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Composite Cold-formed Steel-concrete Structural System

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Composite Cold-Formed Steel-Concrete Structural System

by

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ABSTRACT

Composite beams and columns are developed using soffits made of cold-formed steel sections and cast-in-place concrete.

Soffits made in the form of stiffened channels with embossments performed well as integral parts of composite beams or columns. The combined action of the embossments and the channel's lips lead to very good bond characteristics between the soffit and the concrete. This can be explained, that the concrete has to be lifted up in order to slide over the embossments, while this movement is restrained by the lips of the channel.

Test results show good characteristics of the composite beams, especially with regard to their ultimate load carrying capacity and bond strength. The experimental study shows that by replacing the standard reinforcing bars by cold-formed steel sections of equal areas, the structural performance of the beams could be almost unchanged, while saving is achieved in the cost of forms and shorings.

Similar system is also applied to build columns with channels placed at two parallel faces. Preliminary testing show the performance of the composite columns to be comparable to identical ones reinforced with deformed bars. Using composite columns leads to considerable savings in time and material of construction due to the elimination of the ties as well as part of the forms.

INTRODUCTION

The construction of cast-in-place concrete beams requires the installation and removal of form work and shoring, which constitutes a considerable part of their cost. The construction cost can be reduced by using soffits made of cold-formed steel sections as forms to carry the wet concrete and then to become integral part of the beams, Fig. 1a (1). Experimental studies (1, 3, 8) show good characteristics of these beams especially with regard to their ultimate load carrying capacity, bond strength, as well as the crack width and spacing in the concrete components. The study shows that by replacing the standard reinforcing bars by a cold-formed steel section of equal area, the structural performance of the beam could be unchanged while saving is achieved in the cost of forms and shoring.

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In a similar manner an alternate design for reinforced concrete columns can be made by replacing the longitudinal reinforcing bars by cold-formed steel sections placed at two parallel sides of the column, Fig. 1b (6). Herein, the steel sections replace the forms at two sides. In addition, major saving in the construction cost can be achieved by eliminating the ties required for standard reinforced concrete columns. The main function of the ties is to laterally support the longitudinal reinforcing bars in order to reduce their unsupported length and prevent their local buckling. The need for these ties is eliminated (or reduced) due to the continuous bond between the concrete and the steel soffits, as well as, due to the relatively high rigidity of the soffits when compared with standard reinforcing bars.

The present paper outlines the development of cold-formed steel soffits in composite action with concrete beams and columns. It presents tests conducted showing the feasibility of this type of composite components and provides the basic information on their performance and characteristics.

A - SOFFIT SECTIONS

The cross-section of cold-formed steel soffits can take the form of unstiffened channel, stiffened channel, or two angles connected back to back, Fig. 1a. Experimental studies (3, 6) show that the stiffened channels are the most suitable sections for use in composite beams. Furthermore, the performance of the channels can be considerably improved by forming embossments as proposed in Figs. 2a, b. At bond failure the concrete has to be lifted off the soffit in order to slide over the embossments while this movement is restrained by the lips of the channel. This leads to the increase in the bond strength between the soffit and concrete.

The present paper reports a testing program using soffits of stiffened channels. These channels were either plain (C.P.); or provided with rounded embossments (C.R.); or provided with oval embossments (C.O.). The dimension and detailed properties of the channels tested are given in Table No. 1.

B - COMPOSITE BEAMS

The function of cold-formed steel soffits in composite beams is similar to that of cold-formed steel panels in composite deck floors (7). In both cases the steel performs a dual role as form during construction and as positive reinforcement for the deck or beam. However, it should be noted that the two systems differ in the mechanism of grip and bond between the steel and concrete. The uplift of the concrete and its separation from the steel, which takes place at bond failure in composite decks, is restrained by the lips of the soffits in composite beams. Also, the ratio of depth of the steel deck to the total depth of the deck is considerably high when compared with the corresponding ratio in beams. Therefore, while the concrete tension cracks are covered and have no effect on the deck design, they present a major concern when designing composite beams.

B - 1 TEST PROGRAM

Four test series were conducted on composite beams using stiffened channels for soffits (Table 2). The objective of each series is as follows: Series A examined the effect of variation in the depth, d', and thickness, t, of the soffits made of plain stiffened channel. Series B and C examined the effects of rounded and oval embossments respectively on the behaviour of soffits identical to the ones of series A. Series D tested beams with different dimensions using high tensile soffits of plain and embossed channels.

The beams were cast with the cleaned steel soffits simply supported at the ends of the span, except the beams in test series D, which were provided with additional support at the middle of span. This arrangement simulates the construction conditions in which the steel soffit is to carry the weight of the wet concrete. Equivalent standard beams reinforced with bars were cast on bottom wooden forms resting on the floor. Standard concrete cylinders were also cast out of each mix to determine the actual f_c' for the concrete used in each beam.

All beams were tested as simply supported with span S (Fig. 3, Table 2). The loads were transmitted from a two point loading beam. The top surface of all tested beams were leveled at the point of loading to ensure a line of uniform pressure across the width. The load was applied at a shear distance L' from the centreline of the closest support. Dial gauges were placed at the middle of the span to record the deflections and at the two ends of the beam to record the slip between the soffit and concrete. Fig. 4, 5 and 6 show the load versus deflection of three of the beams tested with observed failure as follows:

Test B-6, tension failure, Figs. 4a, b
Test C-4, shear failure, Figs. 5a, b
Test D-3, bond failure, Figs. 6a, b

Table No. 2 shows the tests conducted together with the experimental and analytically obtained results of the ultimate load.

B - 2: ANALYSIS OF THE COMPOSITE BEAMS

The behaviour and ultimate load carrying capacity of composite beams can be determined by examining their responses to the induced bending-, shear- and bond stresses.

a) Bending Stresses

In general the beams are not shored during construction and the wet concrete is carried by the steel section alone. Bending stresses, f, due to this case of loading are calculated using the section properties of the soffits alone:

$$f = \frac{M_D . y}{I} \tag{1}$$

in which M_D = bending moment due to the weight of wet concrete; y = distance to neutral axis; and I = moment of inertia of soffit about its neutral axis.

After hardening of the concrete, the section acts as a composite and, under superimposed bending moment, M_L, tension is induced over the soffit. The ultimate load carrying capacity is reached when the whole section of the soffit reaches its yield limit (Table VIII, Reference 8). This observation follows the generally accepted characteristics of composite sections in which the initial stresses caused by the weight of wet concrete have no effect on the ultimate load carrying capacity of the composite section. The ultimate moment, M_u', can be calculated using the standard formulas of reinforced concrete beams:

$$M_u' = A_s f_y (d - a/2) \tag{2}$$

in which A_s = cross-sectional area of soffit; f_y = yield stress of soffit material; d = distance from extreme compression fiber to the centroid of soffit; and a = depth of the effective compression zone of concrete:

$$a = \frac{A_s f_y}{0.85 f_c' b} \tag{3}$$

in which f_c' = specified compression strength of concrete; and b = width of compression face of member.

