ANCHORAGE IN LONGITUDINAL JOINTS BETWEEN PRECAST HOLLOW CORE PANELS

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Today, precast hollow core floors are normally not provided with interacting structural toppings. In-plane forces have to be transferred between the floor panels by grouted and concreted joints and by tie bars which are indirectly anchored in joints or cores. There have been doubts whether or not the anchorage of tie bars in grouted joints still is effective when cracks appear along the joint interfaces.

A test series on hollow core floor connections was carried out with the aim to study the effect of initial joint cracks. The anchorage capacity appeared to be considerable in spite of the joint cracks. Based on the test results, recommendations are given for the detailing and the execution of tie bar anchorages in grouted joints.

Keywords: Precast floors, tie bars, grouted joints, joint cracks, diaphragm action.

1. INTRODUCTION

Precast hollow core floors are used extensively in many countries all over the world. The hollow core panels are established as attractive precast products and the interest in hollow core floor construction will probably remain and even increase in the future /1/.

In precast floors individual floor slab panels are mounted together and connected in order to form a completed floor slab. The precast floors are often used in the stabilizing systems as diaphragms for transfer and distribution of the lateral loads to shear walls, cores and other stabilizing structures. With regard to the diaphragm action but also for distribution of vertical concentrated loads and prevention of cracks and relative displacements there are needs for force transfer between the individual panels. Outermore, the need for structural integrity and prevention of progressive collapse will require a certain tensile force capacity within the precast floor.

Hollow core panels are produced by many various manufacturing processes. The long line methods which are commonly used today involve limitations as regards the arrangement of connection details at the joint faces. Therefore, standard solutions for connections in precast construction can not always be adopted for hollow core floors. For instance it is almost impossible to provide the joint faces with protruding tie bars or bends directly anchored into the hollow core panels. It is also difficult if not always impossible to anchor welding plates, threaded inserts etc at the joint faces.

In many applications it has been common practice to provide the hollow core floors with a structural concrete topping. The topping has been designed and reinforced in order to meet the needs for force transfer capacity between the panels. In aseismic design this has been the normal way to secure the in-plane behaviour and the integrity of the completed precast floor. However, in modern design structural topping is normally avoided because of cost and inconvenience at the site, but a thin non-structural topping is often added in order to level out and adjust the top surface of the floor. Even in earthquake-resistant design there is today a tendency to avoid structural toppings /2/, /3/.

Instead, the interaction between the floor panels is provided by making shear transfer possible in the joints and introducing tie bars which are indirectly anchored in joints and/or cores. In Fig. 1 two alternative solutions for the connection at an internal support are exemplified. The tie bars parallel with the span are anchored in concreted cores and in grouted joints respectively.

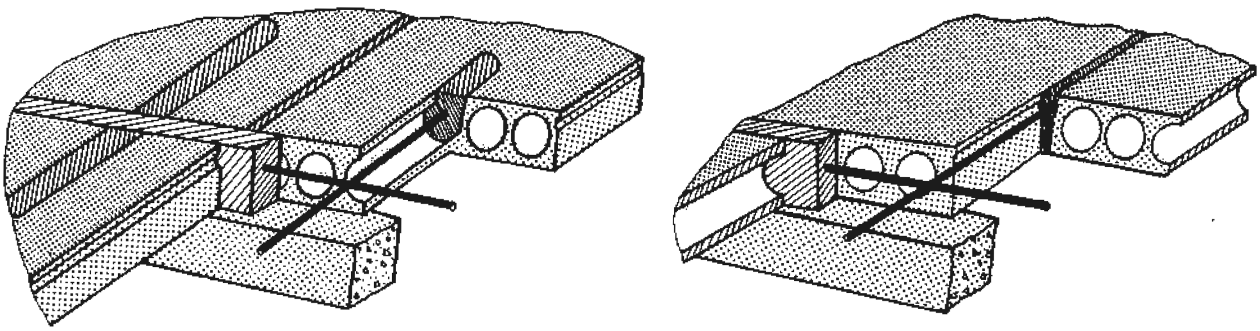

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Fig. 1 Typical solutions for the connection at an internal support
a) longitudinal tie bars anchored in concreted cores
b) longitudinal tie bars anchored in grouted joints

Normally, the anchorage capacity in concreted cores is sufficient for anchorage of an ordinary amount of tie bars. According to observations in tests /4/, the bond at the interface between the concrete filling and the hollow core panel inside the core is not critical for the anchorage. It is rather the splitting strength of the panel itself which may limit the anchorage capacity /4/. In this respect the tie bars can be regarded as if directly anchored into the panels. In practice, other criteria than anchorage will normally be decisive for the length of the concrete filling.

On the other hand, solutions with concreted cores are complex to detail and execute. Openings have to be taken in the top of the panels in order to form slots into which the tie bars can be placed. Furthermore, the stress distribution in the support zone of the panels may be negatively affected by the openings. The connection at the support will also be rather stiff resulting in restraining effects which must be considered in the detailing. For instance, in order to avoid a brittle shear failure in the zone of negative moment, the concrete filling must not be ended too near the support /5/. Because of the abrupt change of stiffness, the cross-section where the reinforced concrete filling is ended will be critical with regard to possible negative bending cracks, which in turn may initiate a shear failure.

It is much simplier and therefore more attractive to place the tie bars in the grouted joints. However, there are some uncertainties as regards the anchorage capacity. The longitudinal joints between hollow core panels are narrow which makes the grouting operation difficult. There is a risk of uncomplete filling of the joint and imperfect encasing of the tie bar. As a result the bond may be insufficient both along the tie bar and at the joint interfaces. Hence, the anchorage capacity will depend on the workability and compaction of the grout and will be sensitive for bad workmanship. In this respect, development of joint grouts and grouting methods is important.

Furthermore, it is known from practice that cracks may develop in the grouted joints because of shrinkage, temperature effects and other restrained movements. It is uncertain how possible restraint cracks will affect the anchorage capacity in the joints. Even if cracks do not occur for normal loading, severe cracking must be expected in case of accidental or seismic loading.

