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Strength Formulations for Composite Slabs

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STRENGTH FORMULATIONS FOR COMPOSITE SLABS

by
L.D. Luttrell* and S. Prasannan**

Introduction

The use of composite concrete-steel slabs depends on an understanding of their responses to load and the ability to predict such responses. Such slabs, formed by placing wet concrete on special steel panel configurations, are generally understood to be flexural members quite sensitive to shear stress transfer between the two components.

Extensive testing programs commonly are used to establish allowable design loads. These programs usually have not focused on behavior so much as on collecting a data base sufficiently large to permit a quasi-empirical design formula development. It is understood that shear transfer is very sensitive to the panel type, its surface conditions, and whether or not it has adequate mechanical interlocking elements - shear lugs.

The purpose here has been to also address a large data base but to focus specifically on the shear transfer mechanism noting that shear is the key to developing tensile forces in the steel section. It is then the key to bending moment and load capacity.

The systems studied have involved slab depths from 4.5 to 7.5 inches, several galvanized steel panel configurations, normal and light-weight concrete, and panels from 16 to 22 gage in thickness. A model is presented here allowing the panel designer to develop an embossment pattern and to assess its merits. He then can predict strengths and place testing in a proper perspective, viz., to use testing for model verification rather than for devleoping the model itself.

Flexural Response

During the loading sequences for composite and essentialy one-way slabs, the response may be quite unlike that of bar-reinforced one-way flat slabs. The difference rests totally within the anchorage development for the tensile reinforcement or, more specifically, in the transfer of forces between the concrete and steel through shear stresses along the interface.

Test load arrangements usually involve line loads P/2 across the slab width B at L' from the center of either end reaction line as in Figure 2. Loads for the present studies were applied using hydraulic rams and controlling displacement throughout the test. In the early load stages, load-deflection response was substantially linear with no major cracks

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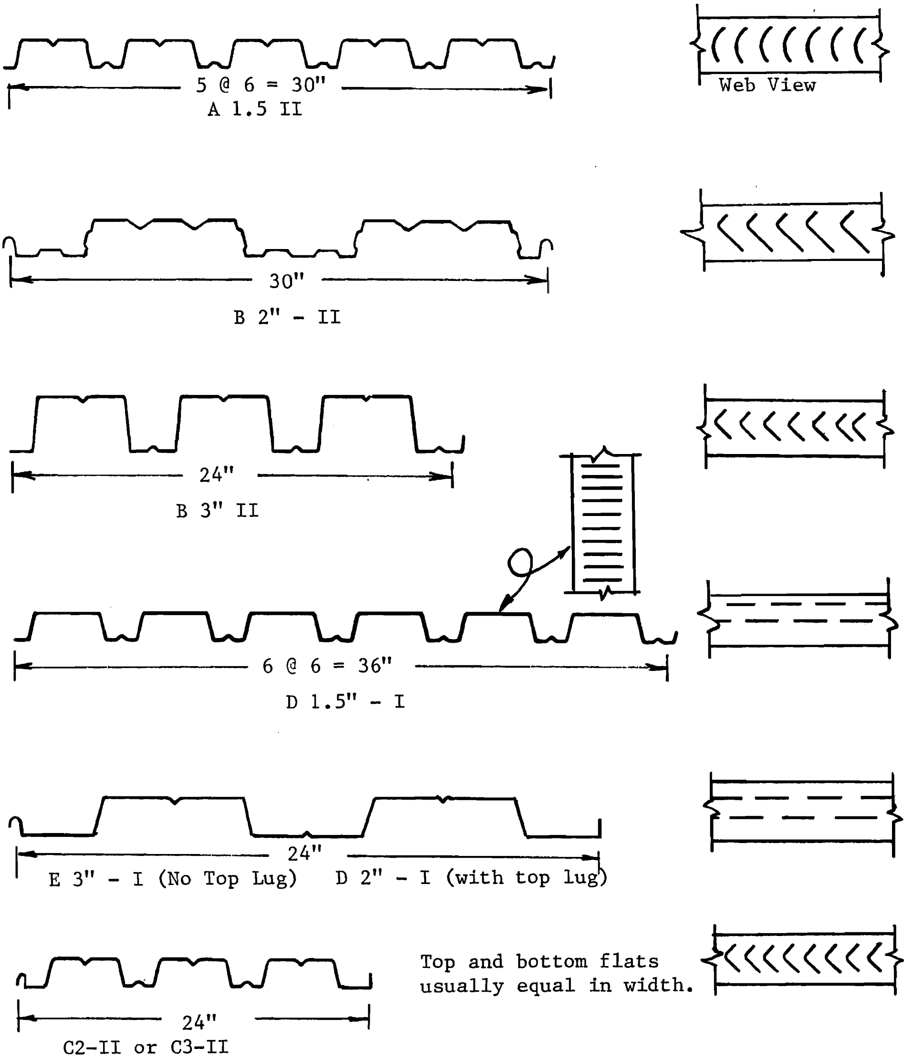

Figure 1. Typical Sections Tested.

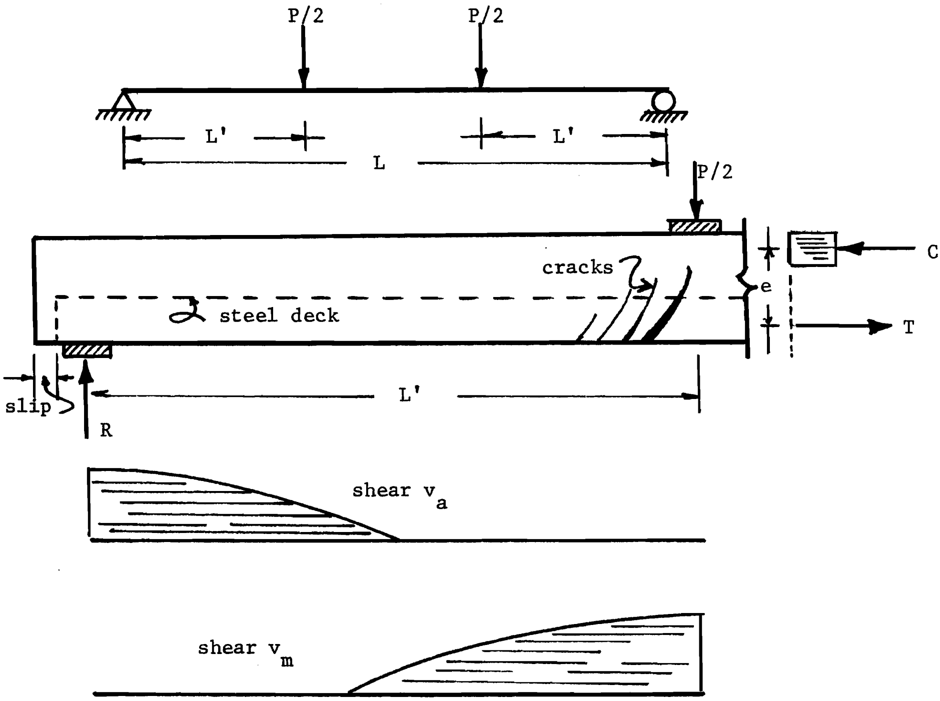

Figure 2. Test slab model.

