JOINTS AND DETAILS IN A PREFABRICATED COLUMN-SLAB SYSTEM - DESIGN BY TESTING

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Synopsis

The development of a column - slab system for use in industrialized apartment buildings is in progress. Experimental testing is used to evaluate the strength and ductility of certain parts of the structure, for which ordinary strength calculations are considered unreliable or even impossible. The paper describes tests on different arrangements of horizontal ties in the floor connections, and tests on the shear capacity of the floor slabs.

INTRODUCTION

In the fall of 1983, the Danish Ministry of Housing invited all companies in the building industry to participate in a competition on the further development of the industrialized apartment building. The purpose of the competition was to develop technical solutions, which can meet new construction and housing needs.

The first prize was awarded to a project prepared by the Arkitektgruppen i Aarhus A/S, Viggo Michaelsen A/S, consulting engineers, and Højgaard & Schultz A/S. Their project is based on a column-slab system, stabilized by concrete walls filled in between some of the columns. The system is illustrated in figure 1.

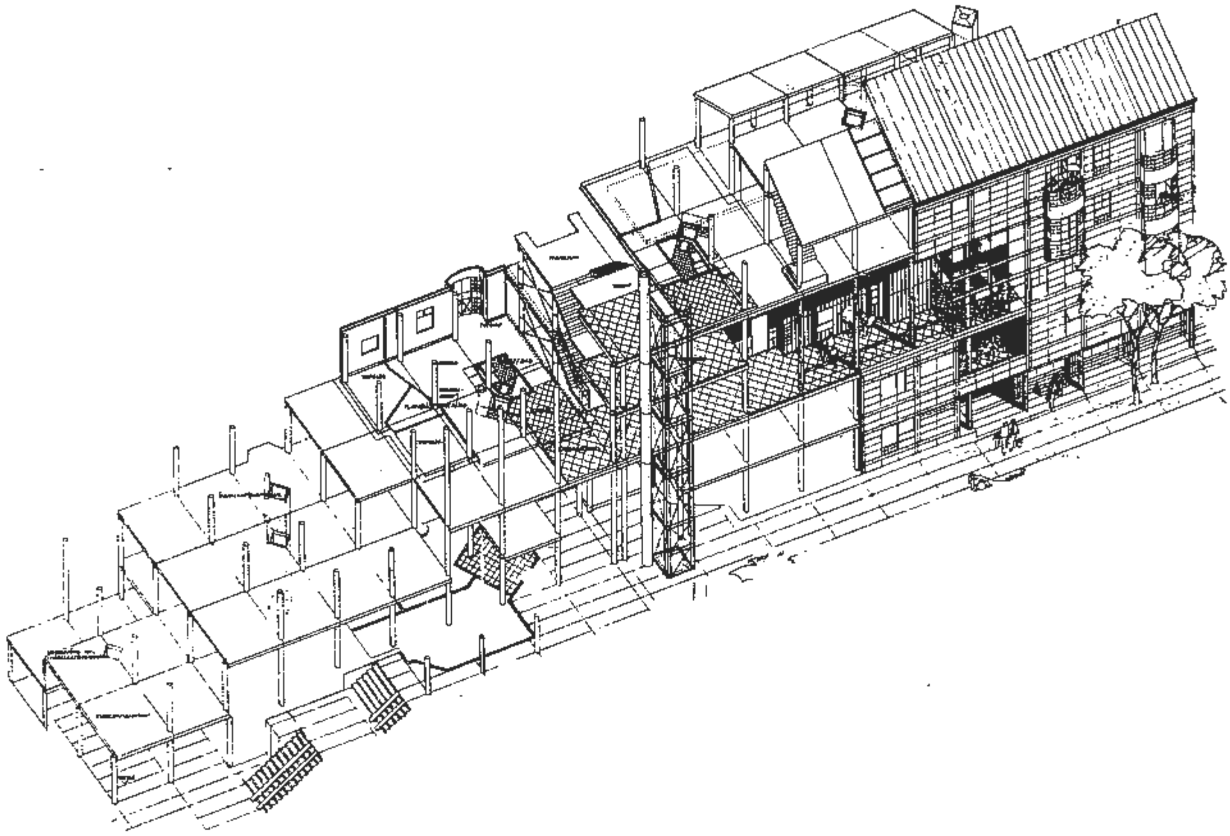

Figure 1: The column-slab system.

Following the competition, the project ideas have been further developed in a number of different areas, part of it being experimental research on the strength and stiffness of the load carrying system. This paper describes the program completed so far.

The system consists of solid reinforced concrete slabs, which are supported in each corner on columns with a 280 mm circular cross section. The columns are placed in a modular system of 3,3 x 3,3 m. The floor slabs are provided with protruding bent bars, so called "hair pins", in each corner so that horizontal ties between the floor slabs can be established in the joint between columns and slabs. Standard keyed shear joints between the floor slabs in combination with these ties, enable the floor to act as a diaphragm which transfers the horizontal loads on the building to the shear walls.

A preliminary evaluation revealed, that the critical points in the project were likely to be:

1) Strength and stiffness of the ties in the diaphragm especially at the facade.
2) The shear strength and ductility of the floor slabs in the support zone.
3) Strength with respect to vertical loads through the column-slab-joint.

The detailed design of the slab corner, and of the column-slab joint, must take into consideration a number of other requirements than just the static ones. Thus requirements and wishes regarding both architecture, production, erection and economy have influenced the solutions which were tested in the research program. A further discussion of this influence falls outside the scope of this paper.

TESTS ON TIES

Program

The maximum tie forces in a slab will normally appear along the facade. As shown in figure 1, the floor slabs along the facade have an overhanging part outside the module line through the facade columns. The tie tests were therefore performed with in an arrangement simulating a facade joint between slabs and columns. The principle is shown in figure 2.

