THE THEORETICAL AND EXPERIMENTAL STUDY OF COMPOSITE SLABS

An Li, Research Ass., Division of Concrete Structures, Chalmers University Technology, Göteborg, Sweden

Krister Cederwall, Professor, Division of Concrete Structures, Chalmers University of Technology, Göteborg, Sweden

ABSTRACT

Seven full-scale composite slabs with profiled steel sheets were tested under a concentrated load. The major parameters of the study were the shear spans, concrete strengths, number of shear connectors and the slabs with or without end studs. A simple analytical method and a step by step computer method are proposed. The comparisons show good agreement between the theoretical and experimental results.

Key-words: composite slab, shear failure, computer analysis, bond

1. INTRODUCTION

In order to design a new type of composite slabs we have to rely on performance tests now /1/. These performance tests can of course only be conducted once the sheet has been produced. However, the design of composite slabs for a new profile is very expensive due to repeated prototype tests. The main objective of a performance test is to determine the shear transfer characteristics of the sheet. Unfortunately, these characteristics do not have direct physical significance.

It is thus obvious that there is a need for an alternative approach to the design of composite slabs. In order to investigate the possibility of an alternative approach we have tested and analysed a number of composite slabs with different bond properties. A simple analytical design method and a step by step computer method are proposed. For both methods it is however necessary to know the ultimate capacity and the slip properties of the connectors. Consequently it seems that it is not possible to avoid small simple tests in the design process but the cost of such tests can be considerably less than performance tests with prototype sheets.

2. DESCRIPTION OF TESTS

2.1 Test Program

The experimental program conducted at Chalmers consisted of seven full-scale composite slabs with profiled steel sheet. The major parameters of the study were the shear spans, concrete strengths, number of shear connectors and with or without end studs. The main purpose was to investigate the possibility of an alternative approach for evaluating the capacity of composite slabs and to deepen our understanding of the real behaviour.

In order to determine the characteristic values of materials bond tests (Fig.1), tensile strength tests for steel sheets and shear force-slip relation tests for screws were carried out. A series of small size specimen tests was performed to determine which kind of shear connector types was the most efficient. The best one was selected and used in the full-scale composite slabs. The details of the test results were presented in Ref./5/.

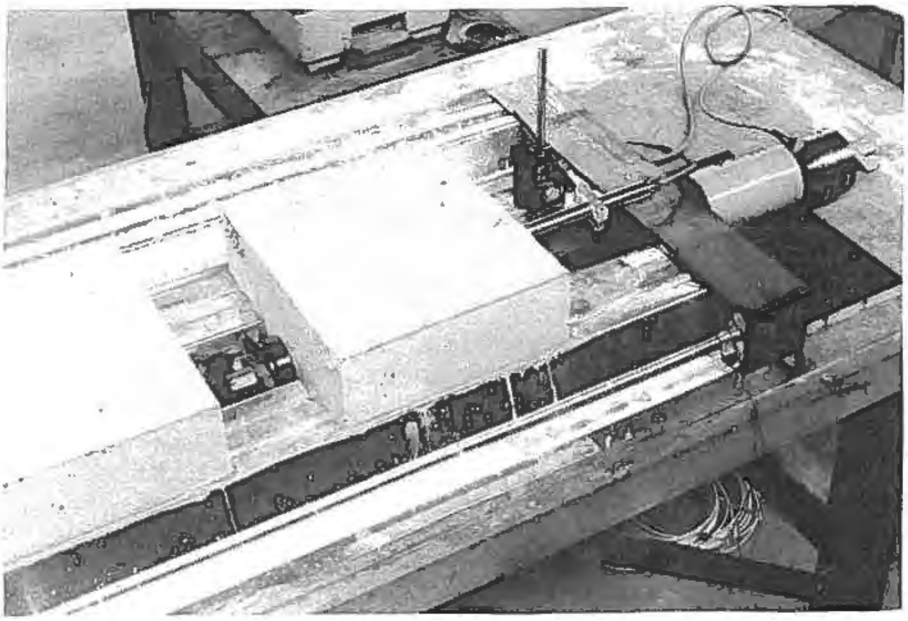


Fig. 1 Bond test

2.2 Test Specimens

The concrete was cast on galvanized steel sheet without embossment. The average yield strength and modulus of elasticity of the steel sheet were found to be 370.6 MPa and 188.8 GPa, respectively. The thickness of steel sheet was 1 mm and the nominal depth was 96 mm. It was produced by Plannja Byggproduker in Sweden.

The composite slabs were 0.79 m wide, 0.2 m high and 4.7 m long. They were tested with a span of 3.3 m. We tested one end first and then changed to the other end.

The shear connectors consist of reinforcement bars of diameter of 10 mm welded to steel plates which are connected to the top face of the steel sheet by self-drilling screws. One connector group consists of 18 screws. It is shown in Fig.2.

Different numbers of shear connector groups were used. The number of shear connector groups needed was determined by the principle that all shear force was taken over by the shear connectors when the steel sheet yielded at complete interaction. Based on that calculation, 40%, 60%, 80% and 100% of that number were used in the design of the specimens. The connector groups were uniformly placed in the shear span for all of the composite slabs except for T14B, in which there were the same connector groups as those in T14A, but most of them were near the ends. The spacing of connector groups in the shear span is shown in Fig.3.