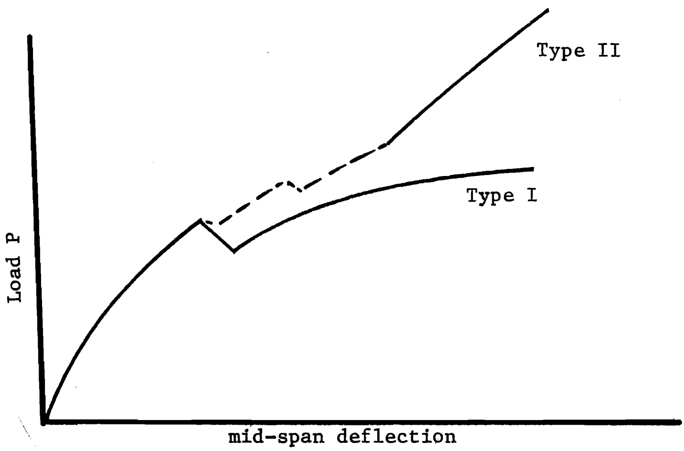

Figure 3. Slab load-deflection response.

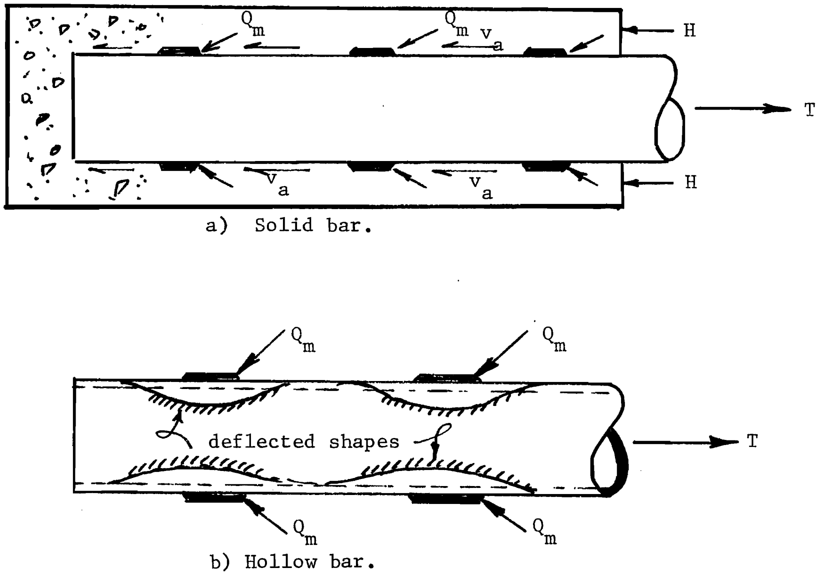

Figure 4. Developed anchor forces Q_m.

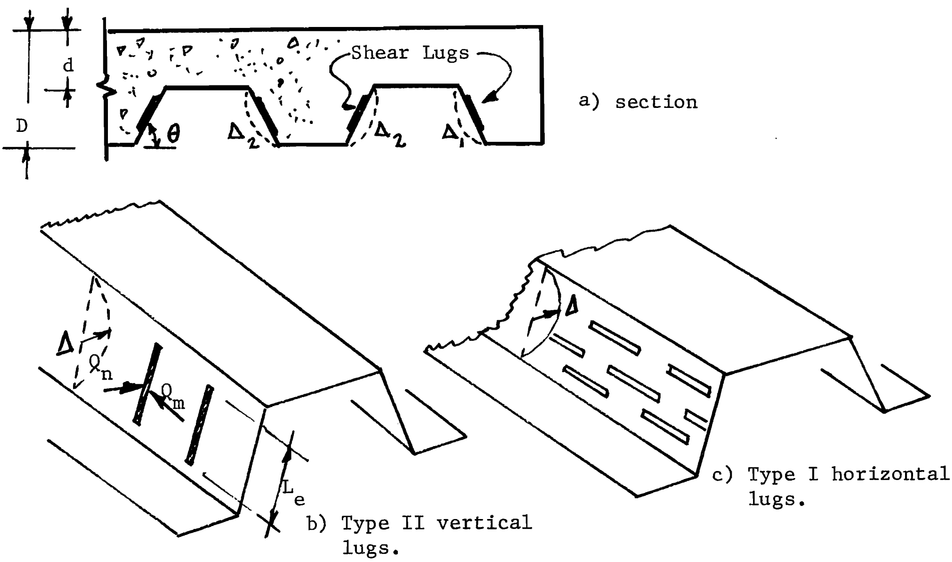

Figure 5. Slab sections.

developing. With the relaxation associated with a crack forming under or near a load line, load resistance dropped and deflections increased. From this point onward, the response was strongly dependent on the types of webs and the embossments used.

In Figure 2, it is clear that the bending moment capacity is M = Te = Ce and that the steel force T is limited by bond anchorage along the steel surface over the shear span L'. The anchorage developed is

$$V_a = bL' \ (Av_a + Bv_m) = T \tag{1}$$

where b is a developed width, L' is the shear span or anchorage length and $Av_a + Bv_b$ is the mix of adhesive and mechanical bond stresses. The mix of these is not known but it is clear that adhesion is the initial strong influence. With minor cracking, adhesion is reduced and stress transfer to shear lugs results if they are sufficient to the task. In Type I response of Fig. 3, they may not be sufficient to assume the transfer and, once slip develops, the slab strength is obtained. When the mechanical lugs or embossments extend well into the concrete and do not move easily away from it, the mechanical bond capacity may be much greater than the adhesive bond resulting in Type II behavior in Figure 3.

The key to obtaining the desired Type II response rests in the lug size and its resistance to lateral movement. Consider a solid deformed steel bar in Figure 4 as being encased in concrete. The force T is resisted by adhesion (v_a) on the surface and forces Q_m at the lugs. Note that Q_m is inclined tending to push the lug away from the concrete and to eliminate the v_a stress transfer. The solid-bar lug is not inclined to move. Were the bar replaced by an otherwise identical but hollow thin tube as in Figure 4b, it is clear that Q_m would tend to disengage the surface more readily.

