Composite Slabs with Steel Deck Panels

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By

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Introduction

Cold-formed steel panels are often used as permanent formwork for concrete floors. Advantages are derived from their ability to support newly poured concrete without shoring, to provide open work space below, and perhaps to act as reinforcing for the slab in a composite system.

The investigation reported here is one that involved tests with 1.5" and 3" deep steel panels under various conditions. Galvanized, painted, and bare metal panels were used as forms for concrete slabs varying in total depth from 3.5 and 6 inches. Most tests were made under simple span conditions with symmetric loading near the third points in the span.

Single panel slabs, 24 to 30 inches wide, were most common in this test series but two tests were made on double width specimens and two in a dual span condition. The latter four were designed to evaluate any continuity effects with width or span.

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A simple evaluation procedure was established which can be extended to design conditions. It is based on a limitation of flexural stresses at the extreme fibers of the cross-section, and on shear stresses at the concrete-steel interface.

Experimental Program

Twenty-five composite steel-concrete slabs were cast using only the steel panels as reinforcement. The panels were Bowman V-Grip production run samples having web indentations one inch apart and about 0.075" deep. These indentations extended over one-half of the web depth as shown in Figures 1 and 2.

Concrete consisted of Type 1 Portland cement with limestone aggregate and was moist cured under plastic sheeting for 7 days. It was then air cured until tests were made, 28 days or more later. The usual concrete cylinders were made and tested at the time of the corresponding slab tests. Material property data from cylinders and steel panel coupons are given in Table 1.

All simple span single width slabs were cast in a shored condition over three equally spaced supports. Slabs were moved into position for testing and supported on a simple beam roller system. Soft wooden strips were used between bearing blocks and the panels and allowed to crush during the test. This removed some effects of the uneveness between adjacent ribs caused by end bearing during casting, a situation that would not exist with in situ concrete placement.

Loads were applied symmetrically about midspan through steel beams extending across the slab width. These loads were distributed to the slab by twelve-inch wide wooden pads as visible in Figure 3. Vertical deflection was measured using dial pages, and end slip was monitored by gages resting against the slab end and supported on angles spot welded to the steel panels. One can be seen at the left in Figure 3. Strain measurements were made at midspan on the lower steel surface, the lower side of the upper steel surface, and on the concrete upper surface.

Dual span slabs were cast in an unshored condition on 1.5" steel panels which rested directly on steel support beams as shown in Figures 3 and 4. No welds were made between the panels and supports.

Test Results

Table 1 contains results from standard tensile coupon tests and concrete control cylinders. Tables 2, 3, and 4 contain slab test results. The test numbers are indicative of panel gage, position, span, and total slab depth. For example, 5 (20R8-4.5G) is slab number 5, has a 20 gage panel in the reversed or wide flange down position, an 8' span, a total depth of 4.5 inches, and a galvanized steel panel. The value m is the shear span measured from the center of the load point to the center of the end support. The span L was always measured from center to center of supports. Other symbols are defined in the Table footnotes.

Figures 5 through 7 show load-deflection and end slip curves from typical tests having 1.5" deep galvanized panels. The dashed curves are theoretical and will be discussed later. The end slip is shown magnified five times. Two conclusions are immediately obvious, the onset of slip does not constitute failure, and failure is not sudden. The behavior of slabs having 3" deep galvanized panels was notably different particularly on 24" wide test specimens. The web, being twice as deep, is more susceptible to lateral displacement and edge curling. A view showing edge curl on a 1.5" panel slab is contained in Figure 4. Since the 3" panels have only three ribs, curl at the edge can destroy bond over large regions of the edge ribs placing severe horizontal shear stresses on the one remaining center rib. This happens, of course, in the 30" wide 1.5" panels but they have 6 ribs and the central portion is not so severely overloaded.

The lower curve in Figure 8 shows results from a 24" wide slab. There was very little resistance to load after the onset of slip. As mentioned, 3" webs are more flexible than similarly embossed 1.5" webs. Once slip begins, the concrete rides up on the lug, displaces the web laterally, and destroys shear resistance.

Panel edges as they are lapped in field installations are not free to curl since some restraint will always be provided by the adjacent panel. Two 48" wide two panel slabs with one interior lap were constructed. Two views of these slabs at failure are shown in Figure 9 and 10. Assuming curl and its attendant bond breakdown is restricted to free edges, this system had 4 effective interior ribs or an average of one per foot. This compares to one per two feet in the single width slab. Results from one of the tests are shown in Figure 8. Each panel was separately instrumented resulting in two slightly different load-deflection curves.

The load capacity beyond slip is much improved in the 48" wide specimen over the 24" slab. Its post-slip behavior of increased load above first slip levels, reflects its relatively larger effective horizontal shear region. This implies that a single panel slab test may not relate well to field conditions. A comparison between Figures 6 and 8 shows that failure occurred at 1.5 times the slip load when 3" panels were used but at 3 times slip load with similar 1.5" deep panels. Plate stiffness of the web against lateral displacement definitely affects horizontal shear capacity.

Several slabs were tested that had painted panels. These are distinguished in Table 2 by the letter P in the test number. None of these exhibited good post-slip behavior. Apparently, the paint acts as a lubricant permitting less shear resistance after first slip. Figure 11 shows a typical load-deflection curve. Comparing this to Figure 5 with the results for a similar galvanized panel slab, shows a two fold difference in strength.

It is possible to make three direct comparisons between slabs with painted and galvanized panels.

	Galvanized		Painted	
Test Type	Slip (lbs)	Ult. (lbs)	Slip (lbs)	Ult. (lbs)
20R8-5.0	5560	14060	5560	8060
20R8-4.5	6430	11430	2530	5530
20R8-4.0	6530	11430	2560	5960

The loads at first slip were scattered but on the average, lower for the painted panels. However, ultimate strengths were consistently higher with galvanized panels being about 90% greater. Other indirect comparisons, considering variables in slab configuration, yielded similar results.

Other direct comparisons can be made when panels were coated with a surface treatment of vinsynite primer indicated by V in the test number.

Test Type	Galvanized		Vinsynite	
	Slip (lbs)	Ult. (lbs)	Slip (lbs)	Ult. (lbs)
22R6-4.5	4530	9530	7180	7580
18N8-5.0	6630	20130	7100	14100
20N8-5.0	5560	14060	6060	7560

In these comparisons, the first slip load is about 20% higher when the vincynite coated panels were used. However, the ultimate strength of the galvanized panel slabs is 50% greater indicating that they are superior to both the vincynite and painted panel slabs.

The essential point to observe when making comparisons as above is the notable difference in load-deflection behavior. The 1.5 inch deep galvanized panel systems show well rounded curves and average ultimate strengths over 2.3 times the first slip load. Panels with other coatings provided much less capacity beyond first slip and erratic load-deflection curves thereafter.