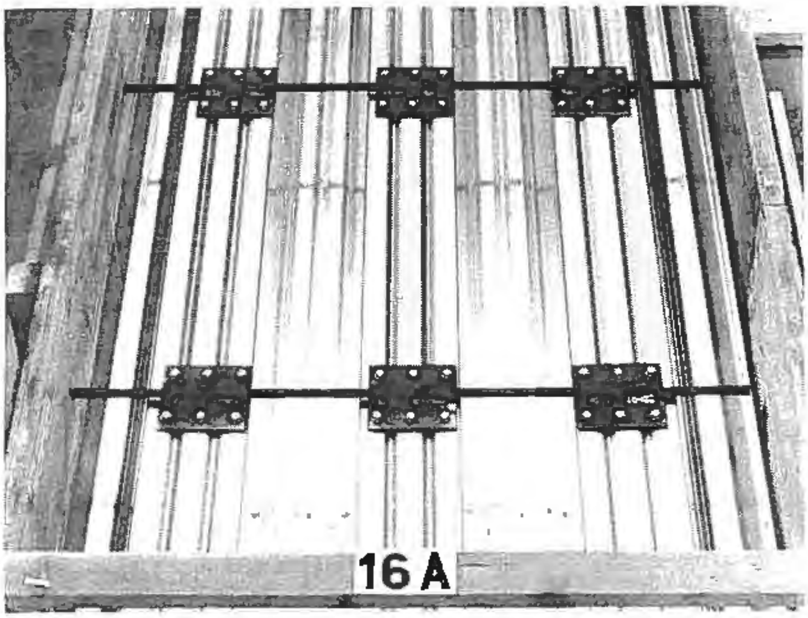


Fig.2 Shear connector groups

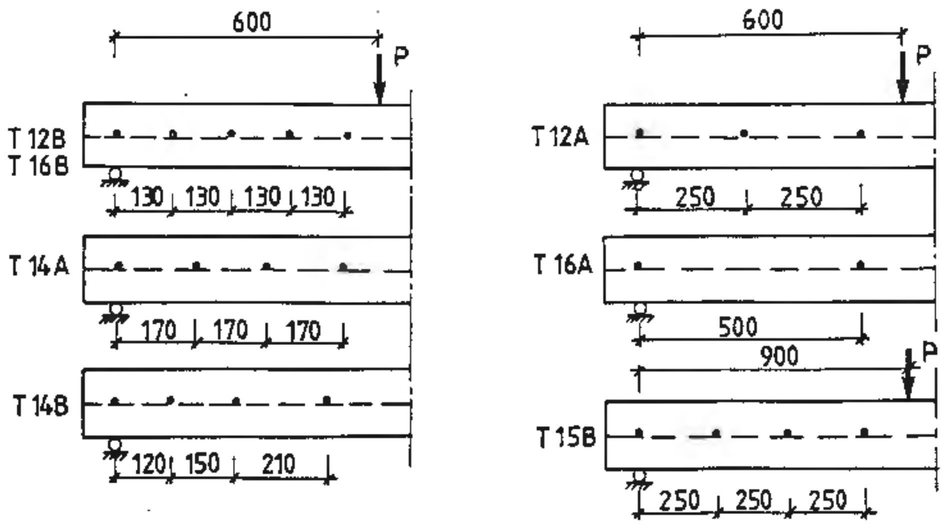


Fig. 3 Distribution of shear connector groups

For two specimens, T13A and T15A, four non-welded studs were placed at the end to provide the composite action. The studs were bolted on a 10 mm thick steel plate, through the steel sheet. The diameter of the studs was 16 mm and the typical cross section is shown in Fig.4. The details and material properties of the slabs are listed in Table 1.

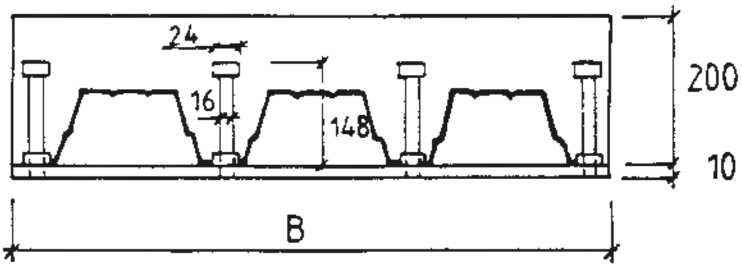


Fig. 4 Typical cross section for studs specimens

2.3 Test Set-up and Instrumentation

The specimens were simply supported and tested under one concentrated line load. Fig.5 shows a typical test set-up and instrumentation. The load was controlled by the deflection at the load section. The rate of loading was 1 mm/min until the maximum capacity was obtained and then double the speed. The composite slab was instrumented to measure the deflection, slip and strains in the steel sheet and the concrete. At each increment all of the measurements were recorded.

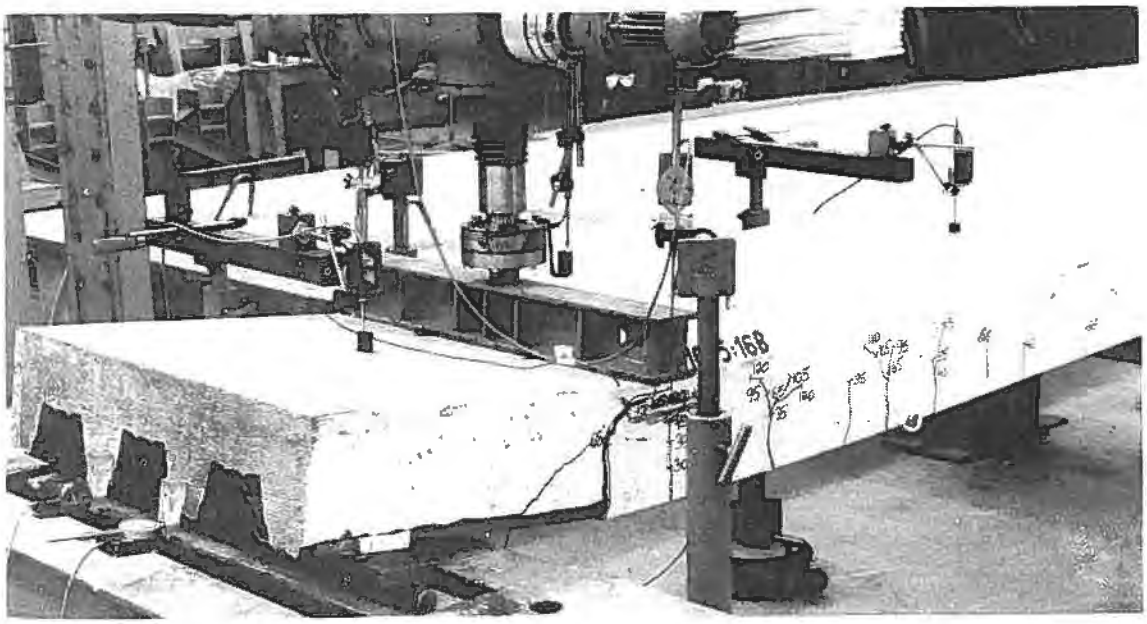


Fig. 5 Test set-up and instrumentation

Table 1 The Details of Specimens

slab	cube strength(1)		cube strength(2)		cylinder strength f_c' (MPa)	width of slab B (mm)	shear span L_1 (mm)	number of connector groups	space of connector (mm)	test order (4)	test P_u (KN)	$P_{u,cal}$ (KN)	
	σ_{cu} (MPa)	σ_{spli} (MPa)	σ_{cu} (MPa)	σ_{spli} (MPa)								Eq.(10)	Eq.(1)
T11A	29.12	3.09	28.94	3.07	-	793	600	-	-	F	50.7	-	-
T12A	29.12	3.09	28.94	3.07	-	793	600	3	250	F	84.3	80.7	86.48
T12B	29.12	3.09	28.94	3.07	-	798	600	5	130	S	126.1	109.5	134.86
T13A	26.84	2.73	26.73	2.83	18.63	797	600	studs	-	F	61.1	58.7	60.66
T14A	26.84	2.73	26.73	2.83	18.63	795	600	4	170	F	106.1	106.1	110.74
T14B	26.84	2.73	26.73	2.83	18.63	796	600	4	(3)	S	107.7	106.4	110.74
T15A	48.7	3.67	46.94	3.57	33.94	795	600	studs	-	S	71.0	71.4	68.57
T15B	48.7	3.67	46.94	3.57	33.94	798	900	4	250	F	85.1	81.2	76.04
T16A	48.7	3.67	46.94	3.57	33.94	790	600	2	500	F	61.6	53.8	61.87
T16B	48.7	3.67	46.94	3.57	33.94	790	600	5	130	S	122.5	117.1	134.88
T17A	48.7	3.67	46.94	3.57	33.94	801	600	-	-	F	40.4	-	-

(1) cubes cured under standard condition.
(2) cubes cured under the same condition as specimens.
(3) non-uniformly distributed.
(4) F — first tested end, S — second tested end.