The ultimate moment is calculated for each of the beams tested and the results are listed in Table No. 2.

b) Bond Strength

The bond force, V_b, (longitudinal shear per unit length) acting between the soffit and concrete can be calculated as follows:

$$V_b = \frac{V.Q_c}{I_c} \tag{4}$$

in which V = the shear force at a given section; I_c = moment of inertia of the transformed uncracked section; and Q_c = the statical moment of the tranformed steel section about the neutral axis of the transformed composite section.

Eq. 4 may govern the design of beams with plain soffits (no embossments) in which the bond forces are resisted only by the chemical bond between the concrete and steel. However, for soffits with embossments, bond failure does not occur until complete slip takes place over the whole shear length L'. Herein, the bond strength is related to the tensile force in the soffit, T, and the shear length L':

$$U_b = \frac{T}{L'} \tag{5}$$

in which $T = M_u/(d - a/2)$.
Considering the test arrangement with, two concentrated loads, $M = V \cdot L'$, therefore:

$$U_{ub} = \frac{V_u}{(d - a/2)} \tag{6}$$

The average ultimate bond strength, U_{ub}, can be obtained experimentally and are listed (underlined) in Table 2 for the cases with observed bond failure.

c) Shear Strength

Tests on standard reinforced concrete beams (2) show that the nominal shear force, V_{cr}, at which diagonal flexure-shear cracking develops is conservatively predicted from:

$$V_{cr} = b.d \cdot (1.9 \sqrt{f'_c} + 2500 \rho \frac{Vd}{M}) \leq (3.5 \sqrt{f'_c}) b.d \qquad (7)$$

in which $\rho = A_s/bd$; V/M = the ratio of shear to bending moment occurring simultaneously.

Once a diagonal crack is formed, the shear force is transmitted by the uncracked portion of the concrete, V_c, and across the soffit, V_d. The force, V_d, creates vertical tension stresses as shown in Fig. 7. These stresses cause the splitting of the concrete along the plane of the lips of the soffits as shown in Fig. 5b (Test No. C-4). It was observed from the test programme that this splitting (or complete failure) occurred under loading that is slightly higher than the one initiating the diagonal crack.

The shear force at the initiation of diagonal cracks, V_{cr}, is calculated using Eq. 7 for each of the beams in Table No. 2. However, it should be noticed that V_{cr} is affected by the soffit especially by its two legs parallel to the web of the beam. The stresses in the soffit at the initiation of diagonal cracks are difficult to determine. Therefore, and in the absence of a proper approach to calculate the shear carried by the soffit, Eq. 7 may be used to determine a lower limit for the shear crack developments. This can be observed in Table 2, which shows the experimental shear capacities in most of the beams governed by shear failure to exceed V_{cr} as calculated using Eq. 7.

d) Shear Bond Strength

Schuster (8) suggested the following expression to calculate the shear bond strength of composite steel deck slabs subjected to the combined action of shear and bending moment:

$$\frac{V_{uc}}{bd} = K_1 \frac{\sqrt{f'_c}}{L'\rho} d + K_2 \qquad (8)$$

in which K_1 and K_2 are constants obtained from experimental results.

Eq. 8 was developed by analytically calculating the tensile stresses induced, in the concrete as a result of the combined effect of shear and bending moment and comparing it with the tensile strength of the concrete. That is to say, Eq. 8 determines the initiations of the diagonal cracks in the concrete. These cracks lead to the uplift of the concrete and to its separation from the steel deck and thus justified considering Eq. 8 as governing the shear-bond failure of composite decks.

With the existence of the lips in the stiffened channels, the diagonal shear failure is not always accompanied with bond failure, Fig. 5b. Herein, the concrete did not lift off the soffit, but failed due to vertical tension in the web. However, the diagonal crack still has an effect on the bond strength, but in different mechanism, since it reduces the shear length L' to (L'-d). Therefore, it is often difficult to separate the pure shear, or bond failures from the combined shear-bond failure. With this understanding Eq. 8 may also be applied to composite beams after determining the constants K_1 and K_2 experimentally. The relation between $V_u/\rho bd$ and $\sqrt{f'_c} \cdot d/\rho L'$ are given in Fig. 8 as obtained from the present testing program.

B - 3 OBSERVATIONS ON BEAM BEHAVIOR

1. The load carrying capacity of composite beams is found to be of equal or higher magnitude than equivalent standard concrete beams reinforced with deformed bars (referred to as S.R.), Table 2. Also, the first crack is developed at a load P_1 which is of the same order in composite and standard reinforced beams (see test series B and D-3 to D-6). However, attention should be paid to the ratio of P_1/P_u which is found to be often lower, and to the number of cracks which is considerably less when comparing the performance of the composite beams to that of standard ones. More studies are required to establish crack control criteria and possible to adjust the load factors for the design of composite beams.

2. The ultimate load carrying capacity of the composite beams may be governed by one or the combined effects of more than one of the failure criteria discussed above. This was observed through the testing program and can be summarized as follows:

   (a) Tension failure in which the soffit reaches its yield limit. Herein, the vertical tension cracks in the concrete propagate reducing the compression zone until failure takes place, Fig. 4b. Tests with observed failure are marked with T in Table No. 2 under type of failure. For these tests the analytically calculated M_u (underlined in Table 2) show reasonable agreement with the test results. Also, the analytical results for M_u are found, in general, to be higher than the experimental ones for the tests in which other criteria were observed to cause failure.

(b) Bond failure in which slip occurs between the soffit and the concrete. This type of failure was observed in test series D in which tension failure was avoided by using soffits of high tensile strength. Bond failure was also observed in a few of the beams with plain channel soffits. Herein, the failure is accompanied with the development of vertical cracks out of which one crack increases in width and propagates through the whole depth of the beam, Fig. 6a. This is accompanied with sliding of the concrete over the soffits as shown in Fig. 6b. The ultimate bond strengths are calculated per unit length and are listed in Table No. 2 and underlined for the cases with observed bond failure.

3. The effect of the depth of the soffit, d', on the shear or shear bond capacity of the beams is examined in test series A, B, and C. The ultimate shear V_u is plotted versus the depth d' in Fig. 9 showing a trend of increase in V_u with the increase of d'.

