Zeitschrift: Schweizerische Bauzeitung

Herausgeber: Verlags-AG der akademischen technischen Vereine

Band: 94 (1976)

Heft: 24

Artikel: Structural connections of prefabricated concrete units - some matters of current concern

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DOI: https://doi.org/10.5169/seals-73114

Spritzbeton in den meisten Fällen verantwortbar, während man sich im weichen Felsgestein auf Haftanker, Armierungsnetze und Spritzbeton beschränken könnte.

Falls hinsichtlich der Abstützung des Traggewölbes gewisse Bedenken bestehen, könnten im Kämpferbereich zusätzlich eine Reihe Alluvialanker angebracht werden.

Zusammenfassung

Die deutsche Tunnelbauweise ist eine sehr lohnintensive Baumethode. Hohe Kosten und bescheidene Ausbauleistungen sind kennzeichnend dafür. Am Beispiel eines Strassentunnels wurden die Baukosten und Vortriebsleistungen aufgezeigt. Eine Rationalisierung durch vermehrten Geräteeinsatz wird in den wenigsten Fällen möglich sein. Wo vor wenigen Jahren die deutsche Bauweise noch wirtschaftlich vertretbar war, sieht sich der Bauherr gezwungen, nach preisgünstigeren Alternativen zu suchen. Als solche wird die neue österreichische Bauweise nicht nur für Felsgestein, sondern auch im Lockergestein zur Prüfung empfohlen.

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Structural connections of prefabricated concrete units – some matters of current concern

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DK 69.067

It is appropriate to consider a structural joint between precast units as composed from a set of characteristic parts.

Joint zones of components and *interface surfaces* exist in all joints. Dependent on the type of joint there are one or more further parts (fig. 1):

- a *joint space*
- a *joint fill* of concrete, grout, pads, plates or sealings
- a *joint crack* and *ties* (or dowels). The ties may be direct, semidirect or indirect dependent on the place where they are anchored; direct when in the component, indirect when in the joint fill.

As all the above parts of the joint can be varied in geometry and mechanical qualities, the design of joints leaves inumerable possibilities. The load-transfer function is one and perhaps the best to classify structural joints into different groups, although the different groups may have common characteristics. Thus there are *simple interfaces* between two concretes, *shear joints, moment stiff joints, compression joints, simple supports* and various combinations of these fundamental groups.

The design of the structural connections decides to a great extent the technology and the economy of precast concrete prefabrication. It is thus of great importance that the solutions chosen not only satisfy the requirements with regard to the transfer of forces, ductility and service function but also are simple, safe and economical during the different phases of the manufacturing and construction procedures. Leaving the design of joints free to economical competition (as has occurred during the early development period of prefabrication) the risk of inadequate solutions has been appearant. On the other hand it is important that

