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# Composite Steel-Deck-Reinforced Concrete Systems Failing in Shear-Bond

Ruine par défaut d'adhérence des plaques composées de tôle d'acier et de béton

Durch Stahlplatten bewehrte Betontragwerke mit Schubbruch

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## INTRODUCTION

Reference is made by Messrs. C. F. Mcdevitt and I. M. Viest[1] to the use of corrugated or ribbed panel light gage steel decks which interact with the concrete slab to form a composite floor system, often referred to as steel-deck-reinforced concrete slab construction. Mention is made by Mcdevitt and Viest of a paper authored by Ekberg and Schuster, included in the final report of the Eighth IABSE Congress[2], describing the state of the art of using steel-deck-reinforced concrete slabs in buildings. Particularly, Mcdevitt and Viest point out the more recent research conducted by Ekberg and Schuster at Iowa State University leading to the development of semi-empirical equations relating to the ultimate shear-bond strength of steel-deck-reinforced concrete systems, pointing out that laboratory tests have shown that most of these specimens exhibit a shear-bond type of failure. Since the initiation of this work in 1967, several unpublished research progress reports related to the shear-bond capacity of steel-deck-reinforced systems have resulted, including references (2), (3) and (4).

The content of this discussion, taken largely from Reference (4), is focused primarily on the development of a concept relating to the ultimate strength of steel-deck-reinforced concrete systems failing in shear-bond. This task is divided into 1. a laboratory test program and 2. an analytical ultimate strength analysis. The laboratory test program was conceived in an effort to provide the necessary experimental data for determining the ultimate shear-bond strength as obtained from a semi-rational-analytical approach, thus requiring a statistical evaluation of experimental data.

## LABORATORY TEST PROGRAM

The philosophy of the test program was to involve the loading to failure of a large number of representative elements of steel-deck-reinforced concrete slabs as simple beams in order to adequately cover a full range of behavioral characteristics. Initial testing indicated that a shear type of failure would be the most predominant mode of failure in most steel-deck-reinforced systems. Based on this observation, beam testing was focused primarily on the nature of shear transfer between the steel deck and concrete.

The test program was designed in an effort to simulate, as closely as practically possible, beam elements of steel-deck-reinforced concrete slabs as found in common construction practice. Therefore, all laboratory beam testing was conducted on simple supports and subjected to a symmetrical mode of loading, consisting of either a single concentrated line load or two concentrated line loads.

Steel decks from four different steel deck manufactures were used in the test program; however, the majority of beam tests were conducted using one particular deck profile, namely that of company I[a]. This was done in an effort to obtain a large number of test results, embodying the most logical parametric variations. To further verify the ultimate strength expressions a representative number of beam tests were conducted on composite units constructed with steel decks E, O and G.

The steel deck profiles tested were divided into two categories based on the pattern of mechanical shear connectors such as embossments, holes or welded wires. The two categories are stated as follows:

Category I - Steel deck profiles that provide horizontal shear capacity primarily by virtue of a fixed pattern of mechanical shear devices, i.e. the center to center spacing of the shear devices is constant for every steel deck thickness and depth of slab.

Category II - Steel deck profiles that have a variable spacing of mechanical shear devices, i.e. the center to center spacing of the shear devices may vary with the depth of slab and steel deck thickness.

Tests were conducted on a total of 145 steel-deck-reinforced concrete beams. All steel decks were of out-to-out depth between 1¼ and 2 in., such that the neutral axis of the composite cross section was located above the top of the steel deck. Span length, shear span, beam depth and width and steel deck thickness were varied with each of the steel-deck-reinforced systems. All beams were supported throughout during the placing and curing of the concrete, except a few selected beam specimens were shored at their ends and at mid-span during placing and curing.

The characterization of a shear-bond failure was identified by the formation of an approximate diagonal crack under or near one of the concentrated loads, resulting in a brittle type of failure at ultimate load. This failure was accompanied by end-slip between the steel deck and concrete, thus, causing the concrete shear span portion, L' (see Fig. 4 for location of L'), to become disengaged, experiencing loss of bond between steel deck and concrete. This simultaneous action of shear and bond is termed shear-bond. See Fig. 1 for typical shear-bond failure.

