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Shear-bond Capacity of Composite Slabs

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by

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SUMMARY

This paper presents an ultimate shear-bond equation for composite slabs failing in shear-bond. The equation is based on recent experimental evidence of composite slabs exhibiting early end slip prior to ultimate load, and contains the steel deck thickness as a parameter which other existing equations do not. The presence of the steel deck thickness parameter can result in a reduction of up to 75% of the presently required number of laboratory performance tests. A total of 196 test results were used to substantiate as well as to compare the results of the equation developed with other existing equations. The results also showed other advantages of the equation developed whose results were always within ∓15% of the corresponding experimental results.

INTRODUCTION

The term "composite slab" in building construction refers to a slab system composed of concrete permanently placed over cold formed steel decking (see Fig. 1). The use of composite slabs results in a valuable reduction of both time and cost of construction. For additional detailed information about the advantages of composite slabs, the reader is referred to Reference (10), which also contains a noteworthy general survey and a state-of-art review of the subject.

In the past 15 years, several equations have been presented to compute the ultimate shear-bond capacity of composite slabs. All equations, known to the authors to date, contain unknown coefficients which have to be evaluated from laboratory performance tests. The number of experimental tests depends in part on the desired level of accuracy of the computed ultimate shear-bond values. This paper presents the results of a recent study, in which a different shear-bond equation was developed. The objective of the study was to develop a shear-bond equation which requires the least possible number of experiments while maintaining the presently accepted level of accuracy of ∓15%. The following part presents a review of three published shear-bond equations which were selected for comparison with the equation derived later in this paper.

REVIEW OF SHEAR-BOND EQUATIONS

As a result of different research carried out on composite slabs, several shear-bond equations have been developed. The first three equations given in Table 1 were selected for presentation and discussion herein. V_u is the

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TABLE 1. - Shear-Bond Equations

Equation No.	Shear-bond Equation	Reference
(1)	$\frac{V_u s}{bd} = m \frac{d\sqrt{f'_c}}{L'} + k\rho$	(6,7)
(2)	$\frac{V_u s}{bd} = m \frac{\rho d}{L'} + k\sqrt{f'_c}$	(4)
(3)*	$\frac{V_u}{bd} = m \frac{1}{L'} + k$	(2,12)
(4)	$\frac{V_u}{bd} = k_1 \frac{t}{L'} + k_2 \frac{1}{L'} + k_3 t + k_4$	(13)

*Coefficients m & k appearing in this equation replace the coefficients F_n and f_f, respectively, which appear in the indicated references.

ultimate transverse shear-bond capacity per unit width of slab, b, and d is defined as the distance from the centroidal axis of the steel deck to the top surface of the concrete compression zone. Also, the term s represents the center-to-center distance between shear devices. For steel deck systems with a fixed pattern of embossments, the term s is set equal to unity. The term ρ is the percent of steel, defined as A_s/bd where A_s is the cross sectional area of the steel deck in slab width b, and f'_c is the concrete compressive strength. The shear span, L', appearing in the denominator of the three equations is defined as the distance between the applied load and the nearest support (see Fig. 2). In the three equations m and k are unknown coefficients which must be determined experimentally for each steel deck thickness of each manufacturer's product type.

Eq. (1) of Table 1 was developed by Schuster (6,7) in 1970, based on the hypothesis that failure is initiated by diagonal tension cracking, since early experiments showed no end-slip prior to ultimate load. Eq. (2) has the same form as the ACI equation for computing the ultimate shear capacity of reinforced concrete members without web reinforcement (1). Eq. (2) was obtained by replacing the two constants (1.9 and 2500) of the ACI shear equation with the two coefficients m and k, respectively. Eqs. (1) and (2) contain the same parameters but in a different arrangement. Both equations were investigated by Porter, et al, in 1976 (5), where the experimental values of the ultimate shear-bond capacities of tested composite slabs were compared with the corresponding computed values using each equation. The investigation showed no appreciable difference between Eqs. (1) and (2).

Eq. (3) was recently developed by Ling (2,12), based on the results of recent experimental evidence showing early end-slip prior to ultimate load. The main difference between Eq. (3) and Eqs. (1) and (2) is that Eq. (3) does not contain the concrete compressive strength and the percent of steel term.

In a recent study, Seleim (13) showed that there is no noteworthy differences between the results obtained by using any of Eqs. (1), (2) or (3), indicating that the two terms (f'_c and ρ) have no significant effect on the ultimate shear-bond capacity. This has also been concluded by Schuster and Ling (12).

DEVELOPMENT OF SHEAR-BOND EQUATION

Recent experiments showed that some composite slab systems failing in shear-bond do exhibit end-slip prior to ultimate load (3,8,9,11,12). This end-slip occurs almost simultaneously with the formation of the first potential failure crack at a load ranging between 50-60% of the ultimate shear-bond capacity.