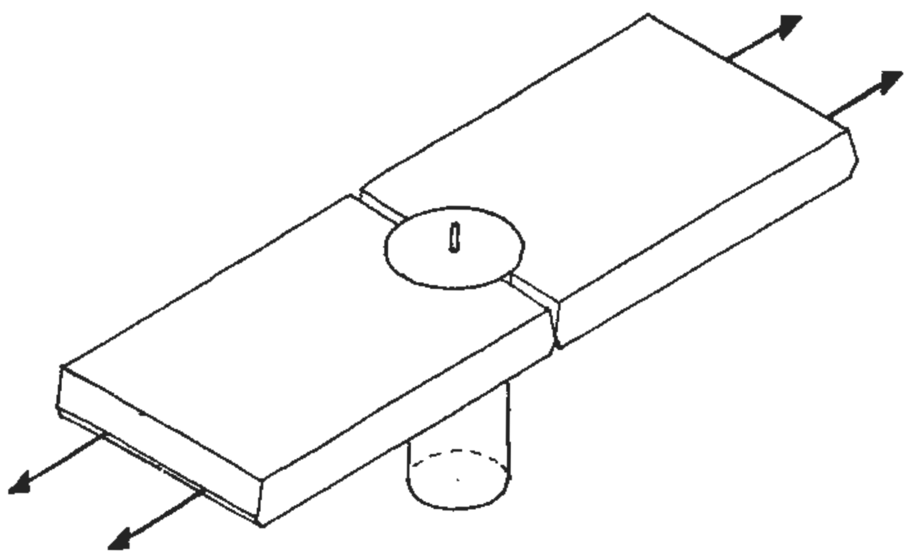

Figure 2: Arrangement for testing of ties.

The specimen was loaded with a tensile force transferred to the ends of the slabs through anchored inserts. The force transfer in the joint was established by means of a protruding horizontal hair pin in each floor slab, a reinforcement unit connecting the two projecting hair pins, and the concrete in the joint. Figure 3 shows examples of the joint before the concrete is cast.

The program includes a total of 12 tests incorporating different hair pins and reinforcement units, c.p. figure 4. Moreover, different joint concretes were used. The size and shape of the hair pins were varied, but in all cases they were made of deformed bars with an anchorage length of ab. 800 mm and they were placed directly on top of the standard reinforcement mesh in the slab.

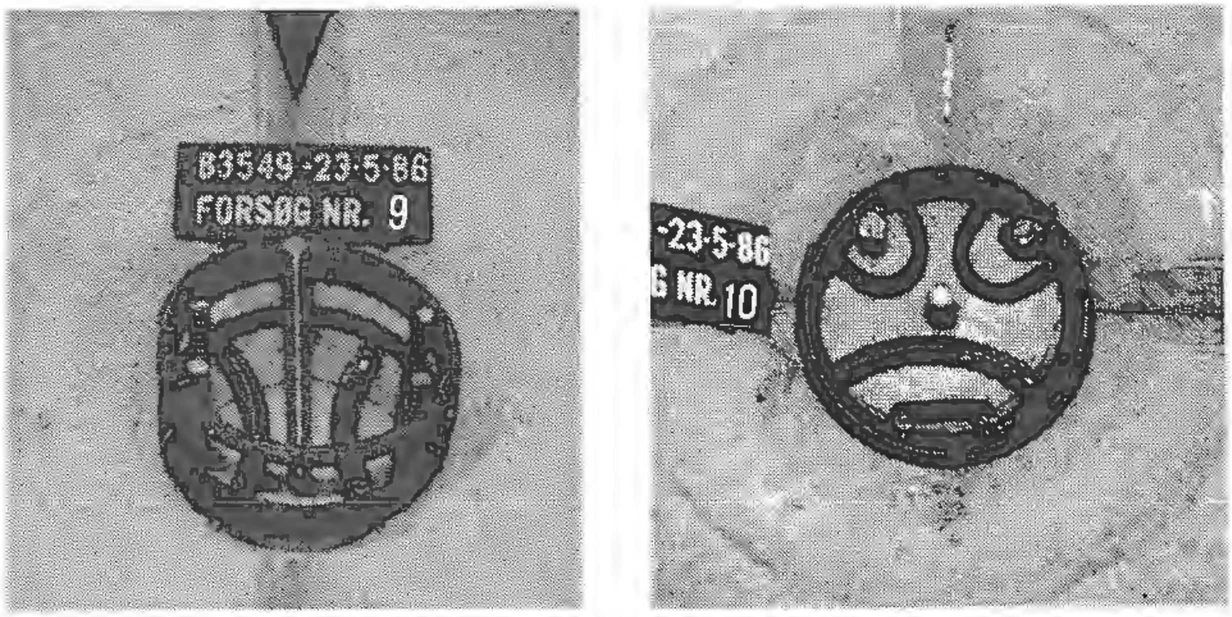

Figure 3: Examples of tie connection, showing protruding hair pins and reinforcement unit.