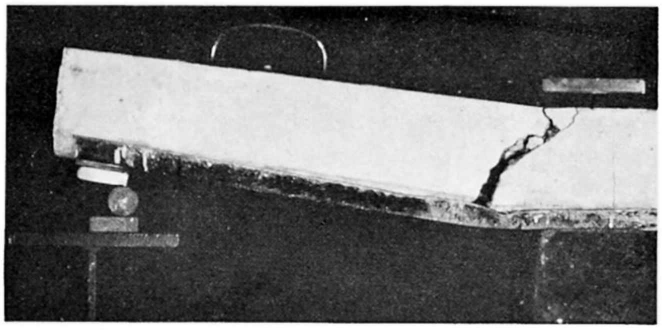

Fig. 1 - Typical shear-bond failure with end-slip of beam constructed with steel deck G

A shear-bond failure may, or may not, have been preceded by the yielding of the steel, depending on the relative values of the percentage of steel, the shear span L' and the inherent load transfer capacity of the shear transfer devices. Yielding of the steel-deck, whenever in occurrance, initiated at the extreme bottom fibers of the steel deck and in some cases progressed towards the top of the steel deck. In no case, however, did the steel deck yield over its entire depth. See Reference (4) for further detail.

[a] Letters were chosen to identify the different steel decks, thus, avoiding direct company comparison.

## ANALYTICAL ANALYSIS

Based on the experimental findings, a semi-rational shear-bond concept was adopted, since a truly rational concept is complicated by the nonhomogeniety and nonisotropy of concrete. Consideration was given to beams constructed with steel decks of Category I with fixed pattern shear transfer devices and Category II where spacing of shear devices is a variable.

In some respects, a shear-bond failure regarding steel-deck-reinforced concrete slabs is similar to a shear and diagonal tension failure in conventional reinforced concrete without web reinforcement. The main similarity lies in the formation of an approximate diagonal tension failure crack, resulting from combined shear and bending. This failure crack is not always diagonal in nature, but for all practical purposes, a diagonal crack can be assumed, leading further to the assumption that this crack is caused by excessive principal tension tresses.

### Category I

Based on the major variables that have been found to influence shear and diagonal tension in conventional reinforced concrete without web reinforcement, a general expression for the ultimate transverse shear may be written as follows:

$$V_{uc} = f(f'_c, L', d, b, p) \tag{1}$$

where $f'_c$ is the compressive strength of the concrete, b is the width of the composite beam cross section, d is the effective depth from the top of concrete to the center of gravity of the steel deck and p is the percentage of steel. To arrive at an expression containing the variables of Eq. (1), it is assumed that the ultimate transverse shear, $V_{uc}$, neglecting dead load, is the result of the concrete and steel deck contributing independently. From Fig. 2 it can be observed, by summation of vertical force components that

$$V_{uc} = V_c + V_d \tag{2}$$

where $V_c$ is the transverse shear carried by the concrete at ultimate load and $V_d$ is the transverse shear carried by the steel deck at ultimate load.

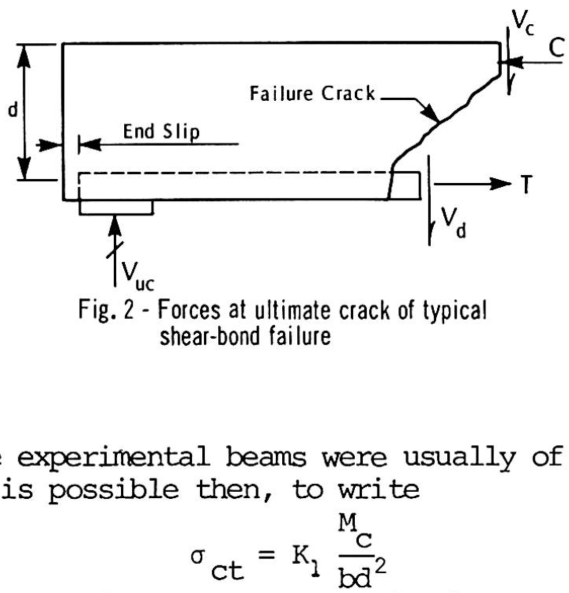

Fig. 2 - Forces at ultimate crack of typical shear-bond failure

The maximum concrete tensile stress, below the neutral axis, is as follows:

