DIAPHRAGM ACTION IN PRECAST HOLLOW-CORE FLOORS

Description of a pilot test series

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This is a report on a pilot test series on precast full scale hollow core slabs. The influence of initial cracks in the joints, of vertical indentations on the joint faces and of different tie arrangements and boundary conditions was studied.

Keywords: diaphragm action, precast hollow core floor, joint, cracks

1. INTRODUCTION

Diaphragm action in precast floor slabs is frequently utilized for distribution of horizontal forces to the stabilizing structures in a building. Thus the floor diaphragms are important parts of the stabilizing system. However, due to the restraint within the floor slab, cracks may occur in the joints between the slab panels. It is uncertain what influence the cracks have on the behaviour of the floor when it acts as a diaphragm and how shear forces can be transferred in the cracked joints.

A pilot test series was carried out in 1984 at Chalmers University of Technology, Division of Concrete Structures, with the aim of studying the behaviour of precast slabs when loaded in in-plane bending.

In this pilot test series the influence of the following parameters was studied:

- initial cracks in the joints
- indentations on the joint faces
- tie arrangements
- boundary conditions of the floor slab

2. DESCRIPTION OF THE TESTS

Six tests were included in the test series. Each test specimen consisted of seven hollow core panels, see Fig. 1.

The joints between the slab panels were grouted and the outermost panels were connected to the edge beams by 8 stirrups (ϕ10 Ks40) anchored in concreted recesses in the panels. The edge beams simulated stabilizing walls.

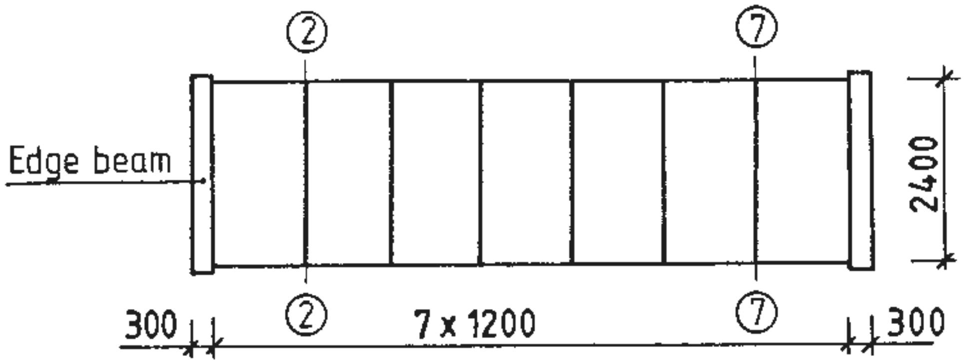


Fig. 1 The test specimen

In each test but one, initial cracks with a width of 0.5 mm were produced in two of the joints, joints (2) and (7) in Fig. 1.

In two of the tests the joint faces of the slab panels were provided with vertical indentations at a depth of about 2 mm. In the other tests the joint faces were unindented, and thus rather smooth.

Two different tie arrangements were used in the test specimens, type A with tie reinforcement placed in cast-in-situ tie beams along the chords and type B where the support beams were used as tie beams. Fig. 2 shows a longitudinal section of the floor slab of type A.

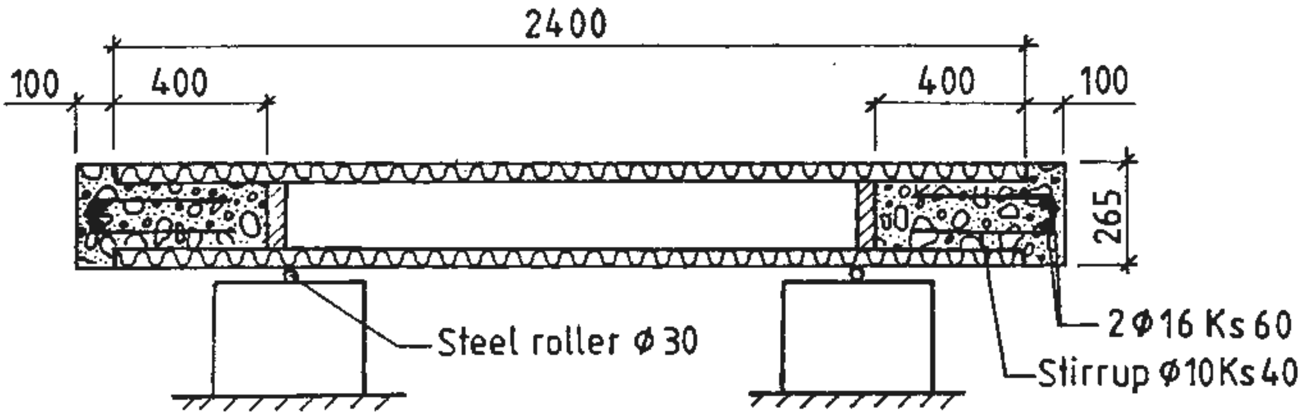


Fig. 2 Longitudinal section of the floor slab of type A

As illustrated in Fig. 2 the tie reinforcement, 2 bars $\phi 16$ Ks60, was placed in a cast-in-situ tie beam along the chords. At the ends of the floor slab the bars were anchored in the outermost cores close to the edge beams, see Fig. 4. The tie beams were connected to the slab with open stirrups placed around the reinforcement bars and anchored in the middle core of every hollow-core panel.