With the aim to study how joint cracks will affect the anchorage capacity in grouted joints a series of tests was carried out at Chalmers University of Technology in 1985/86. The observations and results from these tests will be presented in the following. The presentation will be completed with some essential results from former tests on anchorage capacity in narrow joints without initial cracks.

2. TENSILE TESTS ON HOLLOW CORE FLOOR CONNECTIONS

2.1 Description of the test procedure

In 1985/86 a series of 8 tests on hollow core floor connections was carried out at Chalmers University of Technology. The test specimens consisted of 4 hollow core panels of Spiroll type, which were mounted together in order to form a part of a floor slab around an internal transverse cast-in-situ tie beam, see Fig. 2. The height of the panels was 0.27 m and the width 1.2 m.

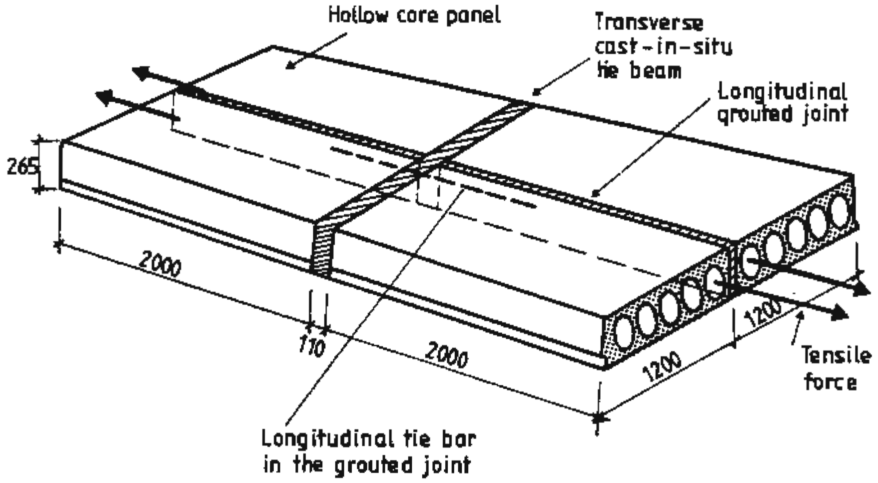

Fig. 2   Test specimen and load arrangement

Two panels were placed side by side with a longitudinal joint in between. Another similar pair of panels was connected at the short ends were the cast-in-situ tie beam was formed by concreting the transverse joint. The hollow core panels were connected indirectly by tie bars anchored in the longitudinal and transverse joints respectively.

The test specimen was exposed to a tensile force acting perpendicular to the transverse tie beam. The tensile force was applied via 2 tensile rods which at each end of the floor slab were anchored by concreting in the cores nearest to the longitudinal joint, see Fig. 3. The tensile force was achieved by a hydraulic jack acting at one end of the test specimen. This end will be called the active end. At the other, passive end the tensile rods were fixed via supports to the laboratory floor.

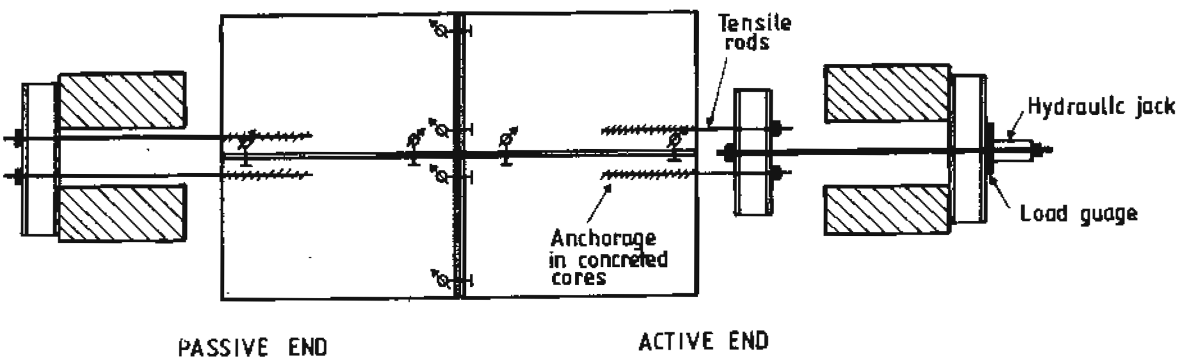

Fig. 3   Load arrangement and instrumentation

The aim of the investigation was mainly to study how the anchorage capacity in the grouted longitudinal joint was affected by initial joint cracks. Also some other parameters of interest for the anchorage capacity were varied in the test series according to Table 1.

Table 1 The test programme

Test no.	Longitudinal tie bar				Transverse tie bar			Transverse joint debonded	Longitudinal joint at active end		
	type	size [mm]	anch. length [mm]	depth [mm]	type	size [mm]	depth [mm]		joint faces a	joint faces b	precracked
1	Ks40	12	600	133	Ks40	12	145	No	III	III+	Yes**
2	"	16	800	"	"	16	149	Yes	III+	I	Yes
3	"	12	600	223	"	12	235	"	I	II+	"
4	"	"	"	133	"	8	143	"	II	I+	"
5	"	"	"	"	"	12	145	"	II+	III	"
6	"	"	"	"	"	"	"	"	I	I+	No
7	"	"	"	"	"	"	"	"	III	IV	"
8	"	16	800*	186	"	16	202	No	III+	IV+	Yes

* provided with end hooks

** the longitudinal joint at the passive end precracked as well, joint faces of category III

\+ joint face where the initial crack appeared

The tie bars used in the tests were always made of ordinary ribbed reinforcement bars, type Ks40 according to Swedish standards. This type stands for a high bond, hot rolled bar with a minimum yield stress of 390 MPa.

The transverse tie bar was always placed just above and tied to the longitudinal tie bar.