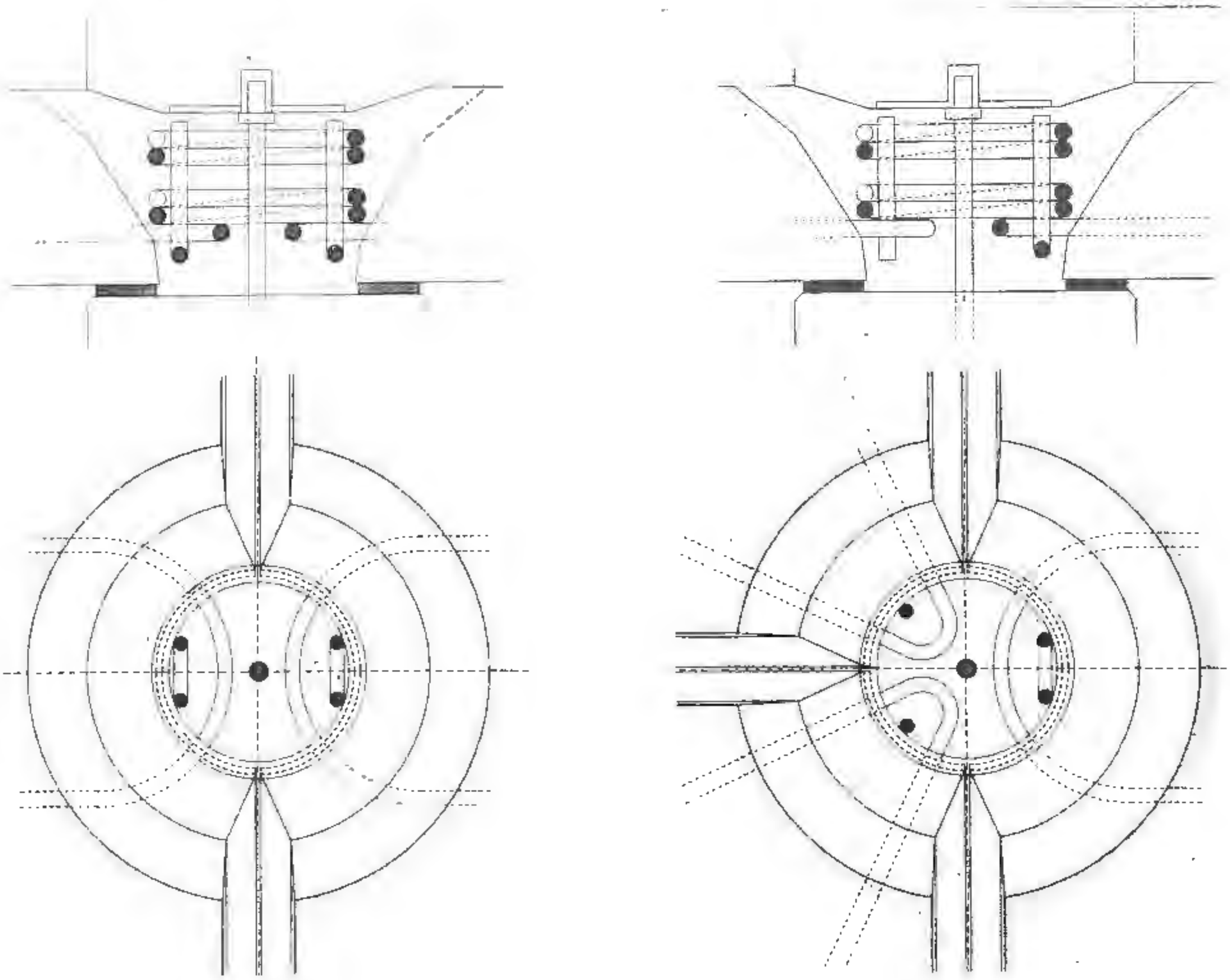

Figure 4: Vertical and horizontal section in tie connections.

The reinforcement unit consisted af two horizontal reinforcement rings, held together by two vertical dowels. In one case, the dowels were of straight bars, and in the other cases they were u-shaped. The dimensions of the rings as well as of the dowels were varied. In the first four tests, Densit mortar was used as joint concrete, and in the remaining tests more ordinary joint mortars of high strength with or without silica were used. All the applied mortars had a consistency that allowed casting without the use of vibration. A survey of the test parameters is given in figure 5.

Test	Hairpin	Reinforcement unit		Joint mortar		Remarks
		Rings	Dowels	Type	Strenght	
no	diam mm	diam mm	diam mm		MPa	
1	12	12	12 (U)	Densit	96	
2	12	12	12 (U)	Densit	96	
3	12	12	12 (U)	Densit	100	Inclined dowels
4	12	12	12 (U)	Densit	100	One ring below hairpin added
5	12	12	16	w.o. MS	46	Straight dowels
6	12	12	12 (U)	w.o. MS	46	
7	12	12	12 (U)	w. MS	80	
8	16	12	12 (U)	w. MS	80	
9	16	12	12 (U)	w. MS	82/69	Construction joint above lower ring
10	16/10	12	12 (U)/16	w. MS	82	Joint as fig. 3b and 4b
11	12	10	12 (U)	w. MS	71	Each ring 2½ revolution Slabs 19 mm apart
12	12	10	12 (U)	w. MS	71	Each ring 2½ revolution

Figure 5: Summary of tie test program.

The load was applied gradually, and at each load step, the relative displacement between the two floor slabs, in the direction of the tensile force, was measured. The measurements were made along the edges of the two slabs at the level of the hair pins.

Results

The results of the tests have been summarized in figures 6 and 7, which shows load deformation curves for deformations up to 10 mm and 1 mm respectively. In all the tests, a load of 20-30 kN could be applied before visible crack formations appeared in the construction joint between one of the slabs and the joint mortar. A further load increase primarily resulted in increasing crack width in the construction joint. Possibly, a few new cracks appeared in the joint mortar itself, or in the other construction joint.

In tests 1-4 (where Densit mortar was used) the maximum load was reached, when the actual failure occurred at a deformation of approx. 40-50 mm. In the remaining tests, the maximum load was attained at somewhat smaller deformations, and larger deformations were obtained under decreasing load.

In tests 3 and 4, one of the protruding hair pins failed. In all other tests the failure occurred by bending of one of the dowels so that the corresponding hair pin would gradually slip out under the deformed dowel.

Each reinforcement ring had overlaps over half the circumference, and could therefore at most rest on the hair pins in 3 out of 4 crossings. It was observed, that there was always a gap between the hair pin and the lower reinforcement ring at one of the crossings in the same side as the dowel which was bent.

The best result from a strength point of view was obtained in test 4 with a combination of a high mortar strength (Densit) and an additional ring in the reinforcement unit placed under the protruding hair pins. In practice, however, the solution is relatively expensive, partly due to the mortar price, partly because the additional ring is difficult to install on site.

The least favourable result was found in test 5, which applied a relatively low mortar strength and straight bars as dowels. A comparison with the result from test 6 seems to indicate, that the most efficient interaction between the joint reinforcement and the joint concrete is obtained with the U-shaped dowel rather than the straight dowel.