In the floor slabs of type B each hollow-core panel was connected to the support beams by dowel pins $\phi20$ Ks40, see Fig. 3.

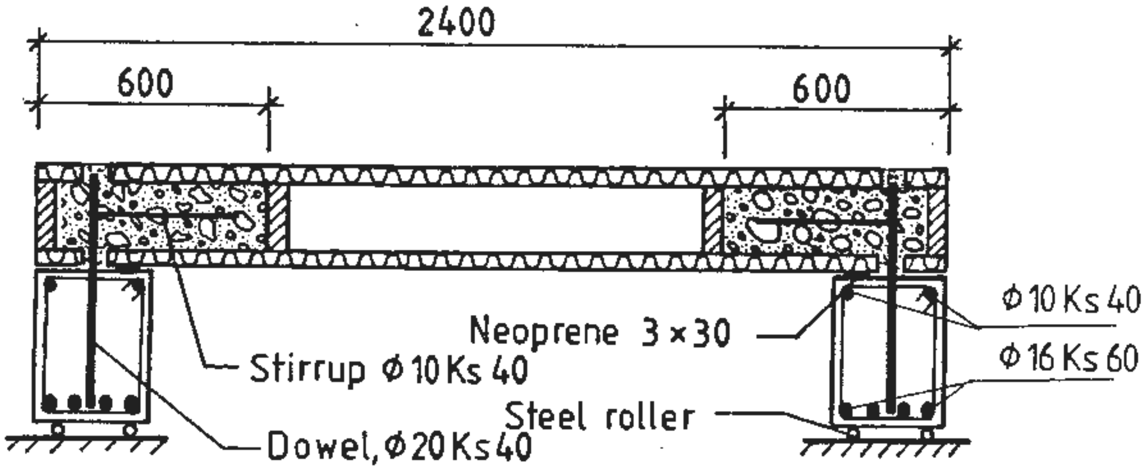


Fig. 3 Longitudinal section of the floor slab of type B

The following concrete qualities were used for the test specimens:

The hollow core panels: K55
The edge beams and the support beams: K40
The cast-in-situ tie beams and complementary concreting around dowels etc: K25
The joint grout: K25

The concrete compressive strength was measured on 150 mm cubes, and the results are presented in Table 1.

Table 2 gives the results from the tensile tests on the reinforcement bars.

Table 1 The compressive strength, [MPa], measured on cubes 150x150x150 [mm^3]. The test results refer to mean values in series of three tests.

Test No.	The joint grout	The tie beams	The support beams
1	30.4	42.5	-
2	33.2	49.1	-
3	29.8	37.0	-
4	31.9	31.6	-
5	35.4	-	58.7
6	28.1	-	47.7

Table 2 The characteristics of the reinforcement bars (Ribbed bars of hot-rolled steel)

Type	Area [mm^2]	Yield strength [MPa]	Modulus of elasticity [MPa]	Ultimate strength [MPa]	Ultimate strain [‰]
ϕ10 Ks40	78.5	468	206	638	178
ϕ20 Ks40	306.7	557	224	711	126
ϕ16 Ks60	196.1	693	222	907	176

The load arrangement and the various boundary conditions are shown in Fig. 4.

With respect to in-plane loading the floor slabs were indirectly supported by the edge beams. Thus the entire load was transferred through the outermost joints from the floor slab to the edge beams, which were separately supported. Since the tie reinforcement in tests no. 1-4 was anchored into the floor slab in the outermost cores it was possible to avoid its being affected by the lateral support forces. In one test (test no. 3) the edge beams were prevented from free rotation in the plane of the floor (type A:2) in order to produce restraining effects at the edges. In the other tests the floor slab was simply supported (A:1, B).

In all the tests the same loading procedure was used. In the initial stage the floor slab was loaded with two equal point loads acting on panels no. 3 and 5 according to Fig. 4. The load was increased step by step using two hydraulic jacks. The initial stage lasted until primary shear failure occurred in one of the joints. After the first joint failure, the jack closest to the ruptured joint was locked at the actual achieved displacement. In the next stage only the other jack was adjustable, so that in order to produce a joint failure also in the other end of the slab. In this stage, too, the load at the active jack was increased step by step. The last stage of the test began when a symmetrical joint failure pattern had been produced. Then both the jacks were used actively but controlled by their mean displacement.

The test programme is presented in Table 3.

