Perfobond Shear Connector for Composite Construction

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INTRODUCTION

Composite construction produces stiffer, stronger sections by the efficient use of both steel and concrete. The desirable composite action, however, is not possible unless adequate shear transfer at the steel member-concrete interface is provided. During the last several decades, several types of shear connectors have been developed to insure this transfer of shear; the most common connector is currently the headed stud. This type of shear connector requires a certain amount of slip to occur before composite action can be established. Note that the slippage can cause cracking of the concrete slab under service loads, as well as, problems with fatigue due to the friction between the steel and concrete (Zellner, 1987). This problem can be pronounced in bridge decks, which are subjected to fatigue truck loading, where the cracks can propagate to the surface and accelerate deterioration.

A relatively new system, referred to as the Perfobond shear connector, has been developed to alleviate some of the problems associated with the headed studs (Leonhardt, Andra, Andra, Saul, and Harre, 1987; Zellner, 1987). The connector is a steel plate containing a number of holes (perforations). As shown in Figure 1, the connector is welded perpendicular to, and along the centerline of, the steel compression flange (similar to a shear tab), and encased in the concrete slab. Concrete dowels are formed through the holes of the plate, eliminating slippage and the resulting cracking under service loading. Construction difficulties associated with conventional studs, which are typically welded across the flange, are virtually eliminated. Note that Perfobond connectors can effectively be used in composite columns or braces; however, they are not easily applicable to beams with metal decks because of construction issues, e.g., it will be difficult to weld such connectors once the metal deck is placed over the steel beam, although a previous study (Veldanda and Hosain, 1992b) has examined performance of Perfobond connectors with metal decks.

The original series of tests aimed at understanding the behavior of the Perfobond connectors was conducted by Leonhardt, Andra and Partners in 1987 (Leonhardt et al., 1987). In the context of proof testing with reference to two railroad bridges, Zellner (1987) tested additional push-out specimens. These tests indicated similar strengths for the Perfobond connectors and headed studs but at a reduced cost. More recently, a comprehensive research program at the University of Saskatchewan (Veldanda and Hosain, 1992a and 1992b; Oguejiofor and Hosain, 1992 and 1994) has focused on developing design equations. As part of this program, push-out specimens as well as full-scale composite beams utilizing the Perfobond connectors were tested. The important aspects of these specimens are summarized in Table 1 and Table 2. Based on regression analysis of the test results from the push-out specimens, Oguejiofor and Hosain (1994) proposed Equation 1 for the shear capacity per each Perfobond rib connector.

$$P = 7.106\,A_{cc}\sqrt{f'_c} + 1233\,A_{tr}\,f_{yr} + 34.58\,nd^2\sqrt{f'_c} \quad (1)$$

where

P = shear capacity per Perfobond rib connector, pounds

A_{cc} = shear area of connector per connector (the shaded area shown in Figure 2),in.^{2}, not to exceed one half of the length times height of the Perfobond rib connector

f'_c = concrete compressive strength, psi

A_{tr} = area of transverse reinforcement per connector, in.^{2}

f_{yr} = yield strength of transverse reinforcement, psi

n = number of holes in the connector

d = diameter of hole, in.

Note that the constants in Equation 1 have been modified from the original values to account for conversion from the SI to English units. The general form of Equation 1 is similar to an equation proposed by Davies (1969) for calculat-

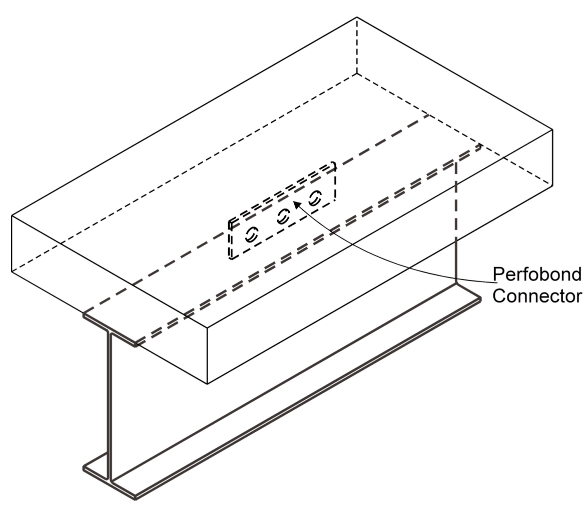

Fig. 1. Typical perfobond shear connector.

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Table 1. Summary of Test Push-out Specimens University of Saskatchewan