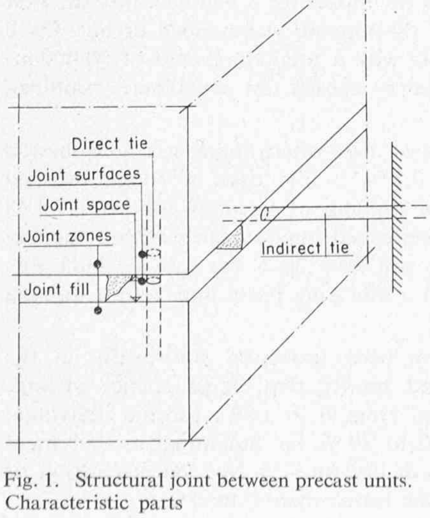


Fig. 1. Structural joint between precast units. Characteristic parts

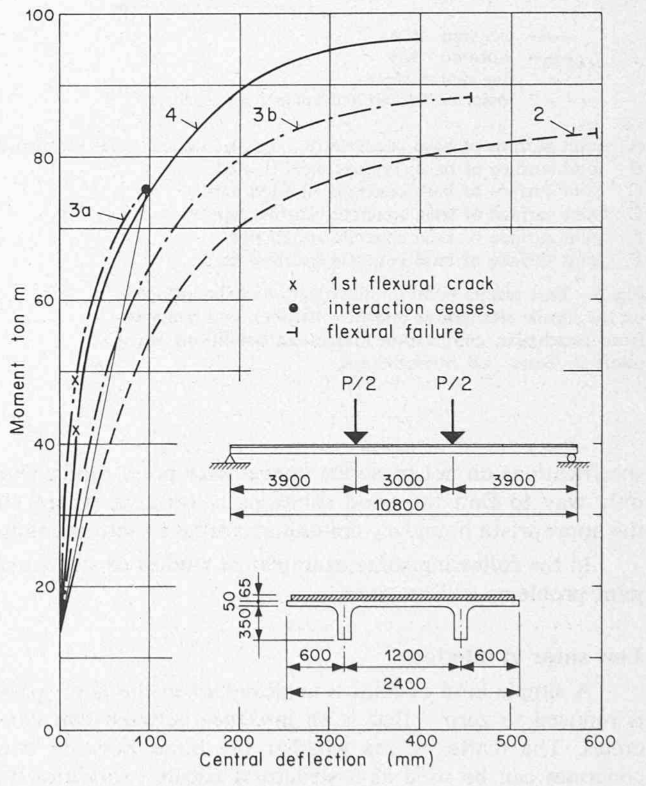
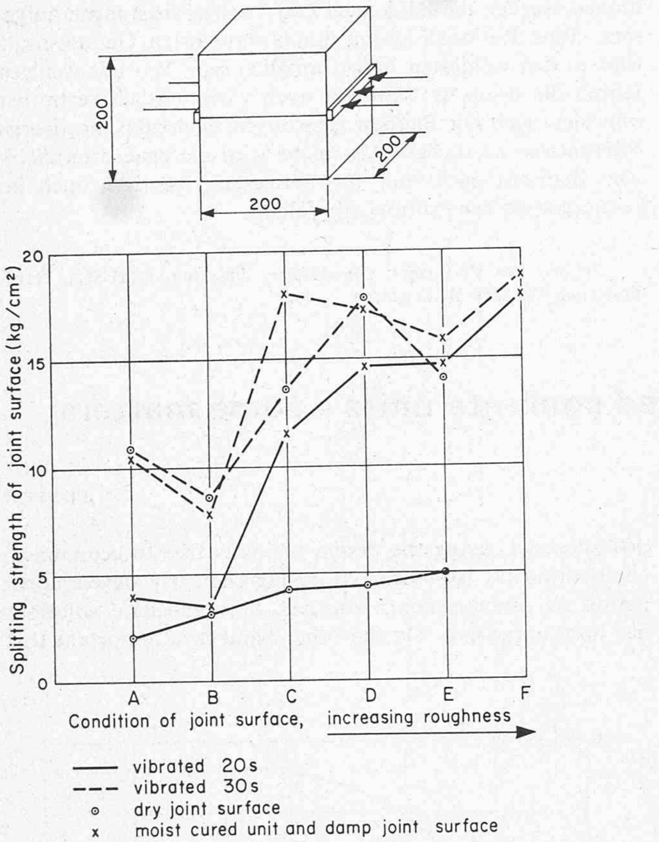


Fig. 2. Full scale tests have been performed on TT-slabs of 10.8 m span with and without interacting concrete topping. Curve 2 in the Figure shows the central deflection of a unit without concrete topping and Curve 4 that of a unit with interacting concrete topping. In the latter case, interaction continued right up to the ultimate stage. Curve 3a shows the central deflection of a similar TT-slab in which adhesion at the joint surface was prevented by oiling it prior to casting of the topping. Interaction persisted for some time above the flexural cracking load, after which complete separation of the topping occurred. After the topping concrete had been anchored at its ends by existing lifting stirrups which had not been surrounded with concrete on first loading (2 No 1/2" prestressing strands at each end), loading was repeated and central deflection took place as shown in Curve 3b. The tests were made at AB Strängbetong and sponsored by the Association of the Swedish Prefabricated Concrete Industries and partially financed by the Swedish Counsil for Building Research