3. THEORETICAL CONSIDERATION

The real behaviour of composite slabs is more complicated than that of ordinary concrete slabs not only because each steel sheet has its own unique shear transferring mechanism, the non-linear and non-elastic behaviour of the concrete and the steel sheet near the ultimate state, but also because there is a slip between the steel sheet and the concrete. Very appreciable strain differences exist at the interface, see Fig.6. This non-linear and non-elastic behaviour of materials and strain differences will be taken into account in the evaluation of strength and deformation of composite slabs.

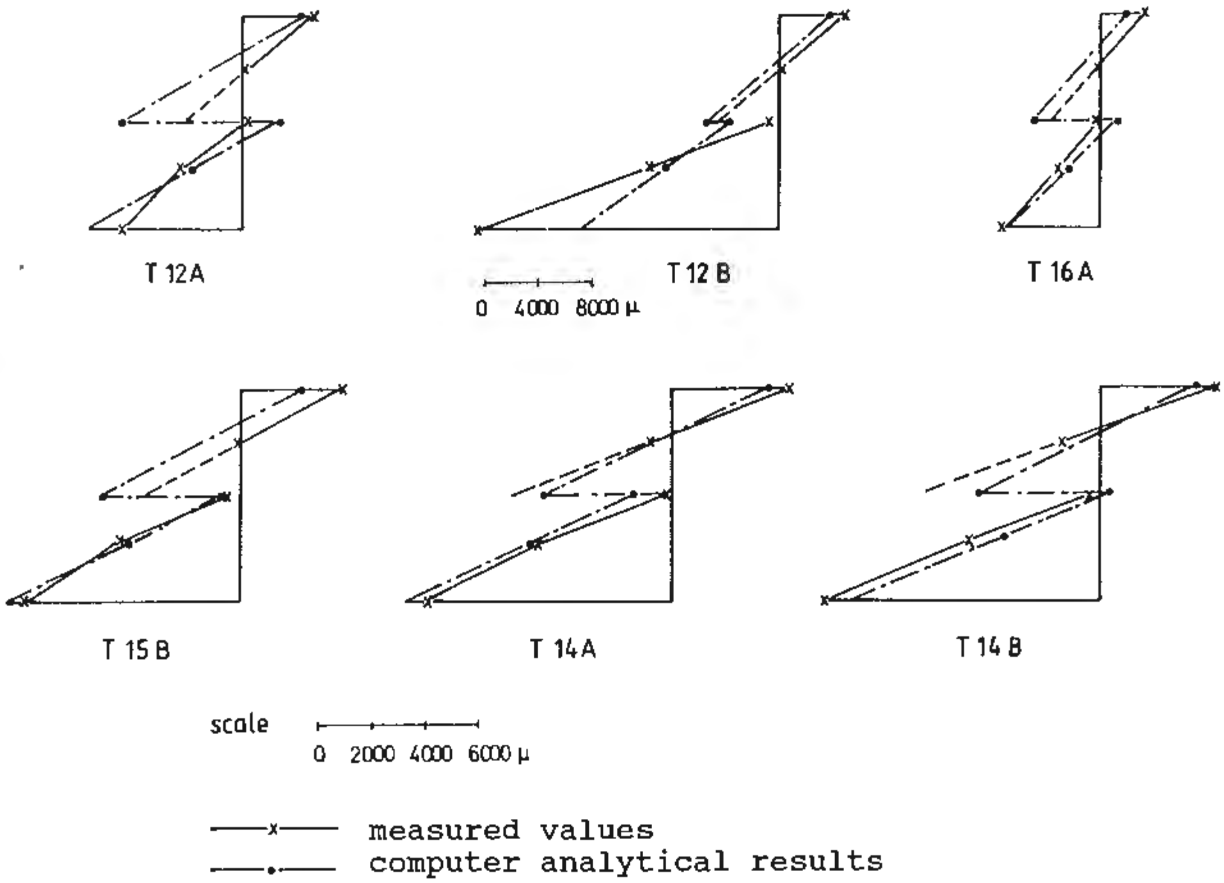


Fig. 6 Strain distribution in the cross section

3.1 Simple Analytical Approach

If we don't consider the influence of concrete located in the trough of the steel sheet and assume that the area of the steel sheet is concentrated at the same level, we obtain the simplified cross section, see Fig.7. It is reasonable to assume that the shear force transferred by connectors is linearly distributed along the shear span and all of the shear connectors develop the maximum capacity at the ultimate state. That is

$$F_x = x \cdot F_{max} / L_1$$

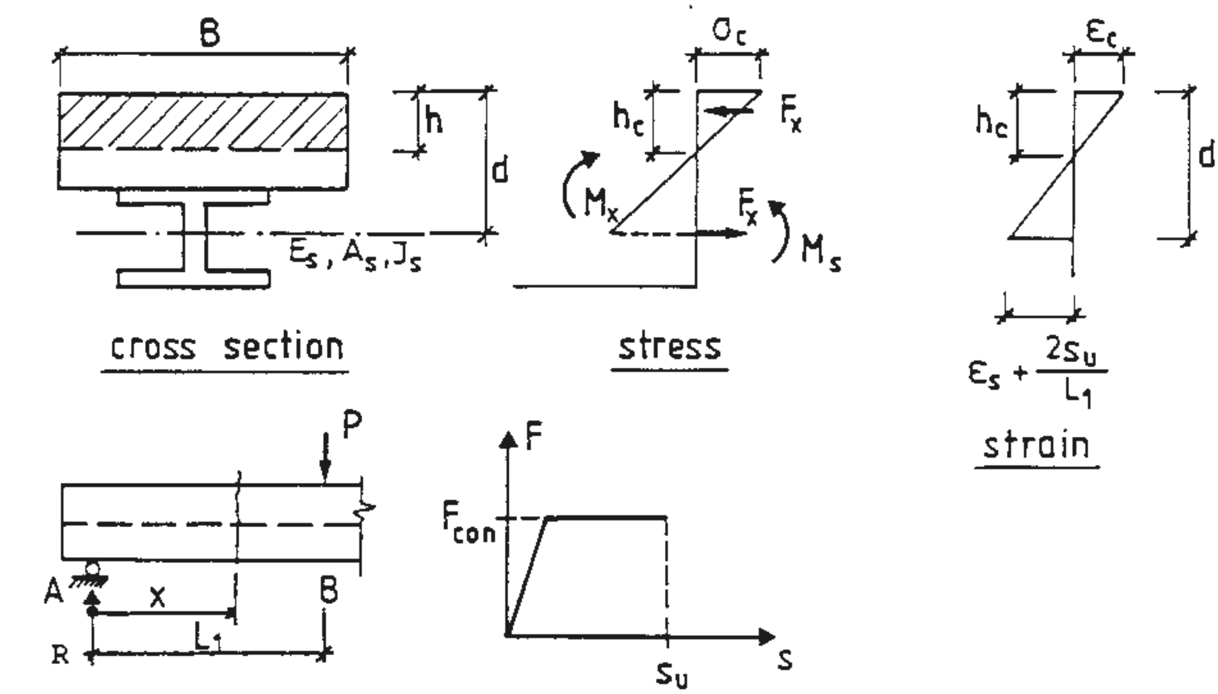


Fig. 7 Simple analytical approach

where F_{max} and F_x are the shear forces carried by connectors at section B and X, respectively, shown in Fig.7. $F_{max} = n \cdot F_{con}$.