Specimen	f'_c (psi)	Connector Thickness (in.)	No. of Holes	Diameter of Holes (in)	Slab Reinforcement	Experimental *P* (kips)	Specimen No.
VS-1	3,829	0.500	4	1.969	2 reinforcing bars & Wire mesh	121	19
VS-3	3,829	0.500	4	1.969	2 reinforcing bars & Wire mesh	111	20
VS-4	3,829	0.500	4	1.969	2 reinforcing bars & Wire mesh	128	21
VSF-2	4,685	0.500	4	1.378	2 reinforcing bars & Wire mesh	98	22
VSF-4	4,685	0.500	3	1.378	2 reinforcing bars & Wire mesh	85	23
EB-1	3,033	0.500	0	----	----	40	24
EB-2	3,033	0.500	2	1.969	----	56	25
EB-3	3,033	0.500	3	1.969	----	62	26
EB-4	3,033	0.500	4	1.969	----	62	27
EB-5	3,033	0.500	0	1.969	3 reinforcing bars*	66	28
EB-6	3,033	0.500	2	1.969	3 reinforcing bars	84	29
EB-7	3,033	0.500	3	1.969	3 reinforcing bars	97	30
EB-8	3,033	0.500	4	1.969	3 reinforcing bars	90	31
EC-1	4,938	0.500	0	1.969	3 reinforcing bars	89	32
EC-2	6,009	0.500	2	1.969	3 reinforcing bars	127	33
EC-3	6,009	0.500	3	1.969	3 reinforcing bars	134	34
EC-4	6,009	0.500	4	1.969	3 reinforcing bars	134	35
EC-5	6,009	0.500	0	1.969	3 reinforcing bars	97	36
EC-6	6,009	0.500	2	1.969	3 reinforcing bars	119	37
EC-7	6,009	0.500	3	1.969	3 reinforcing bars	131	38
EC-8	6,009	0.500	4	1.969	3 reinforcing bars	130	39
ED-1	3,600	0.500	0	1.969	Wire Mesh**	54	40
ED-2	3,600	0.500	2	1.969	Wire Mesh	69	41
ED-3	3,600	0.500	3	1.969	Wire Mesh	77	42
ED-4	3,600	0.500	4	1.969	Wire Mesh	82	43
ED-5	3,600	0.500	0	1.969	3 reinforcing bars & Wire mesh	102	44
ED-6	3,600	0.500	2	1.969	3 reinforcing bars & Wire mesh	120	45
ED-7	3,600	0.500	3	1.969	3 reinforcing bars & Wire mesh	131	46
ED-8	3,600	0.500	4	1.969	3 reinforcing bars & Wire mesh	130	47
EPS-1A	3,855	0.500	3	1.969	3 reinforcing bars & Wire mesh	120	48
EPS-1B	3,855	0.500	3	1.969	3 reinforcing bars & Wire mesh	123	49
EPS-2A	4,030	0.500	3	1.969	Wire Mesh	77	50
EPS-2B	4,030	0.500	3	1.969	Wire Mesh	77	51
EPS-3AR	3,766	0.500	3	1.969	3 reinforcing bars & Wire mesh	113	52
EPS-3BR	3,766	0.500	3	1.969	3 reinforcing bars & Wire mesh	106	53
EPS-4AR	3,894	0.500	3	1.969	3 reinforcing bars & Wire mesh	120	54
EPS-4BR	3,894	0.500	3	1.969	3 reinforcing bars & Wire mesh	111	55
EPS-5A	3,991	0.500	2	1.969	2 reinforcing bars & Wire mesh	88	56
EPS-5B	3,991	0.500	2	1.969	2 reinforcing bars & Wire mesh	96	57
EPS-5C	3,991	0.500	2	1.969	3 reinforcing bars & Wire mesh	104	58
EPS-5D	3,991	0.500	2	1.969	3 reinforcing bars & Wire mesh	109	59
EPS-6A	3,814	0.500	2	1.969	3 reinforcing bars & Wire mesh	109	60
EPS-6C	3,814	0.500	3	1.969	3 reinforcing bars & Wire mesh	124	61

* Diameter of reinforcing bars = 0.444 in.

** Wire mesh: Plain welded wire with area = 0.029 in.^{2} in each direction on a 5.98 in. spacing

Table 2. Summary of Beam Specimens University of Saskatchewan

		Perfobond Connector			Transverse Steel	
Specimen	f'_c (psi)	No. of Holes	Length (in.)	No. of Connectors	No. of Transverse Bars	Wire Mesh
EBS-1	3988	3	14.8	6 @ 30.6" o.c.	10 (f = 0.44 in.)	0.029 in.^{2} E.W. @ 5.98" o.c.
EBS-2	4032	3	14.8	6 @ 30.6" o.c.	10 (f = 0.44 in.)	0.029 in.^{2} E.W. @ 5.98" o.c.
EBS-3	3771	3	14.8	6 @ 30.6" o.c.	10 (f = 0.44 in.)	0.029 in.^{2} E.W. @ 5.98" o.c.
EBS-4	3901	3	14.8	4 @ 45.9" o.c.	10 (f = 0.44 in.)	0.029 in.^{2} E.W. @ 5.98" o.c.
EBS-5	3988	2	9.9	6 @ 30.6" o.c.	10 (f = 0.44 in.)	0.029 in.^{2} E.W. @ 5.98" o.c.

Diameter of holes in perfobond connector = 1.97 in.

ing the capacity of the headed stud connectors. The expression proposed by Davies was essentially the sum of the contribution of the concrete slab and transverse reinforcement. The first two terms in Equation 1 also represent these contributions. The last term accounts for the contribution of the concrete dowels. This empirical design equation is limited to cases with (a) ½-inch thick Perfobond rib connectors, (b) 2-inch diameter holes, (c) center-to-center of the rib holes spaced at least two times the hole diameter, and (d) concrete compressive strength between 2.9 ksi and about 5.8 ksi.

The experimental and analytical study reported herein was undertaken to (a) further examine the fundamental behavior of the Perfobond shear connectors, (b) expand the database of test data for the Perfobond connectors, (c) better understand and examine the potential significant sources of shear resistance in the Perfobond connectors, (d) evaluate Equation 1 for tests other than those used in its development, and (e) develop a more improved design equation if necessary. The focus of this study is on typical steel stringer bridges that include steel girders and reinforced concrete slabs.

RESEARCH OBJECTIVE AND OVERVIEW

To address the stated objectives, a coordinated experimental and analytical study was conducted. A number of specimens, designed to incorporate different experimental variables, were constructed and tested. Using the experimental results, the success of Equation 1 was evaluated. Based on the observed specimen behavior and the results of a finite element analysis, the failure mechanism was established, and a reasonably detailed mathematical model for computing the shear capacity of the Perfobond shear connector was developed. The success of the new design equation was gauged against the test data as well as those obtained by others.

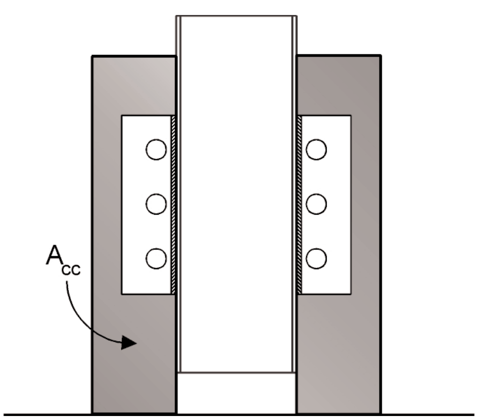

Fig. 2. Definition of A_{cc}.