A joint surface of base concrete cast against smooth steel formwork
B joint surface of base concrete steel floated
C joint surface of base concrete vibrated hard
D joint surface of base concrete brushed lightly
E joint surface of base concrete roughened
F joint surface of base concrete brushed hard

Fig. 3. Part results from an investigation of the influence on the tensile strength at interface between two concretes from roughness, compaction and moist conditions when pouring. Tests: AB Strängbetong

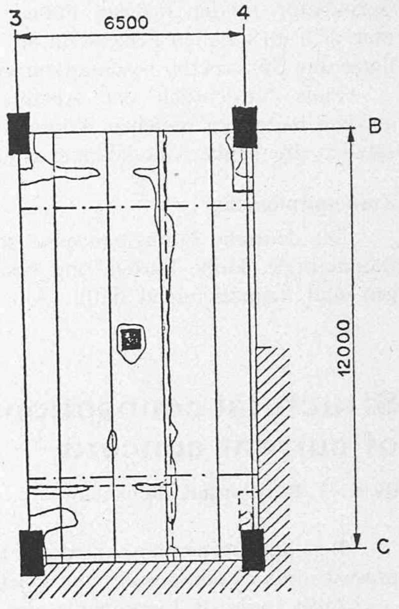


Fig. 4 (right). Examples of non-adhesion areas on a precast floor slab in which the units and topping were to interact. There is no interaction at areas near the support ("k" ends) where the concrete topping was cast on in-situ structural concrete. Other non-adhesion areas are near construction joints in the topping concrete. The survey was made by the Swedish Cement and Concrete Institute, sponsored by the Association of the Swedish Prefabricated Concrete Industries and financed by the Swedish Counsil for Building Research

specifications do not prescribe unnecessary precautions. Our only way to find the good solutions is research where all the appropriate boundary conditions are taken into account.

In the following some examples of studies of structural joint problems will be given.

Low shear at interface

A simple kind of joint is achieved when the joint space is reduced to zero – that is an interface between two concretes. The matter arizes whether the bond between two concretes can be used as a structural tensile connection to secure shear transfer at the interface, thereby allowing composite action.

The matter is of economical significance in e. g. the design of TT-floors. Full scale tests show that composite action is achieved at the ultimate limit state if the execution of the topping procedure is rightly done and also that composite action at the service limit state is satisfied even if a debonding agent is used (fig. 2). Even if doubt in a perfect execution, it seems reasonable to allow for composite design at the service limit state, if certain simple execution rules are followed.

That the ultimate limit state in the case above is reached is not at all astonishing as the shear stresses generated at the interface are small. The potential bond strength at interface is much higher than required as may be seen from the laboratory tests according to fig. 3. These tests show the influence on bond from interface – surface roughness and compaction. They also tell that moistening of the interface surface before pouring the topping reduces the need of compaction.

In Sweden composite action of TT-toppings has been a recognized design procedure until fairly recently, provided low shear (less than 4 kp/cm²) and a limited increase in load bearing capacity (1.2). The change of attitude has been due to a field surway of the state of bond at interface in number of buildings being from 1.5 to 13.5 years old. Preliminary research on test methods indicated that the simple acustic means of impact by a hammer was the best way to find whether the topping was bonded or not. On 8 tested building objects with a total floor area of 90 000 m² totally 4000 m² randomly chosen test areas were examined (fig. 4).

The proportions of area where there was no adhesion were found to be: 0.7 % in the case of rough brushed joint surfaces with a topping of vacuum concrete, 5.8 % in the case of rough brushed joint surfaces with a topping of vibrated concrete and 10.6 % in the case of joint surfaces struck off with a vibrating beam and with a topping of vibrated concrete.

The results have been processed statistically in the report, and it is found, briefly, that the proportion of non-adhearing area varied from 0 to 16 % for the individual buildings and from 0 to 29 % for individual measurement areas, and that in most buildings, 5 No. the proportion of non-adhering area was lower than 3 to 4 %.