The composite deck steel panel may be thought of perhaps as a "steel bar almost out of the concrete". Its resistance to disengagement under Q_m forces rests in its own resistance to changing shape. In Figure 5, two general deck types are shown, one having lugs running across the web and the other along it. The mechanical shear force Q_m, inclined to the surface, obviously is dependent on the projected size L_e presented against slip. Q_n is a component of Q_m acting normal to the steel surface. The tendency toward disengagement under Q_n, as the concrete tries to slip over the lug, is a function of web stiffness. Note that Δ for a flat web would be a beam-like deflection such as

$$\Delta = Q_n d^3/KEI \tag{2}$$

where I is proportional to $t^3/12$ per inch along L'. Then with α as an appropriate constant representing the web angle Θ and web edge conditions,

$$\Delta = \alpha Q_n (d/t)^3 (1/E) \tag{3}$$

Clearly the tencency to disengage is much greater in a deeper thinner panel. Note in Figure 5 however, that Type II embossments spanning

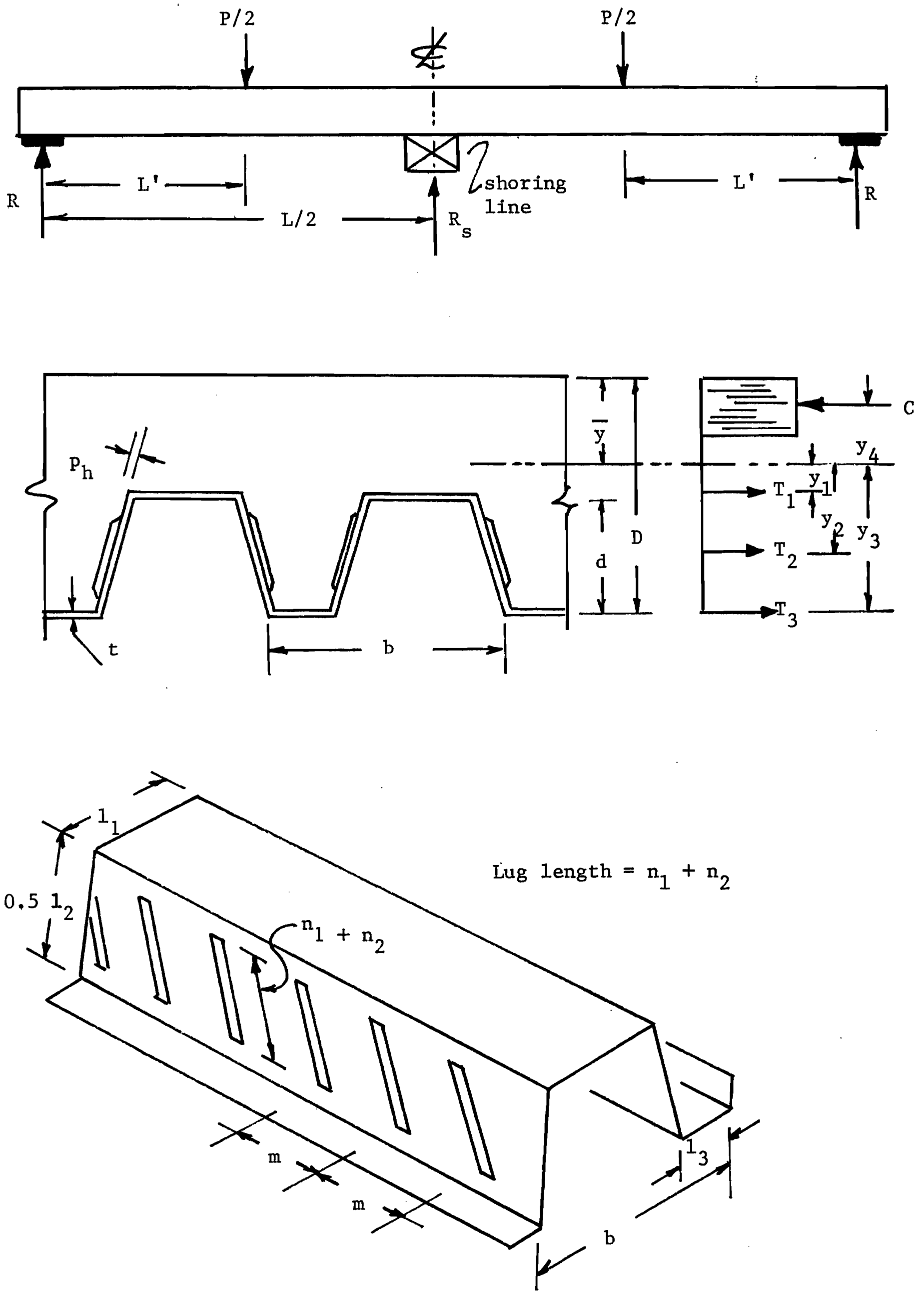

Figure 6. Section details.

generally across the web, are "beam-like" and may stiffen the web appreciably wheras Type I embossments may not. Further in Figure 5, it may be noted that more tendency to disengage exists at slab edges than away from the edge due to anchorage conditions across the bottom flat. The two flute case shown may have two "good" webs of four wheras, if it were twice as wide, it may have six of eight "good webs", etc. Thus the specimen width has much to do with response.

One logical way to approach the formulation for slab strength is to treat the slab as "perfect" and use existing accepted solutions. Then reduce that capacity, through a series of "relaxations" which account for panel shape, lug type, concrete depth, and shear span (or embeddment length for tension reinforcement).

Flexural Strength

The ultimate bending strength of a composite slab may be viewed, in a classic sense, as shown in Figure 6 leading to a flexural capacity,

$$M_f = T_1y_1 + T_2y_2 + T_3y_3 + Cy_4 \tag{4}$$

where the values T_i are forces on steel areas and y their associated lever arms. If no shear-bond limitations existed along L', the sum of $T_1 + T_2 + T_3$ could equal A_sF_y or a value limited by C in compression on the concrete.

Suppose M_f' is a theoretical maximum moment capacity, following Eq. 4, reduced to account of shoring removal moments M_s or other miscellaneous load effects.

$$M_f' = M_f - M_s \tag{5}$$

The theoretical available moment to resist external loads would be some smaller value

$$M_t = KM_f' - k_4S' \tag{6}$$

where $K = k_3/(k_1 + k_2) \leq 1.0$ and is a series of factors accounting for the less-than perfect cross section. The product k_4S' accounts for shear span influences.

Consider Figure 7 where M_f' is the theoretical maximum capacity available. Then M_f'' is the a reduced value accounting for the section itself. S' is a measure and indicator of shear span reduction from the best half-span condition. Though indicated as linear, k_4 probably is not, especially if S' approaches L/2 leaving virtually no shear span on which to develop anchorage.