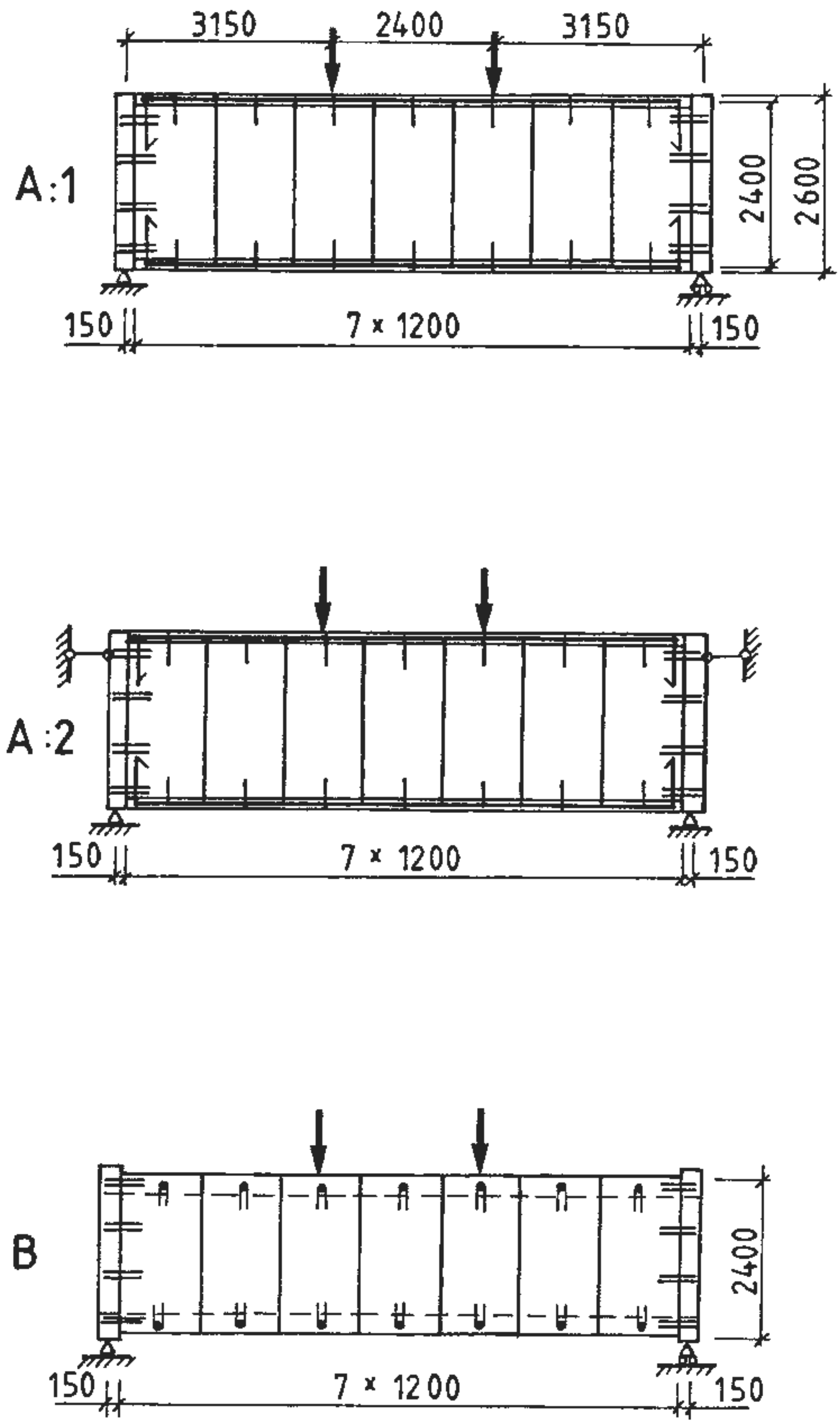


Fig. 4 The load arrangement and the various boundary conditions and tie arrangements

Table 3 The test programme

Test No.	Arrangement	Vertical indentation on the joint faces	Precracked joints (joint No. 2 and 7)
1	A:1	No	No
2	A:1	No	Yes
3	A:2	No	"
4	A:1	Yes	"
5	B	No	"
6	B	Yes	"

3. TEST RESULTS

3.1 Slabs of type A (tests no. 1-4)

Shear failure in a joint was accompanied by simultaneous shear failure in the cast-in-situ tie beams, see Fig. 5.