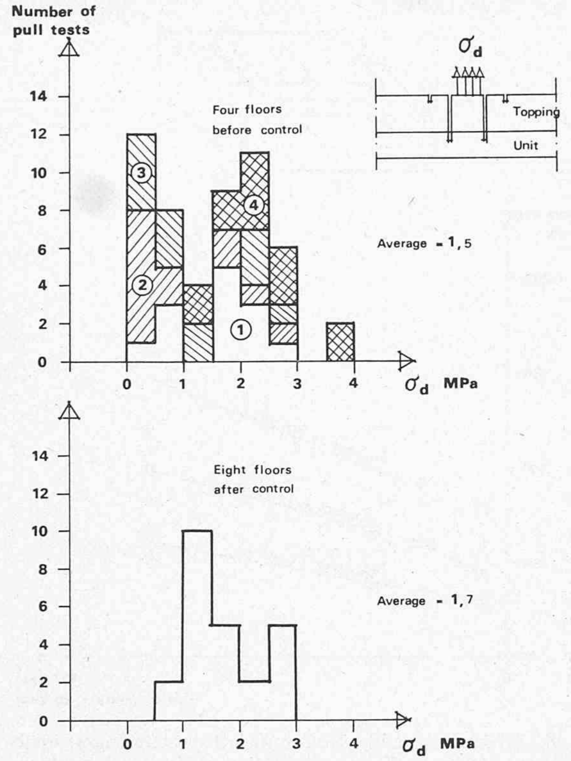


Fig. 5. Tensile bond strength at the interface between TT-slabs and concrete topping measured during construction of an 80 000 m^2 hospital building. The tests were made by pull up of cylinder cores using a method developed at the Swedish Cement and Concrete Institute. Low results were measured at two floors on the slab part of the TT-section. Complementary measurements showed, however, that the corresponding bond above the TT-beams was high

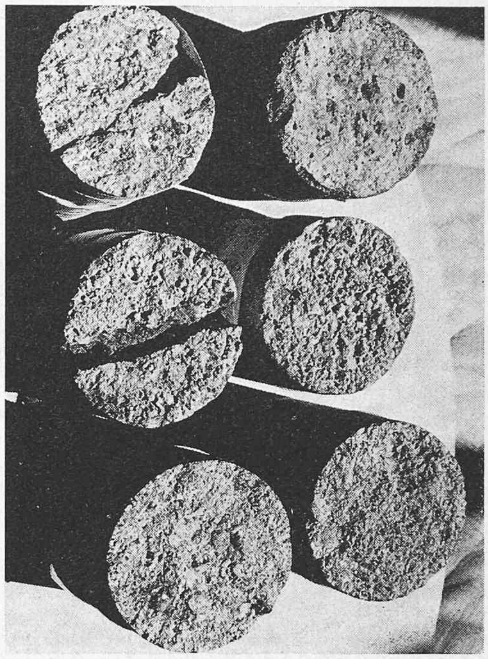


Fig. 6. Examples of rupture surfaces at interface from the tests according to fig. 5. Upwards left ice christals and upwards right a sediment layer from curing can be seen. Insufficient compaction at interface is shown by the two central surfaces and the bottom ones examplify good bond