The development of any shear-bond equation requires the establishment of the basic characteristics of the failure mechanism. In the development of the shear-bond equation presented in this paper, the characteristic behaviour of both steel and concrete was assumed to be the same as in reinforced concrete. Different researchers make different assumptions regarding the cause of a shear-bond failure. In this paper, a shear-bond failure mechanism is characterized in the following manner:

Initially, before crack, the load is carried by both the steel deck and concrete. The mechanical shear devices (embossments) are transmitting shear forces between the steel deck and concrete, so as to maintain the composite action. Under the applied load and before the potential failure crack occurs, both concrete and steel deck will deflect together. With an increase in applied load and due to the difference in flexural rigidity between the steel deck and concrete, a vertical separation is initiated under either of the loads, point A in Fig. 2, where the slope of the bending moment diagram changes from a horizontal to an inclined position. At the location of vertical separation, composite action is no longer maintained and the concrete is essentially carrying additional load. Vertical separation also results in the disengagement of a part of the mechanical shear devices near that location. With the increase of the applied load the process will continue; more vertical separation, more load carried by concrete and more shear devices become disengaged. The second stage of failure will be reached when the additional load carried by the concrete becomes sufficiently large to initiate a diagonal tension crack. Almost at the same time the concrete shear-span portion begins to slide over the steel deck since part of the shear devices, disengaged ones, are not active, resulting in end-slip. Increasing the load beyond this point causes more vertical separation, widening the concrete crack and increasing end-slip. The ultimate load is reached when the size of the crack becomes excessively large and the concrete shear-span becomes disengaged from the steel deck.

The failure mechanism described above, suggests that failure is initiated when the bending resistance of the composite section breaks down near one of the applied loads, where the bending moment is maximum. The break down in bending resistance results in a vertical separation between the steel deck and concrete. The vertical separation leads to a break down in shear resistance since shear devices become disengaged at that location. This will cause more load to be carried by the concrete, leading eventually to a standard diagonal tension crack which is usually experienced in concrete sections without shear reinforcement. The widening of the crack, as well as the disengagement of more shear devices, will result in end-slip. This process continues until both the bending and the shear resistance of the section are exhausted, at which time the ultimate load is reached.

From the failure mechanism described above, one can conclude that the ultimate shear-bond capacity of composite slabs is due to the combined bending and shear resistances. In other words, considering equilibrium of external forces of the diagram shown in Fig. 3-a, one can write the following equation:

$$\frac{P_u}{2} = V_b + V_{sh} \tag{5}$$

where P_u is the ultimate shear-bond capacity, V_b is the ultimate transverse capacity due to bending resistance and V_{sh} is the ultimate transverse capacity due to shear resistance. From equilbrium of external and internal moments of the cracked section shown in Fig. 3-b, one can obtain the following equation for V_b:

$$V_b L' = F_b\ B\ d' \tag{6}$$

F_b is the force in the steel deck per unit width, d' is the distance from the centroidal axis of the steel deck to the centroidal axis of the concrete compression zone, B is the slab width and L' is the shear span as defined before. The force F_b is proportional to the steel deck thickness, t. Hence, by assuming a first degree polynomial to represent the relationship between F_b and t and that d' is directly proportional to d, Eq. (6) can be rewritten as follows:

$$V_b = (k_1\ t + k_2)\ \frac{Bd}{L'} \tag{7}$$

where k_1 and k_2 are unknown coefficients to be determined from laboratory performance tests for each product type.

Again, by considering equilibrium of internal and external moments of the cracked section in Fig. 3-c, the following relationship for the ultimate transverse capacity due to shear resistance is obtained:

$$V_{sh} L' = F_{sh}\ L'\ B\ d' \tag{8}$$

where F_{sh} is the ultimate resisting shear stress between the steel deck and concrete. It can be seen that F_{sh} is proportional to the steel deck thickness, t. Hence, by assuming a first degree polynomial to represent the relationship between F_{sh} and t, and that d' is directly proportional to d, one can rewrite Eq. (8) as follows:

$$V_{sh} = (k_3\ t + k_4)\ Bd \tag{9}$$

where k_3 and k_4 are unknown coefficients to be determined the same way as k_1 and K_2 mentioned before. Substituting Eqs. (7) and (9) into Eq. (5) and rearranging, the following is obtained:

$$\frac{P_u}{2Bd} = k_1 \frac{t}{L'} + k_2 \frac{1}{L'} + k_3\, t + k_4 \qquad (10)$$

Eq. (10) can be rewritten in the form of Eq. (4) of Table 1 by substituting $\frac{V_u}{b}$ for $\frac{P_u}{2B}$.

EVALUATION AND COMPARISON OF SHEAR-BOND EQUATIONS

In order to evaluate and compare the shear-bond equations presented in Table 1, experimental data of 196 one-way composite slabs failing in shear-bond was collected. Slabs were simply supported and subjected to two symmetrically placed line loads. The data comprises nine different groups from four different manufacturers, each of which was given a three-digit identification number. The first digit designates different manufacturers, while the other two digits designate different product types of the same manufacturer. For each group, steel decks with different steel deck thickness were given the same data group number. Each group of the data have the same following characteristics:

1. Same manufacturer's product type.
2. Same coating conditions.
3. Same unit weight of concrete.
4. Same steel deck width.

Differences in shoring conditions were not considered, as it was found (2,6) that the shoring conditions have no adverse effect on the ultimate shear-bond capacity.