Normally the joint interface between the transverse joint and the passive end of the floor was provided with a plastic sheet in order to prevent any adhesive bond and form a crack inducement. There were holes in the sheet in front of the cores so that the joint concrete was allowed to flow into the cores up to a distance of 20 mm from the end face of the panel. Plastic plugs were used to close the cores at this distance, see Fig. 4a. By this arrangement, the transverse connecting ability from the cast-in-situ tie beam remained because of the shear-key effect from the concrete dowels in the cores. However, the joint interface got a considerable tensile force capacity in spite of the plastic sheet.

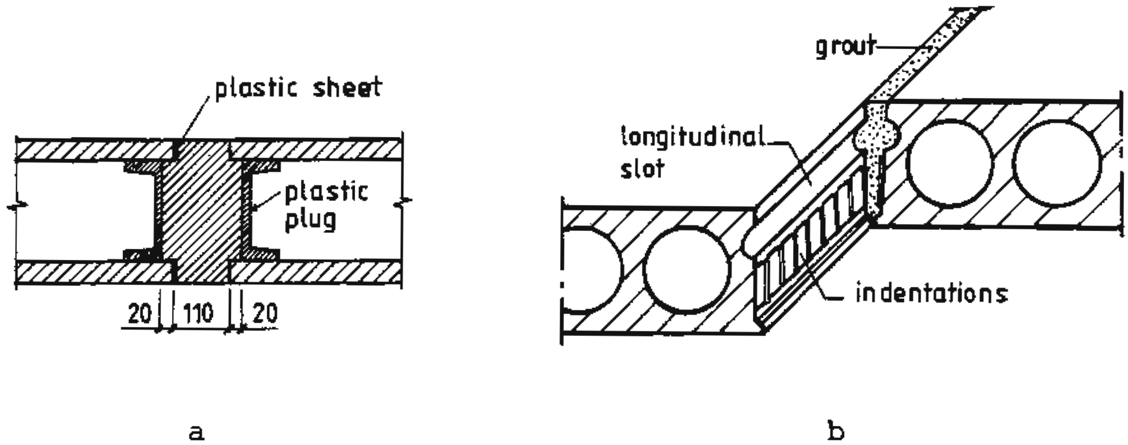

Fig. 4 Joint details
a) the concreted transverse joint forms a tie beam
b) the grouted longitudinal joint

The longitudinal joint faces of the hollow core panels were normally provided with vertical indentations according to Fig. 4b. To produce the indentations the extruding machine is provided with notched wheels, which make impressions in the fresh concrete. The depth of the indentations may vary depending on the adjustment of the notched wheels, the consistence and the composition of the concrete. Because of the variation of the indented joint faces, they were classified in four different groups according to Table 2. Each joint has two joint interfaces to the connected hollow core panels. The types of joint faces which were present at the longitudinal joint at the active end of the test specimens are presented in Table 1. Normally, this was the only joint which was exposed to initial precracking. However, in test no. 1 the longitudinal joint was precracked at both the active and the passive ends of the test specimen.

Table 2 Classification of longitudinal joint faces

Category	Description	Uneveness s	Indentations height [mm]	depth [mm]
I	No distinct indentations Uneven and wavy joint face	0.19-0.34		-
II	Visable indentations or Marked indentations but reduced in height or area	0.25-0.50		0-0.5
III	Marked indentations	-	≅70	0.5-3.5
IV	Marked indentations	-	≅120	0.5-3.5

The uneveness of the joint faces in category I and II was measured according to a Swedish standardized method, SIS 812004 /6/.

At mounting, the transverse and the longitudinal joints were filled in the same time and the tie bars were placed in the joints. The joint concrete in the transverse joint was vibrated while the grout in the longitudinal joint was puddled. The longitudinal tie bar was pressed down in the fresh grout to the right level in order to achieve proper encasing.

About 24 hours after the joint filling, the longitudinal joint at the active end of the specimen was forced to crack by a hand-operated screw. The screw was braced between steel brackets which were mounted in the cores on each side of the longitudinal joint. In order to prevent a sudden, uncontrolled wide crack from occurring, the two panels were compressed transversally by a screw clamp.

When the joint crack was achieved, the screw was turned further on until the width of the crack at the active outermost end of the slab was 1 mm. When the screw was unstressed and removed the crack width decreased due to the action of the transverse tie beam. The remaining crack width at the outermost end was about 0.3-0.4 mm. The crack width decreased along the joint and the crack was not visable near to the transverse tie beam.

The tests were executed 7 days after the joint filling. On the same time the strength of the joint concrete and joint grout was tested on 150 mm cubes. The quality of the joint concrete was K25 and the mean strength varied between 19.9-35.1 MPa with an average value of 25.7 MPa. For the joint grout a dry ready-mix was used. The mean strength varied between 28.6-33.9 MPa with an average value of 30.8 MPa.

The strength data for the tie bars is presented in Table 3.

Table 3 Strength data for the ribbed tie bars of type Ks40

Test no.	Size [mm]	f_{st} [MPa]	f_{stu} [MPa]	ε_{sy} $\cdot 10^{-3}$
4	8	401	654	1.90
1, 3-6	12	525	726	2.42
7	12	439	598	2.22
2, 8	16	416	610	2.08

f_{st} = yield stress

f_{stu} = ultimate tensile strength

ε_{sy} = yield strain

The applied tensile force was measured by a load gauge next to the hydraulic jack and was continuously recorded together with the relative horizontal displacement between the slabs on each side of the transverse tie beam. The relative displacement was measured by a deformation gauge which was mounted across the transverse tie beam, along the grouted joint.

The tensile force was applied in steps of about 5 kN until the yield load was reached. Further loading was controlled by the relative horizontal displacement which was increased in steps of 2-4 mm. At each step the steel strain was measured by strain gauges at several points along the longitudinal tie bar and on the transverse tie bar just where it crossed the longitudinal joint. Outermore, joint openings were measured by deformation gauges at several points along the joints. The placement of instruments appear from Fig. 3.

The tests were divided in two stages, before and after a rupture was achieved in the transverse joint. At this rupture, the transverse joint crack developed along the entire width of the test specimen, as in Fig. 5a, or normally only along one panel, see Fig. 5b.