4. Fig. 8 shows the relation of $V_u/\rho bd$ versus $\sqrt{f'_c} \cdot d/\rho L'$ as obtained from the presented test program. These results are limited and additional testing is required before conclusive evaluation of the validity of Eq. 8 is reached. However, at present the inconsistency and lower level of shear loading capacity of plain soffits can be observed when compared with those of soffits with embossments. This can be understood since only chemical bond is active in plain channels while both chemical and mechanical bond take place in the embossed ones.

5. The test program was conducted on under-reinforced beams with no web (shear) reinforcement. Additional tensile reinforcement and/or web reinforcements may be provided. It is anticipated that the newly developed beams will follow the established behavior of equivalent standard reinforced beams, but in the meantime, further testing may be required to substantiate this assumption.

C - COMPOSITE COLUMNS

Composite columns can be built as shown in Fig. 1b. The channels are placed at two parallel faces while removable forms are used to cover the remaining sides. Lateral ties are eliminated along the column. However, the two ends of the columns should be provided with means of lateral restrain in order to avoid local failure at these ends. Practical applications usually provide such restrain by the beams or footings through which the load is transmitted to the column.

C - 1 TEST PROGRAM

Tests were conducted on composite columns using stiffened channels with round embossments (6). Equivalent standard columns reinforced with bars and ties were also tested for comparison. Axial as well as eccentric loads were applied. The test results are listed in Table No. 3 together with the analytically calculated ultimate load of each column.

A number of columns failed prematurely due to local conditions at the supports, Fig. 10. This affected the load carrying capacity of both the composite and the standard reinforced concrete columns, Tests No. 1 to 4, 7 and 8. The conditions at the supports were improved by providing a steel cap at the loading points. Future tests should be modified to represent the actual conditions at the column ends.

C - 2 ANALYSIS OF COMPOSITE COLUMNS

Test results show that adequate bond exists between the concrete and the steel channels until the columns reach their ultimate load carrying capacity. Therefore, composite columns can be analyzed using the procedures established for standard reinforced concrete columns (5, 9). However, more accurate analysis may be developed in which consideration can be given to the local rigidity of the channel, the channels depth, d', and their orientation with respect to the eccentricity.

C - 3 OBSERVATIONS ON THE BEHAVIOR OF COMPOSITE COLUMNS

The following observations are based on the test results listed in Table No. 3 after excluding columns 1 to 4, 7 and 8 because of their premature failure.

1. The experimental ultimate load carrying capacity compares reasonably well with the analytical results for the cases of axial loading and loading with eccentrically $e_x \neq 0$ and $e_y = 0.0$. Failure takes place by sudden concrete crushing accompanied by bulging of the steel channel, Fig. 11. Herein, separation took place between the channel and concrete. It should be noted that the width of the lip of the channel is in the order of 0.5 in. (12 mm) and that such failure may be delayed by changing the size of the lip. Further study could determine the optimum size of the lips which delays separation without excessive cutting in the concrete section.

2. Columns subjected to eccentric loading with respect to the y-axis ($e_y \neq 0$, Fig. 1b) show considerably low load carrying capacity

when compared with the analytical results (Tests 13 to 15). This is contrary to the case with $e_x \neq 0$. This can be explained since the concrete face with maximum compression is un-restrained in the former case while it is confined in the latter case by the channels section.

CONCLUSION

A testing program was conducted and proved the feasibility and favorable behavior of cast-in-place concrete beams and columns reinforced with cold-formed steel soffits. The proposed soffits are embossed to provide consistent strength in bond. The composite system leads to considerable savings in the cost and time of construction without increasing the area of steel required for reinforcement.

Appendix I REFERENCES

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Appendix II NOTATIONS

A_s = cross-sectional area of the soffit or reinforcing bars on tension side

A'_s = cross-sectional area of the soffit or reinforcing bars on compression side

a = depth of effective compression zone of concrete

b = width of beam or column

d = distance from extreme compression fiber of beam to the centroid of soffit

d' = depth of soffit

e = distance between the centroid and bottom edge of soffit (Fig. 1)

f'_c = compression strength of concrete

f_y = yield stress of soffit or reinforcing bars

h = depth of composite beam or column

H = height of columns

I_c = moment of inertia of the transformed uncracked composite section about its centroid

K_1, K_2 = constants obtained from test

L' = shear length

L = span of beam

M_D = bending moment due to weigth of wet concrete

M_L = superimposed bending moment

M_u = ultimate bending moment

Q_c = statical moment of the transformed steel section about the neutral axis of the transformed composite section

P_1 = load at the initiation of first crack in beams

P_u = ultimate load

T = tensile force in soffit

U_b = bond strength

U_{ub} = average ultimate bond strength

V = shear force

V_u = ultimate shear force

V_{uc} = ultimate shear bond strength

ρ = A_s/bd

Table No. 1

DETAILS OF COLD-FORMED STEEL SOFFITS

Channel No.	b in	d in	t in	A_s in^2	e in
1	4.0	1.0	0.075	0.501	0.31
2	4.0	1.5	0.075	0.576	0.49
3	4.0	1.5	0.600	0.453	0.46
4	4.0	2.0	0.600	0.513	0.66
5	4.0	2.5	0.600	0.573	0.87
6	4.0	2.5	0.048	0.461	0.84
7	4.0	3.0	0.048	0.509	1.05
8	4.0	3.5	0.048	0.557	1.28
9	5.0	2.0	0.074	0.74	0.62
10	6.0	2.0	0.074	0.81	0.65

Soffits are identified with their number and the letters, C.P. for plain channels; C.R. for channels with round embossments (Fig. 2a); C.O. for channels with oval embossments (Fig. 2b).