The remaining tests 6-12 gave rather uniform results, cp. figure 6. The initial deformations in test 6 were, however, somewhat greater than in the remaining tests possibly, because of the relatively low mortar strength. Test 10 gave a higher load capacity and smaller initial deformations, which may be due to the orientation of the overlap of the lower reinforcement ring. All cracks, and finally also the failure in test 10, appeared in that end of the specimen which consisted of one floor slab. The hair pin protruding from this floor slab had contact with the lower reinforcement ring at both crossings. It is therefore likely, that the U-shaped dowel in test 10 - accidentally - was in a more favorable position than the dowels that were bent in the other tests.

Tests 6-9 and 11-12 illustrate the sensitivity of the load capacity to changes, which could appear in practice as accidental variations. The strength of the joint mortar was varied, a horizontal construction joint was placed in the joint mortar, and the overlap of the rings with the protruding hair pins have varied. The effect of these changes on the load - deformation curves is limited, and apparently without any clear systematism, and on the present basis it may be included as scatter on the test results. Somewhat dependent upon which level of deformation has been applied, the corresponding measured load has been determined with a coefficient of variation of approx 10%.

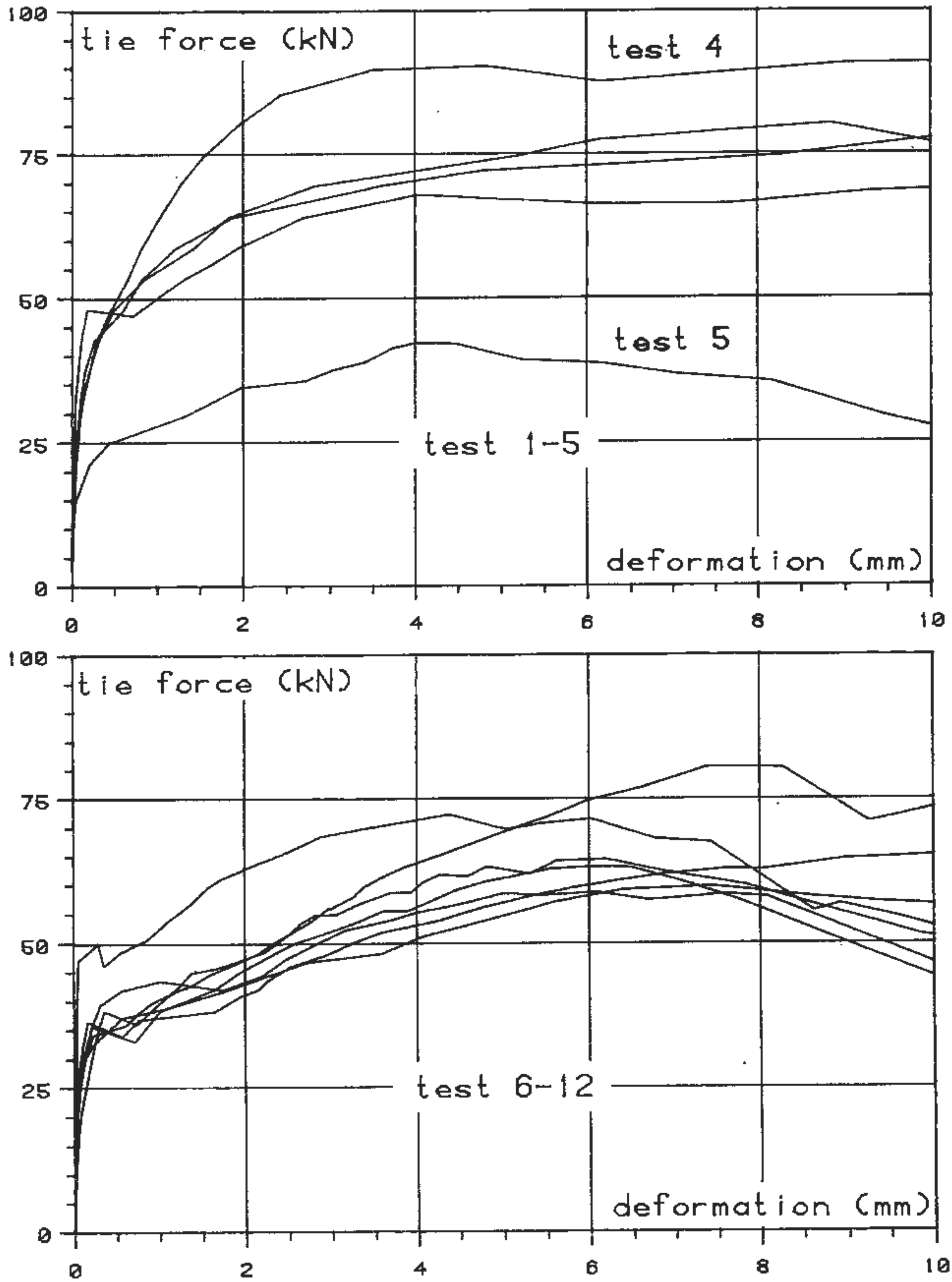

Figure 6: Load-deformation curves measured in tests on ties (Large deformations).