Evaluation of Slab Tests

Several different methods of data analysis have been used in this study. Among these was an incremental load-deflection computer analysis made on composite slabs with 1.5" galvanized panels. The strain distribution over the cross-section was assumed linear, with the possibility of a "slip-strain" existing at the concrete-steel interface. The steel was assumed to be elastic-plastic. Due to the small concrete strains encountered, stresses were assumed to be linearly elastic in compression. Tensile capacity in concrete was ignored. A typical stress distribution over the composite section, with yield penetration into the web, is shown in Figure 12.

It is assumed that flexural stresses in either the concrete or steel will govern. These are reduced appropriately for cases where horizontal shear might control. Referring to Figure 13, the distance k to the neutral axis is found by summing forces on the cross-section. Noting geometry, the lower element force $T_1 = f_1A_1$ where f_1 is the lower element stress and A_1 the element's area. The average web stress is:

$$f_2 = \frac{D - k - d/2}{D - k} (f_1) \qquad (1)$$

and the top steel element would have

$$f_3 = \frac{D - k - d}{D - k} (f_1) \qquad (2)$$

where D is the total depth and d is the steel panel depth. The total steel force is:

$$T = f_1A_1 + f_2A_2 + f_3A_3 \qquad (3)$$

and the cross-section resisting moment is

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

where y values are measured from the position of force resultant C to the corresponding force T.

There are cases where horizontal shear governs, i.e., concrete tends to slip horizontally along the steel panel. Figure 13 shows that the <u>average</u> shear stress on a horizontal plane is:

$$v_h = \frac{C}{mb} = \frac{T_1 + T_2 + T_3}{mb} \tag{5}$$

where m and b are the shear span and section width respectively.

If a permissible average shear value V_h is established, the bending moment capacity can be found in terms of v_h when it is less than V_h, the allowable value. Otherwise, the maximum-flexural stress must be reduced until v_h reaches permissible levels. Under these conditions, the moment governed by shear becomes:

$$M_s = M_f\ V_h/v_h \tag{6}$$

Table 2 shows shear span m, section width b, and applied loads at which certain conditions were reached. These included loads at first slip, strains corresponding to 20 or 30 ksi steel stress, and ultimate strength. It can be seen that P_u/P_s values are high having an average value about 2.33. If the first-slip load were taken as a limiting design condition, an average load factor of 2.33 would result.

The computer program, based on the transformed area concept, has been used to study horizontal shear at initial slip and 20 ksi stress levels. The observed slab loads and strains were used resulting in the following values v_s and v_{20}, shear at slip and 20 ksi flexural stress.

Test No.	Gage	m(in)	D(in)	v_s(ksi)	v_{20}(ksi)
10	16	32	5.0	31.3	43.5
11	16	32	5.5	34.1	40.9
13	22	24	4.5	20.9	22.0
15	18	32	5.0	30.0	24.2
21	20	36	4.5	29.8	21.7
23	20	24	5.0	24.8	25.9
24	20	24	3.5	32.6	26.0
20	20	36	4.0	34.4	23.2

If design values for load are limited to produce flexural stresses to 20 ksi, slip would occur in several cases. The average value of v_s is about 30 ksi for these tests and five other similar tests in an independent study. If the design value for horizontal shear is fixed at 30 ksi or the smaller value related to bending, it can be seen that only Tests 13 and 23 would experience slip under design loads and these only a small amount. The recommended design value for use in Equation 6 for 1.5" galvanized panel slabs is:

$$\text{allowable } V_h = 30 \text{ psi} \qquad (7)$$

A completely theoretical evaluation of these tests at a stress level of 20 ksi reveals the following for Test 10:

Theoretical M_{20} = 60.63 in-k/ft

$T_1 + T_2 + T_3$ = 15,300 lbs

v_{h20} = 15300/(32x12) = 39.97 psi > V_h

M_{des} = 60.63 (30.0/39.97) = 45.50 in-k/ft

From test M_u = (26.67 ft k) (12"/ft)/2.5 ft = 128 in-k/ft

Load Factor = 128/45.5 = 2.81

Using the units above, other tests result in:

Test No.	M_{20}	V_{h20}	M_{des}	Test M_{20}	Test M_u	M_u/M_{des}
10	60.63	40.0	45.50	66.03	128.0	2.81
11	58.15	36.9	47.28	64.45	162.5	3.44
13	30.97	29.8	30.97	22.89	45.74	1.48
15	43.70	30.9	42.42	45.70	128.8	3.04
21	32.82	21.1	32.82	33.72	82.37	2.51
23	31.65	29.3	31.65	27.88	70.88	2.21
24	17.34	23.4	17.34	18.63	55.49	3.20
20	28.02	20.5	28.02	31.78	82.32	2.94

The average M_{20} test values are about 2% higher than the theoretical M_{20} values indicating good agreement and a generally conservative approach. Some attempts have been made to evaluate the flexural behavior subsequent to initial slip. This was done using the concepts embodied in Figure 12 and a slip function related to curvature. Figures 5, 6, and 7 show measured slip values essentially linear over a large portion of their length. The slip function was assumed in a linear form as follows:

$$\varepsilon_s = C_1(\phi-\phi_i) \qquad (8)$$

where ε_s is the slip strain, ϕ is curvature, ϕ_i is the curvature at initial slip, and C_1 is a constant. The initial curvature was taken as that value obtained when horizontal shear stresses corresponded to Equation 7.

Figures 5, 6, and 7 show the theoretical behavior with the slip function taken at zero. Additionally, theoretical load-deflection curves with $C_1 = 2.0$ are shown. Slip is seen to reduce stiffness and strength in significant amounts. The slip function chosen permits a reasonable approximation of load-deflection relationships. It is not suggested that this slip function is applicable to all slabs, but that one does exist and may be used if post-slip behavior must be predicted. Similar but less detailed evaluations have been made for slabs with 3" galvanized panels. Results are in Table 3. It appears that the same evaluation procedure will hold though the average horizontal shear value must be restricted to lower values, perhaps 20 to 22 ksi. This would result in a load factor slightly above 2.0.

The effect of paint on panels is to reduce horizontal shear strength and almost eliminate recovery capacity beyond the loads at first slip. This can be seen in the general information contained in Table 4. The blank entries imply that loads did not reach the indicated levels.

The effect of span continuity was measured using Slabs No. 5 and 6. Refer to Table 2 and Figure 3. These were two span duplicates of single span Tests 21 and 23, the major difference being that the two span slabs were cast in an unshored condition. Cracks developed in the unreinforced concrete at the middle support early in the tests. Strain gages indicated yield stress levels at about 10% of the ultimate load.

No extensive analyses of stress distribution on the steel section have been made. However, the concrete supplies lateral support to all steel elements permitting large strains and restrictions against local buckling. The sections did develop a plastic moment of sorts though its exact value is difficult to determine. In the analysis that was made, it was assumed to be $F_y S$ where S was the elastic section modulus of the steel section. Considering the effect of M_p over the support and shored vs. unshored conditions of the slabs, this assumption yielded favorable comparisons between simple and continuous spans.