Eqs. (1), (2) and (3) in Table 1 can be expressed in the form $y = mx + k$, while Eq. (4) can be written in the form $y = k_1x_1 + k_2x_2 + k_3x_3 + k_4$. The coefficients m and k must be evaluated for each product type and each steel deck thickness separately, using a linear regression analysis. The coefficients k_1 through k_4 must be evaluated for each product type only, regardless of the variation of steel deck thicknesses, by using a multi-linear regression analysis. Experimental data are needed for linear and multi-linear regression analysis. The number of test results required for the analysis depends mainly on the level of accuracy required of the computed ultimate shear-bond values. In order to obtain a level of accuracy of $\mp 15\%$ between computed and experimental ultimate shear-bond values, Porter and Ekberg (4) recommended that data of eight experiments be used in the evaluation of m and k for each steel deck thickness of each product type. For a manufacturer producing 4 different steel deck thicknesses 8 experiments are needed for each of the four steel deck thicknesses produced, resulting in a total of 32 laboratory performance tests in order to obtain that level of accuracy.

In order to determine which equation results in a better accuracy with the least possible number of experimental data, the four equations in Table 1 were studied. The results of Eqs. (1), (2) and (4) only are presented in the following part. Results of Eq. (3) are not presented herein, since they were of the same level of accuracy as Eqs. (1) and (2) as indicated before. The study was carried out in the three steps as illustrated in Table 2. In each step the coefficients of each of the three equations were determined using a number of specified data. The values of the coefficients were then substituted in the corresponding equation in order to compute the ultimate shear-bond capacity for each experiment. Finally, different statistical measures such as the sum of square deviations, the correlation coefficient, percent error, etc. were computed in order to compare the computed with the experimental ultimate shear-bond capacity for each experiment and for each equation.

STEP I

The coefficients of each of Eqs. (1), (2) and (4) were evaluated for each steel deck thickness of each product type by using all of the test data available for that particular thickness (see Table 2). The results of this step, as shown in Table 3, indicate no apparent difference between the three investigated equations. The correlation coefficients are close to unity (never less than 0.938), which indicates a good correlation between the variables of each equation. The sum of square deviations, similarly indicate no significant difference between the three equations. A sample plot of the experimental versus the computed ultimate shear-bond capacities for each equation is shown in Fig. (4). The plots confirm the similarities between the three equations within the adopted scatter band of ∓15%.

STEP II

In this step all of the test data of the different steel deck thicknesses of each product type were used to evaluate the coefficients of each of Eqs. (1), (2) and (4) (See Table 2), i.e., one set of the coefficients of each equation was evaluated for each product type regardless of the variation in steel deck thickness. The results shown in Table 4 indicate that only Eq. (4) maintains the same level of accuracy as obtained in the previous step. For Eq. (1), for example, the correlation coefficient for Deck 730 dropped by 10.6%, from 0.998 (see Table 3) to 0.896 (see Table 4). For Eq. (2), and for the same product type, the correlation coefficient dropped from 0.998 (see Table 3) to 0.874 (see Table 4), which is about 12.4%. The corresponding drop for Eq. (4) is only 0.1% as the correlation coefficient changed from 0.998 (see Table 3) to 0.997 (see Table 4). A similar comparison between the results of Tables 3 and 4 of the other product types shows the same trend. The correlation coefficient gives a quantitative measure of the association between the variables when data of different steel deck thicknesses are combined during the evaluation of the coefficients of each equation. For Eq. (4), the correlation coefficients shown in Table 4, are very close to unity showing very good association between the variables of Eq. (4). But for Eqs. (1) and (2) the correlation coefficients dropped to 0.841 for Eq. (1) of Deck-300 and to 0.874 for Eq. (2) of Deck-730 showing much less association between the variables of these two equations when data of different thicknesses was used together. Also, one can observe from Table 4 that the Sum of Square Deviations (SSD) corresponding to Eq. (4) is much less in comparison to those of Eqs. (1) and (2). For Deck-730, the SSD for Eq. (4) is 1.880 compared with 58.631 and 69.853 for Eqs. (1) and (2), respectively. Furthermore, comparing the SSD values of Table 3 with the corresponding values of Table 4, one can observe that the values corresponding to Eq. (4) have only slightly changed, while the values corresponding to Eqs. (1) and (2) have increased considerably. Plots of a sample of the results of this step are shown in Fig. (5). The plots corresponding to Eqs. (1) and (2) are shown to be more scattered compared with previous plots shown in Fig. (4). For Eq. (4) the plot shown in Fig. (5) is less scattered over the ∓15% band limit and is very similar to that of Fig. (4).

STEP III

Only Eq. (4) was investigated in this step since it gave much better results in the previous step. The coefficients of Eq. (4) were evaluated for each product type using selected sets of data points. As shown in Table 2, each set consisted of a sample of two data points of each steel deck thickness available of each product type. The data points were chosen to cover a wide range of the shear span lengths, L'. Except for Deck-424, the correlation coefficients range from 0.944 to 0.999 (see Table 5) which shows a good correlation. In general, the correlation coefficients only slightly dropped in this step compared with the previous steps. This drop which ranges between 0% for Deck-720 and 2.78% for Deck-412 can be neglected. For Deck-424 the drop is relatively high (from 0.939 in Table 3 to 0.865 in Table 5), which can be attributed to the wide range of shear span lengths of this particular product type. Hence, by only selecting two points of each thickness, it was not possible to cover the entire range of L'. Comparing the SSD values of Table 5 with the corresponding values of Tables 3 and 4, it can be seen that no significant increase occurred. A sample plot is shown in Fig. (6) for the same product type as shown before. The plot demonstrates that still most of the points fall inside the ∓15% band, although they are more scattered. This step demonstrate that for each product type having two or more steel deck thicknesses, only two data points are required from each steel deck thickness. In other words, for a manufacturer producing any product type of 4 different steel deck thicknesses, he is required to perform only 2 experiments for each steel deck thickness or a total of 8 (4 x 2) experiments for the product type. The accuracy obtained from only the eight experiments is expected to be within ∓15% as demonstrated in this step. One should recall here that the same manufacturer is presently required to perform up to 32 experiments to maintain the same level of accuracy as discussed before.