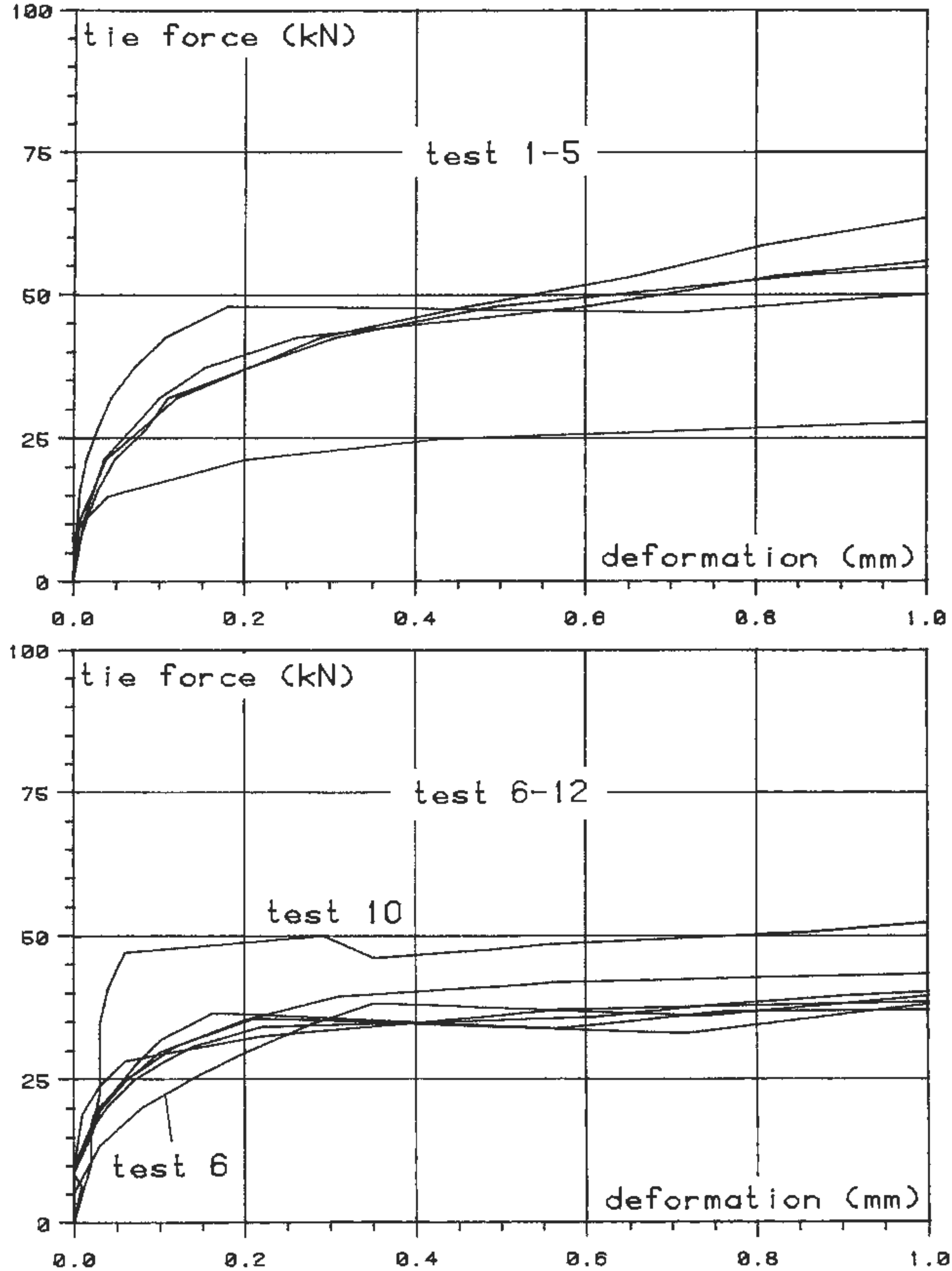

Figure 7: Load-deformation curves measured in tests on ties (small deformations).

TESTS ON THE SUPPORT ZONES OF SLABS

Program

The overall dimensions on columns and floor slabs in the competition project imply, that the floor slabs can only be provided with very small support lengths, and that normal tolerances are allowed in both production and erection. The structural system also implies, that differential settlements in the columns will lead to an uneven distribution of the load from a floor slab to its supports at the four corners. The reinforcement arrangement in the support zone of the slab should therefore prevent a sudden shear failure, and provide a ductility, which will allow a redistribution of forces corresponding to reasonably great differential settlements.

It is intended that the mortar in the column-slab joint in practice shall penetrate also a gap between the top of the column and the bottom of the slab, in order to distribute as well as possible the load to the four corners of the slab. To obtain the gap, the floor slab will be mounted on small "spacers".

Testing of the support zones was performed as shown in figure 8. The slab is supported at one corner in the same way as in practice, and on a simple support close to the opposite corner. A concentrated load is distributed to the top side of the slab through a stiff distributor beam placed perpendicular to the supported diagonal.