Conclusions

Galvanized steel panels with shear lugs as shown in Figures 1 and 2 provide reliable reinforcement for concrete slabs in simple bending. Three inch deep panels will span greater unshored distances than 1.5" panels during concrete placement. However, their composite behavior is not as good. The webs are more flexible and subject to lower critical horizontal shear stresses, they limit shear to about 2/3 the value found for similar panels with 1.5" webs.

A simple transformed area concept seems adequate to predict behavior up to common design limits. The strength thus predicted can be modified to reflect shear limitations that might exist. Though test information is limited to two cases, it appears that panels continuous over intermediate supports will develop appreciable support resisting moments in the absence of other steel reinforcing. The value is approximately $F_y S$, the product of yield stress and the elastic section modulus of the panel.

Painted panels, those with a primer coat, or bare metal surfaces do not behave well compared to panels with galvanized coating. Their behavior after initial slip is erratic often resulting in shear failures at low load. This would result in load factor requirements making them uneconomical for use.

Acknowledgements

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List of Symbols

A	Sub-element area, in^2.
b	Section width, in.
B	Bare metal surface
C	Concrete compression force
C_1	Slip constant
d	Steel panel depth, in.
D	Total slab depth, in.
f	Flexural stress, ksi
f'_c	Concrete strength, ksi
F_y	Steel yield stress, ksi
G	Galvanized surface
k	Distance from top to neutral axis, in.
L	Clear span, in.
m	Shear span, in.
M_f	Bending moment, ft-lbs.
M_s	Bending moment at slip, ft-lbs.
P	Painted surface
V	Vincynite surface
v_h	Calculated horizontal shear stress, ksi
V	Allowable horizontal shear stress, ksi
ε_s	Slip strain (in/in)
ϕ	Curvature

Table 1. Test specimen material properties

Test No.*	t(in)**	F_y(ksi)	F_u(ksi)	% elong.	f'_c(psi)
5 (20R8-4.5G)	0.0340	48.5	58.7	29.7	5000
6 (20N8-5.0G)	0.0339	47.9	58.5	28.9	4940
10 (16R8-5.0G)	0.0564	48.6	59.9	29.7	3540
11 (16N8-5.5G)	0.0564	49.1	60.5	30.5	3760
13 (22-R6-4.5)	0.0312	48.6	59.6	29.7	3370
15 (18N8-5.0G)	0.0489	50.5	60.0	29.7	3370
20 (20R8-4.0G)	0.0337	47.3	57.9	28.2	3370
21 (20R8-4.5G)	0.0333	47.5	58.1	28.9	3370
23 (20N8-5.0G)	0.0338	47.2	58.8	26.6	3370
24 (20N8-3.5G)	0.0339	47.8	57.5	28.9	3370
8 (22R6-4.5V)	0.0288	43.1	54.7	38.3	3450
9 (18N8-5.0V)	0.0466	45.7	57.9	36.3	3370
14 (18R8-5.0B)	0.0456	43.8	51.9	36.7	3750
17 (20R8-5.0P)	0.0353	43.3	53.0	33.6	3750
18 (20R8-4.5P)	0.0352	44.3	52.4	36.0	3750
19 (20N8-5.0V)	0.0346	45.1	53.3	36.0	3750
22 (18-8-5.5B)	0.0448	41.7	52.4	36.8	3750
25 (20R8-4.0P)	0.0344	44.4	53.0	36.8	3750
1 (18-10-6.0G)	0.0444	38.6	50.8	33.6	5595
2 (20-10-5.0G)	0.0356	41.1	53.7	32.1	5566
3 (20-10-5.0G)	0.0360	41.4	54.0	31.3	5585
4 (18-10-6.0G)	0.0440	38.2	50.8	32.8	5470
7 (18-10-5.0G)	0.0439	37.7	50.2	36.0	3565
12 (16-8-6.0G)	0.0614	41.3	50.1	37.5	3880
16 (16-8-5.0G)	0.0614	40.4	50.0	34.4	3370

*The first digit is the assigned laboratory test number.

**Base metal thickness (in.).

Table 2. Results form test with 1.5" galvanized single panel slabs.

Test No.*	m	L	b	P_s	P_{20}	P_{30}	P_u	M_s	M_{20}	M_{30}	M_u
5 (20R8-4.5G)	36	96	30	8415	6078	8481	14875	—	—	—	—
6 (20N8-5.0G)	24	96	30	9390	10420	13400	21990	—	—	—	—
10 (16R8-5.0G)	32	96	30	7400	10312	14405	20000	9842	13756	19216	26666
11 (16N8-5.5G)	32	96	30	8400	10064	14347	25400	11172	13426	19138	33860
13 (22R6-4.5G)	24	72	30	4530	4770	6150	9530	4530	4770	6150	9530
15 (18N8-5.0G)	32	96	30	6630	7140	10000	20130	8838	9520	13333	26830
20 (20R8-4.0G)	36	96	30	6530	4410	5770	11430	9795	6620	8660	17150
21 (20R8-4.5G)	36	96	30	6430	4686	6086	11430	9645	7026	9120	17160
23 (20N8-5.0G)	24	96	30	5560	5808	7153	14060	5560	5808	7153	14060
24 (20N8-3.5G)	24	96	30	4860	3882	5158	11560	4860	3882	5158	11560

*m, L, b: Shear span, span, nominal width respectively (in.).

P_s, P_{20}, P_{30}, P_u: Applied jack load at slip, load at 20 and 30 ksi and ultimate (lbs.).

M_s, M_{20}, M_{30}, M_u: Corresponding bending moments (ft.-lbs.).

Test 5 and 6 were dual span tests.

Table 3. Results from composite slab tests using 3" galvanized panels with single and double width on single spans.

Test No.*	m	L	b	P_s	P_{20}	P_{30}	P_u	M_s	M_{20}	M_{30}	M_u
1 (18-10-6.0G)	42	120	24	5800	4140	5820	7100	10150	7250	10200	12410
2 (20-10-5.0G)	42	120	24	4000	2260	3363	4800	7000	3955	5880	8400
3 (20-10-5.0G)	42	120	48	8100	5051	8105	10200	15400	8850	14200	17850
4 (18-10-6.0G)	42	120	48	9900	9467	11360	15300	16800	16580	19900	26800
7 (18-10-5.0G)**	42	120	24	5600	4940	6510	7600	9800	8645	11393	13300
12 (16-8-6.0G)	32	96	24	7930	6306	8062	11430	10547	8400	10750	15250
16 (16-8-5.0G)	32	96	24	3030	3030	3900	6730	4030	4030	5200	8970

* m, L, b: Shear span, clear span, nominal width respectively (in.).

P_s, P_{20}, P_{30}, P_u: jack load at first slip, load at 20 and 30 ksi and ultimate (lbs.).

M_s, M_{20}, M_{30}, M_u: corresponding bending moments (ft.-lbs.).

**Panel installed wide flange down.

Table 4. Results from composite slab tests with panels having various types of coating.