CONCLUSIONS

As a result of the study presented in this paper, the following conclusions can be made:

1. A number of laboratory performance tests are required in order to determine the ultimate shear-bond capacity of composite slabs.

2. Using Eq. (4), a total of eight suitable laboratory performance tests are required for each manufacturer's product type. This total number of experiments is about 1/4 of the number presently suggested in order to achieve the same level of accuracy of ∓15%.

3. Eq. (4) gives the same level of accuracy as that obtained from existing equations if data of different steel deck thicknesses are considered

separately. However, when data of different steel deck thicknesses were considered together, Eq. (4) was shown to give a much better accuracy.

4. Eq. (4) supports recent work on composite slabs which showed that neither the percent of steel, ρ, nor the concrete compressive strength, f'_c, any appreciable effect on the ultimate shear-bond capacity.

5. The shear span is the only apparant variable affecting the ultimate shear-bond capacity of composite slabs of the same product type and of the same steel deck thickness. This conclusion is obvious from Eq. (4) if the steel deck thickness is set equal to a constant.

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APPENDIX I - REFERENCES

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APPENDIX II - NOTATION

A_s = cross-sectional area of steel deck per unit width of slab;

B = width of slab;

b = unit width of slab;

d = effective depth of slab (distance from the centroidal axis of the steel deck to the top surface of the concrete compression zone);

d' = distance from centroidal axis of steel deck to centroidal axis of concrete compression zone;

F_b = force in the steel deck per unit width;

f'_c = concrete compressive strength;

F_{sh} = ultimate resisting shear stress between the steel deck and concrete;

k = coefficient to be determined from laboratory performance tests;

k_1-k_4 = coefficients to be determined from laboratory performance tests;

L' = length of shear span (distance between load and nearest support);

m = coefficient to be determined from laboratory performance tests;

P_u = ultimate shear-bond capacity of composite slabs;

s = center-to-center spacing of shear transfer devices (for embossments s is set equal to unity);

t = steel deck thickness;

V_b = ultimate transverse capacity due to bending resistance;

V_{sh} = ultimate transverse capacity due to shear resistance;

V_u = ultimate transverse shear-bond capacity per unit width of slab;

ρ = percent of steel, A_s/bd;

TABLE 2. - Different Steps of the Analysis

Step	Equation Investigated	Data of each group used in the evaluation of coefficients
I	(1), (2) & (4)	ALL available data having the same steel deck thickness.
II	(1), (2) & (4)	All available data of different steel deck thicknesses.
III	(4)	The sum of two data points of each steel deck thickness.

TABLE 3. - Statistical Results of Step I

Data Group	Correlation Coefficient			Sum of Square Deviations		
	Eq. 1	Eq. 2	Eq. 4	Eq. 1	Eq. 2	Eq. 4
Deck - 300	0.980	0.982	0.980	7.962	7.027	7.774
Deck - 310	0.982	0.982	0.982	14.160	14.151	14.077
Deck - 320	0.987	0.987	0.987	0.956	0.940	0.944
Deck - 412	0.964	0.969	0.971	15.023	12.991	12.132
Deck - 424	0.942	0.938	0.939	40.076	42.826	42.116
Deck - 710	0.993	0.988	0.991	3.277	5.117	4.142
Deck - 720	0.998	1.000	1.000	1.226	0.326	0.326
Deck - 730	0.998	0.998	0.998	1.349	1.210	1.210
Deck - 800	0.980	0.985	0.986	22.731	17.445	16.095

TABLE 4. - Statistical Results of Step II

Data Group	Correlation Coefficient			Sum of Square Deviations		
	Eq. 1	Eq. 2	Eq. 4	Eq. 1	Eq. 2	Eq. 4
Deck - 300	0.841	0.949	0.977	58.083	19.856	9.067
Deck - 310	0.947	0.970	0.982	41.368	23.446	14.540
Deck - 320	0.955	0.964	0.986	3.183	2.537	1.035
Deck - 412	0.937	0.937	0.971	26.116	26.321	12.433
Deck - 424	0.850	0.902	0.939	98.719	66.244	41.779
Deck - 710	0.938	0.976	0.986	26.478	10.313	6.111
Deck - 720	0.882	0.949	0.999	75.062	33.204	0.727
Deck - 730	0.896	0.874	0.997	58.631	69.853	1.880
Deck - 800	0.964	0.982	0.972	40.802	20.985	31.964

TABLE 5. - Statistical Results of Step III

Data Group	Correlation Coefficient	Sum of Square Deviation
Deck - 300	0.970	11.749
Deck - 310	0.962	30.036
Deck - 320	0.975	1.812
Deck - 412	0.944	23.280
Deck - 424	0.865	89.441
Deck - 710	0.976	10.439
Deck - 720	0.999	0.576
Deck - 730	0.995	2.960
Deck - 800	0.979	24.149