The various values for k follow from some eighty full scale tests and are not simple.

k_1 and k_2 = deck cross-section factors

k_3 = slab width factor

k_4 = shear span factor

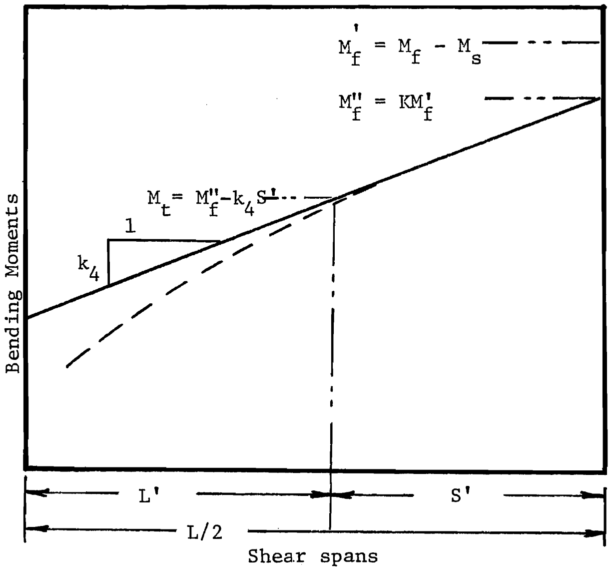

Figure 7. Relaxation model

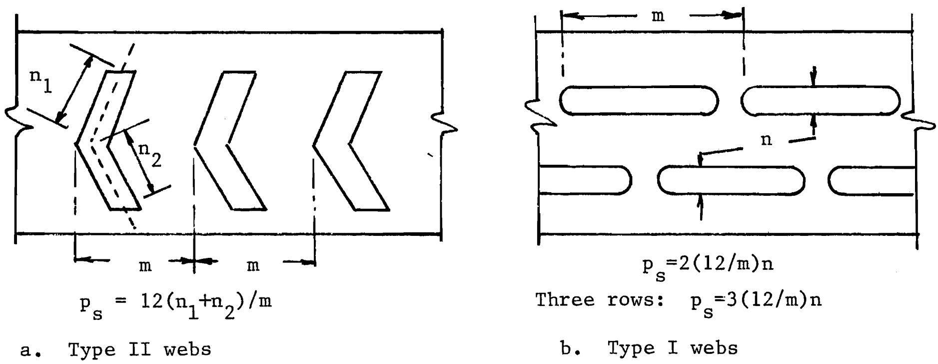

Figure 8. Composite panel webs.

Relaxation Factors (k)

The factor k_1 measures the influence of lug height p_h relative to web dimensions where p_s is the lug length per foot of a web as in Figures 8a and 8b. For all 2" and 3" deep Types I and II panels tested

$$k_1 = \frac{(2.75d - 2.94) - (p_s p_h)^{1/d}}{1.55d - 2.1} \qquad (7)$$

The value can be read directly from Figure 9. Note that lower k_1 values are preferred and that they usually are associated with panels of a shallower depth d.

The factor k_2 is mixed between the stiffness or total thickness of the slab and the steel panel thickness being a general indicator of the tendency of the concrete to separate as a rigid unit from the steel panel. Obviously a smaller k_2 value is desired in Eq. 6. The factor is complex but can be found from Figure 10. For example a 20 gage panel slab having a total depth of 7.5 inches would show an abscissa of 4 and y = +0.30. Then

$$k_2 = 0.30 \sqrt{p_s} d^2/200 p_h \qquad (8)$$

The influence of the number of flutes or corrugations in a system is measured by k_3. As indicated earlier, the edge-most webs may not be too well anchored relative to those away from the edge. The k_3 factor is shown in Figure 11 and ranges up to a maximum of 1.4 which would be expected in a slab made of several properly edge-lapped steel panel units.

None of the k factors are totally independent and k_4 is no exception. It is mixed in both shear span and steel section depth and is expressed as:

$$k_4 = 3 \times 10^5 p_h^4 + (0.9 + 16 p_s p_h^2/\sqrt{d})(30 - X) \qquad (9)$$

where X is the abscissa of Figure 12.

Some clarity to finding the k factors can be added by the following example.

See Figure 6. Deck Type C-2.

L = 12' D = 6" t = 0.0359" (20 gage)

b = 12" d = 3" l_1 = 5" l_2 = 6.4"

l_3 = 5" $A_s = \Sigma lt = 16.4(t) = 0.589$ in.^{2}

F_c = 4000 psi concrete; F_y = 40 ksi

Lug length = $n_1 + n_2$ = 2.40" spaced at m = 9/16"

Lug depth: p_h = 0.075"

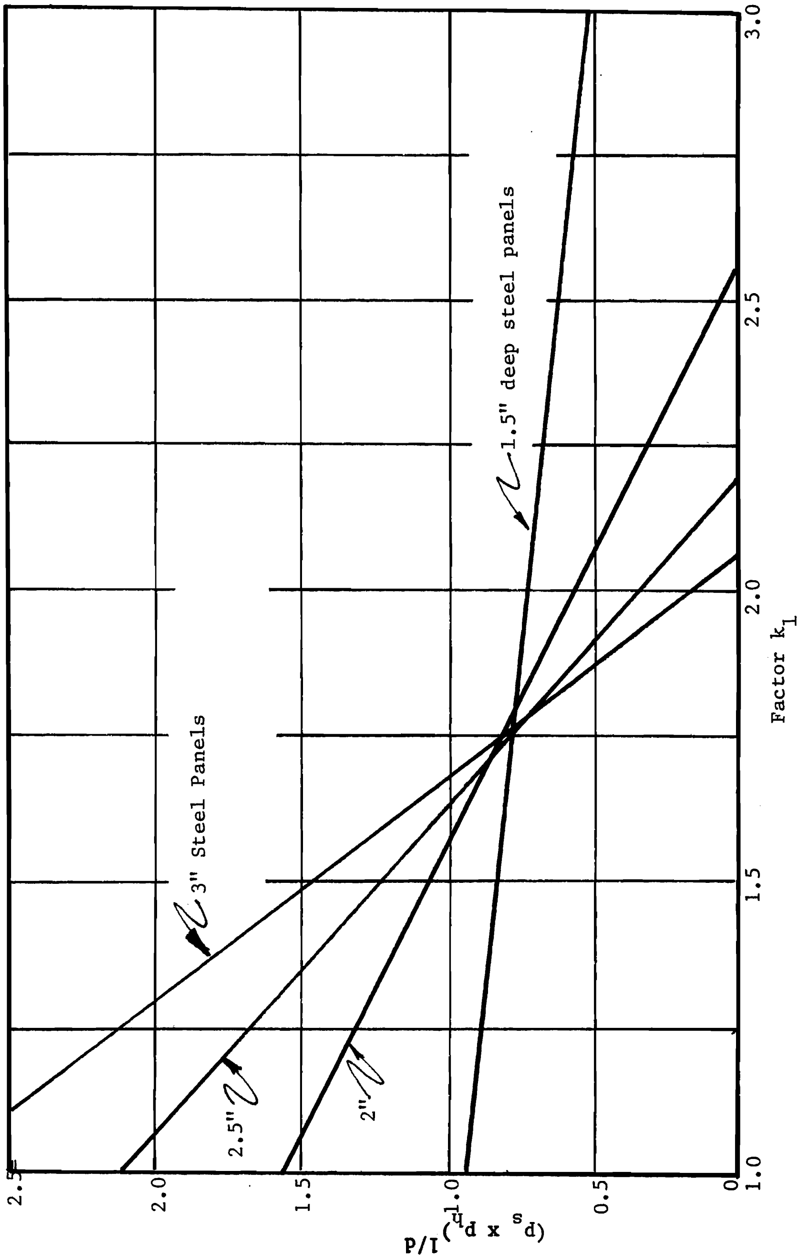
1.5" deep steel panels
3" Steel Panels
2.5"
2"
Factor k_1
$(p_s \times p_h)^{1/d}$
1.0
1.5
2.0
2.5
3.0
0
0.5
1.0
1.5
2.0
2.5

Figure 9.