Test No.*	m	L	b	P_s	P_{20}	P_{30}	P_u	M_s	M_{20}	M_{30}	M_u
8 (22R6-4.5V)	24	72	30	7180	—	—	7580	7180	—	—	7580
9 (18N8-5.0V)	32	96	30	7100	7470	10838	14100	9443	9950	14436	18782
14 (18R8-5.0B)	24	96	30	5810	10430	14385	17810	5810	10430	14385	17810
17 (20R8-5.0P)	24	96	30	5560	—	—	8060	5560	—	—	8060
18 (20R8-4.5P)	36	96	30	2530	4380	5074	5530	3795	6750	7600	8300
19 (20N8-5.0V)	24	96	30	6060	—	—	7560	6060	—	—	7560
22 (18-8-5.5B)	32	96	24	6560	—	—	6810	8725	—	—	9090
25 (20R8-4.0P)	36	96	30	2560	3642	4560	5960	3840	5130	6840	8940

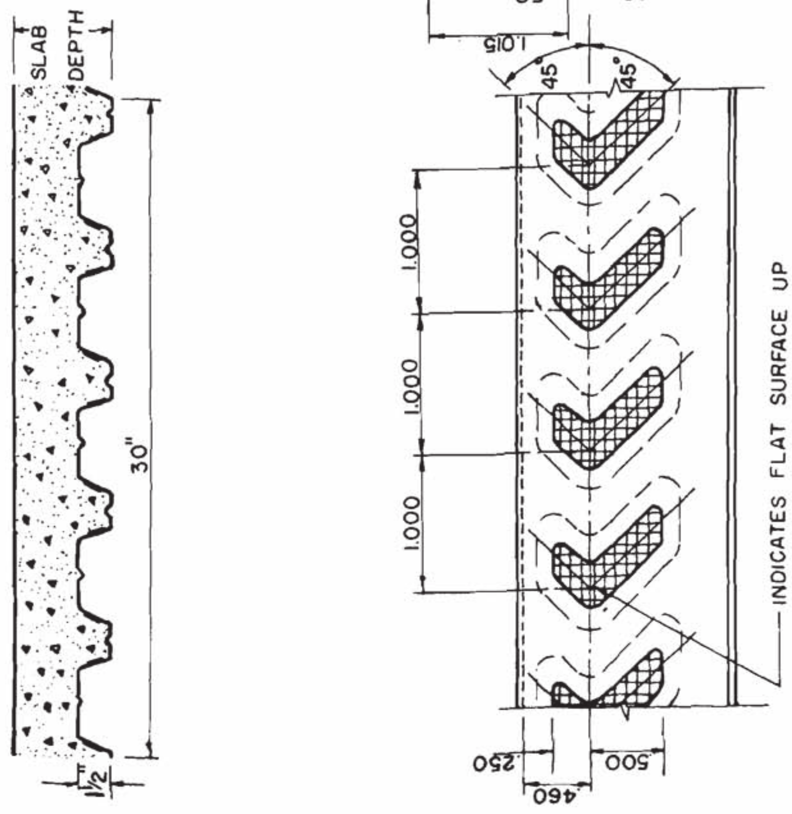
* m, L, b: Shear span, clear span, nominal width respectively (in.).

P_s, P_{20}, P_{30}, P_u: Jack load at first slip, load at 20, 30 ksi and ultimate (lbs.).

M_s, M_{20}, M_{30}, M_u: Corresponding bending moments (ft.-lbs.).

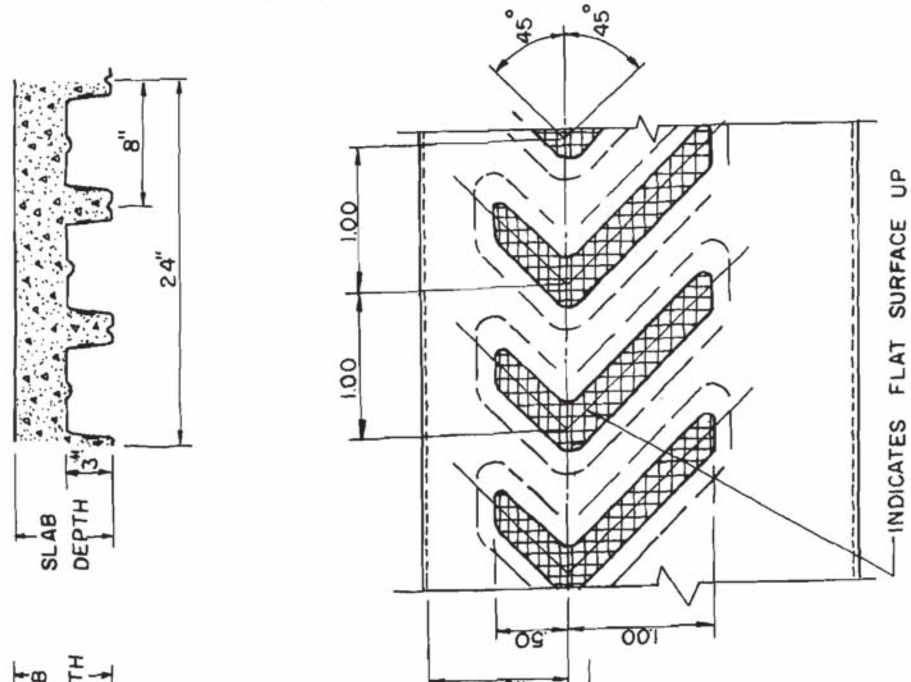
No. 22 was fabricated using a 3" deep panel.

Blank spaces in load columns indicate these loads were not attained.



1.5" COMPOSITE PANEL

Fig. 1. Composite system with 1.5 inch panels.



3.0" COMPOSITE PANEL

Fig. 2. Composite system with 3.0 inch panels.

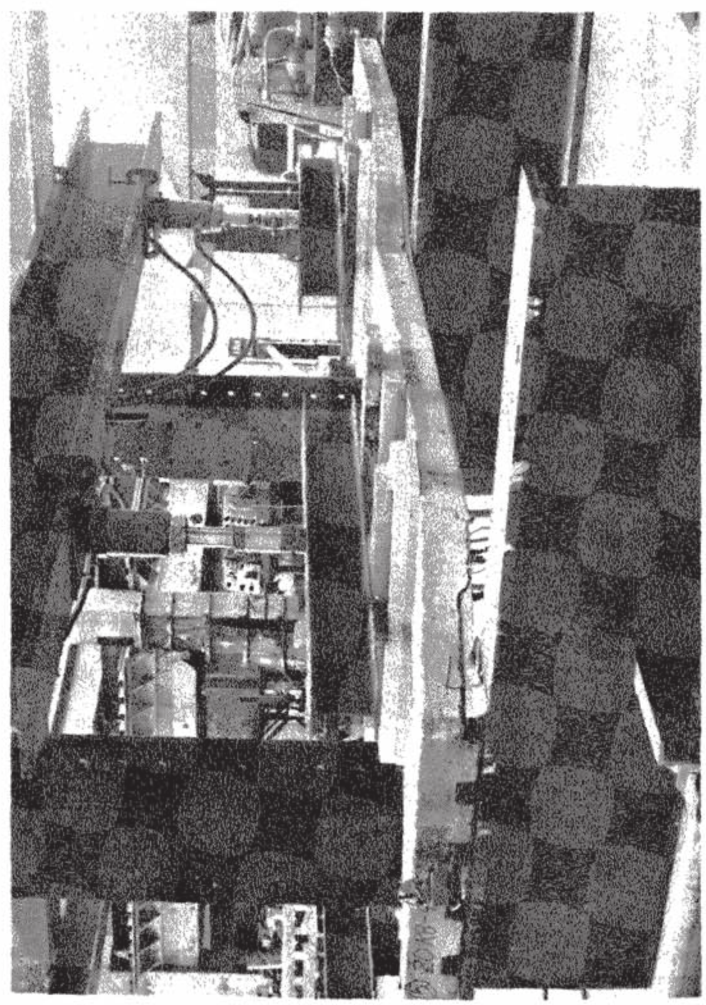


Fig. 3. Dual span Slab 20R8-4.5 at failure.