$$\sigma_{max} = \frac{\sigma_{ct}}{2} + \sqrt{\left(\frac{\sigma_{ct}}{2}\right)^2 + v_c^2} \tag{3}$$

where $\sigma_{ct}$ is the normal stress in the concrete and $v_c$ is the vertical, or horizontal, shear stress in the concrete. The magnitude of the tensile bending stress, $\sigma_{ct}$, is influenced by the presence of tensile cracks, and may consequently be computed either from an assumed cracked or uncracked section. For this analysis, $\sigma_{ct}$, is based on the uncracked section theory. The reason for this being that cracks in the experimental beams were usually of a hairline nature even near ultimate load. It is possible then, to write

$$\sigma_{ct} = K_1 \frac{M_c}{bd^2}$$

where $M_c$ is the moment carried by the concrete at ultimate load and $K_1$ is a constant as well as $K_2$, through $K_8$ to follow.

The shear stress, $v_c$, in the concrete is assumed to be proportional to the average intensity of shear stress on the total cross section. Thus

$$v_c = K_2 \frac{V_c}{bd} .$$

When the maximum principal stress $\sigma_{max}$, exceeds the tensile strength of concrete, $f'_t$, at the location of the potential diagonal failure crack, shear-bond failure is assumed to impend. The tensile strength of concrete is assumed proportional to the square root of the compressive strength, $f'_c$, of the concrete. Thus,

$$\sigma_{max} = K_3 \sqrt{f'_c} .$$

Substituting the above stress relationships into Eq. (3) and rearranging in terms $V_c/bd$ results in the following:

$$\frac{V_c}{bd} \cdot \frac{1}{2K_3} = \frac{1}{\frac{K_1}{d\sqrt{f'_c}} \cdot \frac{M_c}{V_c} + \frac{1}{\sqrt{f'_c}} \sqrt{\left(\frac{K_1}{d} \cdot \frac{M_c}{V_c}\right)^2 + (2K_2)^2}}$$

Now, factoring the term $(K_1/d \cdot M_c/V_c)$ from the square root and substituting $K_4 = 2K_2/K_1$, the expression reduced to

$$\frac{V_c}{bd} \cdot \frac{1}{2K_3} = \frac{1}{\frac{K_1 M_c}{d\sqrt{f'_c}\, V_c} \left[1 + \sqrt{1 + \left(K_4 \frac{V_c d}{M_c}\right)^2}\,\right]} . \tag{4}$$

A study of Eq. (4) indicates that the magnitude of the term $(K_4 dV_c/M_c)$ can be assumed to approach zero for most practical cases. Letting $K_5 = K_3/K_1$ and solving for $V_c$, Eq. (4) reduces to

$$V_c = \frac{K_5 bd^2 \sqrt{f'_c}\, V_c}{M_c} \tag{5}$$

The transverse shear carried by the steel deck is assumed to be proportional to the cross-sectional area of the steel deck ($A_s$). Thus,

$$V_d = K_6 A_s . \tag{6}$$

Since the concrete is placed directly over the steel deck, the transverse shear contribution can be quite appreciable, particularly when the cross-sectional area of the deck is large and the depth of the concrete is at a minimum.

Combining Eqs. (5) and (6) in accordance with Eq. (2) and expressing in terms of unit nominal ultimate shear stress, with $v_{uc} = V_{uc}/bd$, the following general equation results:

$$v_{uc} = K_5 \frac{\sqrt{f'_c}\, dV_c}{M_c} + K_6 p \tag{7}$$

Based on actual experimental beam testing of this investigation, Eq. (7) is expressed more specifically for the special case of symmetrical concentrated loads. The terms $V_c/M_c$ and $1/L'$ are synonymous since $M_c = V_c L'$ and

$$v_{uc} = \frac{V_{uc}}{bd} = K_5 \frac{\sqrt{f'_c}\, d}{L'} + K_6 p . \tag{8}$$

where $p = A_s/bd$.

Equation (8) gives the parameters to be investigated and takes into account the three most important variables that affect the shear-bond strength of flexural members subjected to combined bending and shear; these are the compressive strength of concrete, ratio of reinforcement, and the ratio of external shear to the maximum moment in the shear span.