The equilibrium of moments gives the bending moment of the steel sheet,

$$M_s = R \cdot x - x \cdot F_{max} (d - H_c/3)/L_1$$

Considering the strain difference,

$$dS/dx = -x \cdot [R(d-H_c/2)/(E_s \cdot J_s) - F_{max}/(L_1 \cdot r^2)]$$

and slip boundary condition,

$$S_B \approx 0 \quad \text{for} \quad x = L_1$$
$$S_A = S_{max} \quad \text{for} \quad x = 0$$

we can obtain the expression of slip as,

$$S = S_{max} - 0.5 \cdot x^2 \cdot [R(d-H_c/2)/(E_s \cdot J_s) - F_{max}/(L_1 \cdot r^2)]$$

where $1/r^2 = 1/(E_s \cdot A_s) + 1/(B \cdot E_c \cdot H_c) + (d-H_c/2)(d-H_c/3)/(E_s \cdot J_s)$ and

$$S_{max} = 0.5 \cdot L_1^2 \cdot [R(d-H_c/2)/(E_s \cdot J_s) - F_{max}/(L_1 \cdot r^2]]$$

The maximum reaction force at the support will be

$$R_u = E_s \cdot J_s [2 \cdot S_u/L_1^2 + F_{max}/(L_1 \cdot r^2)]/(d-H_c/2)$$

when the failure criterion is $S_{max} = S_u$.
Therefore

$$P = L(R_u - W)/(L - L_1) \qquad (1)$$

The derivation has shown that the capacity of the specimens depends on both the interface slip and the force in the connectors.

Now we have to provide a way to get the value of H_c. If we assume elastic materials and the strain distribution shown in Fig.7 the following relations are obtained,

$$\epsilon_c = 2 \cdot F_{max} / (H_c \cdot B \cdot E_c)$$
$$\epsilon_s = F_{max} / (A_s \cdot E_s)$$

and

$$\epsilon_c (d - H_c)/H_c = \epsilon_s + 2 \cdot S_u / L_1$$

We can obtain,

$$B \cdot E_c [F_{max}/(A_s \cdot E_s) + 2 \cdot S_u / L_1] H_c^2 + 2 \cdot F_{max} \cdot H_c - 2 \cdot F_{max} d = 0 \quad (2)$$

The value of H_c is obtained from Eq.(2). The estimated capacities for composite slabs found from Eq.(1) are listed in Table 1.

3.2 Step by Step Computer Method

Fig.8 shows the step by step computer method. The consideration of equilibrium, compatibility and constitutive equations for concrete, steel sheets and shear connectors will give the following results.

Equilibrium:

$$M = T \cdot Z = C \cdot Z$$
$$C = T = F \quad (3)$$

where $C = 0.8 \cdot n_1 \cdot B \cdot \bar{\sigma}_c$ $\quad T = \sigma_s \cdot A_s \quad F = \Sigma n_i \cdot F_i \quad (4)$

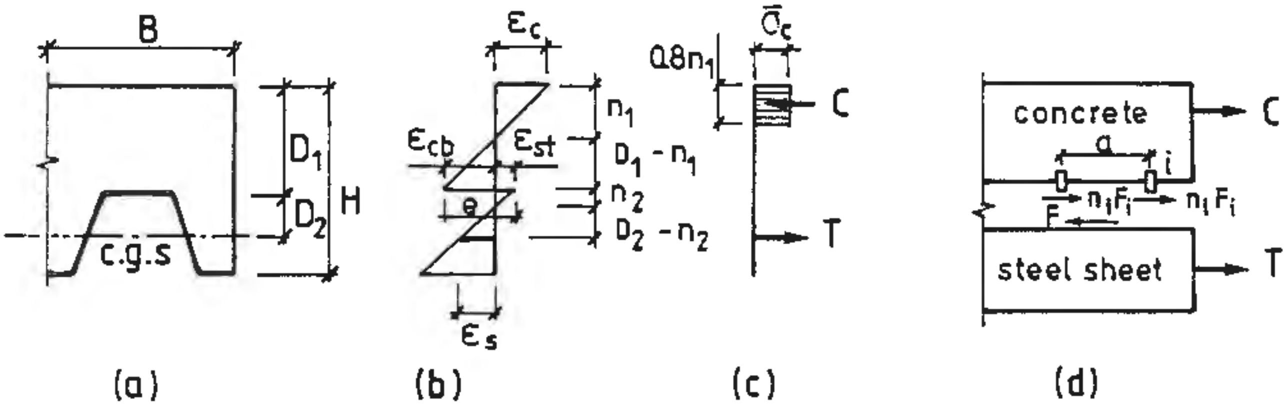


Fig. 8 Computer method

Compatibility:

Assumption:

1. The same curvature, ϕ, is assumed in the steel sheet and the concrete.
2. The distribution of strain is linear at the section.
3. The bond at the interface and the tensile strength of

concrete are neglected.
The following equations are obtained,

$$\epsilon_c = \phi \cdot n_1 \qquad \epsilon_s = \phi \cdot (D_2 - n_2) \tag{5}$$
$$\epsilon_{cb} = \phi \cdot (D_1 - n_1) \qquad \epsilon_{st} = \phi \cdot n_2$$

and

$$\epsilon_{cb} - \epsilon_{st} = e \tag{6}$$

where $\epsilon_{st} \geq 0$ in tension
$\epsilon_{st} < 0$ in compression

Constitutive Equations:

The following relations are used in the analysis (Fig.9).
1. The stress-strain relation for concrete.
The stress-strain distribution for the concrete in the cross section is assumed to be a parabola and a straight line. The mean value of the concrete compressive stress in the section is given by Ref./2/.

$$\bar{\sigma}_c = f_c'[\epsilon_c/\epsilon_p - (\epsilon_c/\epsilon_p)^2/3] \qquad \epsilon_c \leq \epsilon_p$$
$$\bar{\sigma}_c = f_c'[1 - \epsilon_c/\epsilon_p/3] \qquad \epsilon_c > \epsilon_p \tag{7}$$

where $\epsilon_p = 0.002$

2. The stress-strain relation for the steel sheet.
Based on the tension test results for the steel sheet the following expressions can be obtained by regression,

$$\sigma_s = F_y[2.2(\epsilon_s/\epsilon_s') - 1.2(\epsilon_s/\epsilon_s')^2] \qquad 0 \leq \epsilon_s \leq 0.0039$$
$$\sigma_s = F_y \qquad 0.0039 \leq \epsilon_s \leq 0.0065 \tag{8}$$
$$\sigma_s = F_y + 2100(\epsilon_s - 0.0065) \qquad \epsilon_s \geq 0.0065$$