Fig. 8a: $p_s = 12(n_1 + n_2)/m = 51.2$

Fig. 6: $C = 0.85(0.85)(f'_c)\ b\bar{y} = 34.68\bar{y}(kips)$

$T_y = A_s\ F_y = 23.56$ kips

Then $\bar{y} = 23.56/34.68 = 0.679''$

$y_3 = D - \bar{y} = 5.321''$

$y_2 = y_3 - d/2 = 3.821''$

$y_1 = y_3 - d = 2.321''$

$y_4 = \bar{y}\,(1 - 0.85/2) = 0.390''$

From Eq. 4, $M_f = \Sigma\ T_i y_i + C_4 y_4$ and

$M_f = 0.0359(40)[5(2.321) + 6.4(3.821) + 5(5.321)]$
$+ 34.68(0.679)(0.390) = 99.2$ in.k/unit width.
$= (99.2)(1000)/12 = 8267$ ft.lbs/ft. of width.

With one shoring line at mid-span, the shore would have supported about 5/8 of the slab weight which is about,

$W_D = 150(D - d/2)/12 = 56.3$ psf.

On a one foot strip: $R_s = 0.625(12')(56.3psf) = 422$ lbs. Then the shore removal bending moment is about

$M_s = R_s L/4 = 422(12/4) = 1266$ ft. lbs/ft. of width.

Eq. 5: $M'_f = 8267 - 1266 = 7000$ ft. lbs.

Relaxations: $(p_s p_h)^{1/d} = (51.2 \times 0.075)^{\frac{1}{3}} = 1.566$

Eq. 7: $k_1 = \dfrac{2.75(3) - 2.94 - 1.57}{1.55(3) - 2.1} = 1.47$

Fig. 10: 20 gage 6" Deep Bar, Read y = +0.077

Eq. 8: $k_2 = 0.077\sqrt{51.20}\ (3)^2/(200 \times 0.075) = 0.331$

Fig. 11: Field Conditions (very wide slab), $k_3 = 1.4$

Fig. 12: 20 gage 6" Deep Bar, Read x = 16

Eq. 9: $k_4 = 59.34$ (C-2 curve)

Eq. 6: $K = \dfrac{k_3}{k_1 + k_2} = \dfrac{1.4}{1.47 + 0.33} = 0.78 \leq 1.0$

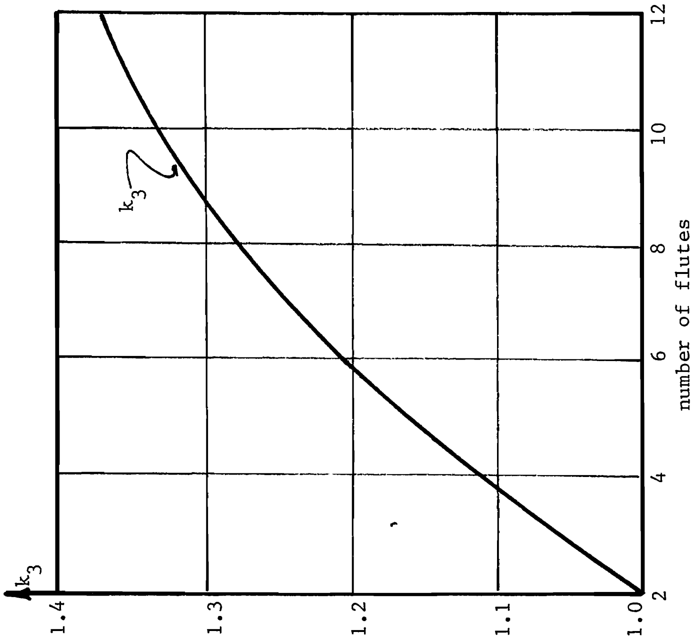

Figure 11. k_3 factor.

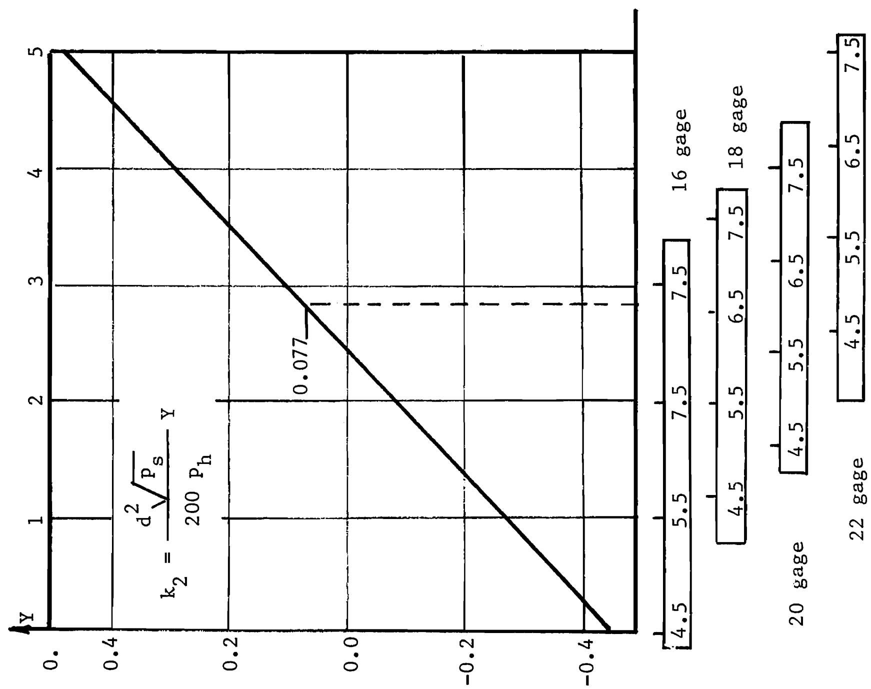

Figure 10. k_2 v. slab depth D.