Table No. 2

TEST AND ANALYTICAL RESULTS FOR COMPOSITE COLD-FORMED STEEL CONCRETE BEAMS

Test No.	Soffit ID	A_s in^2	F_y Ksi	S in	b in	h in	d in	100 ρ	f'_c Ksi	L' in	P_1 Kips	Ultimate Load Test									Analytical	
												N	P_u Kips	V_u Kips	M_u Kips·ft	Failure	$\frac{V_u}{\rho bd}$	$\frac{\sqrt{f'_c}\,d}{\rho L'}$	$\frac{P_1}{P_u}$	$\frac{V_u}{(d-a/2)}$	V_{cr} Eq. 7	M_u Eq. (3) Kips·ft
A-1	CP-1	0.501	40	114	4.	12.	11.69	1.07	4.83	43	4.40	7	9.3	4.65	16.7	S.B.	9.28	1766	0.47	0.42	6.18	18.50
A-2	CP-2	0.576	39	114	4.	12.	11.51	1.25	4.65	43	6.90	5	10.8	5.40	19.3	S.B.	9.38	1460	0.64	0.50	6.35	20.22
A-3	CP-3	0.453	42	114	4.	12.	11.54	0.98	4.50	43	3.40	6	7.4	3.70	13.3	S.B.	8.17	1837	0.46	0.34	6.19	18.74
A-4	CP-4	0.513	42	114	4.	12.	11.34	1.13	4.70	43	3.90	4	7.4	3.70	13.3	B	7.21	1600	0.53	0.35	6.25	18.29
A-5	CP-5	0.573	42	114	4.	12.	11.13	1.29	3.92	43	9.3	6	12.8	6.40	22.9	B	11.17	1256	0.73	0.62	5.67	19.62
A-6	CP-6	0.461	42	114	4.	12.	11.16	1.03	5.67	43	2.0	3	4.4	2.20	7.9	B	4.77	1897	0.45	0.21	6.68	17.20
A-7	CP-7	0.509	43	114	4.	12.	10.95	1.16	4.30	43	3.4	6	10.9	5.45	19.53	S.B.	10.71	1440	0.31	0.53	5.78	18.61
A-8	CP-8	0.557	41	114	4.	12.	10.72	1.30	5.41	43	4.4	4	8.9	4.45	15.9	B	7.99	1411	0.49	0.44	6.36	20.51
B-1	SR	0.61	66.0	96	4.	12.	10.50	1.40	4.67	34	6.4	20	13.36	6.68	18.9	S.			0.48		5.91	29.87
B-3	CR-3	0.453	39.0	96	4.	12.	11.54	0.98	5.08	34	5.4	7	11.30	5.65	16.0	S.	12.47	2468	0.48	0.51	6.58	17.60
B-4	CR-4	0.513	42.0	96	4.	12.	11.34	1.13	4.66	34	4.9	6	12.76	6.38	18.1	S.	12.44	2015	0.38	0.60	5.89	19.14
B-6	CR-6	0.461	42.0	96	4.	12.	11.16	1.03	4.70	34	5.4	5	13.25	6.63	18.8	T			0.41	0.63	5.82	17.12
B-7	CR-7	0.509	43.0	96	4.	12.	10.95	1.16	4.64	34	6.0	5	14.85	7.43	21.0	T			0.40	0.72	6.08	18.71
B-8	CR-8	0.597	41	96	4.	12.	10.72	1.30	4.97	34	5.4	6	15.22	7.61	21.6	S.B.	13.66	1710	0.35	0.76	6.18	20.38
C-3	CO-3	0.453	39	96	4.	12.	11.54	0.98	4.50	34	4.95	6	11.28	5.64	15.98	S.	12.45	2323	0.44	0.51	6.26	16.14
C-4	CO-4	0.513	42	96	4.	12.	11.34	1.13	4.53	34	5.44	7	13.86	6.93	19.63	S.B.	13.51	1987	0.39	0.65	6.23	19.11

Table No. 2 (cont'd)

TEST AND ANALYTICAL RESULTS FOR COMPOSITE COLD-FORMED STEEL CONCRETE BEAMS

Test No.	Soffit ID	A_s in^2	F_y Ksi	s in	b in	h in	d in	100 ρ	f'_c Ksi	L' in	P_1 Kips	Ultimate Load Test									Analytical	
												N	P_u Kips	V_u Kips	M_u Kips·ft	Failure	$\frac{V_u}{\rho bd}$	$\frac{\sqrt{f'_c}}{\rho L'}d$	$\frac{P_1}{P_u}$	$\frac{V_u}{(d-a/2)}$	V_{cr} Eq. 7	M_u Eq. (3) Kips·ft
C-6	CO-6	0.461	42	96	4.	12.	11.16	1.03	4.58	34	4.06	5	13.11	6.56	18.59	S.B./T	14.23	2157	0.31	0.62	6.12	15.34
C-7	CO-7	0.509	43	84	4.	12.	10.95	1.16	4.48	34	4.95	8	14.35	7.18	20.33	T			0.34	0.75	5.98	17.35
C-8	CO-8	0.557	41	96	4.	12.	10.72	1.30	4.66	34	4.45	6	14.43	7.22	20.44	S.B.	12.96	1656	0.31	0.72	6.60	19.03
D-1	CO-9	0.74	74.4	121	5.	14	13.38	1.1	4.24	38	5.47	7	18.91	9.46	29.9	B	12.78	2103	0.29	0.80	8.92	54.38
D-2	CP-9	0.74	74.4	121	5.	14	13.38	1.1	3.84	38	5.47	4	12.44	6.22	19.7	B	8.41	2001	0.44	0.51	8.52	53.65
D-3	CO-10	0.81	74.4	121	6.	16	15.35	0.87	3.94	38	8.71	5	25.63	12.82	40.6	B	15.83	2943	0.34	0.93	11.79	69.56
D-4	CR-10	0.81	74.4	121	6.	16	15.35	0.87	3.80	38	8.71	5	30.6	15.30	48.5	B	18.89	2890	0.28	1.11	11.60	69.28
D-5	CP-10	0.81	74.4	121	6.	16	15.35	0.87	4.21	38	7.96	3	21.4	10.70	33.9	B	13.21	3042	0.37	0.77	12.16	70.04
D-6	SR	0.81	67.3	121	6.	16	14.80	0.96	4.11	38	10.54	20	21.89	10.95	34.66	S.B.			0.48		10.98	57.7