SPECIMEN SELECTION

Twenty-eight push-out specimens were selected, fabricated, and tested. The specimens were designed such that the shear connection was the only variable. Each specimen was constructed of two 8½-in. thick slabs 2 ft.-wide × 3 ft.-long with a 3 ft.-long W12×19 steel section. The general layout of the test specimens is illustrated in Figure 3.

The experimental program was developed such that with each successive series, one test variable was introduced; hence, its contribution towards the total shear strength could be examined. Series A specimens, designed to obtain the chemical bond strength at the steel flange-concrete interface, utilized no connectors and the slab was not reinforced. The flanges of the steel section were greased in series B specimens, and a solid plate connector was used. Since the mode of failure was expected to occur in the concrete, this series was intended to determine the shear resistance of the slab. Series C also contained solid connectors, but the beam flanges were not greased. This series was intended to examine the combined contributions of the concrete slab and chemical bond. Series D utilized connectors with holes to evaluate the resistance of the concrete dowels formed within the holes of the connector. In series E, No. 4 steel reinforcing bars were added to increase the strength and ductility of the concrete. Note that the bars did not pass through the connector. The slabs contained both top and bottom reinforcement, similar to typical bridge decks. In series B through E, both ½-in. and ¾-in. thick plates were used to determine the effect of connector thickness. A com-

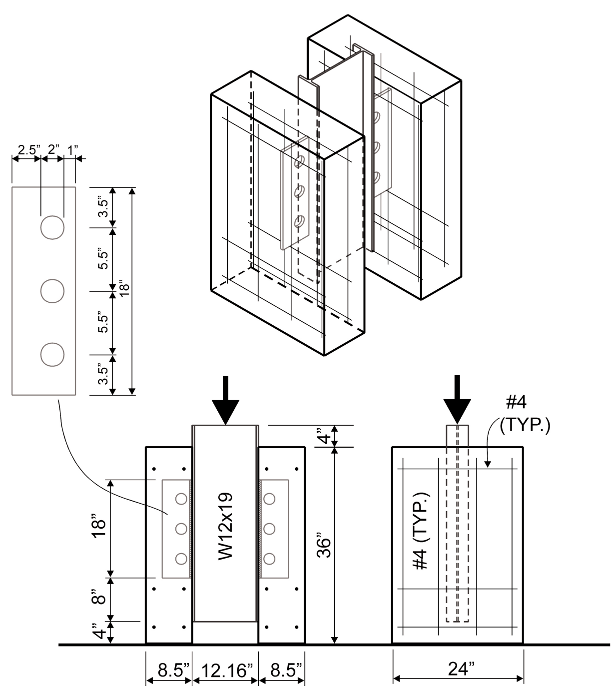

Fig. 3. Push-out specimens.

panion set of specimens using headed shear studs, Series F, was also tested for comparison of ductility and mode of failure to specimens with Perfobond connectors. Four $\frac{1}{2}$-in. × $5\frac{5}{16}$ -in. Nelson shear studs were used in Series F, and 6 No. 4 transverse reinforcing bars were also included in each slab.

A total of 10 specimens, one specimen from each series with each connector thickness, were fabricated at a time. Simultaneous casting of each specimen type insured that the concrete strength remained constant between successive series. Three concrete pours produced 30 total specimens. The concrete matrix consisted of $\frac{3}{8}$ in.-aggregate. The average concrete compressive strength for pours 1, 2, and 3 was 6,600 psi, 6,276 psi, and 5,743 psi, respectively.

Previous studies (Veldanda and Hosain, 1992a, 1992b; Oguejiofor and Hosain, 1992 and 1994) point to the importance of connector height. If the plate is not tall enough, failure will occur above the connector, thus making it ineffective. Maintaining a 3-inch concrete cover over the connector, within the $8\frac{1}{2}$-inch thick slab, the connector height was selected to be $5\frac{1}{2}$ inches. In this investigation, the number and diameter of the holes were kept constant. Each connector was assumed to have three holes. Using ASTM C31-88 standards (ASTM, 1989) for sizing concrete cylinders, the diameter of each hole must be greater than three times the size of the maximum aggregate. To meet this requirement, each hole was set to 2 inches in diameter. The connector length (18 inches) was established by meeting LRFD (AISC, 1994) requirements for minimum edge distance and minimum spacing of holes. Vertical placement of the holes was determined by the LRFD minimum edge distance required to the top of the plate. The final connector design, with its resulting dimensions, is shown in Figure 3. The connector was welded along its entire length, both sides, to the beam flange with E70XX fillet welds. The weld size ($\frac{1}{4}$ in. for $\frac{1}{2}$ in. thick connectors and $\frac{5}{16}$ in. for $\frac{3}{4}$ in. thick connectors) was selected based upon the expected capacity of the connector. Series B and C specimens utilized solid plates equal in size to the Perfobond connector designed for Series D and E specimens.

EXPERIMENTAL PROGRAM

The specimens were tested in a self-equilibrating frame, as detailed in Figure 4. A steel plate was placed atop the steel section of the specimen to evenly distribute the load to the entire cross section. A hydraulic jack, placed above the plate, was used to load the specimen. A swivel spherical bearing plate was positioned between the jack and the crossbeam of the testing frame, to eliminate eccentric loading of the specimen and to maintain a concentric axial load on the steel section. The load was applied in ten-kip increments until failure.