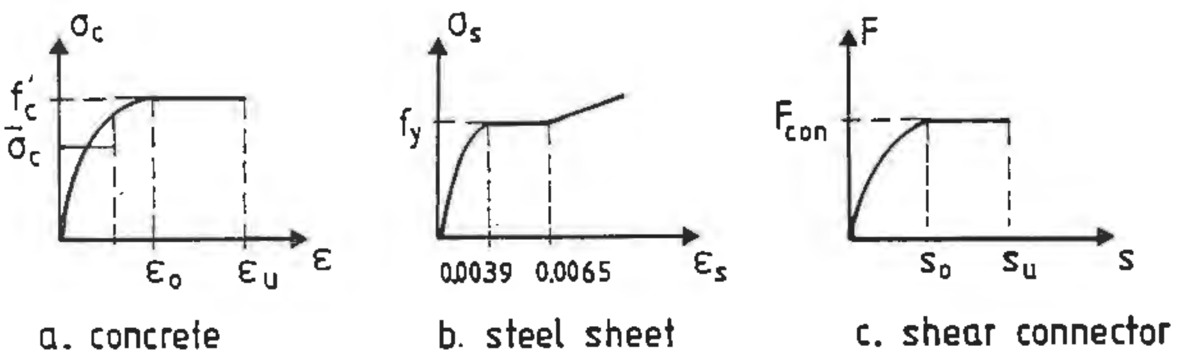


Fig. 9 Constitutive equations for materials

3. The shear force-slip relation for the connector groups.
Only limited tests have been carried out and we have not enough information to be able to get a reliable expression for the

relation of the shear force-slip. That relation is assumed, partly from our results.

$$F_i = F_{con} \; [2(S/S_o) - (S/S_o)^2] \quad 0 \le S \le S_o$$
$$F_i = F_{con} \qquad\qquad\qquad\qquad S > S_o \qquad (9)$$

For specimens with studs we take the same expression except for the value of S_o and F_{con}, which were derived according to other researcher's investigation /3/.

Failure Criteria:

In the computer analysis the maximum capacity will be reached when the following equations are satisfied.

1. $\epsilon_c = \epsilon_u = 0.0035$

or 2. $S = S_u = 1.5$ mm

In principle there will be a failure criterion for the steel sheet. However, in our special situation the steel sheet is so ductile that the yielding of the steel sheet is not the main reason for causing failure. Thus we do not consider it in our model.

In order to carry out the calculation we have to provide a link between the slip S and the strain difference e in the section. This link is given by

$$e = dS/dx = (d\eta_s - d\eta_c)/dx = \epsilon_s - \epsilon_c$$

as shown in Fig.10.

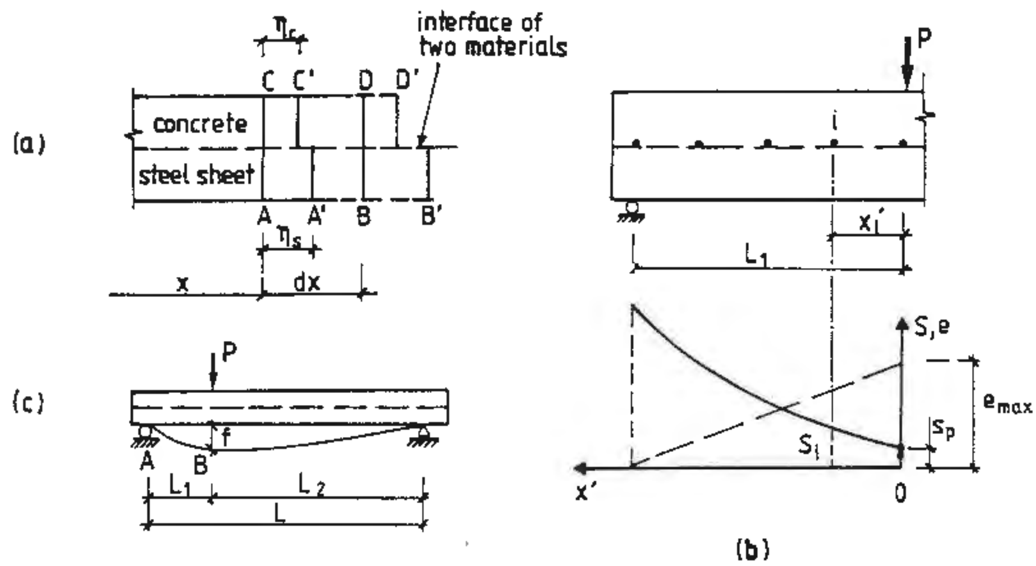


Fig.10 Consideration of slip and strain difference

According to the elastic analysis of composite beams the distribution of the strain difference along AB (Fig.10c) is found to be a hyperbolic sine /4/. It is convenient to use the coordinate system x' and to assume a linear strain difference distribution along the shear span,

$$e = e_{max} \cdot (L_1 - x_i') / L_1$$

Together with the boundary condition,

$$S = S_p \qquad \text{when } X_i' = 0$$

we can obtain,

$$S_i = e_{max} \cdot (L_1 \cdot X_i' - X_i'^2 / 2) / L_1 + S_p \tag{10}$$

where S_p and e_{max} are the slip and strain differences at section B, respectively. The value of S_p is approximately taken from the elastic analysis and has a little effect on the results.

The elastic beam theory gives the moment-curvature relation,

$$\phi = M/EI$$

The load-moment and load-deflection relation are

$$P = L(M/L_1 - W)/(L - L_1) \tag{11}$$

$$f = P \cdot L_1^2 (L - L_1)^2 / (3 \cdot EI \cdot L)$$

where EI is the equivalent stiffness.

In the computer program, first, the depths of compression zone in the concrete n_1 and in the steel sheet n_2 are assumed for a given curvature and the forces in the concrete, the steel sheet and the shear connectors are calculated according to Eqs.(4)-(10). Equilibrium equation (3) is then checked. If Eq.(3) is satisfied, the curvature is increased and the calculation proceeds. Otherwise the values of n_1 and n_2 are adjusted and calculated again. The results satisfying the equilibrium, compatibility, constitutive equations and slip distribution are obtained by using iterative procedure and include the strain distribution over the depth, stress in the concrete and the steel sheet, the load and the deflection of the slabs. Some results are shown in Figs.6 and 12. The details of the analytical procedure are presented in Ref./5/.

4. TEST RESULTS

4.1 Behaviour of Composite Slab

Typical load-deflection curves are shown in Fig.12. Also shown in the figure are the results from computer analysis.

Before the bond failure the behaviour of the slabs with or without shear connector groups or studs was not very different. The big drop in the load-deflection curves was caused by a loss of bond at the interface for the first tested end of specimens. When the test of the first end was completed the bond between the two materials was almost destroyed. Normally there was no drop in the load-deflection curves for the second tested end of specimens.

The difference between the specimens with and without shear connectors can be seen from comparisons of T11A and T17A with other specimens. The maximum capacity is obtained when the bond fails for specimens without shear connectors. The separation of the steel sheet and the concrete at the interface is observed in the shear span. Most specimens with different shear connector groups have better performance. Forces are taken over by the connectors and studs after the loss of bond. For specimens with 4 or 5 shear connector groups which correspond to 80% and 100% complete interaction, the behaviour is very ductile. At first, a bending crack is formed near the loading point. The width of the crack increases with load. Then the steel sheet yields. The maximum capacity is reached when a critical diagonal crack is formed. The slip and deformation of the steel sheet at the end and tearing of the steel sheet around the screws are observed, see Fig.11 and 13. The more shear connector groups in the shear span, the better the ductility and capacity of the composite slabs.