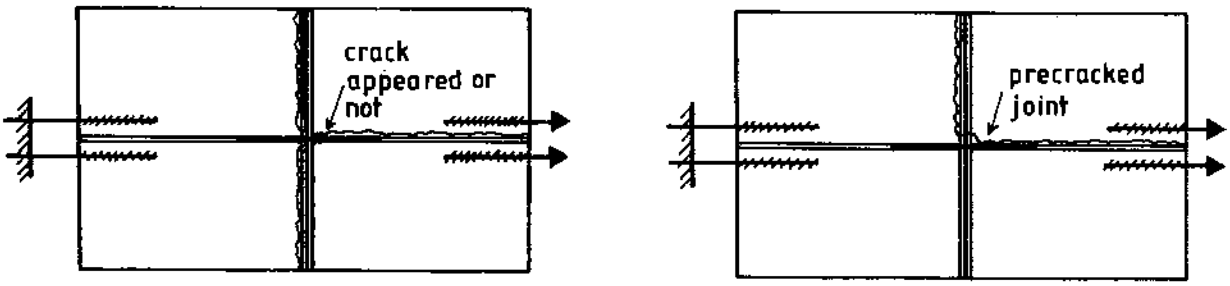

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Fig. 5 Alternative ruptures in the transverse joint
a) crack along the entire width of the specimen (tests no. 6, 7)
b) crack along one panel only meeting the initial crack in the longitudinal joint (tests no. 1-5, 8)

For the tests with a precracked longitudinal joint, the width of the initial crack increased successively for increased tensile loading. For the panel next to the precracked joint interface, small horizontal displacements were recorded across the transverse joint. However, no transverse crack was visable for the moment. This panel seemed to be slightly rotating under action of the tensile force according to Fig. 6a.

The longitudinal tie bar was more and more strained during this phase but the maximum strain recorded in the first stage was normally far below the yield stress, see the results in Table 4. The first stage ended when the transverse joint suddenly ruptured according to Fig. 5b. The transverse joint crack appeared along the interface of one panel only, crossed the cast-in-situ tie beam and met the initial crack in the longitudinal joint. At the rupture, the concrete dowels in the cores normally failed in tension just at the joint-interface in spite of the short length of 20 mm, see Fig. 4a. Only exceptionally, the concrete dowels were pulled out from the cores. The tensile force at rupture was considerable, especially having in mind the plastic sheet in the ruptured joint interface.

After the partly rupture, the panel next to the longitudinal joint crack could not be loaded in tension any more. However, the longitudinal joint grout filling remained intact and the tie bar was still connecting the two floor slabs on each side of the transverse joint. However, any tensile force in the tie bar could now be transferred at one joint interface only. For the second stage of the test, the load arrangement was modified according to Fig. 6b.

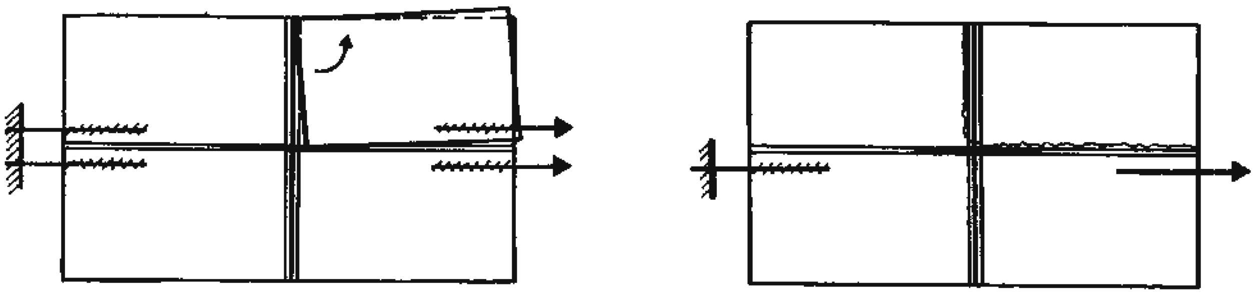

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Fig. 6 Test specimen with precracked longitudinal joint
a) In the first stage, the panel next to the initial crack tends to rotate and the width of the longitudinal crack increases successively
b) Modified load arrangement in the second stage after partly rupture in the transverse joint

For the two test specimens without initial crack in the longitudinal joint (tests no. 6 and 7) the transverse crack appeared along the entire length of the tie beam, according to Fig. 5a. For test no. 6, the longitudinal joint at the active end ruptured at the same time why the modified load arrangement was used for the second stage. Whereas in test no. 7, the longitudinal joint remained uncracked and the original load arrangement could be kept also in the second stage.

In the modified load arrangement normally used in the second stage, the test specimens were loaded slightly excentrically. The transverse joint had been partly ruptured in the first stage and now the remaining part of the joint was forced to crack. This was normally the start of the plastic phase. The crack then appeared also across the longitudinal tie bar. However, the tie bar was still able to connect the slabs on each side of the transverse joint opening.

Normally, the second stage ended either because of fracture or pull-out of the longitudinal tie bar. In none of the tests, anchorage failure occurred due to splitting of the joint grout. However, in test no. 8, the ultimate failure was determined by bond failure at a joint interface.

2.2 Observations and results

2.2.1 Main test results

The main results as regards crack loads, ultimate loads, failure modes and maximum steel strains are summarized in Table 4.