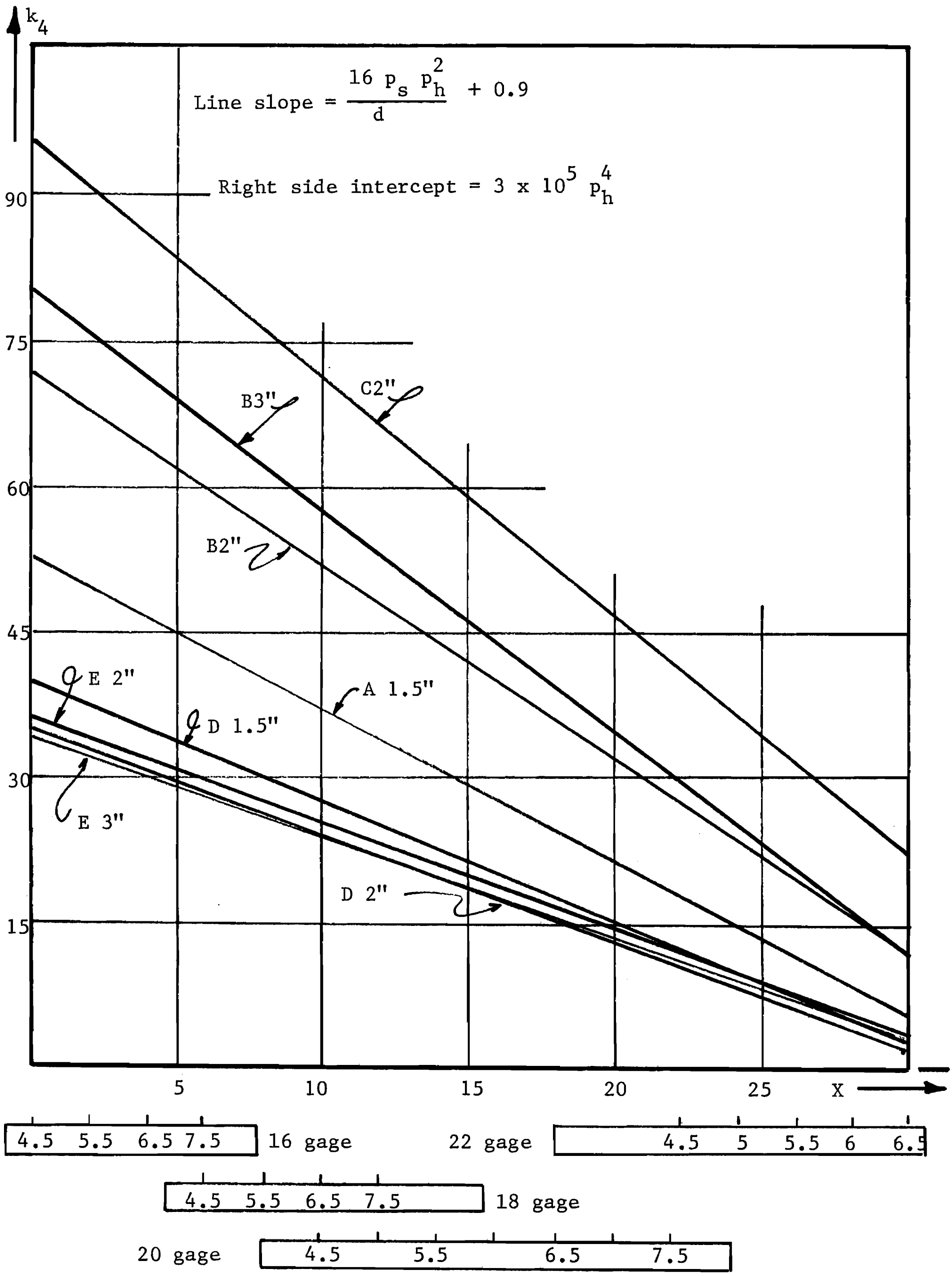

Figure 12. Variations in k_4 with slab depth.

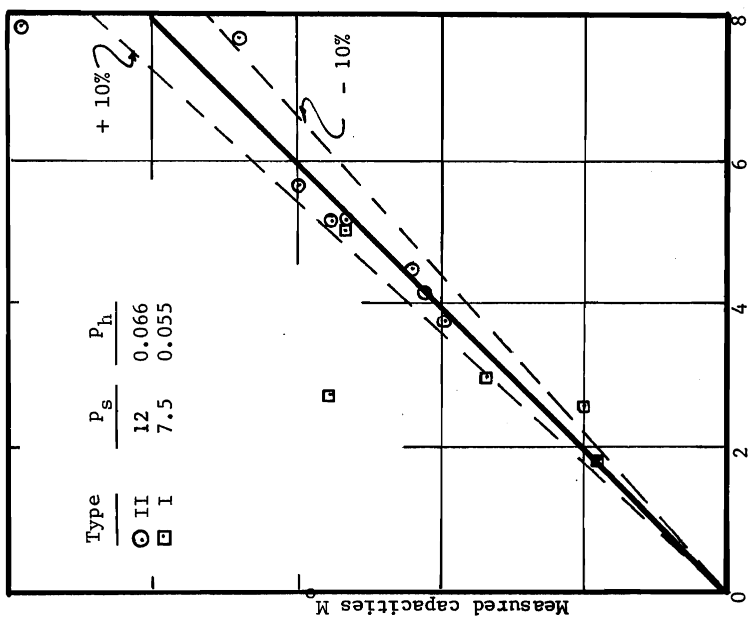

Figure 13. M_o v. M_t comparisons in slabs with 1.5 " steel panels.

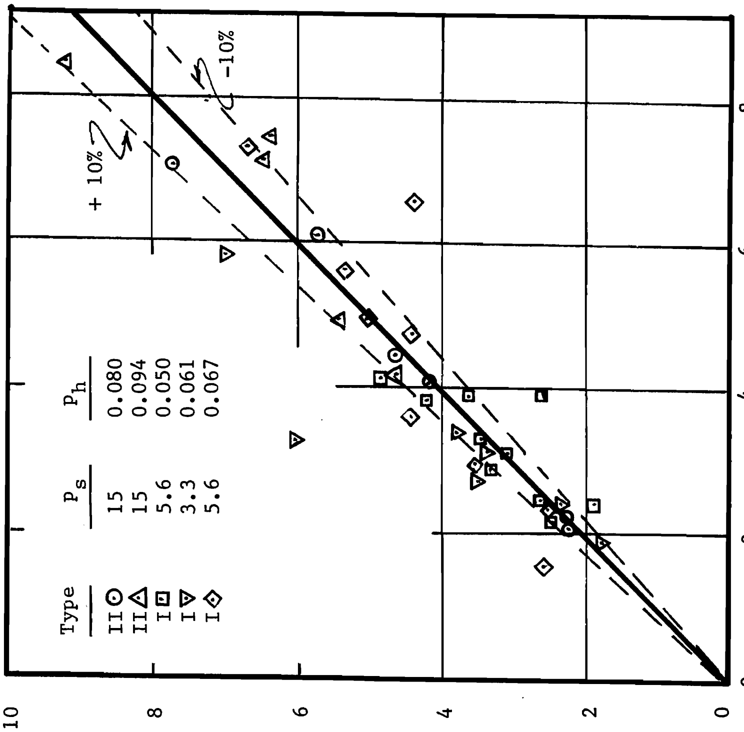

Figure 14. M_o v. M_t comparisons in slabs with 2" steel panels.