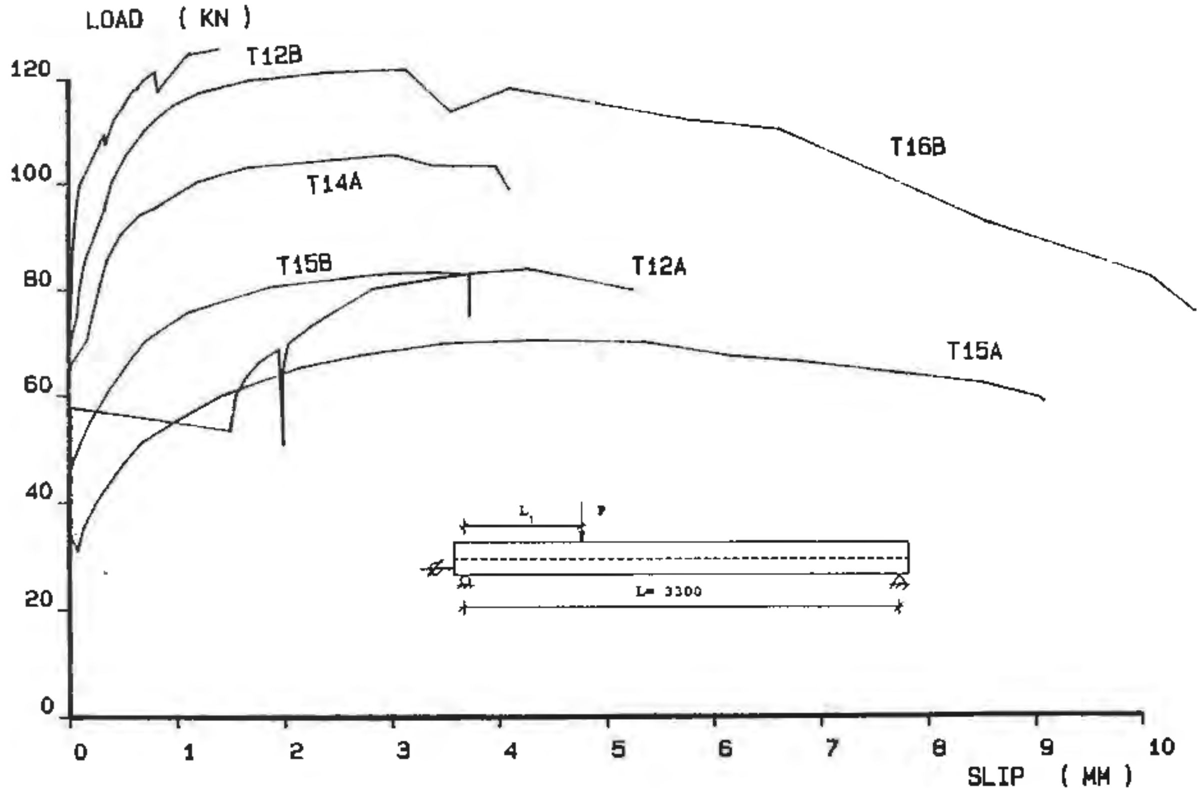
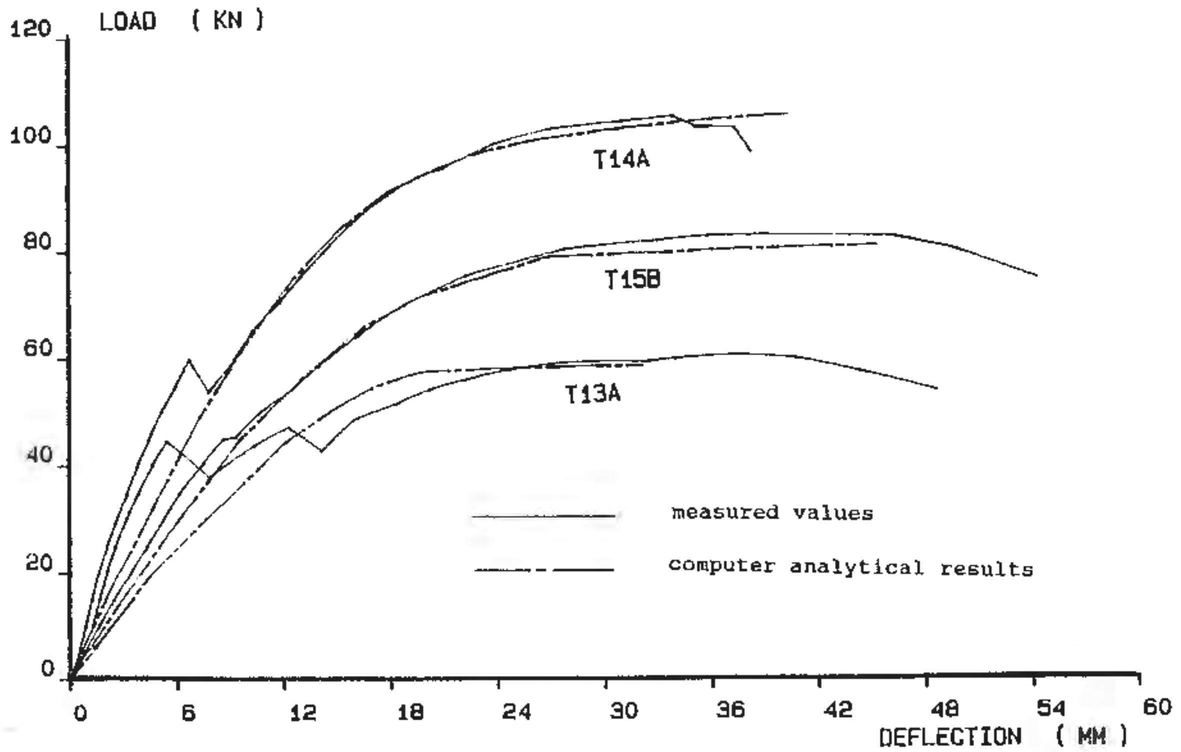