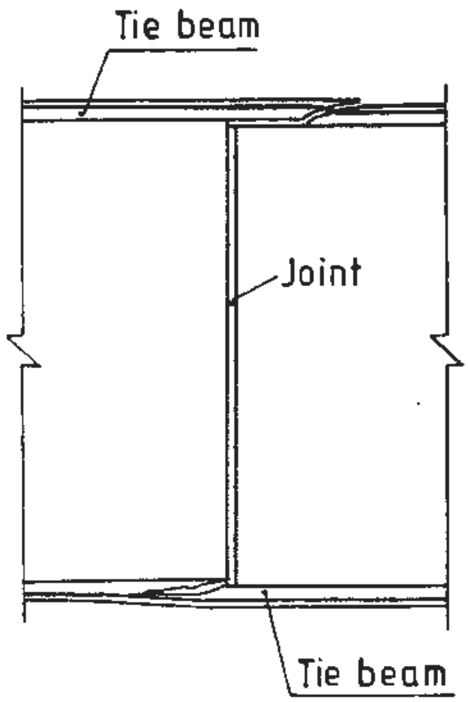
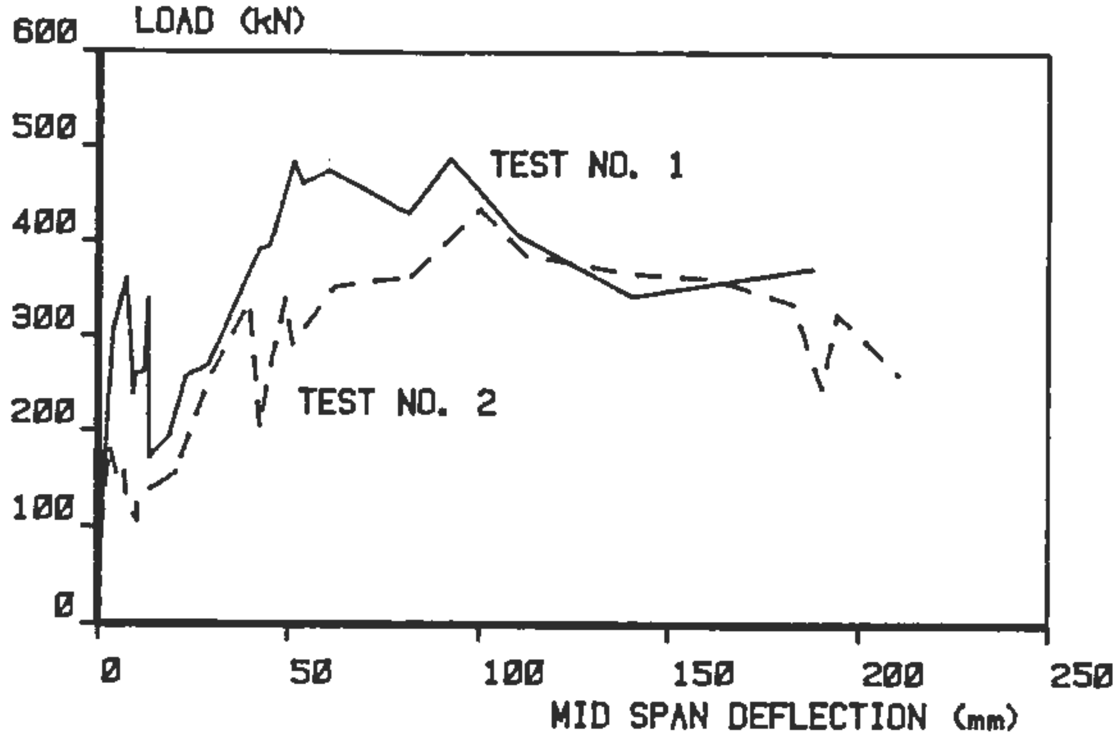


Fig. 5 Example showing shear failure in a joint and in the tie beams

The tie reinforcement had not in any test reached the yield strength when the joints failed in shear.

After the first combined shear failure it was possible to increase the load as the floor slab was deformed in in-plane bending, see Fig. 6. The deformations involved large relative displacements in the joints. Finally the load capacity reached maximum and decreased when the compression zone of a slab panel was destroyed by severe cracking.