P_1 = load at the development of first crack

N = number of cracks developed at the ultimate load

Table No. 3

TEST AND ANALYTICAL RESULTS FOR COMPOSITE COLD-FORMED STEEL-CONCRETE COLUMNS

Test No.	Soffit I.D.	A_s A'_s in^2	F_y Ksi	f'_e Ksi	b in	h in	H in	Eccentricity e_x in	Eccentricity e_y in	Ultimate Load Exper. Kips	Ultimate Load Analytical Kips	Type of Failure
1	SR	0.47	58.0	5.5	4	8	72	0.0	0.0	75.5	145.2	Pre-mature
2	CR-4	0.51	59.0	5.45	4	8	72	0.0	0.0	65.7	144.5	Pre-mature
3	SR	0.47	58.0	5.43	4	8	72	0.0	0.0	52.5	144.7	Pre-mature
4	CR-4	0.51	59.0	5.76	4	8	54	0.0	0.0	69.1	212.2	Pre-mature
5	CR-5	0.74	74.4	6.51	5	10	90	0.0	0.0	253.3	246.7	Bulging of Channel
6	CR-4	0.51	59.0	6.44	4	8	72	0.0	0.0	157.8	157.0	Bulging of Channel
7	CR-4	0.51	59.0	6.44	4	8	72	1.5	0.0	50.9	134.5	Pre-mature
8	CR-4	0.51	59.0	6.43	4	8	54	1.5	0.0	52.2	144.1	Pre-mature
9	CR-6	0.81	74.4	6.60	6	10	90	2.5	0.0	208.5	225.7	No failure
0	CR-6	0.81	74.4	6.6	6	10	90	4.0	0.0	178.7	165.1	Bulging of Channel
1	CR-6	0.81	74.4	6.7	6	10	90	4.0	0.0	199.5	167.4	Bulging of Channel
2	CR-6	0.81	74.4	6.67	6	10	72	4.0	0.0	181.7	175.3	Bulging of Channel
3	CR-4	0.51	59.0	6.96	4	8	54	0.0	1.0	42.8	70.0	Concrete failure
4	CR-4	0.81	74.4	6.26	6	10	90	0.0	2.0	59.6	98.7	Concrete failure
5	SR	0.78	58.0	5.72	6	10	72	0.0	2.0	64.5	115.3	Concrete failure