Table 4 Main test results

Test no.	First stage					Second stage			
	Crack load* [kN]	Remarks	$\varepsilon_{s\ell,max}$ $\cdot 10^{-3}$	$\varepsilon_{st,max}$ $\cdot 10^{-3}$	$\varepsilon_{st,r}$ $\cdot 10^{-3}$	Crack load** [kN]	Ult. load [kN]	Failure mode	$\Delta\varepsilon_{st}$ $\cdot 10^{-3}$
1	90	partly	3.19	0.63	-	80	53	pull-out	-
2	144	"	0.70	0.04	0.86	101	119	"	1.09
3	127	"	0.89	0.10	1.30	63	75	fracture	1.24
4	143	"	1.11	0.04	1.89	71	74	"	1.26
5	113	"	0.93	0.02	1.22	74	72	"	0.52
6	190	complete	0.58	-0.06	-0.02	-	71	"	0.23
7	200	"	0.13	-0.02	0.01	-	88	"	0.02
8	112	partly	0.31	-0.01	0.05	70	70	interface	0.02

* partly or complete rupture of the transverse joint

** rupture in the remaining part, if any, of the transverse joint

$\varepsilon_{s\ell,max}$ = maximum steel strain in the longitudinal tie bar measured at the load step just before rupture

$\varepsilon_{st,max}$ = maximum steel strain in the transverse tie bar measured at the load step just before rupture

$\varepsilon_{st,r}$ = steel strain in the transverse tie bar measured just after rupture in the transverse joint

$\Delta\varepsilon_{st}$ = increase of steel strain in the transverse tie bar during the second stage

2.2.2 The effect of initial cracks in the longitudinal joint

In spite of the initial crack in the longtitudinal joint and, outermore, the plastic sheet at one of the transverse joint interfaces, the floor connection could always sustain a tensile force which was higher than the ultimate capacity of the longitudinal tie bar at fracture, see the crack loads in Table 4. When a partly rupture occurred in the transverse joint, the anchorage of the longitudinal tie bar remained effective on both sides of the transverse joint.

In the second state, when the transverse joint had been forced to crack completely, it was possible in five tests out of eight to achieve fracture of the longitudinal tie bar without any damage of the joint grout. The fractural load was about 70 kN in the actual tests (ϕ12 Ks40).

Due to the plastic sheet in all of the actual tests, the transverse joint ruptured in the joint interface next to the passive end of the specimen. The longitudinal joint was always precracked in the active end, if precracked. Accordingly, it was concluded that the anchorage capacity of the longitudinal tie bar was not limited due to the crack in the longitudinal joint if the adjacent panels still were bonded to the tie beam, see Fig. 7a. It was even possible to transfer the fractural load of the tie along one joint interface only. Furthermore, the anchorage capacity of the tie bar in an uncracked longitudinal joint was not limited due to the rupture along the adjacent interface to the transverse tie beam, see Fig. 7b.

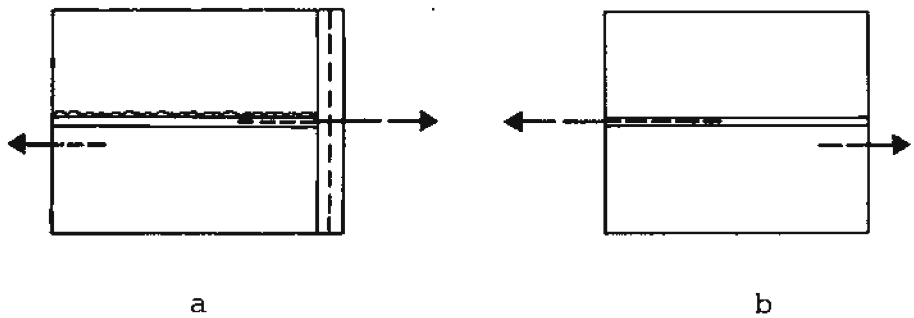


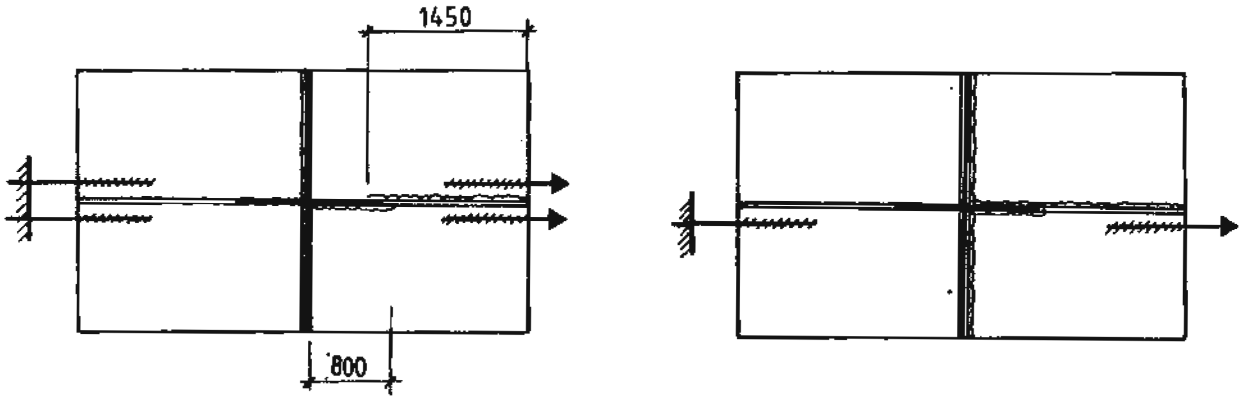
Fig. 7 Severe conditions where the anchorage capacity in the longitudinal joint still remained
a) crack in the longitudinal joint, but the transverse tie beam still in action
b) uncracked longitudinal joint but no effect of the transverse tie beam

However, in three of the tests anchorage failures were achieved. In test no. 2, there was a pure pull-out failure which was not affected by the initial crack. With an increased anchorage length it would probably have been possible to anchor the tie bar until fracture, see 2.2.5. However, the longitudinal joint, with crack formations according to Fig. 7, resisted a tensile force of 119 kN before the pull-out failure.

The specimen in test no. 1 was exposed to more severe initial cracking than the other tests. At the active end, the longitudinal joint had an initial crack width of about 3,0 mm, at the passive end it was about 0,1 mm. Furthermore, small initial cracks were present also in the transverse joint. In the second stage when the transverse joint was completely ruptured, the joint grout in the longitudinal joint was damaged because of a sudden in-plane deflection of the specimen. The tie bar lost the grout cover within a lenth of approximately 0.35 m at the passive end. The remaining anchorage length of approximately 0.25 m was unsufficient for anchorage of the fractural load, see Table 4. In an ordinary precast floor, a similar in-plane deflection would be impossible because of the restraint from adjacent floor panels.