Fig. 4. Dual span slab showing edge curl and end slip.

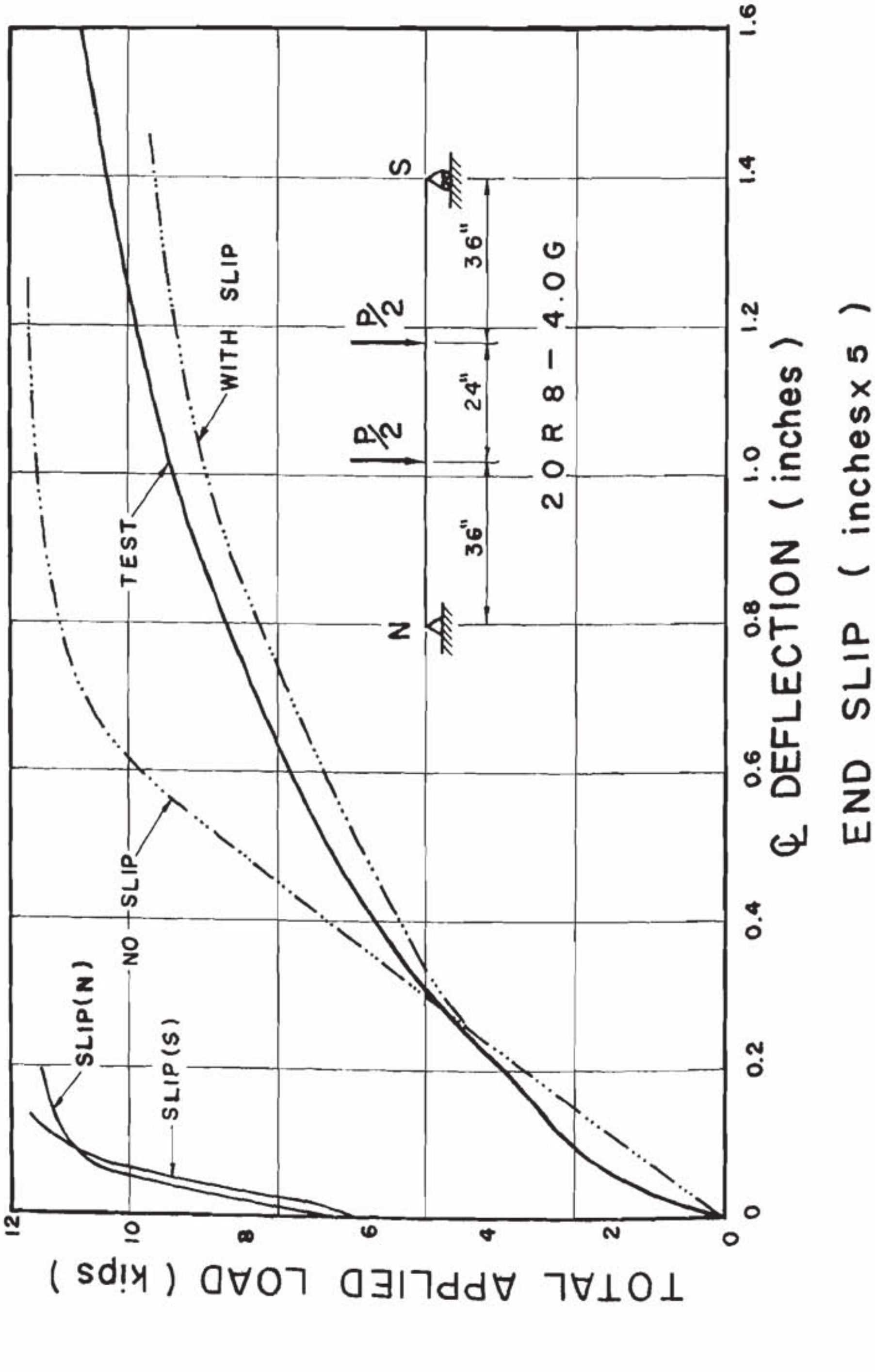


Fig. 5. Load-deflection curves for 20R8-4.0G.