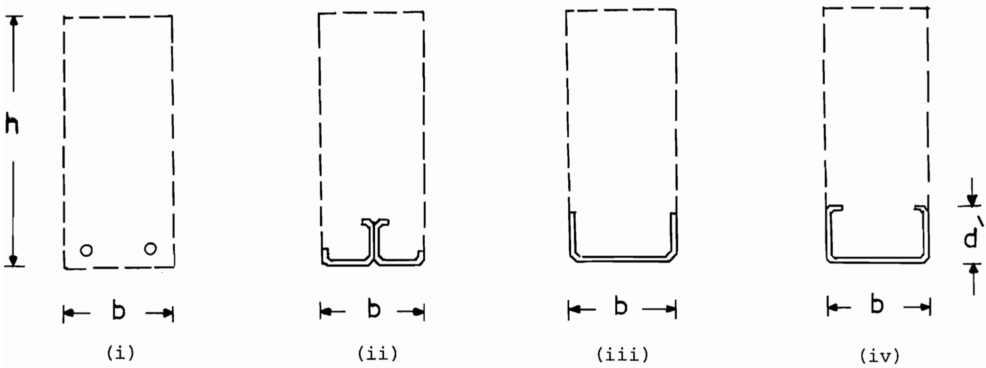

Fig. 1a: Beam Cross-Section

(i) Standard Reinforced

(ii) Soffit Made of Two Angles Back to Back

(iii) Soffit of Unstiffened Channel

(iv) Soffit of Stiffened Channel

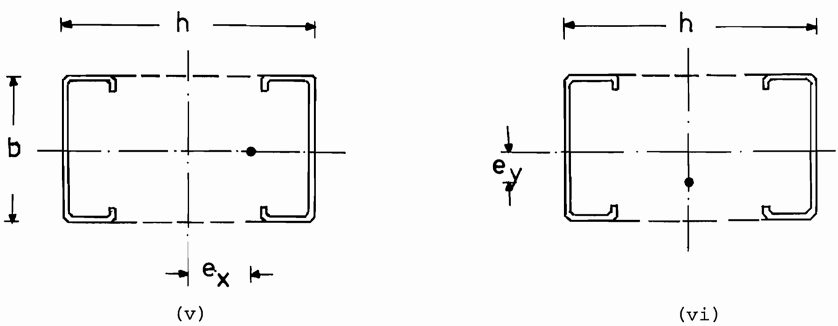

Fig. 1b: Column Cross-Section

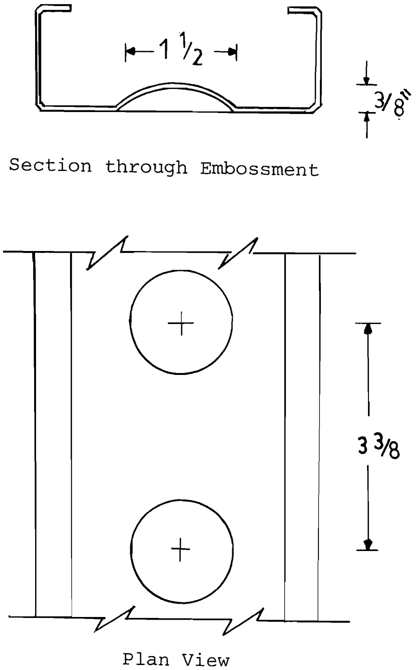

Fig. 2a: Channels with Round Embossments

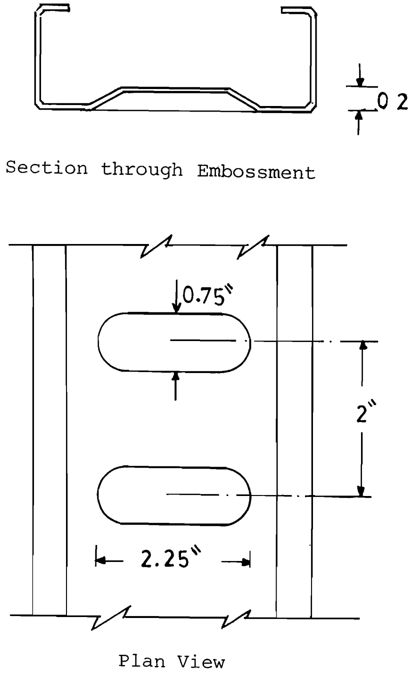

Fig. 2b: Channels with Oval Embossments

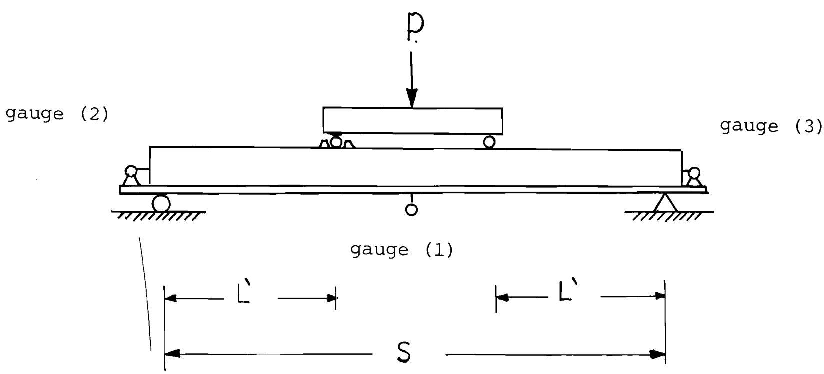

Fig. 3: Load Conditions in Testing

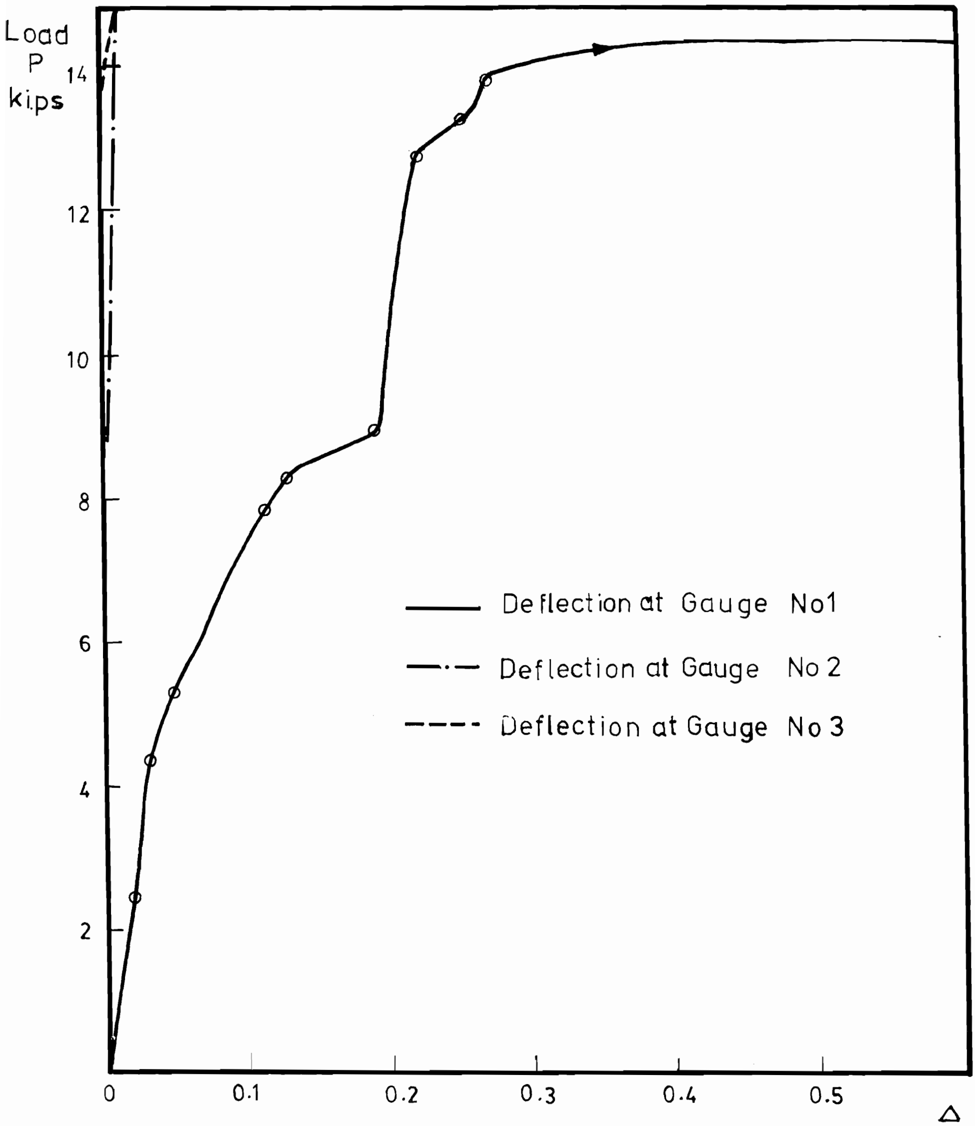

Fig. 4a: Load, P, versus Deflection, Δ, for Beam No. B-6

Fig. 4b: Beam No. B-6 after Failure (Tension Failure)

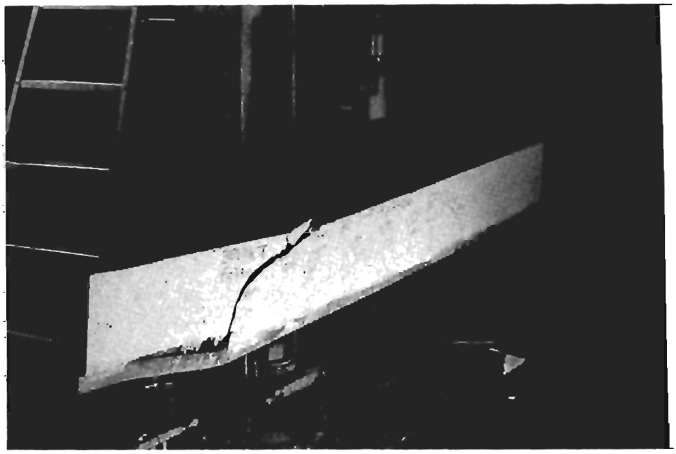

Fig. 5b: Beam No. C-4 after Failure (Shear-Bond Failure)