## Category I

The same concept was employed in the development of a shear-bond expression for Category II as was used in Category I. However, the resulting expression now contains one additional parameter, namely, the spacing of the shear transfer devices, s.

Figure 3 shows a typical steel deck profile of Category II where the shear transfer device spacing, s, is subject to change. Summing forces between the horizontal interface of the concrete and top of the steel deck where the shear transfer devices are located, see Fig. 3, an expression that satisfies statics may be written as

$$v_{uc} bs = \frac{b}{g} m_u$$

where $m_u$ is the ultimate load carrying capacity per weld

or,
$$\frac{V_{uc}}{bd} = \frac{1}{s} \cdot \frac{m_u}{g} = v_{uc} \; . \qquad (9)$$

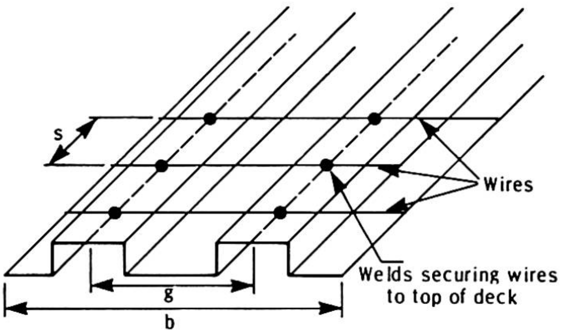

Fig. 3 - Typical steel deck profile of category II

Equation (9) indicates that the ultimate shear-bond stress, $v_{uc}$, is inversely proportional to the shear device spacing, s, and directly proportional to the ratio $m_u/g$. Since the dimension, g, of any given deck profile is constant, the ratio $m_u/g$ can be assumed to be directly proportional to the shear-bond capacity. The following expression results

$$\frac{V_{uc}}{bd} = \frac{1}{s} \left(K_7 \frac{\sqrt{f'_c}\, d}{L'} + K_8 p\right) . \qquad (10)$$

Note that $K_7$ and $K_8$ correspond to constants $K_5$ and $K_6$ of Eq. (8). The determination of these constants depends upon experimental beam test data.

## TEST RESULT EVALUATION

The data resulting from the numerous beam tests was assimilated so that the ultimate shear-bond capacity could be related to the various parameters as expressed by Eqs. (8) and (10); and a statistical regression analysis was used in evaluating the respective regression constants. Equations (8) and (10), applicable to beams of Category I and II respectively, provided the necessary variables for the determination of these regression constants.

Figure 4, for example, shows a plot of ultimate strength shear-bond relationships for beams constructed with steel deck I - 22 gage. All beams were supported throughout, except three beams which were shored at midspan prior to placing of the concrete. No appreciable difference exists between beams supported throughout and those shored at midspan. Also, it can be observed that the change of width of beams produces no apparent effect on the shear-bond load carrying capacity.

Figure 5 exhibits the same linear shear-bond relationship and similar behaviorial characteristics as Fig. 4.

Figure 6 represents a plot of shear-bond relationships of beams constructed with steel deck E resulting from a combined regression analysis of tests conducted in this investigation and by company E. The reason for combining the 20 and 22 gage parameters in one regression is because the difference in steel deck thickness being very small.

Beams constructed with steel deck G are placed in Category II, and Eq. (10) applies for the shear-bond regression analysis. Figure 7 represents the ultimate shear-bond relationship for beams constructed with steel decks G-20 and 24 gage. Figure 7 reveals that the ultimate shear-bond strength of beams constructed with deck G-24 gage is greater than that of beams constructed with 20 gage decking. In the case of beams with 20-gage deck, there was a shearing action at the connectors which left the deck itself relatively intact. On the other hand, there was an actual tearing of the steel deck in the areas of the welds, with the beams constructed with 24-gage steel. This led to the conclusion that more complete