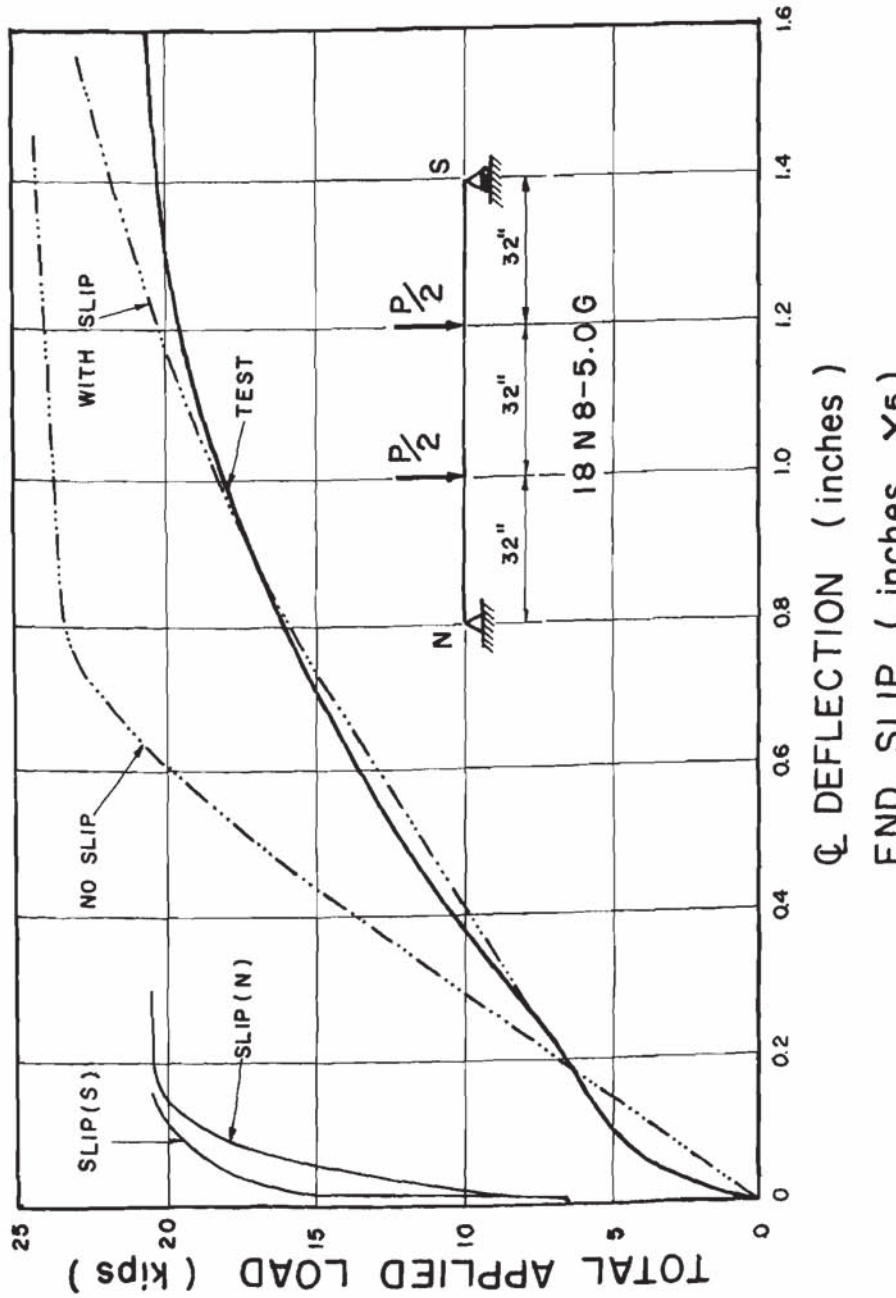


Fig. 6. Load-deflection curves for 18N8-5.0G.

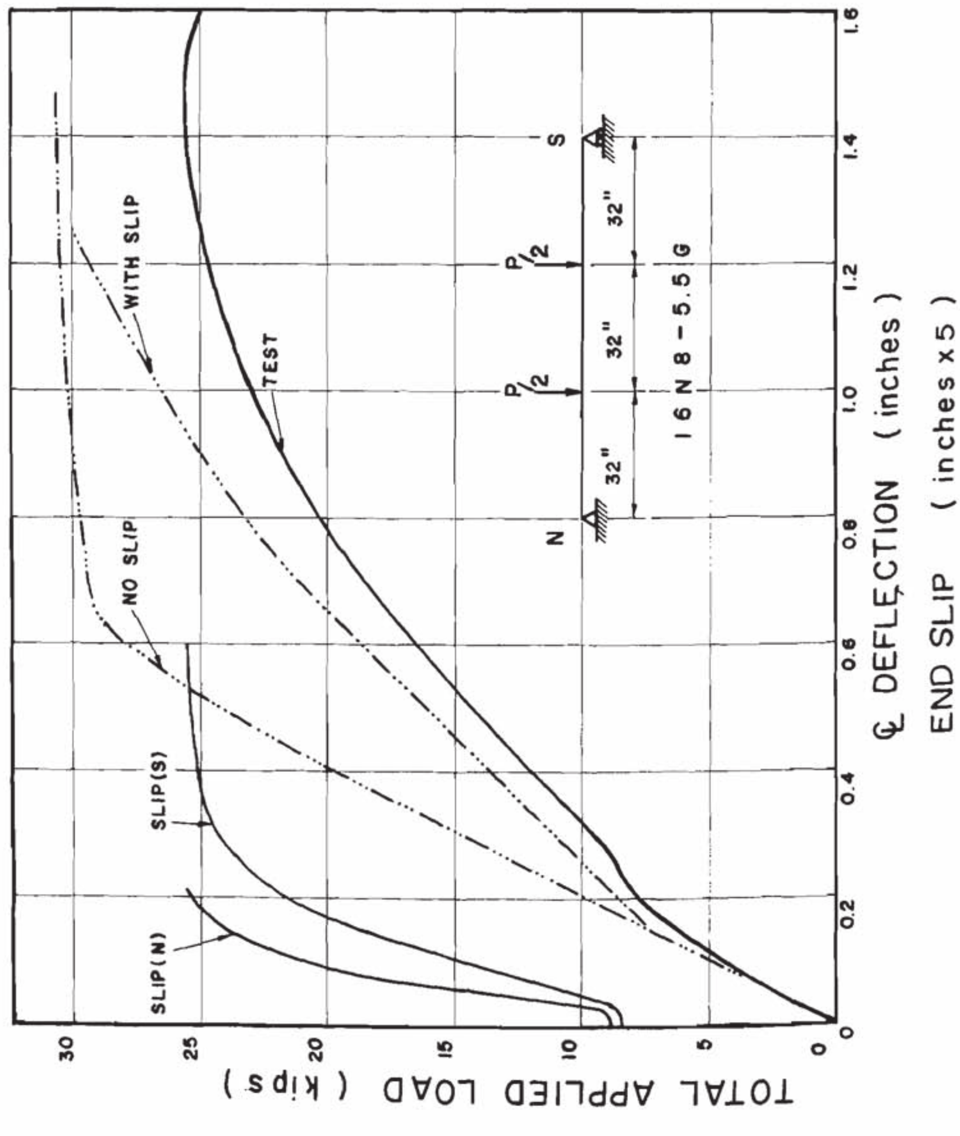


Fig. 7. Load-deflection curves for 16N8-5.5G.

