{vf}f_y + N_x)$ is greater than or equal to $K_1A_c/1.45$ lb (or V_n is greater than or equal to 1.55 K_1A_c lb)

$$V_n = A_cK_1 + 0.8(A_{vf}f_y + N_x) \qquad (6)$$

but not greater than $K_2f_c'A_c$ not K_3A_c

b. When $(A_{vf}f_y + N_x)$ is less than or equal to $K_1A_c/1.45$ lb (or V_n is less than or equal to $1.55K_1A_c$ lb)

$$V_n = 2.25(A_{vf}f_y + N_x) \text{ lb} \qquad (7)$$

Table 3—Initially cracked normalweight concrete specimens subject to sustained load before static load test to failure

Researcher	Specimen	$\rho_{vf}f_y$, psi	σ_{Nx}, psi	f_c', psi	v_n, psi	$0.3f_c'$, psi	K_1, psi	v_n(calc), psi	Test/calc
Frenay[11]	2	747	0	6751	1617	2025	675	1273	1.27
	3	893	0	6724	2154	2017	672	1387	1.55
	4	747	0	6732	1961	2020	673	1271	1.54
	6	893	0	6751	1489	2025	675	1390	1.07
	7	1121	0	7051	2142	2115	705	1602	1.34
	9	1121	0	8044	2036	2413	800	1697	1.20
	10	1121	0	7567	1906	2270	757	1654	1.15
	11	1121	0	7567	1856	2270	757	1654	1.12
	12	1787	0	7556	2676	2267	756	2185	1.22
	13	747	0	9437	1698	2831	800	1398	1.21
	14	747	0	9443	1777	2833	800	1398	1.27
	15	893	0	9902	2384	2971	800	1514	1.57
	16	893	0	9882	1774	2965	800	1514	1.17
	17	747	0	9144	1753	2743	800	1398	1.25
	19	747	0	9144	1561	2743	800	1398	1.12
	20	1121	0	9882	2371	2965	800	1697	1.40
	21	1121	0	9882	2332	2965	800	1697	1.37
	22	1121	0	9225	2273	2768	800	1697	1.34
	23	1121	0	9225	2181	2768	800	1697	1.29
	24	1787	0	9818	3197	2945	800	2230	1.43

Note: Average test calculation = 1.1297; standard deviation = 0.148.

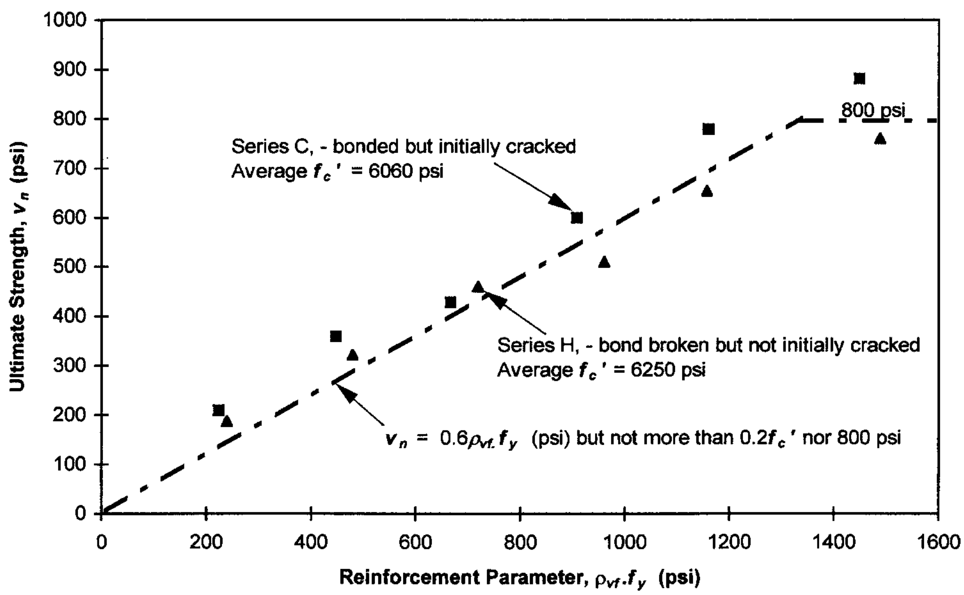

Fig. 7—Normalweight concrete, composite specimens, 2 (concrete placed against smooth surface).

where for normalweight monolithic concrete, $K_1 = 0.1f_c'$ but not more than 800 psi; K_2 = 0.3; and K_3 = 2400 psi. For the case of concrete placed against hardened concrete with its surface intentionally roughened, K_1 = 400 psi; K_2 = 0.3, where f_c' shall be taken as the lower of the compressive strengths of the two concretes; and K_3 = 2400 psi.

For sand-lightweight concrete, K_1 = 250 psi; K_2 = 0.2; and K_3 = 1200 psi. For all lightweight concrete, K_1 = 200 psi; K_2 = 0.2; and K_3 = 1200 psi. N_x = permanent normal force acting across the shear plane, positive if compression, and negative if tension.

2. For concrete placed against hardened concrete not intentionally roughened

$$V_n = 0.6\lambda A_{vf} f_y \quad (8)$$

but not more than $0.2A_c f_c'$ nor $800A_c$ lb

3. For concrete anchored to clean, unpainted, as-rolled structural steel by headed studs or by reinforcing bars

$$V_n = 0.7\lambda A_{vf} f_y \quad (9)$$

but not more than $0.2A_c f_c'$ nor $800A_c$ lb

where λ is as defined in ACI 318-99.