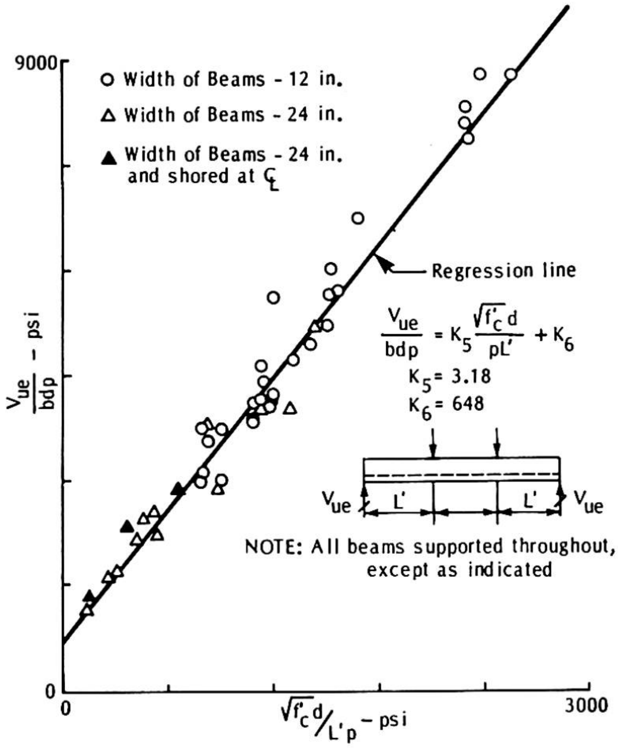


Fig. 4 - Relationship between $V_{ue}$/ bdp and $\sqrt{f'_c}\,d$ / L'p for beams constructed with steel deck I-22 gage

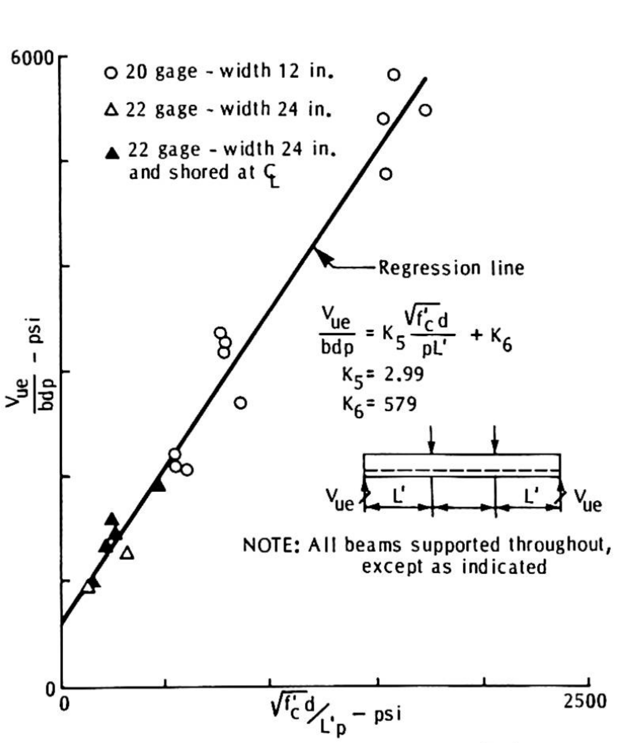


Fig. 5 - Relationship between $V_{ue}$/ bdp and $\sqrt{f'_c}\,d$ / L'p for beams constructed with steel deck O-20 and 22 gage

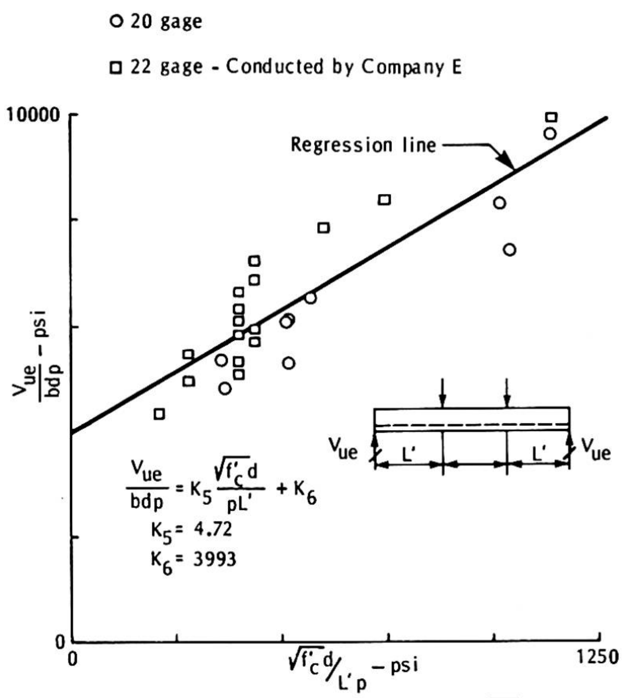


Fig. 6 - Relationship between $V_{ue}$/ bdp and $\sqrt{f'_c}\,d$ / L'p for beams constructed with steel deck E-20 and 22 gage