Then $M_t = KM_f' - k_4 S' = 0.78\ (7000) - 59.34\ S'$

$$S' = L/2 - L' = 72'' - 48'' = 24''$$

$$M_t = 5460 - 59.34(24) = 4035 \text{ ft.k.}$$

M_t is the theoretical ultimate flexural strength accounting for the panel type and loading conditions. An appropriate safety factor would be required to find the permissible working loads.

Test Program.

Some eighty tests were made using both normal weight and structural light-weight concrete. The former had average strengths f_c' about 4200 psi while the light-weight concrete had $f_c' = 3900$ psi and a weight of 114 pcf. The investigation covered slab depths from 4.5 to 7.5 inches, shear spans L' from 20 to 34% of the span, panel thicknesses from 22 to 16 gage, and several steel panel configurations. The steel panels were from 1.5 to 3 inches deep and had four distinct embossment patterns - two each of Type I and Type II.

The use of Eq. 6 has been demonstrated earlier. Its application to the test data from this program is presented in Figures 13 to 15 where the ordinate M_o represents the observed bending capacity and M_t on the abscissa is the value predicted from Eq. 6. Were the results all to fall on the 45° line, both perfection and euphoria would result.

In Figure 13 the scatter is greatest with Type I systems where there was little or no capacity beyond that at first slip. Such systems are very dependent on adhesive bond and therefore less predictable. Type II slabs, with higher p_s and p_h values, are controlled by mechanical bonding and are less sensitive to slip and sudden failure.

Figures 14 and 15 present similar results for slabs with 2", 2.5", and 3" deep steel panels. Again, the Type II systems respond more predictably.

Specific Comparisons

Figures 13 through 15 do not indicate relative strength values from Type I to Type II slabs. Even with eighty tests, few one-on-one comparisons are possible and the following are presented as indicators though they may be slightly dissimilar.

By comparing the paired entries in Table I, it is clear that the embossment pattern has a very marked influence. Indeed in these particular comparisons, the Type II systems were about 48% stronger than Type I and this is fairly typical of the entire series.

The question of using structural light weight concrete arises. Given that adhesive strength may be more a function of cement paste than some $\sqrt{f_c'}$ compressive strength of the concrete; f_c' may be relatively unimportant to composite slabs. (It's hard to distinguish between $\sqrt{3000}$ and $\sqrt{4000}$ in a world having 10% scatter anyway.)

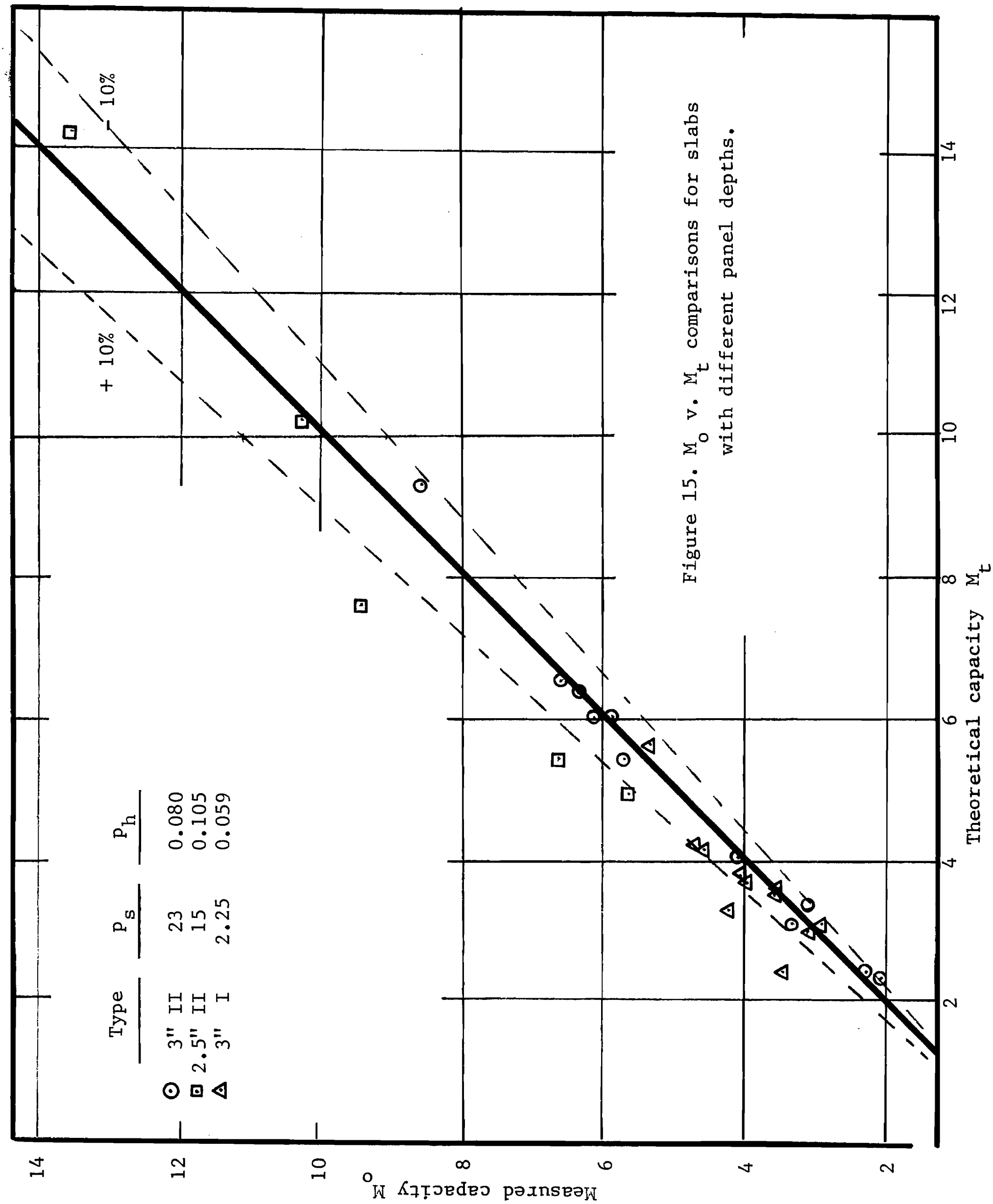

Figure 15. M_o v. M_t comparisons for slabs with different panel depths.