Only in one of the tests, no. 8, the initial cracks had a direct negative effect on the tie bar anchorage. At the initial cracking, the longitudinal crack did not follow one joint interface only, but changed from one side to the other according to Fig. 8a. The floor behaved satisfactory in the first stage. In this test, the transverse joint was not provided with a plastic sheet. At the end of the first stage, a partly rupture was achieved at the joint interface between the tie beam and a panel at the active end. At the same time the initial longitudinal crack at the outermost end of the floor slab propagated to the transverse joint and met the crack there. As a result the longitudinal joint had cracks along both the interfaces within the anchorage length of the tie bar. Due to the partly rupture in the transverse joint there was now little transverse restraint from the tie beam.

In the second stage it was still possible to transfer a considerable tensile force across the transverse joint. The strain in the longitudinal tie bar almost reached the yield strain. However, when the remaining part of the transverse joint ruptured for a load of 70 kN also the longitudinal joint ruptured according to Fig. 8b. The joint grout around the tie bar was still undamaged but the bond at the joint interfaces was totally broken why the floor slab fell apart. It is difficult to say whether or not a transverse restraint from adjacent floor panels could prevent this type of failure.



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b

Fig. 8 Crack formation in test no. 8
a. Result of initial cracking in the longitudinal joint
b. Crack pattern at ultimate failure. The tie bar had no connection any more to the panels at the active end of the specimen

The ruptures which occurred in the transverse joint at the end of the first stage were also affected by the initial cracking in the longitudinal joint. When the longitudinal joint was uncracked, tests no. 6-7, the transverse joint ruptured completely for a tensile force of 190 kN and 200 kN respectively, see Fig. 5a. In test no. 7, the longitudinal joint remained uncracked until the tie bar fractured, but in test no. 6, the longitudinal joint at the active end ruptured at the same time as the transverse joint.

In the tests where the longitudinal joint at the active end was precracked, the transverse joint was always partly ruptured at the end of the first stage. The crack load varied between

112-144 kN. In test no. 1, the crack load was lower, 90 kN, probably due to the more severe precracking.

It is noticeable that the plastic sheet in the transverse joint did not seem to reduce the crack load. In tests no. 1 and 8, the transverse joint was not provided with a plastic sheet, but still the crack load was not higher than normal. Probably the crack load was determined by the tensile capacity of the concrete dowels in the cores.

2.2.3 The effect of joint faces

In the various tests, the longitudinal joint faces were more or less indented. With exception from tests no. 1 and 6, the two joint faces along the longitudinal joint at the active end were of different types in one and the same test specimen. The various types of joint faces are defined in Table 2. It is easy to believe that the initial crack which was forced on the joint should appear at the joint interface with the smallest indentations. However, this occurred only in two of five tests.

In tests no. 6 and 7 the longitudinal joint was initially uncracked. However, in test no. 6 with rather even joint faces a crack appeared when the transverse joint ruptured. Whereas in test no. 7, with marked indentations, the longitudinal joint remained uncracked until the end of the test.

In one of the tests with considerably marked indentations, test no. 8, the initial crack changed position from one interface to the other along the joint, see Fig. 8a. This may be an effect of a high bond capacity at the joint interfaces. The same phenomenon was also observed in another investigation /7/.

In the test specimens with least pronounced indentations, tests no. 3, 4 and 6, the general behaviour was not different from that in the other tests. The bond at the joint interfaces was never critical and it was possible to anchor the fractural load of the tie bar, about 70 kN.

As a result from the extrusion process, the hollow core panels are normally naturally rough at the joint faces. This natural roughness seemed to be sufficient with regard to the bond capacity at the joint interfaces. The uneveness of the joint faces without marked indentations were measured and is given in Table 2.

2.2.4 The effect of the transverse tie bar

The essential results from the strain measurements on the transverse tie bar are presented in Table 4. Except from test no. 4, the transverse tie bar was always of the same type and size as the longitudinal tie bar.

From Table 4 it appears that the transverse tie bar was normally only slightly affected during the first stage of the test.

Except from test no. 1 where the specimen was exposed to severe precracking, the maximum steel strain recorded just before rupture in the transverse joint, never exceeded $0.10 \cdot 10^{-3}$. The strain was measured where the tie bar crossed the longitudinal joint section. During the first stage, a considerable transverse connecting ability was provided by the uncracked reinforced tie beam which was connected to the panels by stiff concrete dowels in the cores, see Fig. 4a.

For tests no. 1-5, the first stage was ended by a partial rupture at the transverse joint interface next to the passive end of the specimen. As a result, a crack was formed across the tie beam so that the transverse crack met the initial crack in the longitudinal joint, see Fig. 5b. The transverse connecting ability was now provided by the transverse tie bar only. Accordingly, the steel strain in the transverse tie bar increased at this first joint rupture. The maximum strain recorded in tests no. 2, 3 and 5, just after the rupture varied between $0.86 \cdot 10^{-3}$-$1.30 \cdot 10^{-3}$.

In test no. 4, the ratio between the yield capacities of the transverse and the longitudinal tie bar respectively was only 0.34. The maximum steel strain in the transverse bar just after the rupture reached $1.89 \cdot 10^{-3}$, but was still below the yield strain.

In tests no. 1-5, the steel strain in the transverse tie bar increased during the second stage of loading. However, in none of those tests, the yield strength was reached. In tests no. 7-8 where the tie beam remained intact also in the second stage, the transverse tie bar was almost not affected at all.

It can be concluded from the observations that the transverse separating effect from the longitudinal tie bar loaded in tension is rather small. When the longitudinal tie bar was anchored in an uncracked joint, the anchorage behaved satisfactory until fracture of the tie bar without any transverse restraint at all, see Fig. 7h. On the other hand, if the longitudinal joint was precracked and, furthermore, a crack was formed across the adjacent tie beam, the transverse tie bar was strained due to the separating effect of the longitudinal tie bar. However, the capacity of the transverse tie bars was always sufficient and no anchorage failure occurred because of lacking transverse connecting ability of the tie beam.