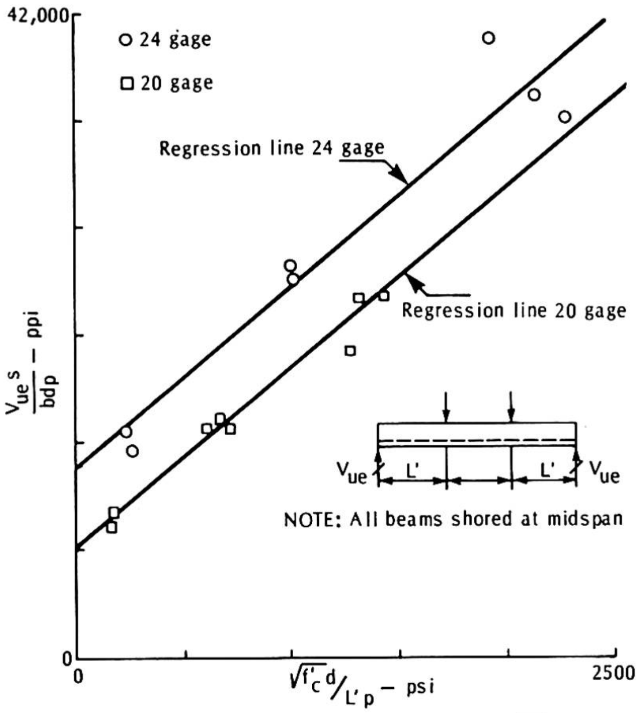


Fig. 7 - Relationship between $V_{ue}$s / bdp and $\sqrt{f'_c}\,d$ / L'p for beams constructed with steel deck G-20 and 24 gage

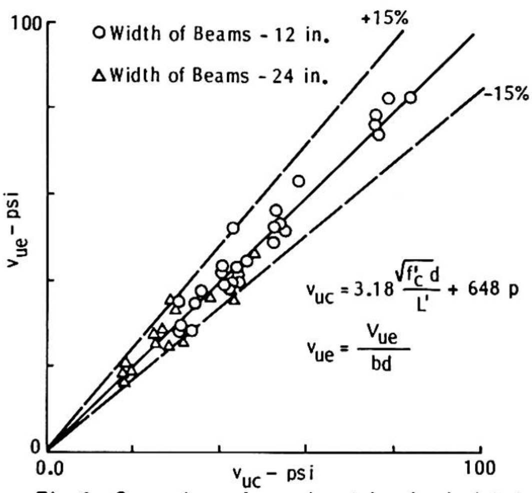

Fig. 8 - Comparison of experimental and calculated ultimate shear-bond stresses for beams constructed with steel deck I-22 gage

penetration of connector welds existed with beams of 24-gage steel decks than with those of 20-gage decking. The better penetration undoubtedly caused a greater strength at each individual weld in the case of the 24-gage decking.

Figure 8 illustrates a comparison of experimental and calculated ultimate shear-bond stresses for beam test data of Fig. 4. The calculated shear-bond stresses, $v_{uc}$, are obtained from Eq. (8) with constants $K_5$ and $K_6$ resulting from Fig. 4. Similar comparison curves were plotted and are recorded in Reference (4). In all cases, a maximum correlation of ±15% exists between experimental and calculated shear-bond values.

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## SUMMARY

Shear-bond may be classed as a brittle type of failure and is characterized by the formation of an approximate diagonal crack, resulting in end-slip and loss of bond between the steel deck and concrete. The ultimate shear-bond capacity of steel-deck-reinforced systems is a function of the compressive strength of concrete, the depth and width of slab, the thickness of steel deck, the shear span and 2 constants to be evaluated from experimental test data. A ±15% correlation between experimental and calculated shear-bond stresses existed.

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