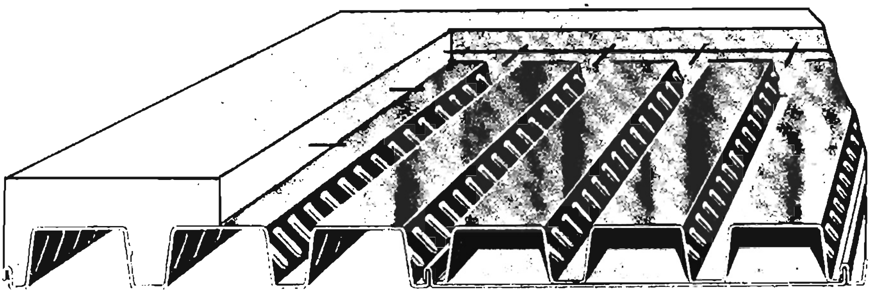

Fig. 1 - Example of Steel Deck, courtesy of Westeel-Rosco Limited, Toronto, Ontario, Canada.

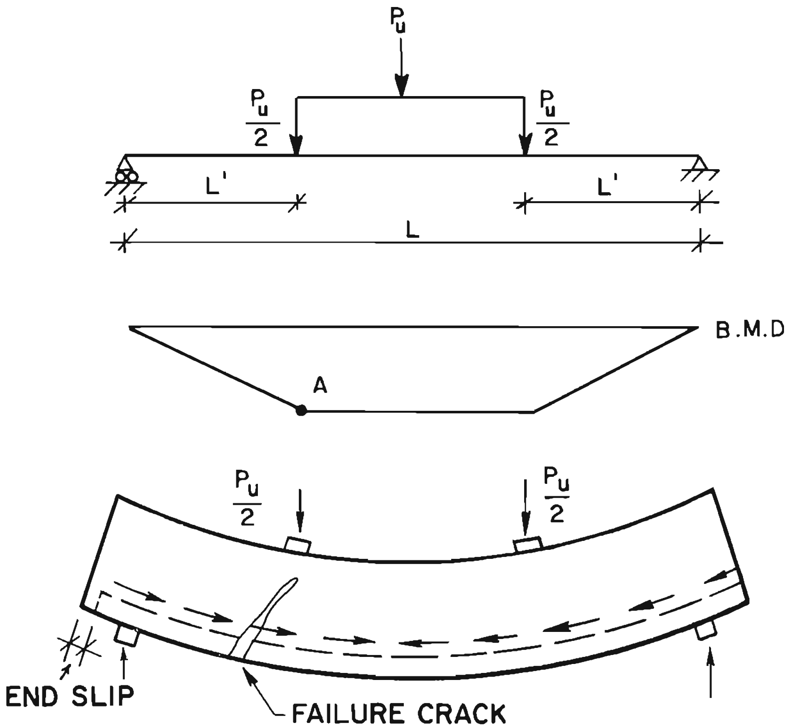

Fig. 2 - Typical Shear-Bond Failure

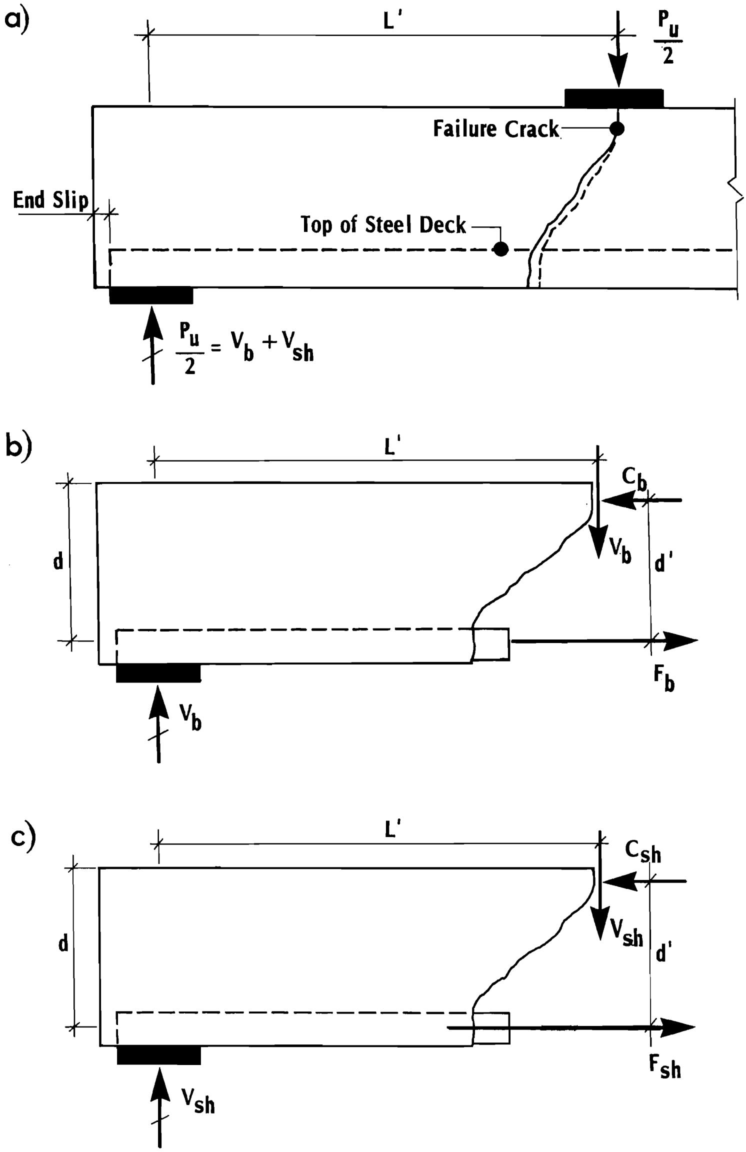

Fig. 3 - Internal Forces at Region of Crack