Fig. 11 Load-slip curves

LOAD-DEFLECTION



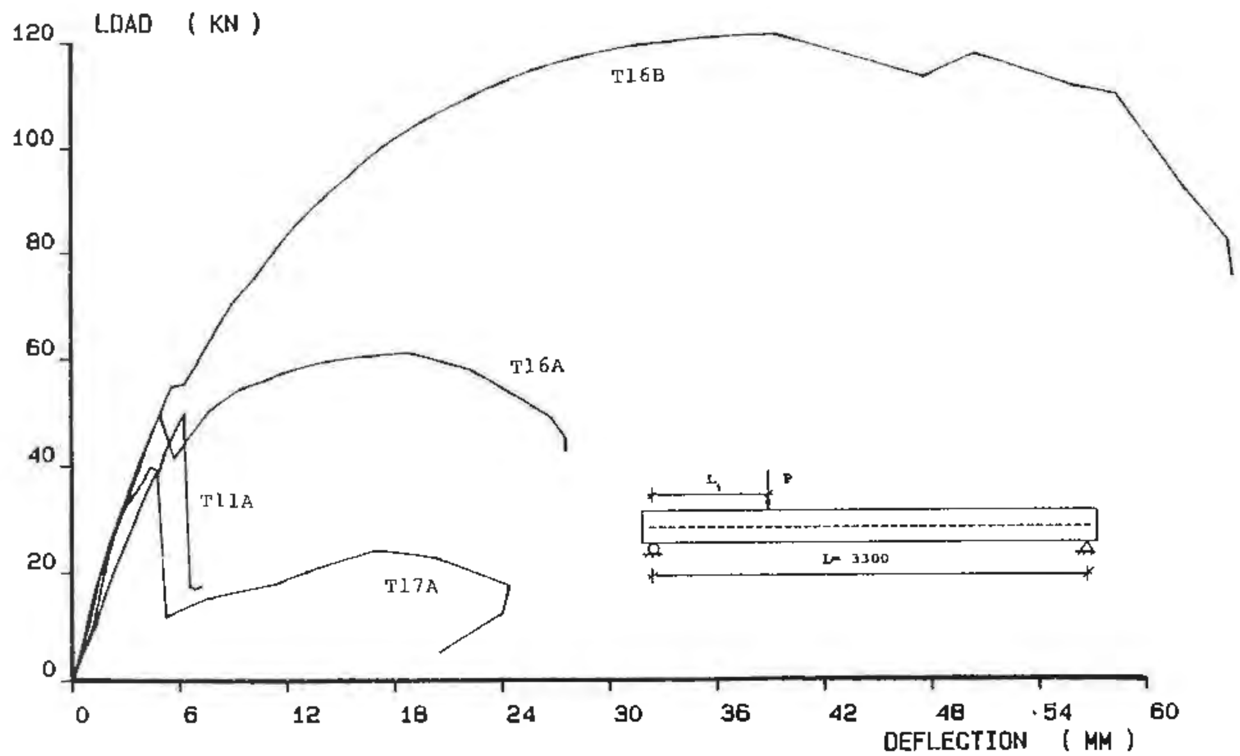


Fig. 12 Load-deflection curves

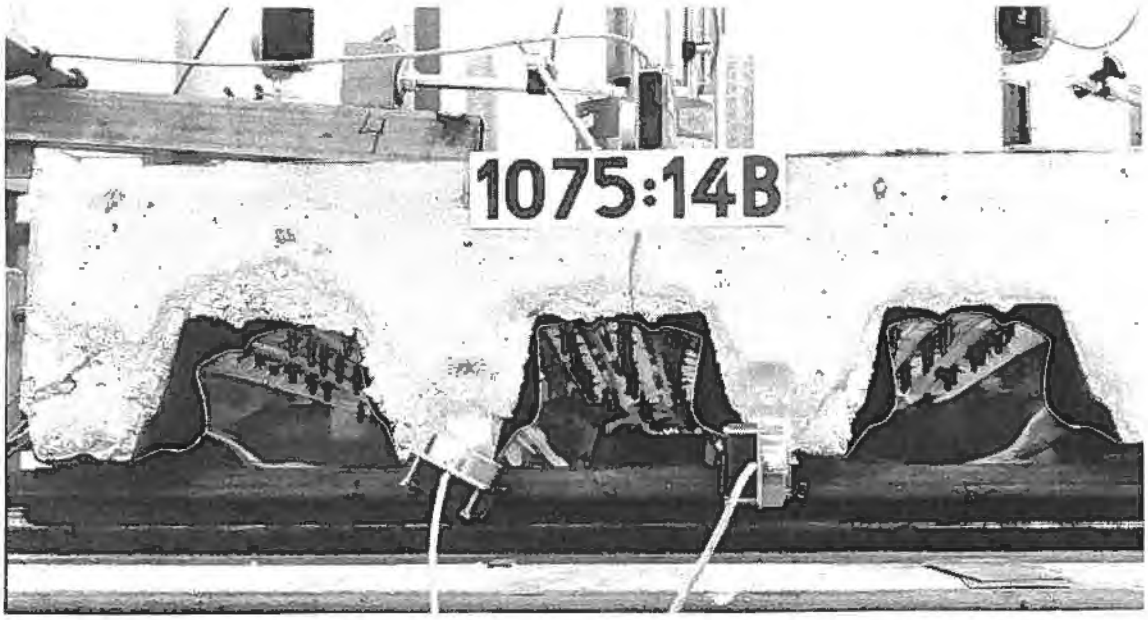


Fig. 13 Deformation of the steel sheet

4.2 Comparison between the Test Results and Theoretical Analysis.

Comparisons are listed in Table 1, and also shown in Fig.6 and 12. They agree quite well. The ultimate capacity calculated from the simple analytical method is a little higher than the real value in most cases. This method can estimate the capacity and is easy to use. In the step by step computer method, because the bond between the two materials and the tensile capacity of the concrete are neglected the computer results are lower than the test results in the beginning branches of the curves. The maximum capacity of the specimens is given according to the failure criteria and lower than the test value in most cases. This method not only gives the capacity of composite slabs, but also the results during loading.

In both the simple theoretical method and the computer method the shear force-slip relation for connectors greatly influences the calculated results. Unfortunately only very limited tests are carried out to determine this relation. Based on the test we chose one group of parameters which agrees quite well with the test results in our calculation. More tests are needed to determine a more accurate the shear force-slip relation.

The effect of the concrete strength on the ultimate strength of composite slab is difficult to explain (Fig.14 and Table 1). The capacity for K40 is higher than that for K20 for non-welded stud specimens. However, the situation is the opposite for specimens without any shear connectors because of the sudden failure for higher concrete strength. If we compare specimen T12B with T16B, in which everything is the same except the concrete strength, the capacity is almost the same. However, the greater the concrete strength, the better the deformation ability of the specimens (Fig.12).