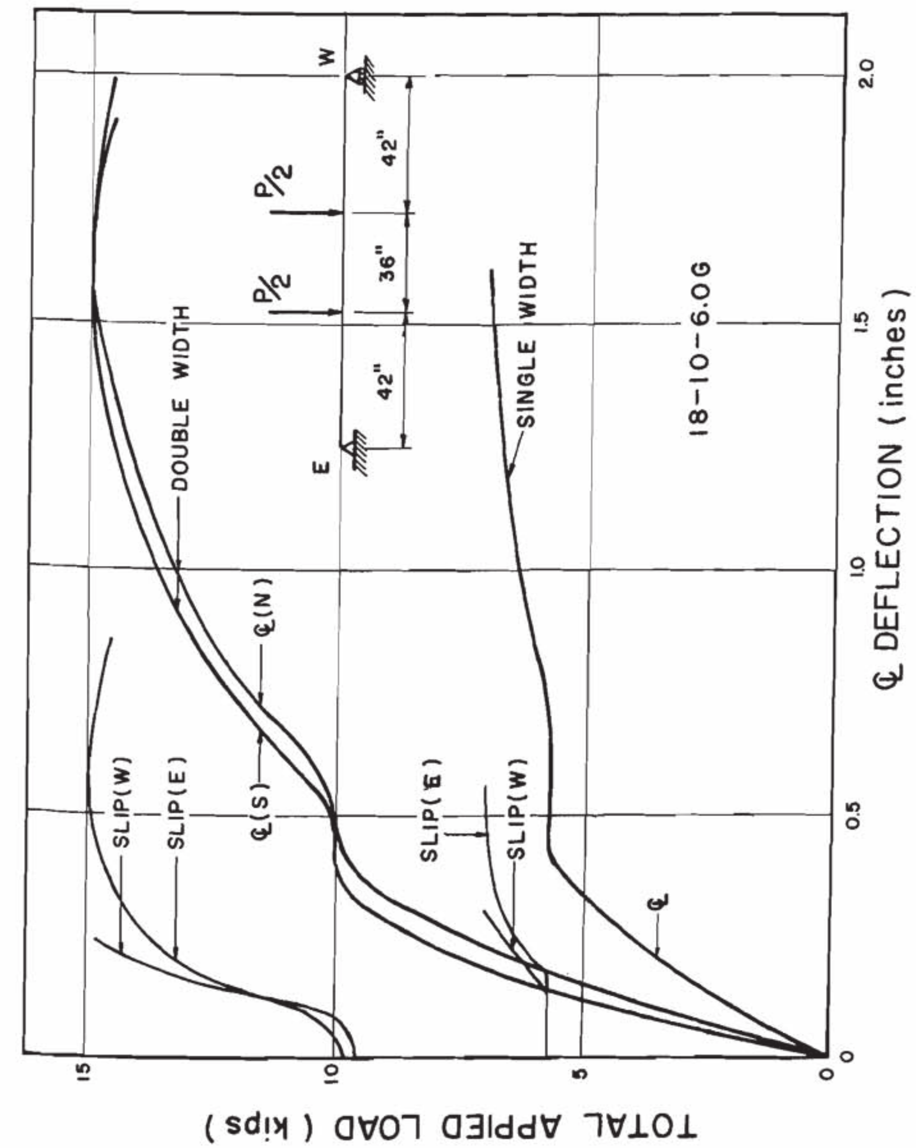


Fig. 8. Single and double width slab comparisons.

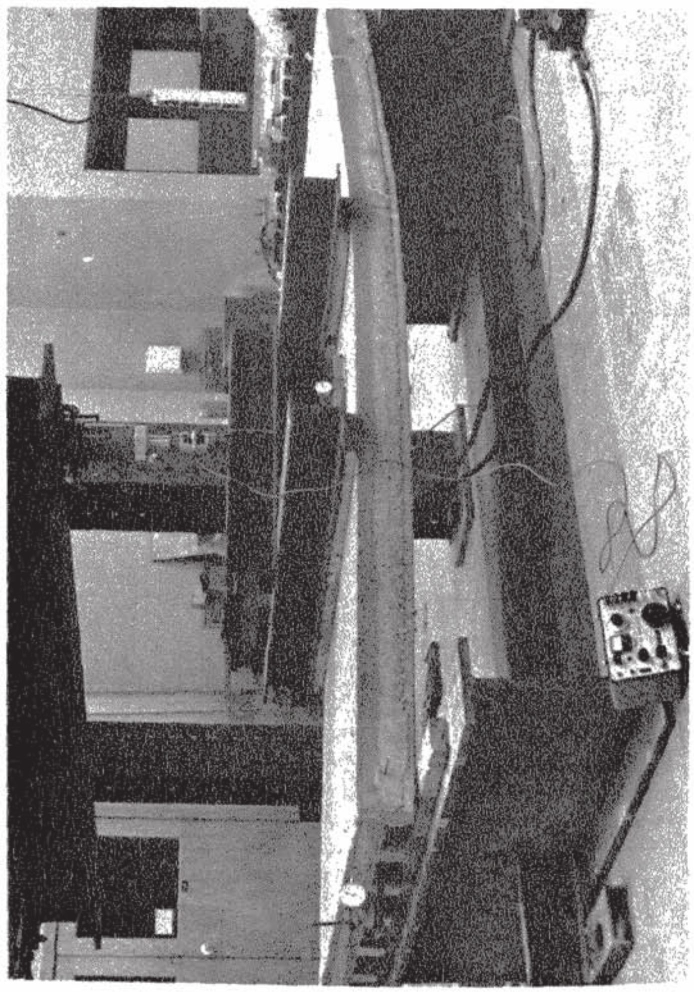


Fig. 9. Test arrangement for double width (48") slabs.

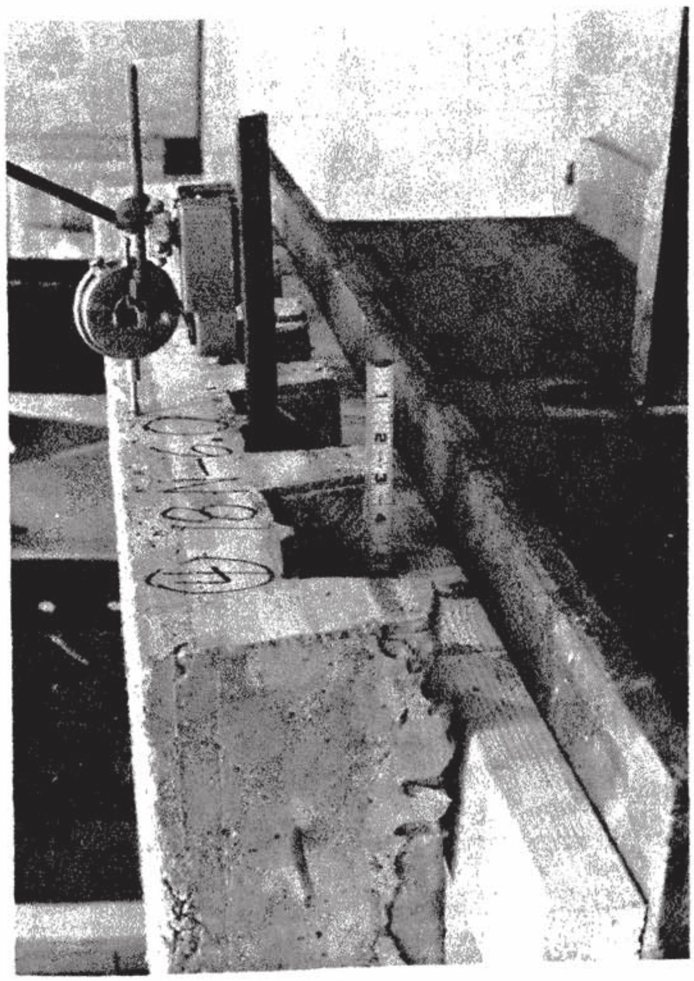


Fig. 10. Slip condition at failure for slabs with 3" panels.