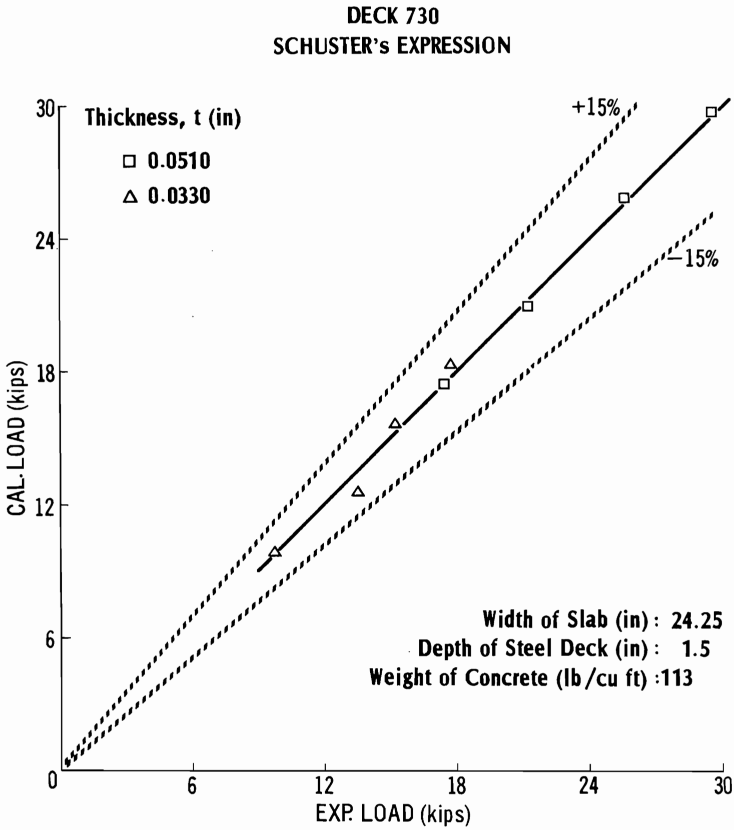

Fig. 4a - EXPerimental and CALculated Ultimate Shear-Bond Capacities of Deck-730, Eq. (1), Step I.

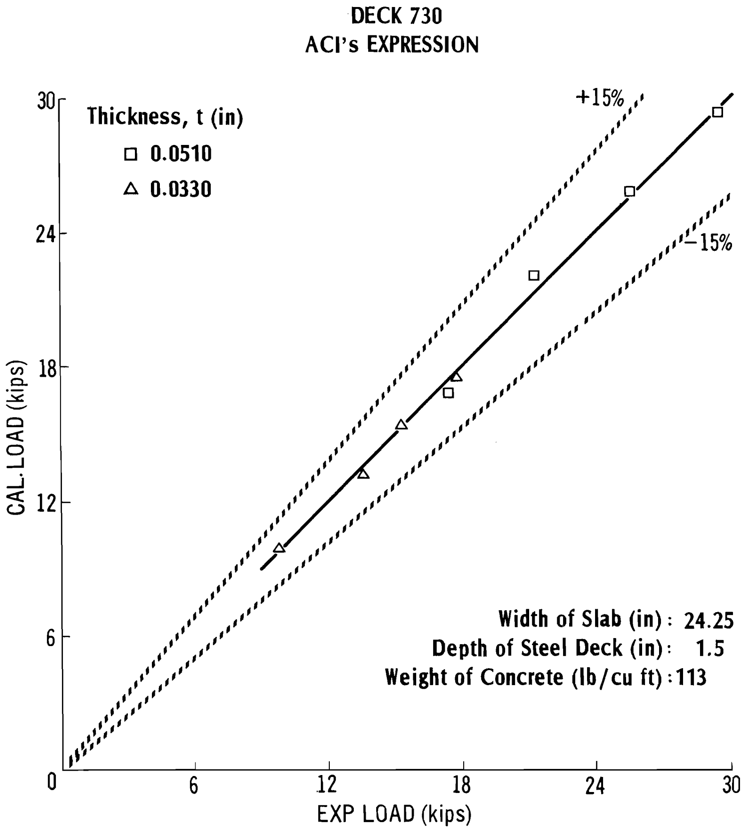

Fig. 4b - EXPerimental and CALculated Ultimate Shear-Bond Capacities of Deck-730, Eq. (2), Step I.