Each specimen was instrumented with two displacement transducers (DCDTs), one located at each flange, as shown in Figure 4. The DCDTs provided the displacement of the steel relative to each of the concrete slabs, thus providing a measure of slip at the steel-slab interface. The applied load was measured by a pressure transducer, which had been calibrated against a load cell. The load and displacements were recorded by a data acquisition system at ten-second intervals.

EXPERIMENTAL RESULTS

The concrete slab separated from the steel flange in two specimens of Series A during the removal of the forms; hence, only one such specimen could be tested. Cracking was heard instantly upon loading the specimen; slip occurred immediately and increased rapidly until detachment of the slab from the steel flange. Each specimen utilizing a solid plate or Perfobond connector experienced very little slip up to approximately 50% of its ultimate capacity. At higher loads, cracking was heard and the slip increased at a greater rate. This trend is apparent in Figure 5 in which a representative load-slip curve from Series B, C, and D is plotted. The mode of failure of the specimens in these series, which did not contain any transverse steel reinforcement within the slabs, was very brittle. Failure

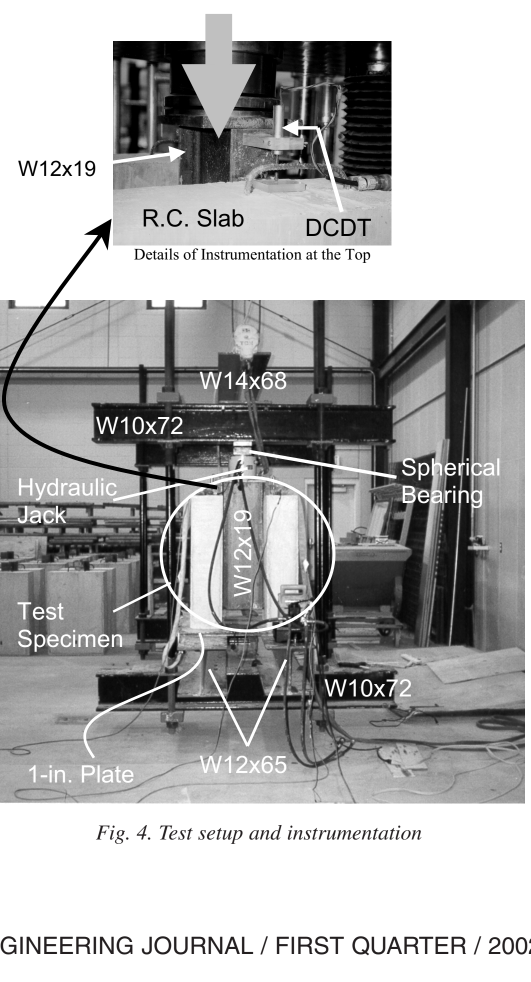

Fig. 4. Test setup and instrumentation

occurred rather abruptly due to splitting of the concrete slab above and parallel to the connector, as seen in Figure 6. Failure occurred typically in one slab. Although both experienced cracking, one slab simply had the tendency to fail first. (Simultaneous splitting of both slabs did occur in one specimen.) Analyses conducted as part of this study (Medberry and Shahrooz, 1998) and by others (Johnson and Oehlers, 1981) reveal that tensile stresses develop in the concrete at the loaded edge of the connector. These stresses tend to pry the concrete slab apart, resulting in the cracking and eventual splitting of the concrete above and along the length of the connector. The presence of such stresses illustrates the need for transverse reinforcement similar to that provided in Series E. The crack pattern in Series E, apparent in both slabs of the specimen, was similar to the splitting pattern in the specimens without reinforcement, as seen in Figure 7. The ductility observed in the load-slip curve for this series (see Figure 5) is attributed to the presence of the steel reinforcement within the slab. The reinforcement prevented splitting of the concrete, thereby allowing the specimen to attain a larger amount of slip prior to failure. Aggregate interlock between the cracked concrete surfaces provided additional load-carrying capacity. Loading of Series E specimens was stopped when additional load could not be sustained. In contrast to the splitting failure of series B, C, and D, these specimens did not truly fail. Three specimens of series F were tested until they began to unload. Greater levels of slip and ductility characterized the specimen behavior, as illustrated in the representative load-slip curve in Figure 5.

Figure 5 illustrates the difference between the brittle concrete failure in specimens utilizing plate and Perfobond connectors with unreinforced concrete slabs, the ductile steel failure inherent in specimens with headed studs, and

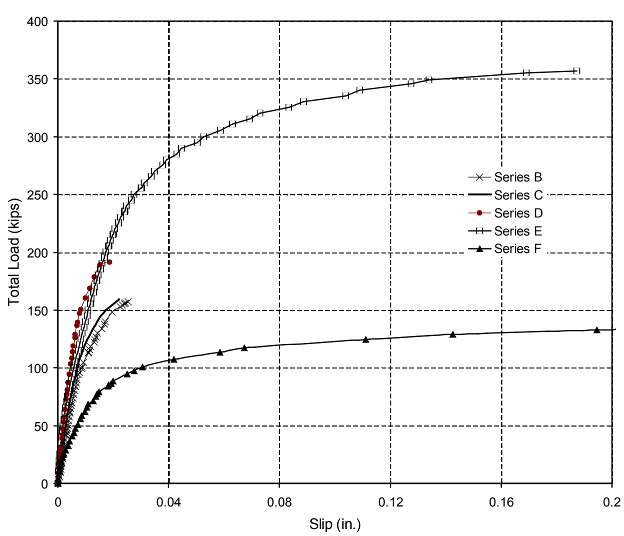

Fig. 5. Typical load-slip curves.