2.2.5 The effect of anchorage length and depth of the longitudinal tie bar

In all the tests the anchorage length of the longitudinal tie bar was 50ϕ, where ϕ = the diameter of the bar. The anchorage length was measured behind the transverse joint interface.

In none of the test specimens with a 16 mm tie bar, it was possible to achieve a fracture of the tie bar. In test no. 8, the anchorage failed because of bond failure at the longitudinal joint faces within the anchorage length. The failure was initiated by the specific crack pattern which occurred in this test, see Fig. 8a-b. Probably, the actual type of failure would not have been possible to prevent by an increased anchorage length.

However, in test no. 2 a pull-out failure occurred at the active end where, because of the precracking, only one joint interface was active for transfer of the tensile force. The pull-out failure occurred in a late plastic state for a tensile force of 119 kN which was about the ultimate capacity of the bar. With a somewhat increased anchorage length it would probably have been possible to anchor the tie bar until fracture. From the strain measurements it was estimated that about half the anchorage length was affected by yielding when the pull-out failure occurred. The elongation of the tie bar was about 41 mm for the moment.

Except from test no. 1, the anchorage length 50ϕ was sufficient for all the tests with a 12 mm tie bar.

In test no. 1 a pull-out failure occurred when the yield load was reached. However, the pull-out failure was initiated by damage of the grouted joint, see 2.2.2. Due to the damage, which followed the rupture in the transverse joint, the tie bar became uncovered within a length of 0.35 m nearest to the transverse joint crack. The remaining anchorage length of about 20ϕ was unsufficient for anchorage of the fractural load, about 70 kN.

Normally, the longitudinal tie bar was placed mid depth, which always was satisfactory with regard to the anchorage capacity.

In one of the tests, no. 3, the tie bar was placed with the centre 223 mm above the bottom level. The top cover was then 36 mm, or 3ϕ, where ϕ = the diameter of the longitudinal tie bar. In spite of the relatively small top cover and, in the same test, an initial crack in the longitudinal joint, the tie bar fractured without any damage of the joint grout.

3. BENDING TESTS ON HOLLOW CORE FLOOR CONNECTIONS

3.1 Description of the test procedure

In a previous investigation at Chalmers University of Technology a series of bending tests was executed on tie connections between hollow core panels. The aim was to study the behaviour of floor connections when exposed to large angular deformation. The anchorage capacity in grouted joints was of special interest to investigate. In /8/ the tests are reported in detail and results and conclusions are also presented in /9/.

Hollow core slabs of Spiroll type were used for the test specimen. A floor slab strip, width 1.2 m, was placed horizontally on three supports with a transverse joint over the middle one. The connection was loaded in bending by lifting the middle support beam upwards according to Fig. 9a. The two outermost supports were restricted from lifting, but horizontally movable.

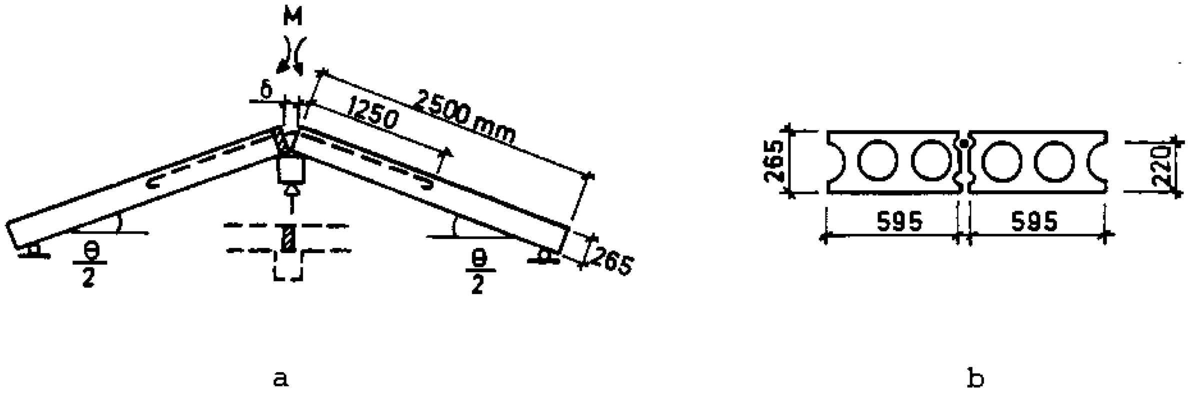


Fig. 9 The test specimen and load arrangement
a. longitudinal section
b. cross-section with a tie bar in the longitudinal grouted joint

The longitudinal tie bar was placed in the grouted joint across the transverse tie beam which was cast in-situ over the middle support. Plastic plugs prevented the joint concrete from flowing into the cores and thus affecting the bending stiffness.

During the test procedure the angular deformations were always concentrated to the transverse joint. Bending cracks occurred only in the interfaces between the joint concrete and the end faces of the slab panels. The elongation of the tie bar could easily be estimated from crack width observations.

3.2 Observations

In the first series of bending tests, the longitudinal tie bars were going straight through the test specimen from one end to the other. Thus, the anchorage length was 2.5 m. The tie bars were normally placed with the depth d = 0.22 m. The upper grout cover varied between 47-51 mm depending on the size of the tie bar.

In spite of the considerable anchorage length, pull out failures were received in four tests out of five. Only in one test it was possible to anchor the tie bar, $\phi 8$ Ks40, until fracture. When 12-16 mm bars of the same type were used, it was possible to anchor the yield load. However, in the plastic state, pull-out failures occurred. A similar behaviour was received for a 16 mm smooth bar of mild steel (Ss26) without end-anchors. The type Ss26 according to Swedish standards refers to a hot rolled, plain, smooth reinforcement bar with a minimum yield stress of 260 MPa. From the strain measurements on the actual tie bars, it was obvious that the bond capacity was extremely low even before the yield stress was reached. When the joints were examined afterwards the grout filling and the encasing of the tie bar were found to be imperfect.