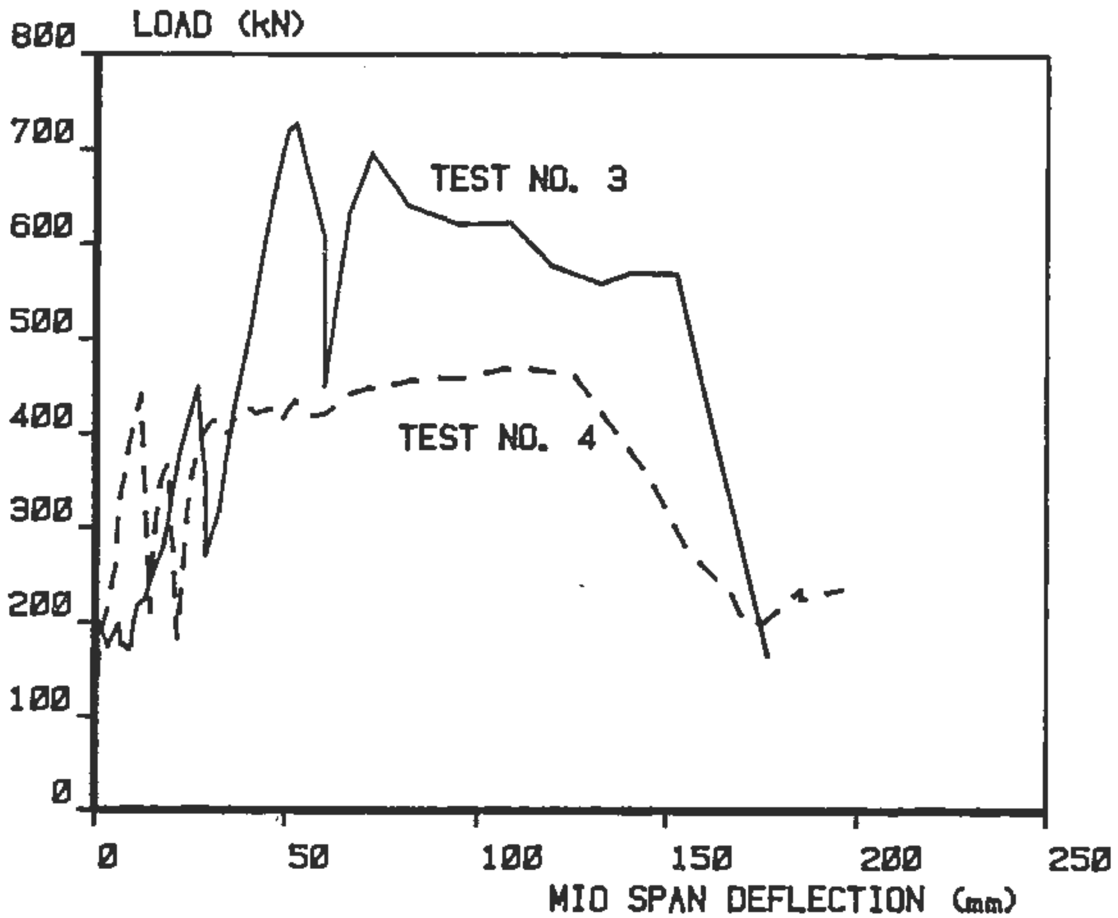


Fig. 6 The relations between the mid-span deflection and the total load for tests no. 1-4

As an example Fig. 7 shows the slab of test no. 2 at the maximum load. A symmetrical crack pattern has been produced and the in-plane deformations forced on the slab have involved joint failures and relative displacements in several joints.

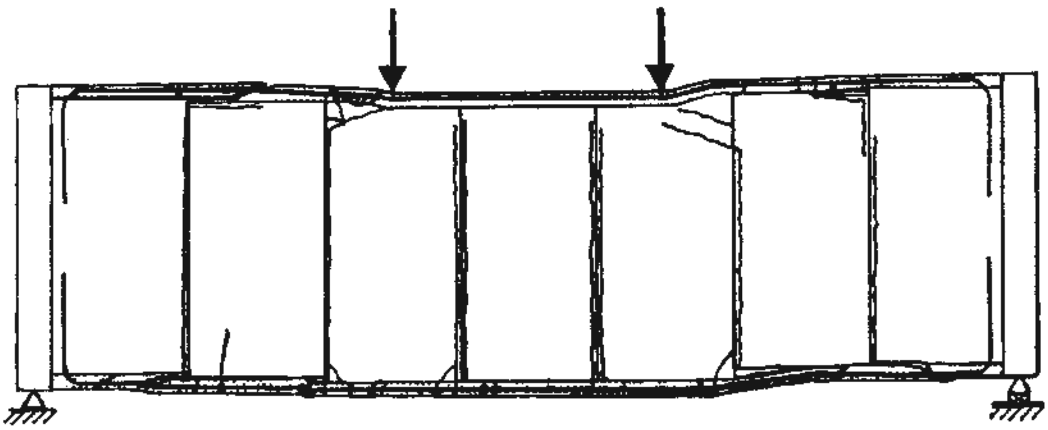


Fig. 7 Slab at the maximum load (test no. 2)

Table 4 gives the actual shear forces which produced combined shear failures in the joints and the tie beams. The numbers in parentheses indicate the sequence of the joint failures. In tests no. 1 and 2 the support reactions were not measured. Therefore the shear forces presented in Table 4 have been estimated from the registered forces at the hydraulic jacks and are presented in parentheses in Table 4. However, in tests no. 3 and 4 the actual support reactions were measured and, as is obvious from Table 4, there were some discrepancies from the estimated values.