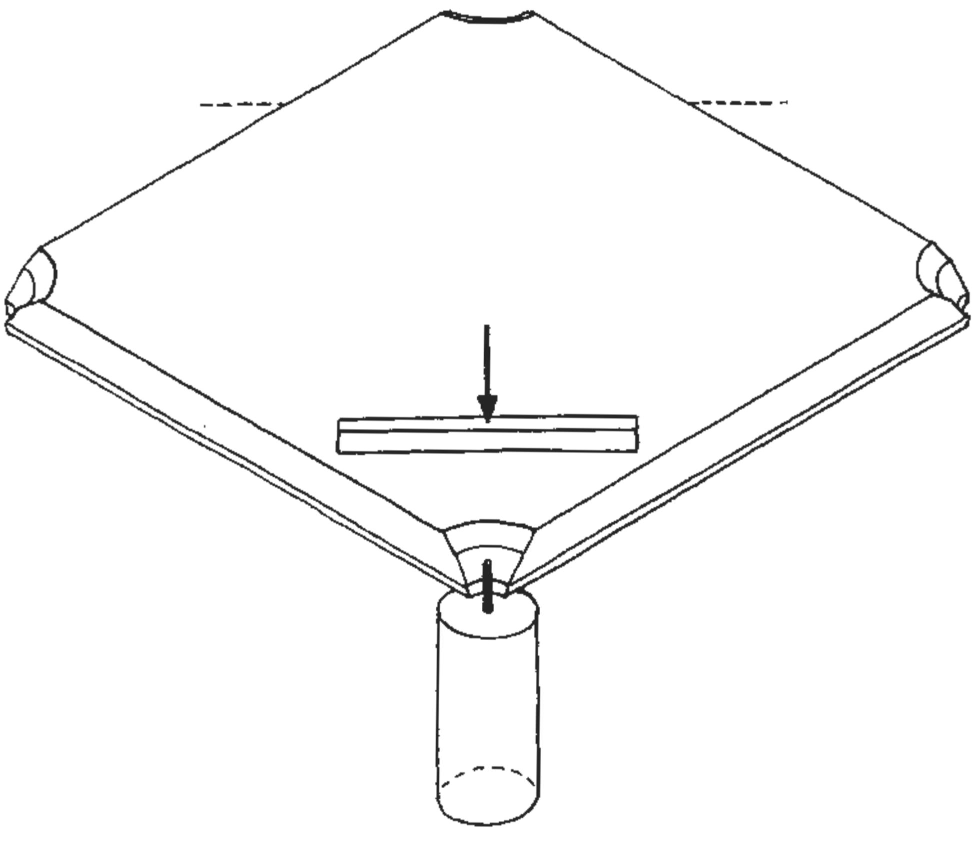

Figure 8: Arrangement for testing the shear strength of the support zone.

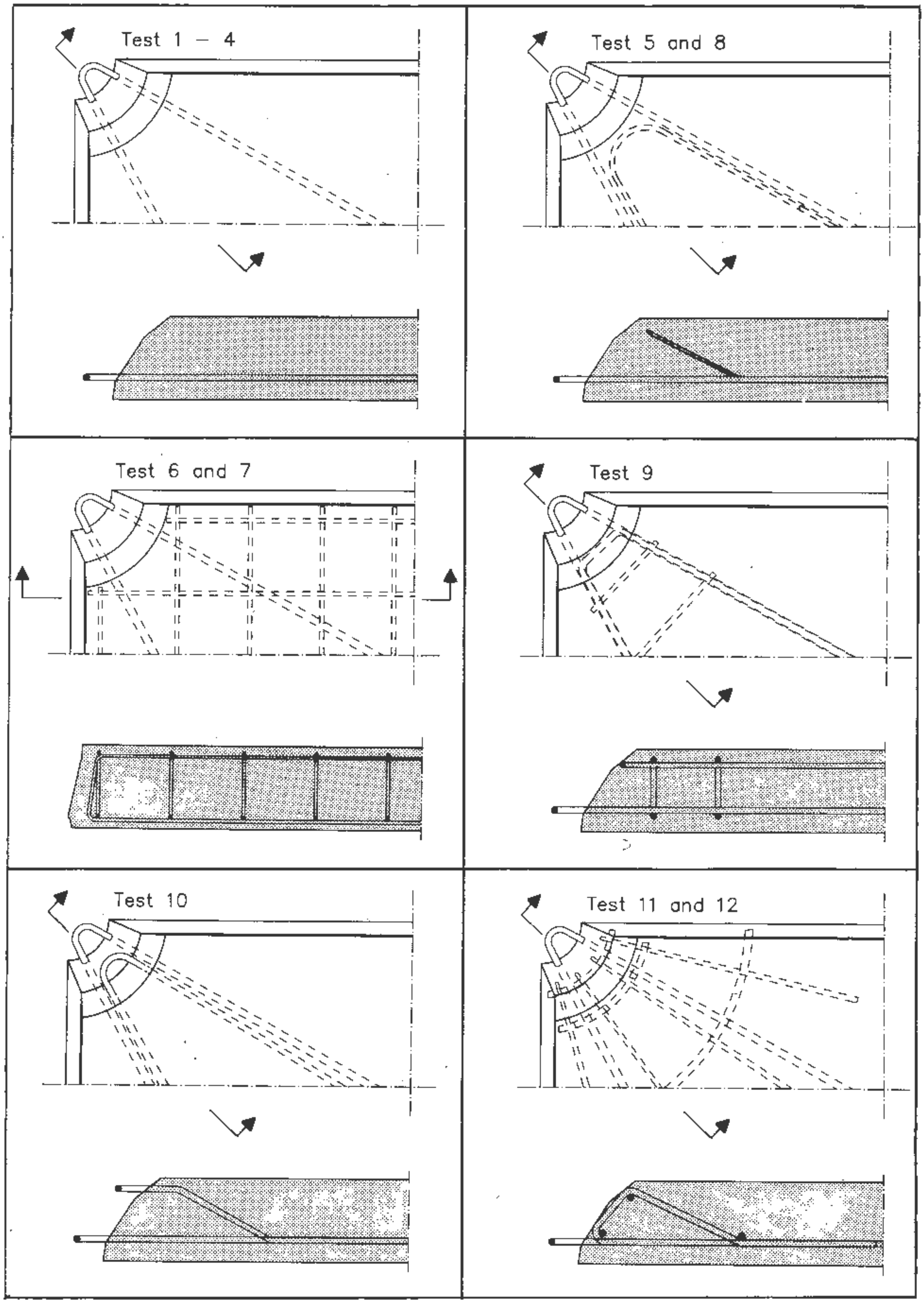

Figure 9: Reinforcement arrangements in the slab corners. The ordinary mesh reinforcement is not shown.

The test program includes (as of Sep 1, 1986) a total of 12 tests performed on four corners of three floor slabs. Six different reinforcement arangements in the corners of the floor slabs have been tested, and the support conditions as well as the concrete strength of the floor slabs have been varied.

The reinforcement arrangements are shown in figure 9. All the tests incorporated a protruding hair pin, placed horizontally on top of the reinforcement mesh in the floor slab. In tests 5-12 different additional reinforcements were used in order to add suitable ductility to the expected shear failure in the corner of the slab.

The specimen in test 1-8 did not contain the joint mortar, but a layer of (wet) plaster was used between the column and the slab. This plaster layer was interrupted in tests 6-8 by a polysterene spacer covering an area of 15x50 mm.

Test specimens 9 to 12 incorporated the casting of a joint mortar, with (roughly) the same geometrical shape as appears in practice in the column-slab joint, and spacers were used.