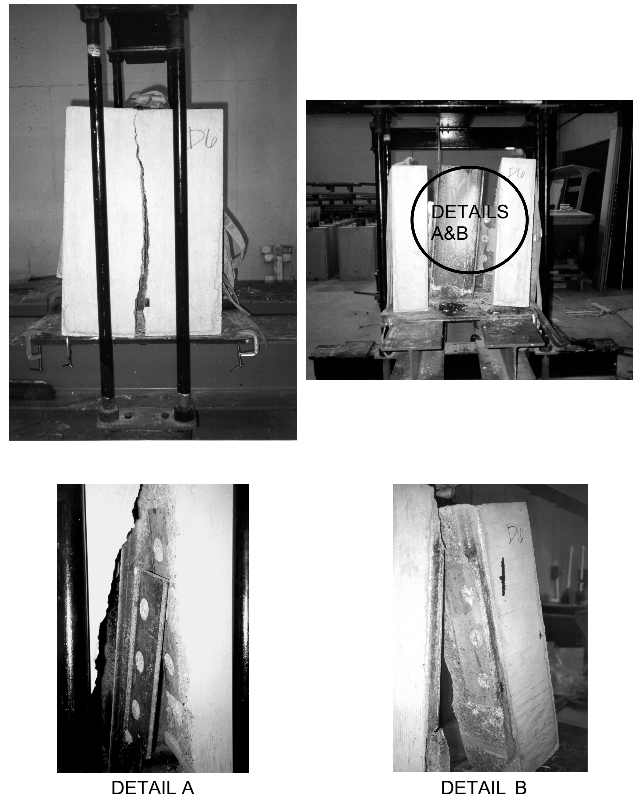

Fig. 6. Typical splitting failure (specimens without slab reinforcement).

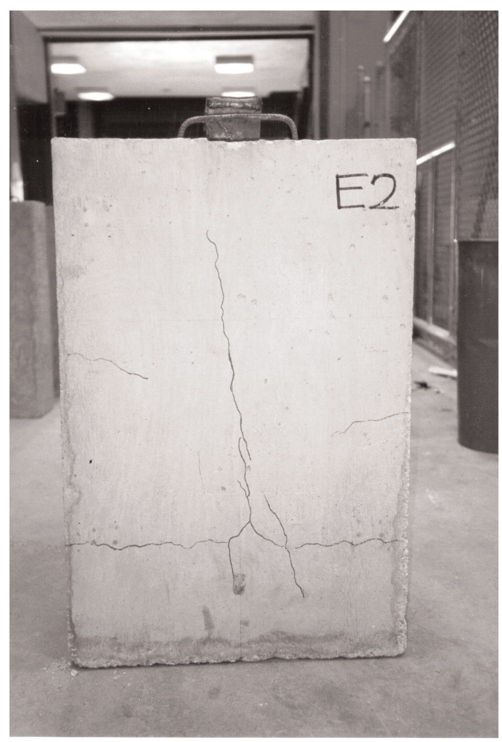

Fig. 7. Cracking pattern (specimens with slab reinforcement).

Table 3. Summary of Push-out Specimens
University of Cincinnati

Series	Specimen	Greased Flanges?	f'_c (psi)	Connector Thickness (in.)	No. of Holes	Diameter of Hole (in.)	No. of Bars in each Slab	Experimental P (kips)	Specimen No.
B	B1 (r)	Yes	6,600	0.5	0	----	0	114	----
	B2	Yes	6,276	0.5	0	----	0	157	1
	B3	Yes	5,743	0.5	0	----	0	132	2
	B5	Yes	6,600	0.75	0	----	0	153	3
	B6	Yes	6,276	0.75	0	----	0	158	4
	B7	Yes	5,743	0.75	0	----	0	137	5
C	C1 (r)	No	6,600	0.5	0	----	0	124	----
	C2	No	6,276	0.5	0	----	0	160	6
	C3	No	5,743	0.5	0	----	0	159	7
	C5 (r)	No	6,600	0.75	0	----	0	148	----
	C6	No	6,276	0.75	0	----	0	168	8
	C7	No	5,743	0.75	0	----	0	141	9
D	D1 (r)	No	6,600	0.5	3	2	0	154	----
	D2	No	6,276	0.5	3	2	0	181	10
	D3	No	5,743	0.5	3	2	0	190	11
	D5 (r)	No	6,600	0.75	3	2	0	158	----
	D6	No	6,276	0.75	3	2	0	192	12
	D7	No	5,743	0.75	3	2	0	201	13
E	E1	No	6,600	0.5	3	2	6	295	14
	E2	No	6,276	0.5	3	2	6	297	15
	E3	No	5,743	0.5	3	2	6	314	16
	E5	No	6,600	0.75	3	2	6	357	17
	E6	No	6,276	0.75	3	2	6	354	18
	E7 (*)	No	5,743	0.75	3	2	6	266	----

(r) = rocked

(*) = lower than expected

the increased strength and ductility achieved in specimens using Perfobond connectors with reinforced concrete slabs. Because current composite construction practices utilize reinforced concrete slabs, the brittle failure of the specimens in Series B, C and D should not be of concern. The reinforced concrete slabs with Perfobond shear connectors in Series E, proposed for the actual design of composite construction, achieve the desired ductility with significantly higher capacities than those specimens with headed studs in Series F.

As expected, an increase in shear strength was obtained with each successive series of specimens. The ultimate load achieved by each specimen in Series B, C, D, and E is provided in Table 3. A number of the measured data were considered erroneous when the specimen failed at a lower capacity than expected, either for no apparent reason or due to uneven distribution of the load between the two slabs and shear connectors which led to rotation of the steel assembly. The rocking mechanism was evident in the measured load-slip curves, i.e., the slip between the concrete slab and steel flange steadily increased at one connector, while it reversed direction at the other connector, leading to "rocking" of the steel beam. This rocking lowered the ultimate shear capacity of the specimen. The five specimens that experienced rocking (B1, C1, D1, C5, and D5) are noted by (r) in Table 3. Specimen E7 was also discarded, as the measured capacity was lower than expected, i.e., its capacity was less than that for E3 which had a thinner connector and also lower than the remaining specimens in Series E.