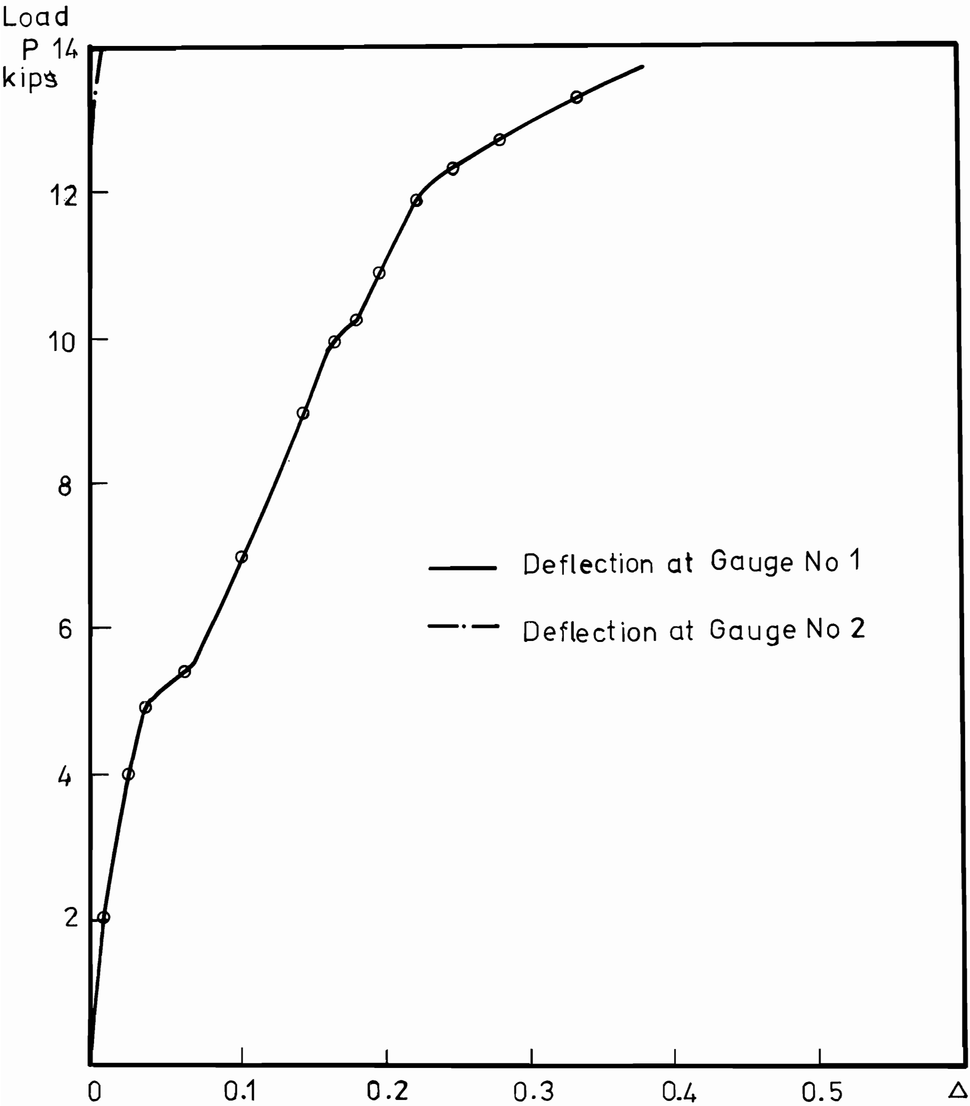

Fig. 5a: Load, P, versus Deflection, Δ, for Beam No. C-4

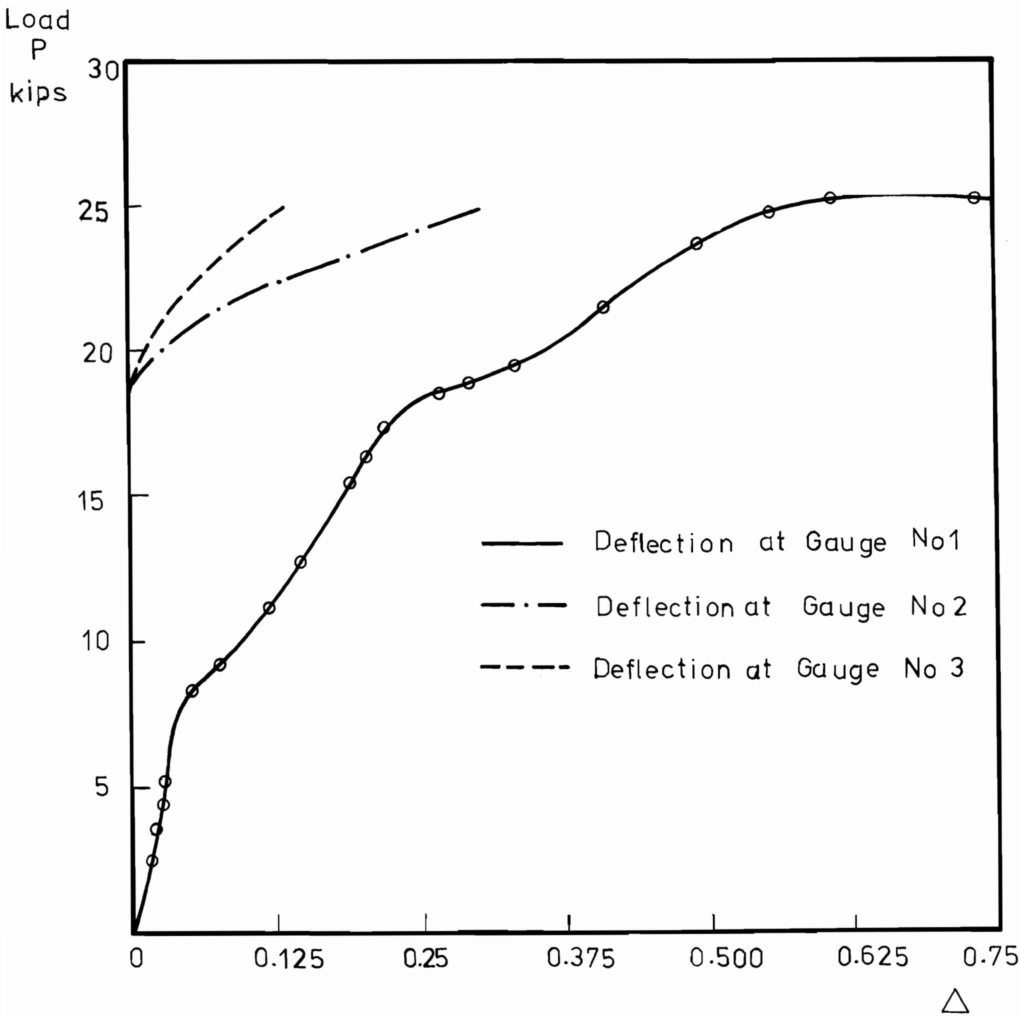

Fig. 6a: Load, P, versus Deflection, Δ, for Beam No. D-3

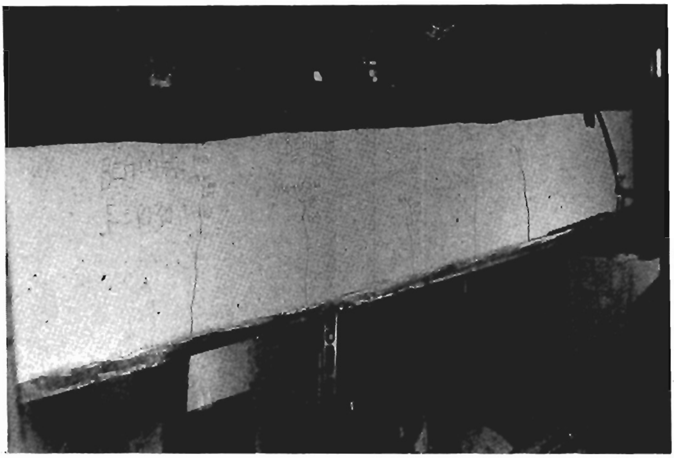

Fig. 6b: Beam No. D-3 after Failure (Bond Failure)