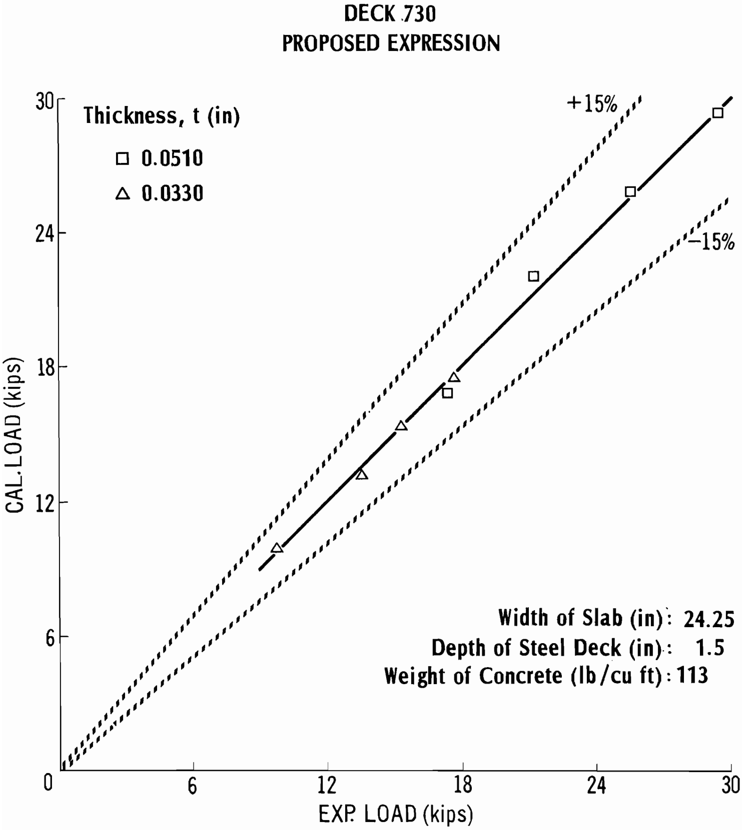

Fig. 4c - EXPerimental and CALculated Ultimate Shear-Bond Capacities of Deck-730, Eq. (4), Step I.

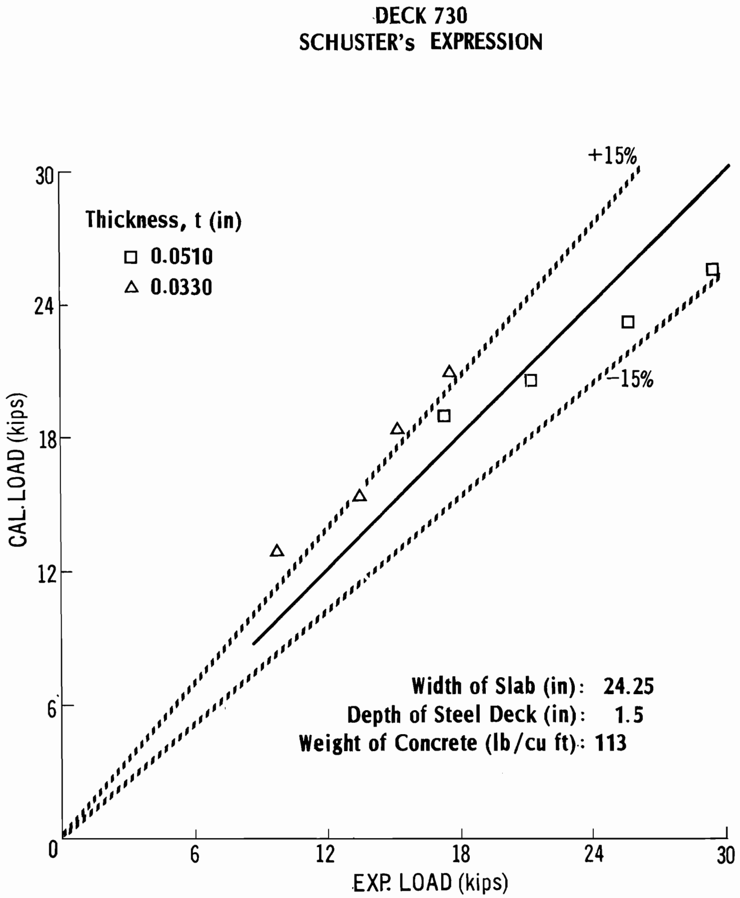

Fig. 5a - EXPerimental and CALculated Ultimate Shear-Bond Capacities of Deck-730, Eq. (1), Step II.

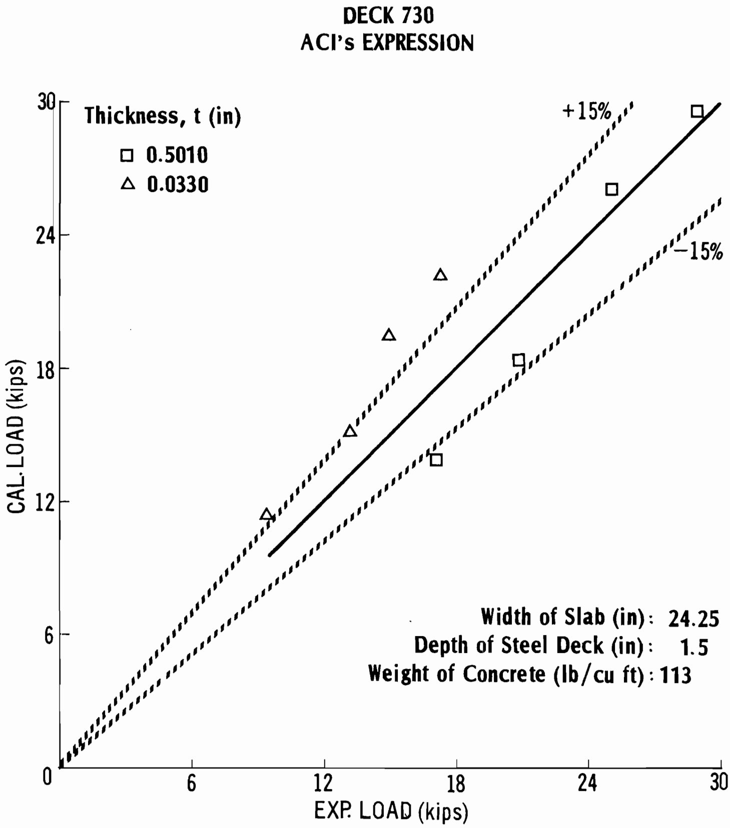

Fig. 5b - EXPerimental and CALculated Ultimate Shear-Bond Capacities of Deck-730, Eq. (2), Step II.