EVALUATION OF EXISTING EQUATION

The capacities of the specimens tested at the University of Cincinnati and University of Saskatchewan were computed from Equation 1. The ratio of the measured capacity to computed capacity is plotted in Figure 8, and the corresponding values of the average, standard deviation, and coefficient of variation are summarized in Table 4. The specimen numbers shown in Figure 8 are tabulated in Tables 1 and 3 (far right hand column). The level of correlation is, expectedly, good for the specimens tested at the University of Saskatchewan; however, for more than half of the specimens the capacity is overestimated as evident by the results shown in Figure 8. Moreover, Equation 1 is not nearly as successful in replicating the capacities of the specimens tested at the University of Cincinnati. The capacities

Table 4. Ratio of Measured Capacity to Computed Capacity

	Univ. of Cincinnati	Univ. of Saskatchewan	Total Data
(a) Eq. 1			
Average	0.716	0.990	0.909
Std. Dev.	0.304	0.096	0.220
C.O.V.	0.425	0.097	0.242
(b) Eq. 2			
Average	1.017	1.088	1.067
Std. Dev.	0.066	0.106	0.101
C.O.V.	0.064	0.098	0.095

of the specimens without transverse reinforcement are overestimated by as much as 22 percent, and for cases with transverse reinforcement the computed capacity is about 4 times larger than the measured capacities. These major differences are attributed to several reasons, including (a) the thickness of some of the connectors exceeds 0.5 in., (b) the concrete compressive strength exceeds 5.8 ksi, (c) different levels of transverse reinforcement—the maximum area of steel in the University of Saskatchewan's specimens was about 0.2 in.^{2} whereas 1.2 in.^{2} of transverse steel was used in the University of Cincinnati's specimens, and (d) the slab thickness for the University of Cincinnati's specimens was about 1.5 times thicker than that used for the University of Saskatchewan's specimens. In view of the results predicted by Equation 1, a new design equation is needed.

FORMULATION OF DESIGN EQUATIONS

The primary mechanisms contributing towards the capacity are (a) the concrete slab, (b) the chemical bond between the concrete and steel beam flanges, (c) the concrete dowels,

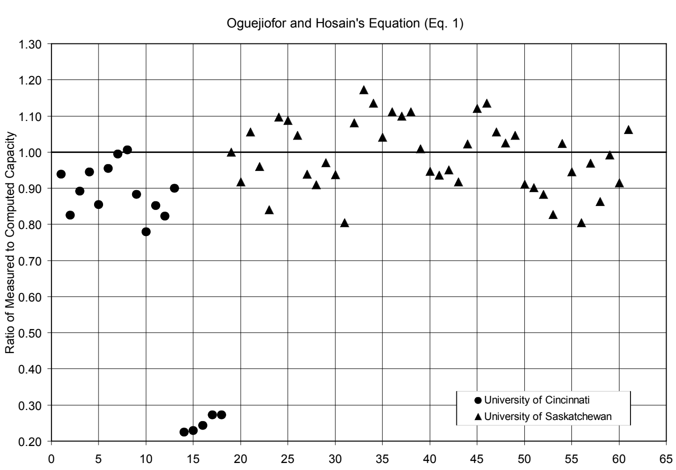

Fig. 8. Evaluation of available design equation.

and (d) the transverse reinforcement. These contributions were calculated as the difference in specimen capacities between the successive series. The capacity provided by the concrete slab, established in Series B, was the only test variable that remained constant in each series. Note that the slab capacity between sets of specimens varied due to the difference in concrete compressive strength. The contribution of the chemical bond was computed by subtracting Series B from Series C. The shear resistance of the concrete dowels was calculated by subtracting Series C from Series D, while subtracting Series D from Series E gave the contribution of the steel reinforcement within the slab. The experimental data suggest that the resistance provided by the concrete slab and the additional strength provided by the slab reinforcement are the primary resistance mechanisms. The concrete dowels provided additional strength, however, not quite as significant as proposed in earlier investigations (Oguejiofor and Hosain, 1994). The chemical bond, as expected, provided the least amount of shear resistance.

The total shear capacity was assumed to be the sum of the capacities for each of the contributing mechanisms. A simple superposition tends to violate displacement compatibility between various resisting mechanisms. However, for simplicity and consistency with previous equations developed by others (Oguejiofor and Hosain, 1994 for the Perfobond connectors, and Davies, 1969 for headed studs), the capacity is computed as the superposition of each of the resisting mechanisms. The validity of this assumption will be evaluated when the computed and measured capacities are compared.

(a) Concrete Slab

The contribution of the slab was taken as the tensile splitting force. Simple linear, elastic finite element analyses reported elsewhere (Medberry and Shahrooz, 1998) indicate that the tensile stresses below the connector are distributed as those shown schematically in Figure 9. Note that linear, elastic analysis cannot clearly model the behavior after cracking; however, these analyses are deemed acceptable to capture the distribution of stresses up to cracking which immediately preceded failure in specimens without transverse reinforcement. The total tensile splitting force is equal to the volume under the stress distribution. For design purposes, a simpler idealized stress distribution is desirable. The simplified method must be calibrated such that it yields an equal value of total tensile force.

In this study, the stress distribution shown in Figure 9 was idealized as two rectangular blocks, as illustrated in Figure 10. Six parameters are required to identify the equivalent stress block:

- h, the total height of the slab below the connector,
- b, the depth of the slab,

- σ_t, the maximum tensile stress of the stress block,
- α and β, factors defining the height of each portion of the stress block as a function of h, and
- γ, the factor relating the stress in the lower portion of the stress block to the maximum tensile stress of the upper portion of the stress block.