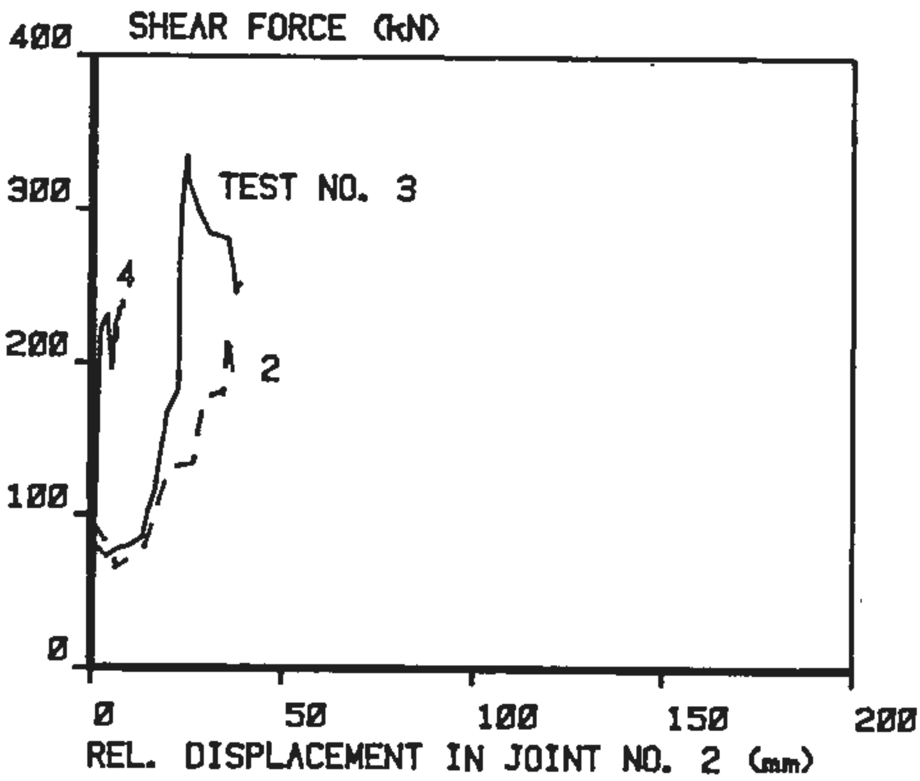
Table 4 also gives the registered maximum values of the total load resisted by the floor slab in in-plane bending.

Table 4

Test No.	Shear force at joint failures [kN] Joint No. 2[1)]	3	6	7[1)]	Maximum total load on the floor slab [kN]
1	-	(198) (2)	(183) (1)	(146) (3)	488
2	(93) (1)	(170) (3)	(169) (4)	(100) (2)	435
3	81 (1)	202 (3)	369 (4)	86 (2)	727
	(101)	(227)	(360)	(98)	
4	230 (2)	230 (2)	222 (1)	-	469
	(235)	(235)	(218)	-	

1) In tests no. 2-4 joints no. 2 and 7 were precracked.

Examples from the tests showing the relations between the shear force and the relative displacement in the joint are presented in Fig. 8.



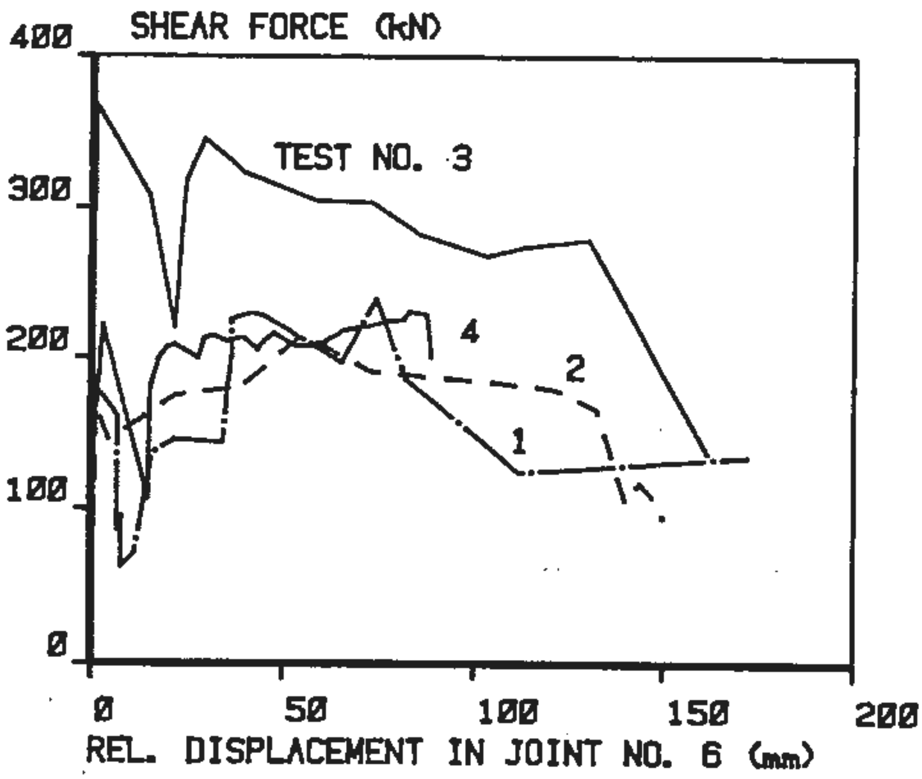


Fig. 8 Examples from the tests showing the relations between the shear force and the relative displacement in the joint. In tests no. 2-4 joint no. 2 was precracked. In test no. 1 there was no failure in joint no. 2. In test no. 3 the edge beams were prevented from free rotation. In test no. 4 the joint faces of the panels were provided with vertical indentations.

The test results indicated that

- When two of the joints were precracked the first combined shear failure in a joint and the tie beams occurred for a much lower shear force than with an uncracked floor slab.

- The restraint on the edge beams in test no. 3 had very little effect on the shear force at the first combined shear failure in the precracked joints (joints no. 2 and 7) but involved markedly increased resistance at the uncracked joints (joints no. 3 and 6).