Besides the load, vertical displacements of the slab were measured in three points along the supported diagonal. The two points were placed as close as possible to the supports, whereas the third was placed close to the applied load. The differential deflection has been used as a relative measure of the ductility of the support zone, although the absolute value is hardly of any relevance.

Results

The test results have been summarized in figures 10-13. Figure 10 gives the main results and figures 11-13 show the load deflection curves for each of the three slabs.

Test no	Shear strength kN	Support length mm	Concrete strength MPa
1	59	60	28
2	58	45	28
3	56	45	28
4	64	60	28
5	58	45	23
6	42	40	23
7	50	40	23
8	46	40	23
9	68	40	30
10	76	40	30
11	96	40	30
12	81	40	30

Figure 10: Main results of shear tests on slabs.

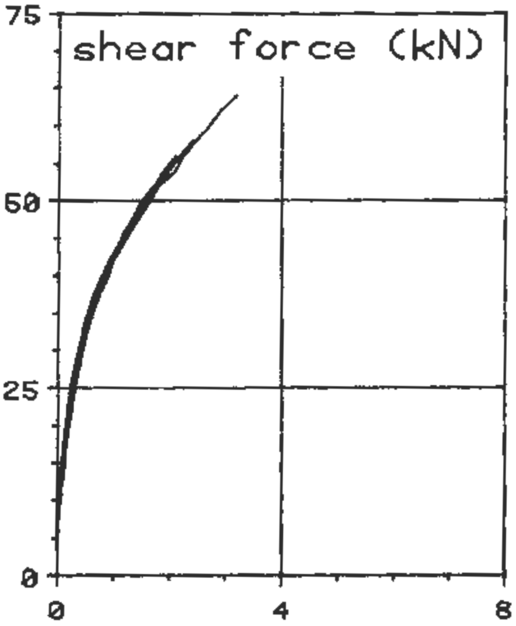

Figure 11: Load-deflection curves for shear tests 1-4.

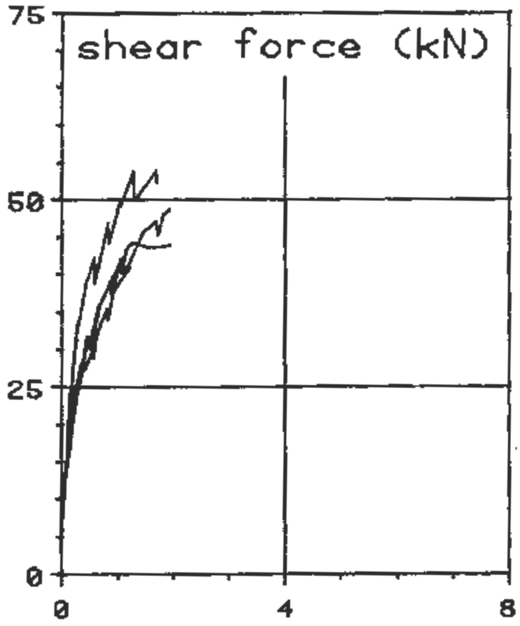

Figure 12: Load-deflection curves for shear tests 5-8.