The total volume of the stress block (i.e., the total tensile force) equals $\alpha h(\sigma_t)(b) + \beta h(\gamma\sigma_t)(b)$. The finite element analysis provided the volume of the stress block to which this equation was set to equal. The values of h and b were known parameters from the specimen design. The value of $\gamma\sigma_t$ was selected based upon the computed stress profiles, as the average stress of all the elements in the bottom row below the connector. After a number of permutations, the values of α and β were obtained, and the contribution of each slab was found to be $1.8(b)(h)(\sigma_t)$. By correlating this equation with the experimental results, the value of σ_t was determined to be on the average approximately 70 percent of the tensile strength of the concrete, taken as $7.2(f'_c)^{1/2}$. Therefore, the contribution of each slab, in pounds, equals $9(b)(h)(f'_c)^{1/2}$, in which b = the depth of the slab, h = the height of the slab below the connector, and f'_c = the concrete compressive strength in psi.

(b) Bond

The chemical bond of the concrete to the steel flange does contribute a small, but important, portion towards the overall specimen capacity. Designs are often conservative because this contribution is ignored. A limited number of previous studies have been conducted to examine the transfer of bond stresses between concrete and steel structural shapes (Brown, 1966; Roeder, 1984). Based upon experimental data derived from push-out specimens, Roeder developed an equation calculating the bond strength between steel wide flange sections and concrete. However, the steel member in Roeder's research was completely encased in concrete with transverse reinforcement. The transverse steel confines the concrete and thus is expected to increase the bond strength. Due to the differences between the design details of the specimens reported here and those used in Roeder's study, the equations developed by Roeder (1984) are not applicable for this investigation.

The product of an average bond stress and the contact area was used to compute the contribution of the bond of the concrete to the steel flange. Based on the experimental results, the average bond stress was found to be 60 psi. Therefore, the contribution of the bond at each flange equals $(60)(b_f)(L_c)$, where b_f = the width of the flanges and L_c = the length of contact between the concrete and the steel at one flange.

(c) Concrete Dowels

The concrete dowels formed within the holes of the Perfobond connector are analogous to bolts in double shear. The shear strength of a steel bolt is the product of its nominal shear strength and cross-sectional area. This methodology was applied to determine the capacity of the concrete dowels. An average shear stress, equal to $20(f'_c)^{1/2}$, was established from the experimental capacities. The contribution of the concrete dowels for each connector, therefore, is equal to $[20(f'_c)^{1/2}](n)(\pi)(d/2)^2$, where f'_c = the concrete compressive strength in psi, n = the number of holes in the

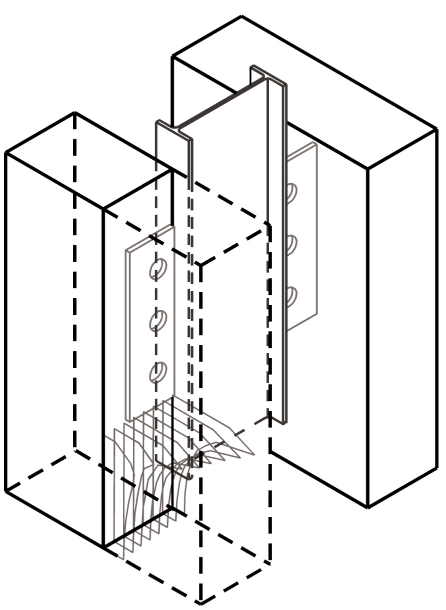

Fig. 9. General distribution of tensile stresses below the connectors.

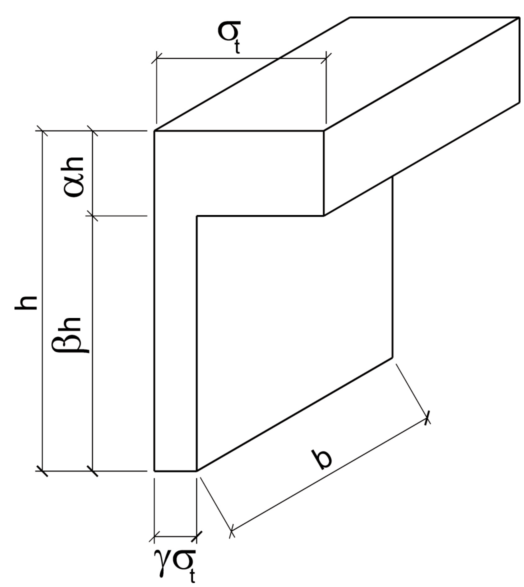

Fig. 10. Idealized stress block.

connector, and d = the diameter of the holes.

(d) Steel Reinforcement

Slippage between the adjacent cracked surfaces of the slab mobilizes aggregate interlock. The shear-friction method (ACI, 1997) was used to account for the contribution of the transverse slab reinforcement towards the capacity of the specimen, i.e., $V_n = \mu(A_{vf})(f_s)$ where μ = the coefficient of friction taken as 1.0, A_{vf} = the cross-sectional area of the steel reinforcement, and f_s = the stress of the steel. In order to match the measured contribution of the slab reinforcement, the value of f_s was taken as $0.9f_y$ where f_y = the yield strength of the transverse reinforcement. Hence, the contribution of the steel reinforcement towards the capacity of each slab equals $0.9(A_{vf})(f_y)$.

EVALUATION OF DESIGN EQUATIONS

Adding the contributions of the slab, chemical bond, concrete dowels, and steel transverse reinforcement, Equation 2 is proposed to compute the shear capacity of each slab in a push-out specimen utilizing Perfobond shear connectors.

$$P = 9bh\sqrt{f'_c} + 60b_f L_c + 20\,n\pi\sqrt{f'_c}\left(\frac{d}{2}\right)^2 + 0.9\,A_{vf}\,f_y \quad (2)$$

in which

P = the load capacity per slab, lbs.
b = the depth of the slab, in.
h = the height of the slab below the connector, in.
f'_c = the concrete compressive strength, psi
b_f = the width of the steel flanges, in.
L_c = the length of contact between the concrete and the steel at one flange, in.
n = the number of holes in the connector
d = the diameter of the holes in the connector, in.
A_{vf} = the total cross-sectional area of the transverse steel per slab, in.^{2}
f_y = the yield strength of the steel, psi

This design equation, multiplied by two, calculates the total shear capacity of a push-out specimen.