In one of the tests the tie bar was placed in the bottom of the joint, d = 0.05 m. Anchorage failure occurred as the bar was tearing out through the bottom cover. The mode of action is illustrated in Fig. 10a. Thus, if the tie connection will be loaded in negative bending, the bottom concrete cover must not be too small.

In the tests which followed, several measures were taken in order to prevent anchorage failures. The most important step was to use another composition for the joint grout in order to get slower hardening and better workability. Outermore, the tie bars, even ribbed bars, were provided with end-hooks. However, the anchorage length was now reduced to 1.25 m on each side of the transverse joint.

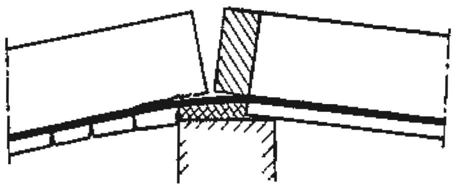
In most of the tests it was now possible to anchor the ties until fracture and the plastic deformations were large, especially for the smooth bars of mild steel. The anchorage capacity in the uncracked longitudinal joint was estimated to be at least 130 kN, see /8/, /9/.

Because of the transverse bending crack at one or both the interfaces at the cast-in-situ tie beam, at least one side of the floor slab lost the transverse restraint from the tie beam, see Fig. 9a. Still, anchorage failure never occurred due to transverse separation of the two adjacent panels.

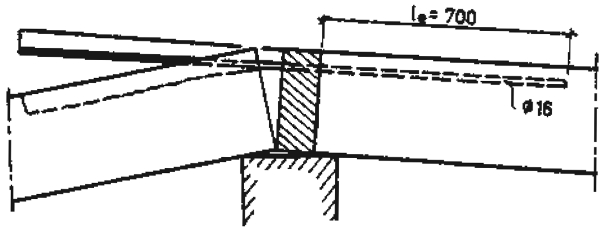
However, in some few tests anchorage failures occurred in spite of end-hooks on the ties. In one of the tests the end-hooks were bent upwards in a sharp bend and the tie bar was placed near the top of the joint. Thus, the anchorage force was concentrated to the upper part of the longitudinal joint, initiating longitudinal splitting cracks. However, when anchorage failure occurred, fracture was almost reached in the tie.

In another test two ties $\phi 16$ Ss26 were placed together in the joint just above each other. Both the ties were provided with end-hooks and the hooks of the lower tie were situated near the bottom of the joint. When the yield stress was reached the end-hooks were straightened and this movement of the end leg caused splitting cracks and separation in the bottom part of the joint.

In a similar investigation /10/ another type of anchorage failure was observed. The anchorage length, 44ϕ, of a straight tie bar was unsufficient to prevent the joint grout from splitting when the floor connection was loaded in bending, see Fig. 10b.



a



b

Fig. 10 Anchorage failures of the longitudinal tie bar
a. The tie bar tears out through the bottom cover
b. Splitting of the joint grout because of a too short anchorage length /10/

4. CONCLUDING REMARKS AND RECOMMENDATIONS

o The anchorage in grouted joints is sensitive for a poor grout operation. Therefore, in order to enable complete filling of the joint and a proper encasing of the tie bar,the consistence of the fresh grout should not be too harsh. However, in order to prevent leakage at the bottom of the joint, the consistence should not be too fluid .

The joint shculd preferably be filled to the full height in one operation.

In order to achieve a good encasing of the tie bar, it should preferably be pushed down in the fresh grout to the final level, instead of being placed before grouting.

o In the tests reported above, the average width of the joint was 25-30 mm. In order to facilitate the grouting operation it is important, that the joint width is not too narrow. It is recommended that the joint width should be at least 2ϕ, not less than 20 mm at the level of the tie bar (ϕ = the diameter of the anchored tie bar).

o The longitudinal joint faces of the hollow core panels should be sufficiently rough in order to provide for bond capacity at the joint interfaces. From the tests it seemed to be satisfactory with the ordinary roughness, which is a natural result of the extrusion process. There were no indications implying that indented joint faces are necessary with regard to the anchorage of longitudinal tie bars. However, of other reasons, indentations may be justified /5/.

o When the joints were properly grouted it was possible to achieve a considerable anchorage capacity in the grouted joints. It was possible to anchor 16 mm ribbed bars (Ks 40) to fracture without any cracks or other damage in the joints, if initially uncracked.

- A crack in a longitudinal joint face did not seem to affect the anchorage capacity negatively if separation of the adjacent panels was prevented by transverse restraint.

- Transverse restraint can be provided by the transverse cast-in-situ tie beam. With regard to the anchorage capacity it is recommended that the yield capacity of the transverse tie bar should be the same as for the longitudinal tie bar.

- Because of the risk of poor quality of the grouting operation, which may result in poor bond capacity along the tie bar, it is justified with certain limitations as regards the anchorage of longitudinal tie bars. Pull-out failures were observed in tests also when the anchorage length was considerable. However, in all the tests the yield strength was resisted and any pull-out failures occurred in the plastic state.

  For tie bars which are designed for ordinary loads, straight ends can be accepted. However, an anchorage length of 100ϕ is recommended, where ϕ = the diameter of the tie bar.

  For tie bars which are mounted for prevention of progressive collapse a ductile behaviour is demanded. It is then essential that the fractural load can be anchored, in order to take advantage of the entire plastic state. Therefore, it is recommended that even ribbed bars should be provided with end anchors. The anchorage length should be at least 75ϕ.

  It is also recommended that the yield load of the tie bars with straight ends should be limited to 30 kN. For tie bars with end anchors, a yield load of 80 kN can be accepted.

- For tie bars anchored by bond only, the tie bar should preferably be placed within or below the longitudinal slot of the joint faces, see Fig. 4b. A placing mid-depth should be aimed at.

- Any end-anchors should be placed mid-depth in order not to initiate splitting of the grouted joint.

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Several of the tensile tests were executed by the student Saeed Sadr as part of his diploma work /11/ with the author as guide.

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