  The restraint on the edge beams also involved an increased total load capacity.

- When the joint faces of the panels were provided with vertical indentations, the shear resistance of the joints increased compared with smooth interfaces. This was most obvious in the precracked joints.

3.2 Slabs of type B

Fig. 9 shows the relation between the mid span deflection and the registrated total load for the floor slabs of type B.

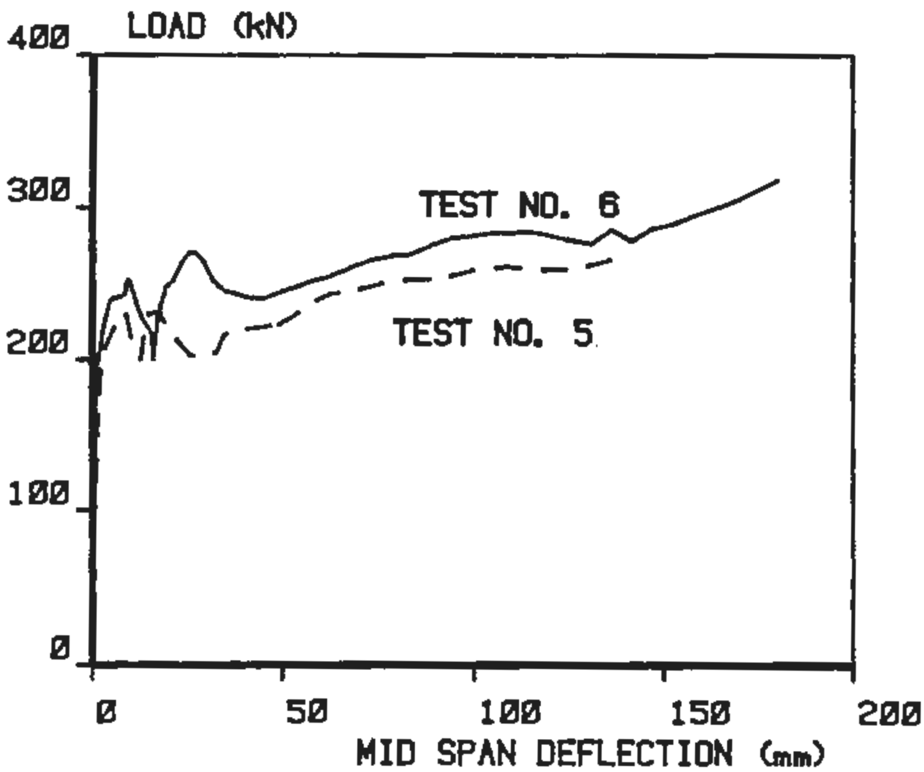


Fig. 9 The relation between the mid-span deflection and the total load for tests no. 5 and 6

The first joint crack was observed for a total load of about 200 kN. This crack could be characterized as a bending crack, and occurred at the tensile edge near mid-span. Then, after a small further increase of the load, a sudden relative displacement occurred in one of the precracked joints. Later during the test procedure relative displacements were also produced in other joints. The maximum load was determined by a sudden rupture of the outermost panel. The rupture was brittle and caused by a crack running almost parallel with the edge beam, see Fig. 10.

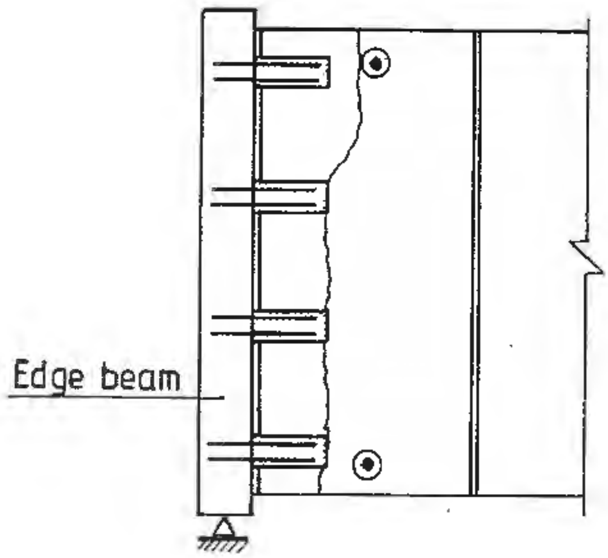


Fig. 10 Rupture of the outermost panel (test no. 5)

In test no. 6 the joint faces were provided with vertical indentations but the effect of the indentations was negligible (see Fig. 9). This may be because this type of tie arrangement does not give a sufficient connecting effect orthogonal to the joint faces.

3.3 Comparison between the type A and type B slabs

The results from the test series have proved that the two different tie arrangements lead to quite different slab behaviour.

In the type A slabs shear failure in a joint was accompanied by simultaneous shear failure in the cast-in-situ tie beams. Thus the shear failure level is dependent on the capacity of the tie beams. However, it is not clear how the shear force transfer is distributed between shear transfer in the joint faces and shear transfer in the tie beams. This distribution depends on the shear stiffness of the tie beams. In the case of smooth interfaces and initial cracks in the joints it is reasonable to assume that the main part of the shear force is transferred in the tie beams.