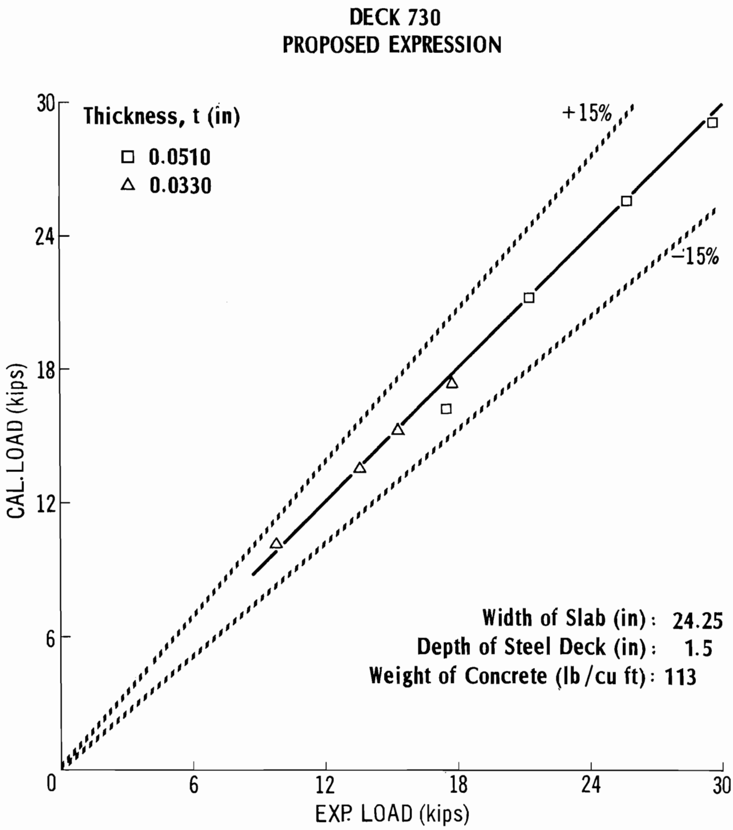

Fig. 5c - EXPerimental and CALculated Ultimate Shear-Bond Capacities of Deck-730, Eq. (4), Step II.

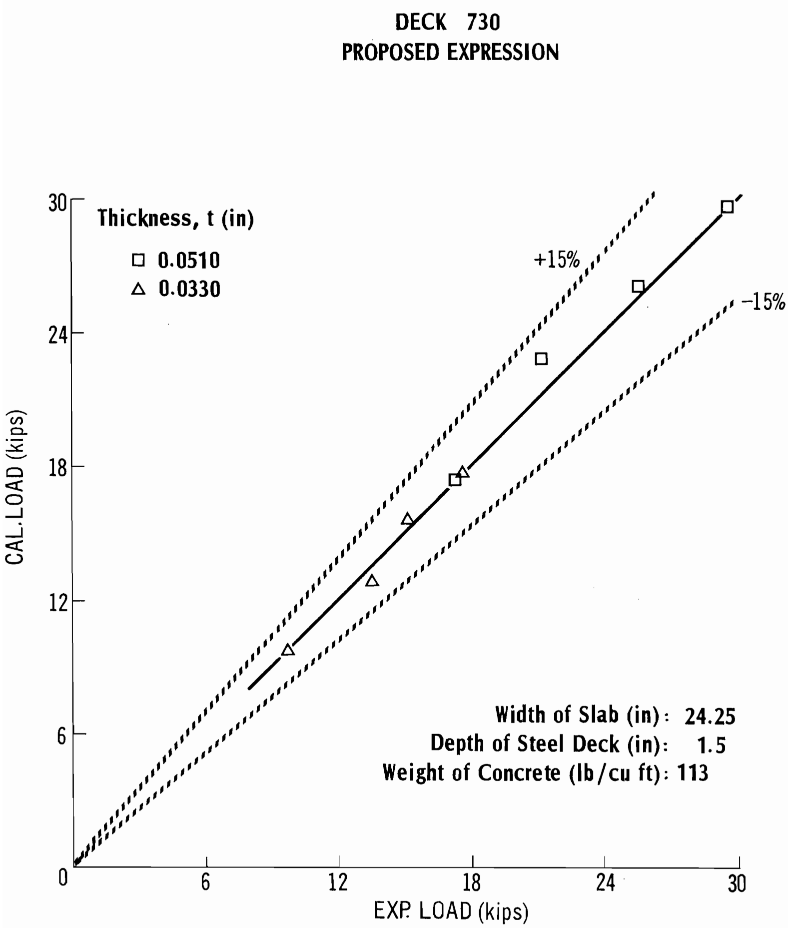

Fig. 6 - EXPerimental and CALculated Ultimate Shear-Bond Capacities of Deck-730, Eq. (4), Step III.