The ratio of the measured capacity to the capacity computed from Equation 2 for all the specimens in the database is plotted in Figure 11. The average, standard deviation, and coefficient of variation are summarized in Table 4. As noted previously, some of the University of Cincinnati's specimens (B1, C1, C5, D1, D5, and E7) were discarded because of concerns about the validity of the measured capacities. By comparing Figures 8 and 11, it is evident that the capacities computed from the proposed equation (Equation 2) tend to match the experimental values significantly better than those obtained from Equation 1. The average value of the measured to computed capacity conservatively gets closer to 1 (1.07 versus 0.909), the standard deviation becomes smaller (0.1 versus 0.2), and the capacities are overestimated by at most 9% versus 77% from Equation 1. The proposed equation, however, tends to underestimate the capacities of the specimens tested at the University of Saskatchewan slightly more than Equation 1, i.e., 17% versus 28% from Equation 2. Considering the collective trend of the computed capacities, Equation 2 clearly provides a reasonable and conservative estimate of the measured capacities, and the problems associated with over prediction of the capacities by Equation 1 are also rectified.

APPLICATION OF DESIGN EQUATION TO COMPOSITE BEAMS

The proposed design equation (Equation 2) was calibrated and gauged solely based upon available data from push-out specimens. An attempt was made to adapt the equation for composite beams utilizing Perfobond shear connectors. In order to use the proposed design equation (Equation 2), certain variables were changed: h was assumed to equal one-half the distance between connectors, in inches; L_c was set to equal the length of contact between the concrete and the steel at the compression flange per connector (i.e., the distance between connectors plus the connector length), in inches; and A_{vf} was taken as the total cross-sectional area of the slab transverse steel per connector (i.e., the total cross-sectional area divided by the number of connectors), in square inches. Using this procedure, the equation provides the capacity of one connector. The adapted equation was applied to the composite beams previously tested at the University of Saskatchewan (Oguejiofor and Hosain, 1994). The important features of the specimens are summarized in Table 2.

The shear capacity of the Perfobond connectors was computed from Equation 2. Following standard AISC LRFD (AISC, 1994) provisions for composite beams, the

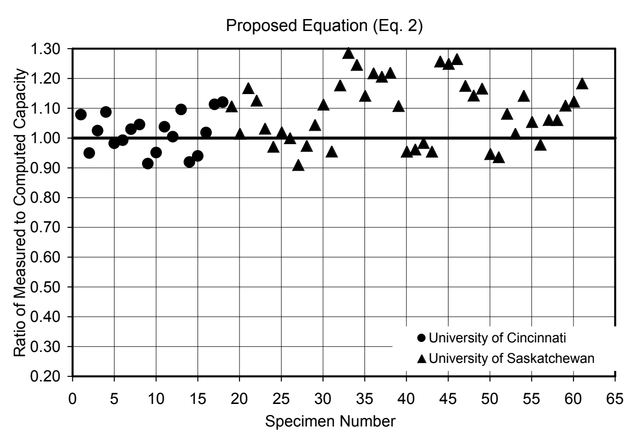

Fig. 11. Evaluation of proposed design equation.

capacity of each beam was calculated. The results summarized in Figure 12 indicate that the computed moment capacities are reasonably close to the measured values. The measured moment, on the average, is 1.02 times its experimental counterpart with a coefficient of variation of 0.063. The lowest ratio is 0.95 (for specimen 2), which implies that the measured capacity was overestimated by 5%. Therefore, the proposed equation provides a reasonable and conservative estimate of the capacity of Perfobond connectors for composite beams.

SUMMARY AND CONCLUSIONS

A new type of shear connector for composite construction, called the Perfobond shear connector has recently been developed to eliminate some of the problems and difficulties associated with standard shear studs. A coordinated experimental and analytical study was undertaken to further examine the behavior of composite members utilizing the Perfobond shear connectors, and to develop a reasonably simple, yet reliable design equation.

Thirty push-out specimens were fabricated and tested at the University of Cincinnati to study the contribution of (a) the chemical bond between the steel flange and the concrete, (b) the concrete slab, (c) the concrete "dowels" formed within the holes of the Perfobond connector, and (d) the steel transverse reinforcement within the slab. The experimental data from these tests as well as those from previous studies were utilized to propose a design equation (Equation 2). This equation was found to provide a reasonable, conservative estimate of the shear capacity of Perfobond connectors, and composite beams utilizing such connectors. The small contributions of chemical bond and concrete dowels, which were included in the development of Equation 2, may be omitted for design.

The experimental data and observations indicate that Perfobond connectors are a viable alternative to shear studs.

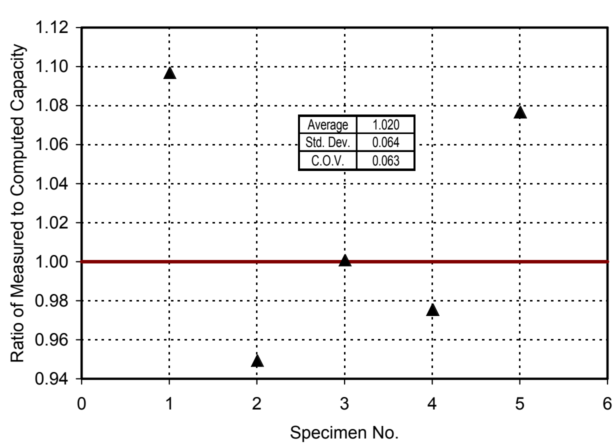

Fig. 12. Evaluation of the proposed equation for composite beams.

For composite beams utilizing reinforced concrete slabs, Perfobond shear connectors exhibit adequate ductility and substantially higher capacities.

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