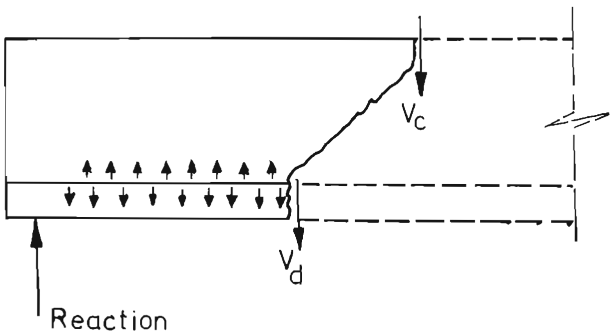

Fig. 7: Shear Forces After Development of Diagonal Crack

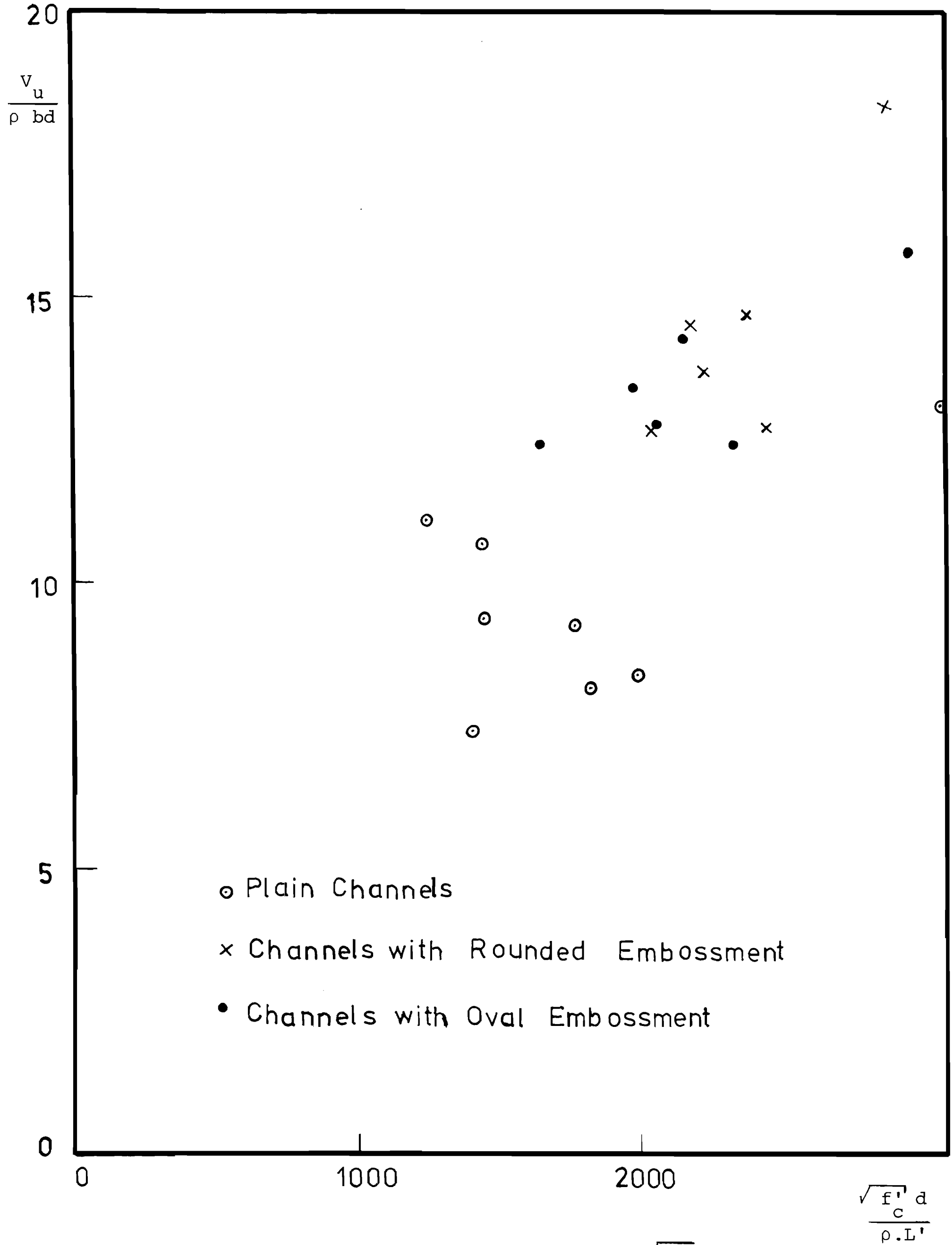

Fig. 8: Relation between $\frac{Vu}{\rho \ bd}$ and $\frac{\sqrt{f'_c}\ d}{\rho \ L'}$

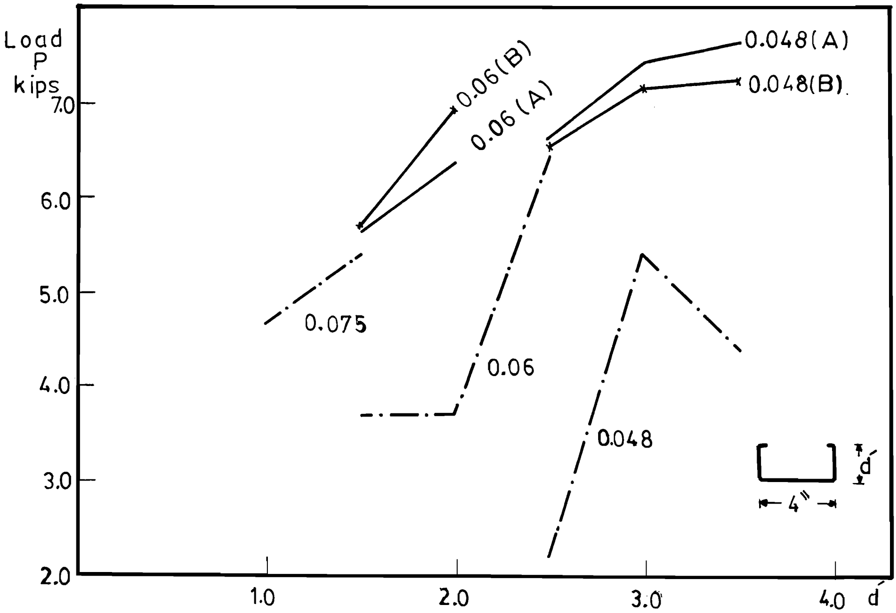

Fig. 9: Ultimate Shear Force, Vu, versus the Depth of the Channel, d'

Fig. 10: Premature Failure at Support (Column No. 4)

Fig. 11: Bulging Failure (Column No. 5)