In the next stage, after the shear failure in the tie beams has occurred, the shear force is transferred in the joint faces. As the shear slippage goes on, the shear force capacity in the joint increases, see Fig. 11.

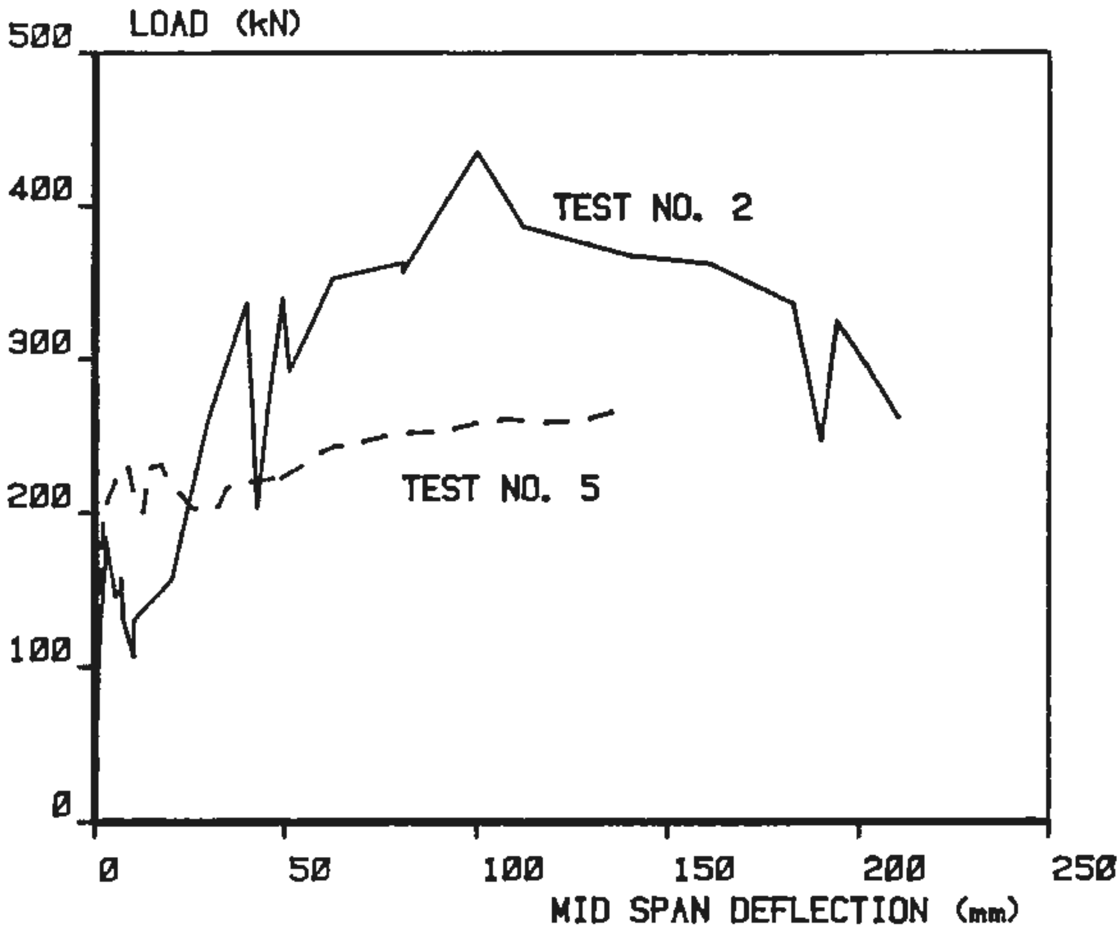


Fig. 11 The relation between the mid-span deflection and the total load for test no. 2 (type A slab) and test no. 5 (type B slab)

On the assumption that the capacity of the dowels is sufficient, the behaviour of the type B slab is, greatly dependent on the behaviour of the support beams. Since the panels are fixed at the dowels, the movement of the panels is determined by the deflection of the support beams. When the deflection of the support beams increases the crack widths on the tensile edge of the slab increase and, gradually, relative displacements between the panels occur. The panels then just rotate around the dowels.

After the reinforcement in the support beams has reached the yield strength the capacity of the support beams becomes almost constant. Therefore the load capacity of the slab becomes almost constant, as the floor slab is deformed in in-plane bending, see Fig. 11.

This pilot test series will be supplemented by further tests. This work will also include theoretical modelling of failure mechanisms. The results will be presented in a forthcoming article.

ACKNOWLEDGEMENTS

I would like to thank Professor Krister Cederwall, Head of the Division, and Research Assistant Björn Engström for their valuable help in connection with the layout of the test series and the preparation of this paper.

REFERENCE

/1/ Svensson, S: "Skivverkan i elementbjälklag. En serie pilotförsök". Chalmers tekniska högskola, avd Betongbyggnad, Rapport 85:2, Göteborg 1985.

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