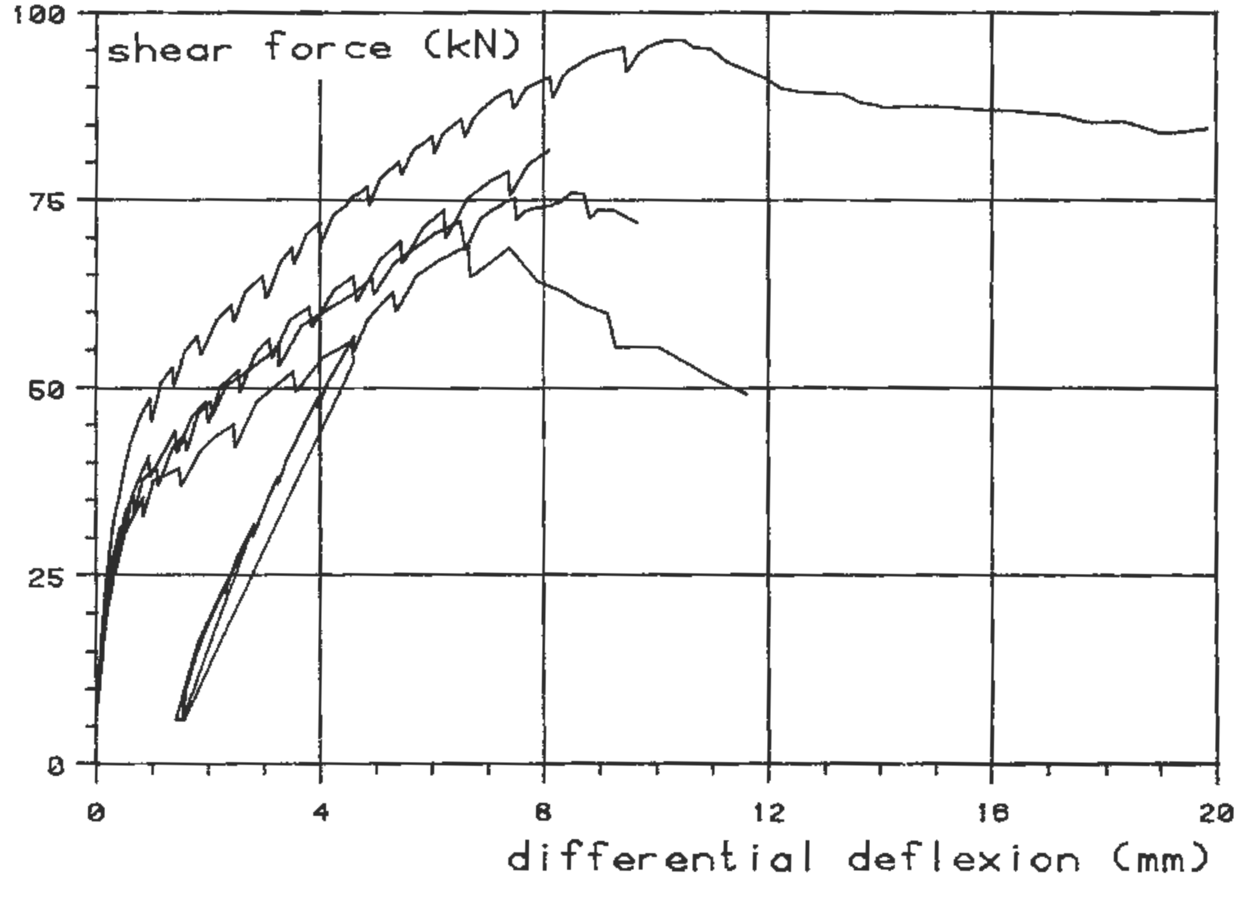

Figure 13: Load-deflection curves for shear tests 9-12

In the first four tests, a sudden shear failure occurred. After the formation of the shear crack, the load capacity had practically disappeared.

Tests 5-8 resulted largely in the same failure mode. The additional reinforcement, apparently, was not placed near enough to the edge of the slab and the shear failure could therefore still occur without being affected by the additional reinforcement to any appreciable extent.

The details in tests 9-12 had been changed, that the additional reinforcement had no cover, and in some cases extra protruding reinforcement was provided. In test 11, a regular bending failure occurred in the slab, following the formation of shear cracks as in the other tests. Test 12 was identical to test 11 except that joint mortar had not been cast. Also in this case a substantial improvement of both strength and ductility was obtained. The strength was reached when the column in principle sheared off the exterior corner of the slab, so that the slab could move freely along the column. By this time the usual shear cracks had been formed. A comparison between tests 11 and 12 indicates that the joint concrete around the protruding hair pin restrains the movements which would otherwise result from a shear crack at the support. This restraint may explain the improved results in both tests 9, 10 and 11. In this context it should be mentioned that the shear strength in tests 11 and 12 is comparable to the punching shear strength to be expected in a corresponding, solid concrete slab supported on a circular column. A further improvement of the strength is not likely to be obtained.

TESTS ON COLUMN-SLAB-COLUMN JOINT

Program

The transfer of vertical load through the slab-column joint was studied in the testing arrangement shown in figure 14. The specimen included two column pieces, and two pieces of a facade slab, and the same protruding hair pins and the same joint reinforcement was used as for instance in stringer test No 6 (figure 4 and 5). No further additional reinforcement was used.

As in the stringer tests, the slabs were supported directly on top of the lower column, without any intermediate distributing layer. The joint concrete was cast in two operations with a construction joint right above the top of the joint reinforcement. The upper column was mounted after the first operation and the second operation then filled the space between the bottom of this column and the top of the first casting.

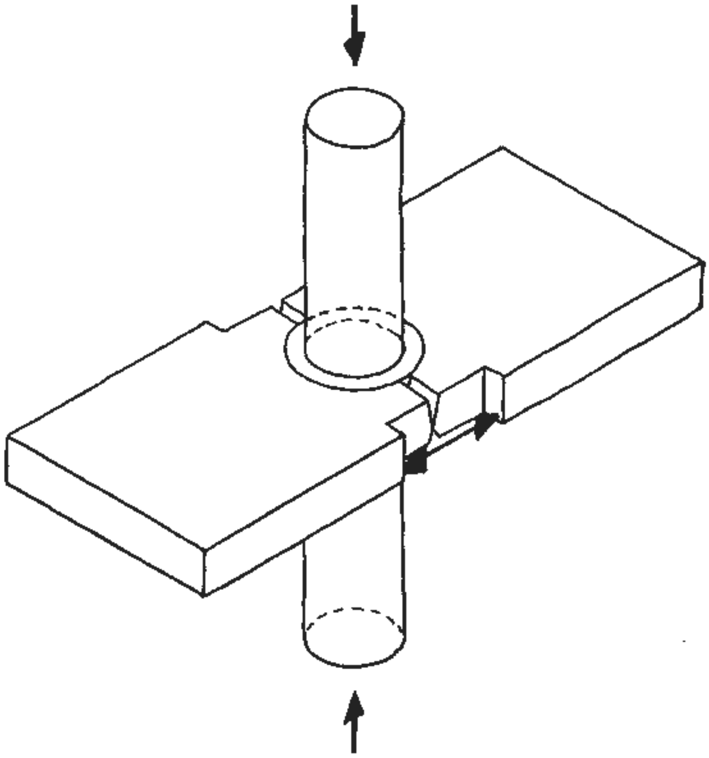

Figure 14: Arrangement for testing of column-slab-column joint.

Results

The floor slabs were first forced ab. 1 mm apart in the horizontal direction in order to simulate the effect of a stringer force in the slab. This imposed deformation means that transfer of vertical load from column to column through the ends of the floor slabs is very unlikely. Having forced the slabs apart, vertical load was applied to the column ends. The maximum capacity of the load equipment was 2 MN and at this load level, a number of vertical cracks had developed in the columns. There was no indication however af an imminent failure. Having unloaded the vertical load, the horizontal deformation was gradually increased to ab. 7.5 mm and the vertical load of 2 MN was again applied, but still no failure occurred.

The test specimens were demounted, and it could be established that the column concrete outside the ring reinforcement was spalling. Moreover, the joint mortar had been pushed 0,5 - 1 mm down into the top of the bottom column. Vertical cracks (probably due to the stringer load) were found in the mortar but outside the core formed by the joint reinforcement. Apart from that the mortar was intact.

The strength obtained in the two tests was well beyond the strength required by the proposed system, and therefore further testing was terminated.

CONCLUDING REMARKS

The test results are presently under evaluation and comparisons are made between the results obtained and the properties required in actual building schemes. The outcome of these comparisons will determine the need for further testing.