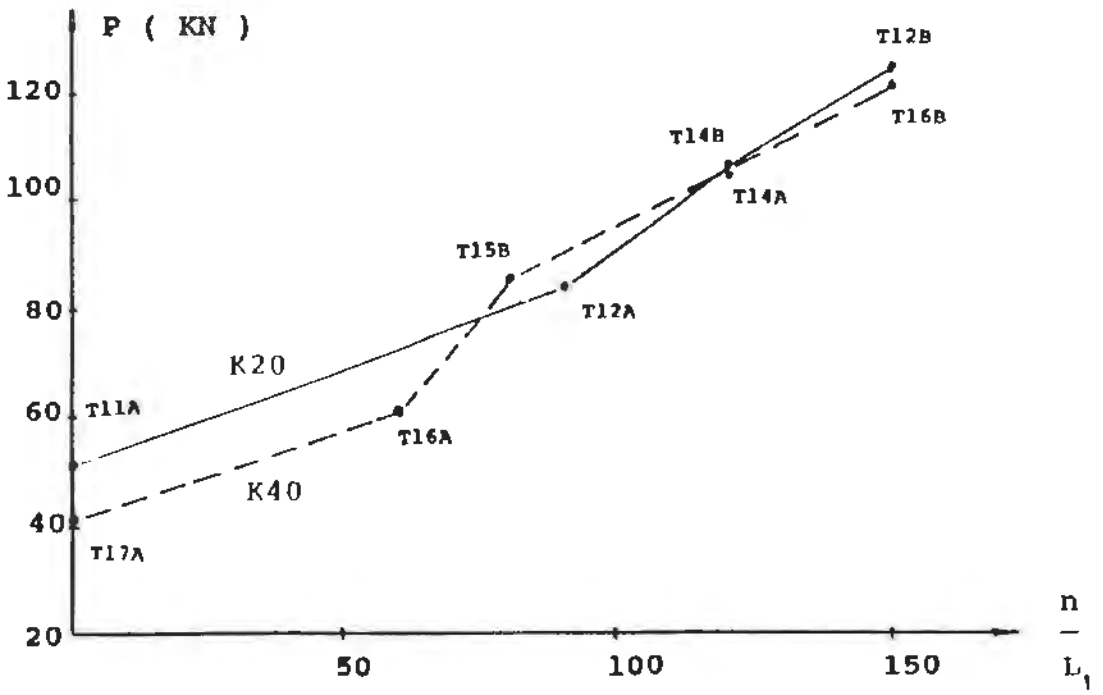


Fig. 14 The test results

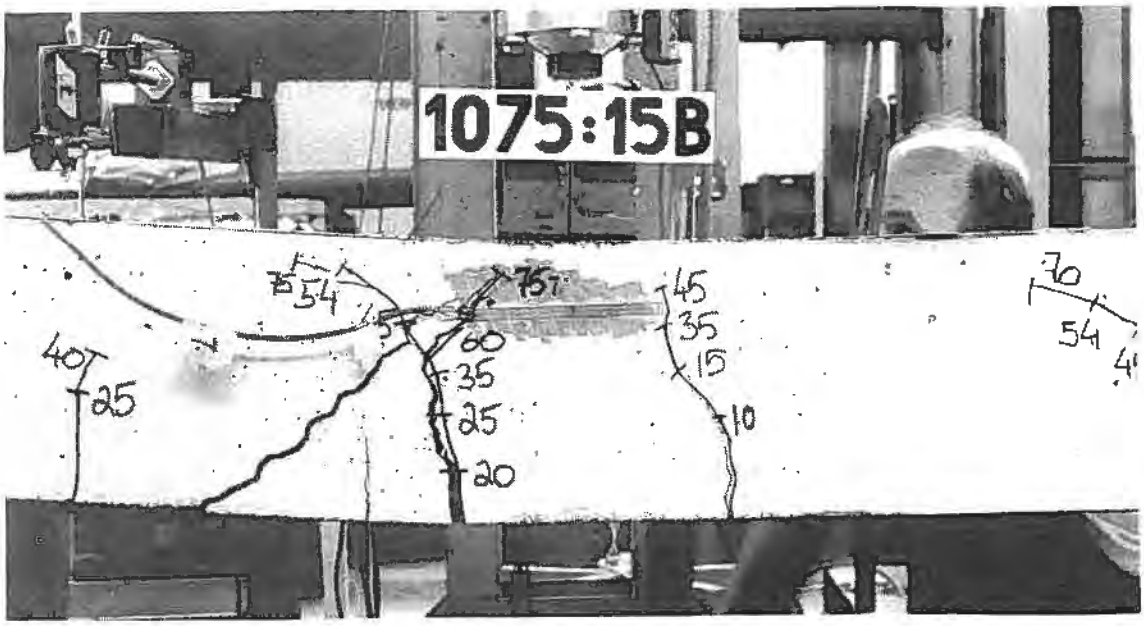


Fig. 15 Failure of composite slab

4.3 Failure Mode

The failure mode for specimens without shear connectors is the shear-bond without yielding of the steel sheet. For most specimens different number of shear connectors are used and the failure mode is rather complex. If the concrete is relatively weak, the failure reason is the crushing of the concrete, such as in specimen T12B. Otherwise the failure will be due to anchorage, that is, the shear failure along the horizontal direction. The non-welded stud specimens fail due to the tearing of the steel sheet around the studs. At the ultimate state, a critical diagonal crack is formed at the loading point, see Fig.15.

5. CONCLUSION

1. The results obtained from the theoretical analyses agree quite well with the test results, and this implies that the assumptions made in the analysis are reasonable. These theoretical analyses provide methods which have more direct physical meaning for predicting the capacity of composite slabs with different shear spans, different numbers and spacings of shear connectors and different concrete strengths. A simple analytical method can give the ultimate capacity and be easy to use. The computer method can follow the loading procedure until failure.

2. The failure mode for specimens without shear connectors was shear-bond without the yielding of the steel sheet.

3. The behaviour and failure mode are rather complicated in the specimens with different number of shear connectors and studs and depend on both the concrete compressive strength and anchorage at the interface. The slip and yielding of the steel sheet are experienced prior to failure. The bond has little effect on the capacity of the specimens.

4. The ductility and strength of specimens increase with number of shear connectors. For concrete K20, the slab with complete interaction increases the carrying capacity by 250% in comparison with slab without connectors. For concrete K40, the increase is 300%.

5. In our theoretical analytical models which take the strain difference at the interface and deformation of shear connector into account, the shear force-slip relation of connectors has a great influence on the results. More research is needed in order to obtain more accurate results.

6. ACKNOWLEDGEMENT

This study was carried out at the Division of Concrete Structures, Chalmers University of Technology. We are grateful to The Foundation for Swedish Concrete Research and The Swedish Council for Building Research for their financial support. We would also like to express our thanks to the Faculty Staff in the laboratory for their assistance.

7. REFERENCE

/1/ "Steel-Concrete Composite Structures, stability and strength". Edited by R. Narayanan, 1988.

/2/ Cederwall, K " Some ideas and studies concerning the ultimate capacity of steel and concrete elements" Nordisk Betong 5, 1988, p8-14.

/3/ R.P.Johnson "Composite structures of steel and concrete, Volumn 1. Beams, Columns, Frames and Application in Building" Reprinted 1984 by Granada Publishing.

/4/ R.P.Johnson " Loss of Interaction in Short-Span Composite Beams and Plates" Journal of Constructional Steel Research, Vol.1, No.2, January 1981.

/5/ An Li, Cederwall,K "The Experimental Study on the Composite Slabs with Steel Sheet", Division of Concrete Structures, Chalmers University of Technology, Report 89:3.

8. NOTATION

A	area of the steel sheet
B^s	width of the composite slabs
C	compression force in the concrete
d	the distance from the extreme compression fibre to the centroids of the steel sheet
D	depth of concrete over the steel sheet
D^1_2	the distance from the centroid of the steel sheet to the interface of two materials
E	the modulus of elasticity of concrete
E^c	the modulus of elasticity of the steel sheet
e^s	strain difference at the interface
e	strain difference in the loading section
F^{max}	shear force transferred by a single screw
f^{con}	the yield strength of the steel sheet
f^y	deflection
f'	the cylinder strength of concrete
L^c	length of span
L	lengh of shear span
J^1	moment of inertia of the steel sheet
M^s	resistance moment by the steel sheet
n^s_1, H_c	depth of compression zone in the concrete
n^1	depth of compression zone in the steel sheet
S^2	slip
S	slip at the loading section
S^p	ultimate slip for connectors
R^u	the force in the support
W	dead weight
σ	the mean value of concrete stress
σ^c	stress in the steel sheet
ϕ^s	curvature
ϵ	strain in the concrete
ϵ^c_s	strain in the steel sheet