Table I. Strength Comparisons.

No. & Type	t(in.)	d(in.)	D(in.)	L'(in.)	M_o(ft.k)
A1.5 II	0.0359	1.50	4.63	28	4181
D1.5 I	0.0365	1.50	4.38	30	1804
A1.5 II	0.0329	1.50	5.56	36	5500
D1.5 I	0.0355	1.50	5.52	30	1930
B2 II	0.0348	2.00	4.51	32	2674
D2 I	0.0375	2.00	4.50	30	2388
B2 II	0.0465	2.00	6.49	28	5723
F2 I	0.0465	2.00	6.50	36	4431
B3 II	0.0345	3.00	5.50	25	3327
E3 I	0.0350	3.00	5.41	24	2851
B3 II	0.0479	3.00	5.49	28	5667
E3 I	0.0460	3.00	5.47	24	3792
B3 II	0.0479	3.00	7.51	30	6043
E3 I	0.0460	3.00	7.50	36	5215

Table II. Lightweight v. Normal Wt. Concrete Slabs.

No. & Type*	t(in.)	d(in.)	D(in.)	L'(in.)	f_c(psi)	M_o(ft-k)
B2 - II N	0.0472	2.00	4.48	36	4802	4622
B2 - II L	0.0475	2.00	4.56	36	3982	4500
B2 - II N	0.0479	3.00	7.51	30	4802	6043
B2 - II L	0.0472	3.00	7.38	30	4920	5451
R2 - I L	0.0343	2.00	6.42	36	5298	3394
R2 - I L	0.0358	2.00	6.71	36	3892	2917
R2 - I N	0.0466	2.00	5.52	24	5287	3732
R2 - I L	0.0475	2.00	5.51	24	2987	5838
R2 - I N	0.0350	3.00	7.49	36	4635	4669
R2 - I L	0.0351	3.00	7.51	36	3790	3007

*N = normal wt.; L = light weight concrete

In the Type II deck slabs, the N-slabs were about 7% stronger than those having lightweight concrete and the Type I slabs averaged about equal in resistance. While this is an insufficient group from which to reach a conclusion, it at least indicates that structural light-weight concrete slabs are not greatly different from those with normal weight concrete.

Conclusions

The major problem facing a composite panel manufacturer is just how to design the panel and then how to establish safe working-load tables. A very large number of tests may have been required for either of these missions. The commonly used test programs and regression analysis on data do not focus sharply on the deck characteristics and how they change with panel thickness.

The performance factors presented here are cumbersome and efforts continue to simplify them. However, the approach presented does allow for the assigning of composite characteristics to geometry and materials and then directly to predict strength. With such results in hand, a test program can then be used for verification rather than to use it both to predict and verify.

Acknowledgements

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APPENDIX I - NOTATION

A_s	Area of steel deck (in.^{2}/per width b)
b	Width of steel-deck flute (in.)
B	Total width of steel-deck panel (in.)
C	Compression force in concrete (kips)
d	Depth of steel-deck (in.)
d'	Average depth of concrete in slab (in.)
D	Depth of slab (in.)
E	Modulus of elasticity (ksi)
f'_c	Concrete cylinder compressive strength (ksi)
F_y	Steel yield stress (ksi)
k_1, k_2	Deck-type relaxation factor
k_3	Flute relaxation factor
k_4	Shear span relaxation factor
K	Deck 'diameter' relaxation factor = $k_3/(k_1+k_2) \leq 1.0$
P_h	Lug height (in.)
P_s	Projected surface length of lugs (in.), (See Figure 6)
L	Span length of slab (ft.)
L'	Shear span length (in.)
M_f	Ultimate "ideal" flexural moment capacity (ft.k/ft. of slab width)
M_f'	Adjusted flexural moment capacity (ft.k/ft.)
M_o	Observed moment capacity (ft.k/ft.)
M_t	Theoritial moment capacity (ft.k/ft.)
M_s	Moment due to removal of intermediate shoring (ft.k/ft.)
P	Applied external load (kips)
Q_m	Force transfer to web on embossment
R	Slab end reaction
R_s	Shoring line reaction (force/ft. of B)
S'	Conjugate shear span = (0.5L - L') (inches)
t	Base metal thickness (in.)
T	Tensile force in steel panel (kips/width b)
V_a	Anchorage development in shear
v_a	Adhesive shear stress
v_m	Mechanical shear stress

APPENDIX II - REFERENCES.

1. Friberg, B.F., "Combined Form and Reinforcement for Concrete Slabs", Proceedings, American Concrete Institute, Vol. 25, May 1954, pp. 677-716.

2. Johnson, R.P., "Composite Structures of Steel and Concrete", Vol. 1, Halstead Press.

3. Karoubas, K.I., "Dimensional-Statistical Analysis of Steel-Deck Slab Systems", MSCE Thesis submitted to the faculty of West Virginia University, May, 1982.

4. Luttrell, L.D., and Davidson, J.H., "Composite Slabs With Steel Deck Panels", Proceedings of Second Specialty Conference on Cold-Formed Steel Structures, Department of Civil Engineering, University of Missouri-Roulla, 1973, pp. 573-602.

5. Prasannan, S., and Luttrell, L.D., "Flexural Strength Formulations For Steel-Deck Composite Slabs", W.V.U. Civil Engineering Studies, January 1984.

6. Porter, M.L., and Ekberg, CE. Jr., "Design Recommendations for Steel-Deck Floor Slabs", Journal of the Structural Division, ASCE, Nov. 1976, pp. 2121-2136.

7. Porter, M.L., Ekberg, C.E. Jr., Greimann, L.F., Elleby, H.A., "Shear Bond Analysis of Steel-Deck Reinforced Slabs", Journal of the Structural Division, ASCE, December, 1976, pp. 2255-2268.

8. Porter, M.L., and Ekberg, C.E. Jr., "Compendium of ISU Research Conducted on Cold-Formed Steel-Deck Reinforced Slab Systems", Engineering Research Institute, Iowa State University, Ames, Iowa, Bulletin 200, 1978.

9. Plooksawasdi, S., "Evaluation and Design Formulations for Composite Steel-Deck-Concrete Slab Systems", Thesis submitted to West Virginia University in partial fulfillment of the requirements for the degree of Doctor of Philosophy. 1977.

10. Schuster, R.M., "Composite Steel-Deck Concrete Floor Systems", Journal of the Structural Division, ASCE, Vol. 102, No. ST5, May 1976, pp. 899-917.

11. Schuster, R.M., and Ling, W.C., "Mechanical Interlocking Capacity of Composite Slabs", Proc. of Fifth Spec. Conf. on Cold-Formed Strs., UMR, Rolla, 1980.