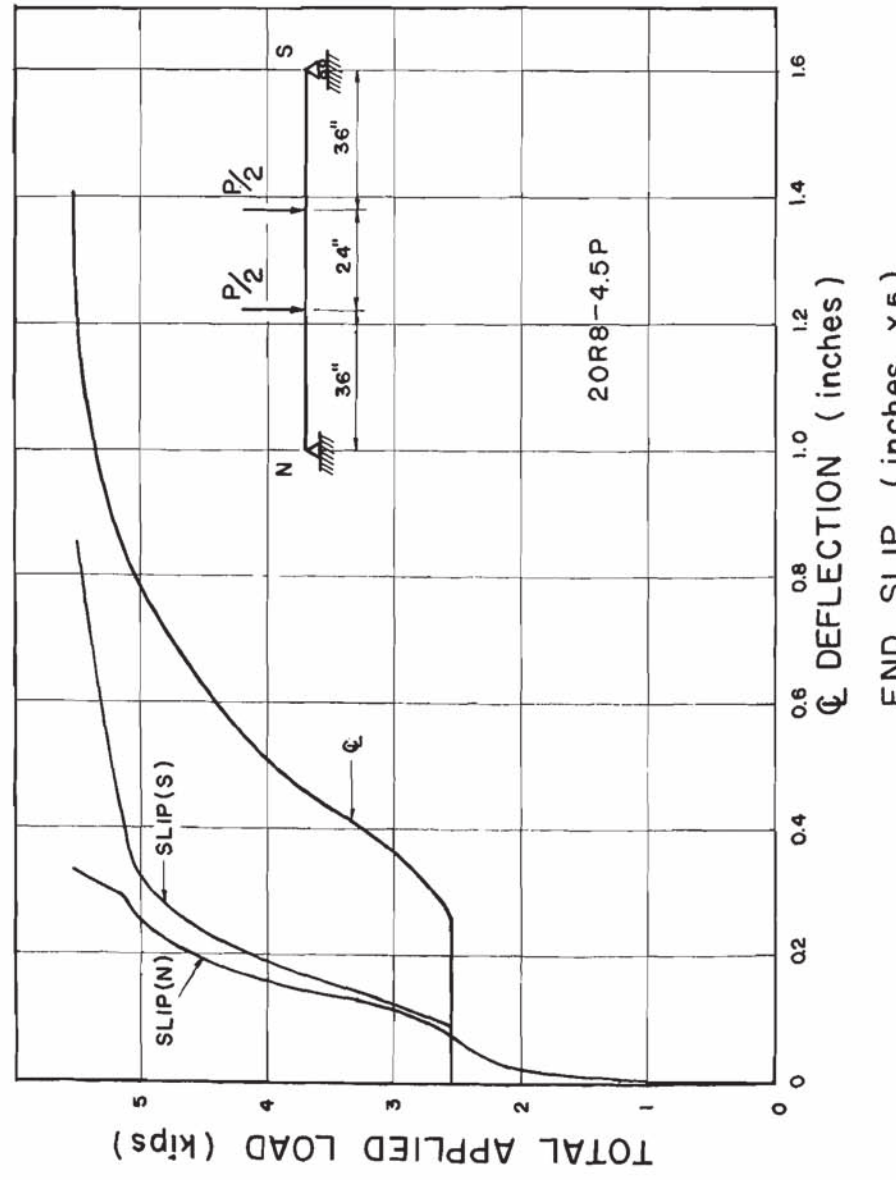


Fig. 11. Load-deflection curve when painted panels were used.

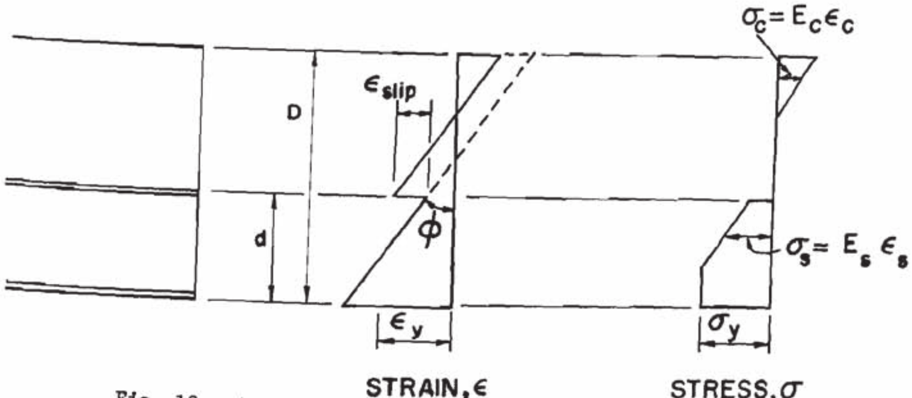


Fig. 12. Assumed strain distribution with slip strains.

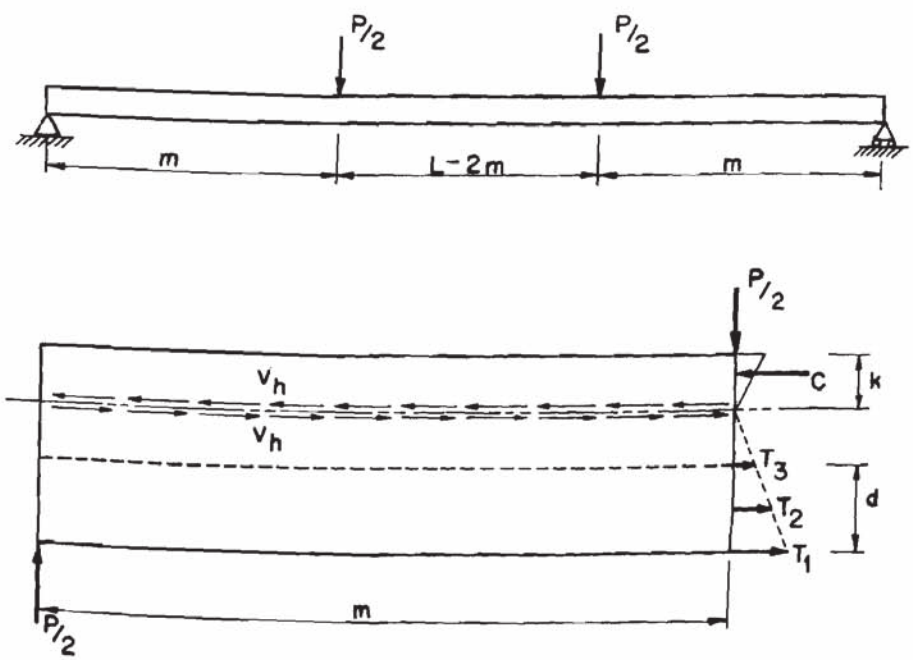


Fig. 13. Equilibrium condition in shear span.

COLD-FORMED STEEL RACK STRUCTURES

by

T. Peköz[1] and G. Winter[2]

INTRODUCTION

The process of moving almost all manufactured goods from the producer to the consumer involves the need for storage somewhere along the line. Nationally, a tremendous volume of material is being stored in this process at any given time, and the cost of such storing is by no means a negligible part of the total cost to the consumer. While earlier storage and warehousing relied on made-to-order shelving and other devices, the need for improvement in storage and in handling efficiency called for increasing mechanization on the one hand and increasing density of storage on the other. Both these requirements could be met only by a highly engineered development of industrial storage rack facilities. In consequence, the establishment of the storage rack manufacturing industry as an identifyable branch of the nation's multi-billion dollar materials handling industry occured during the early 1960's.

The requirements for a storage system vary widely with the nature of the storage situation. However, a general criterion is the ability to store as much material as possible in a given volume

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