Table 4—Shear friction in initially cracked sand-lightweight concrete

Researcher	Specimen	$\rho_{vf} f_y$, psi	f_c', psi	v_n, psi	$0.2f_c'$, psi	v_n(calc), psi	Test/calc
Mattock, Li, and Wang[6]	B1	218	3740	450	748	424	1.06
	B2	448	3360	652	672	608	1.07
	B3	672	3910	840	782	782	1.07
	B4	864	4100	940	820	820	1.15
	B5	1111	3960	1000	792	792	1.26
	B6	1368	4250	1154	850	850	1.36
	C1	218	2330	364	466	424	0.86
	C2	472	2370	514	474	474	1.08
	C3	672	2000	525	400	400	1.31
	C4	921	2050	560	410	410	1.37
	D1	228	5995	370	1199	432	0.86
	D2	460	5995	668	1199	618	1.08
	D3	690	5710	772	1142	802	0.96
	D4	920	5710	1022	1142	986	1.04
	D5	1151	5600	1082	1120	1120	0.97
	D6	1368	5600	1220	1120	1120	1.09
	Average test/calc		1.099	Standard deviation			0.158
Hoff[12]	LWC1-1	281	8490	287	1698	475	0.60
	LWC1-2	281	8510	365	1702	475	0.77
	LWC1-3	281	8290	413	1658	475	0.87
	LWC1-4	648	8490	761	1698	768	0.99
	LWC1-5	657	8510	680	1702	776	0.88
	LWC1-6	648	8290	727	1658	768	0.95
	LWC2-1	280	9270	489	1854	474	1.03
	LWC2-2	280	8760	335	1752	474	0.71
	LWC2-3	280	8730	299	1746	474	0.63
	LWC2-4	648	9270	739	1854	768	0.96
	LWC2-5	652	8760	692	1752	772	0.90
	LWC2-6	652	8730	680	1746	772	0.88
	LWC3-1	378	10,310	668	2062	552	1.21
	LWC3-2	378	10,910	548	2182	552	0.99
	LWC3-3	378	11,020	585	2062	552	1.06
	LWC3-4	632	10,310	870	2182	756	1.15
	LWC3-5	640	10,910	870	2182	762	1.14
	LWC3-6	636	11,020	894	2204	759	1.18
	Average test/calc		0.939	Standard deviation			0.180
Overall average test/calc			1.014	Standard deviation			0.186

Table 5—Shear friction in initially cracked all lightweight concrete

Researcher	Specimen	$\rho_{vf} f_y$, psi	f_c', psi	v_n, psi	$0.2f_c'$, psi	v_n(calc), psi	Test/calc
Mattock, Li, and Wang[6]	F1	234	4150	450	830	387	1.16
	F2	460	4030	530	806	568	0.93
	F2A	448	3970	620	794	558	1.11
	F3	690	4065	734	813	852	0.98
	F3A	678	3970	702	794	742	0.95
	F4	896	4040	870	808	808	1.08
	F5	1140	4115	920	823	823	1.12
	F6	1404	4050	982	810	810	1.21
	H1	219	4145	400	829	375	1.07
	H2	456	3880	620	776	565	1.10
	H3	684	4100	866	820	747	1.16
	H4	912	4420	940	884	884	1.06
	H5	1111	3950	990	790	790	1.25
	H6	1315	4080	1042	816	816	1.28
	Average test/calc		1.104	Standard deviation			0.106

Table 6—Shear friction in initially cracked normalweight concrete composite specimens with roughened interface

Researcher	Specimen	$\rho_{vf} f_y$, psi	f'_{c1}, psi	f'_{c2}, psi	v_n, psi	$0.3f'_{c2}$, psi	v_n(calc), psi	Test/calc
Mattock[7]	B1	226	6330	5840	487	1752	509	0.96
	B2	445	6330	5840	700	1752	756	0.93
	B3	676	6055	6225	1054	1868	941	1.12
	B4	947	6055	6225	1276	1868	1158	1.10
	B5	1262	6040	5895	1570	1769	1410	1.11
	B6	1576	6040	5895	1700	1769	1661	1.02
	D1	225	6245	3770	590	1131	506	1.17
	D2	451	6245	3770	920	1131	761	1.21
	D3	739	5910	2940	1010	882	955	1.06
	D4A	950	6085	2495	994	749	955	1.04
	D4	985	5910	2940	1002	882	955	1.05
	Average f'_{c2} for Series D			3183	Average $0.3f'_{c2}$ for Series D			955
	Average test/calc			1.070	Standard deviation			0.084

Table 7—Shear friction in initially cracked normalweight concrete composite specimens with smooth interface

Researcher	Specimen	$\rho_{vf} f_y$, psi	f'_{c1}, psi	f'_{c2}, psi	v_n, psi	v_n(calc), psi	Test/calc
Mattock[7]	Specimens bonded but initially cracked along interface						
	C1	224	6190	5870	210	134	1.56
	C2	448	6190	5870	360	269	1.34
	C3	667	5980	5980	428	400	1.07
	C4	908	5980	5980	600	545	1.10
	C5	1160	6165	6185	780	696	1.12
	C6	1448	6165	6185	882	800	1.10
	Specimens had bond broken, but not initially cracked						
	H1	240	6330	5825	188	144	1.31
	H2	480	6170	6080	322	288	1.12
	H3	720	6170	6080	460	432	1.06
	H4	960	6720	6075	510	576	0.89
	H5	1157	6650	6180	654	694	0.94
	H6	1488	6535	5900	760	800	0.95
	Average test/calc			1.130	Standard deviation		0.190

CONVERSION FACTORS

1 ksi = 6.90 MPa (N/mm^2)
1 kip = 4.45 kN

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