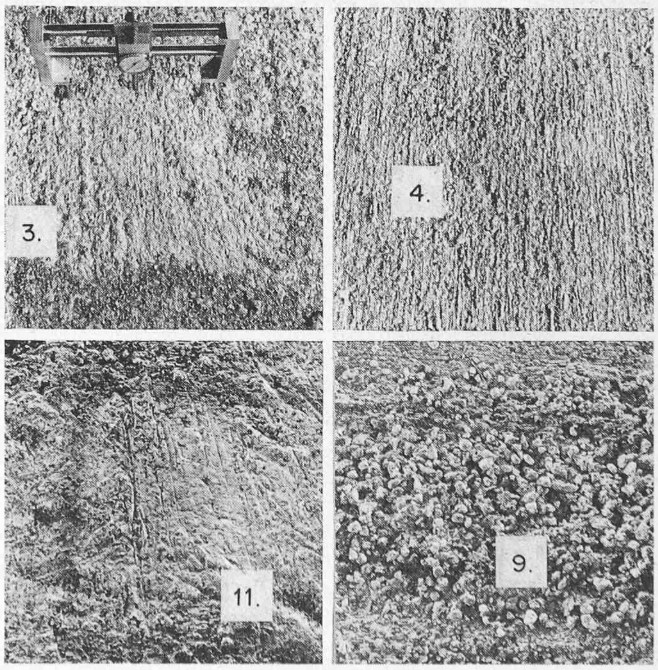
Since it is obvious that there are several reasons for the occurence of non-adhesion areas in these buildings, statistical treatment alone of the results is not sufficient. The value of the investigation consists primarily in its spotlighting the most usual defects which occur when topping concrete is cast. If we disregard cases where lack of interaction was intentional, and one measurement area in a building where workmanship was evidently faulty, in casting topping concrete onto a non-roughened surface, the non-adhesion areas found are mainly confined to

- areas next to joints between courses of topping concrete laid at different times
- topping concrete on the in-situ connection concrete laid on the "k" ends and
- around joints between floor units in conjunction with the use of sealing strips above the joints.

The above described survey gave simply a qualitative indication whether bond was present or not. A recent study at a large 80 000 m^2 building object by means of pull up of bored cylinder cores aimed at a quantitative analyses of the tensile bond strength. Measurements have been made at randomly chosen test areas and test points partially before and partially after special execution control was set in. From fig. 5 it may be seen that test results did improve due to the control.

An analyses of the reasons of measurements indicating lack of bond or low bond values revealed a set of different reasons. The two predominant ones were lack of compaction at the interface and various kind of sediment layers

Fig. 7. Examples of test surfaces in an actual building object for the evaluation of the influence on bond tensile strength at interface from surface roughness and surface quality. At top left the measure device for establishing the surface roughness is seen. This area had a sediment layer which partially was removed by machine working. Top right indicates a normal interface, bottom right an extraordinary rough surface and bottom left one having a laitance skin



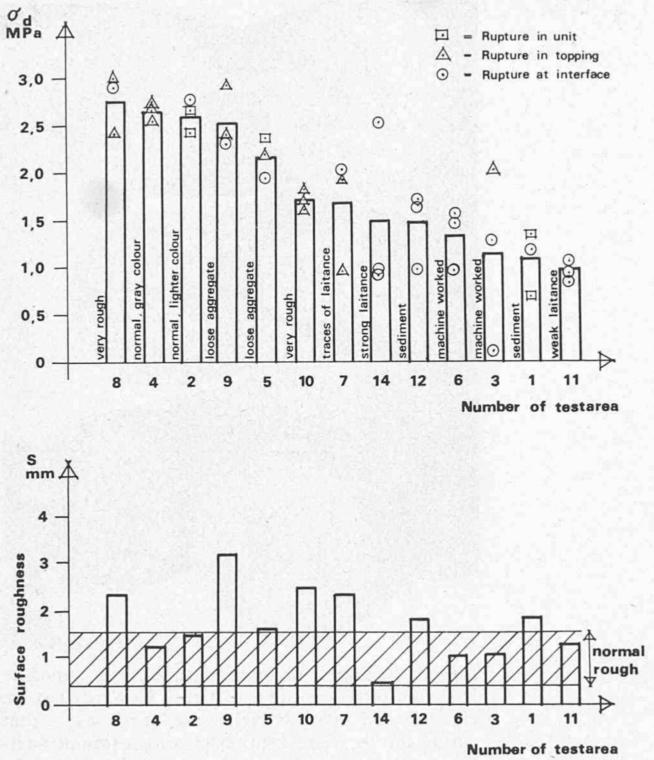


Fig. 8. Pull up core tensile measurements from surfaces examplified in fig. 7. The tests were sponsored by the Precast Industry and carried out by the Swedish Cement and Concrete Institute and are financed by the Swedish Council for Building Research

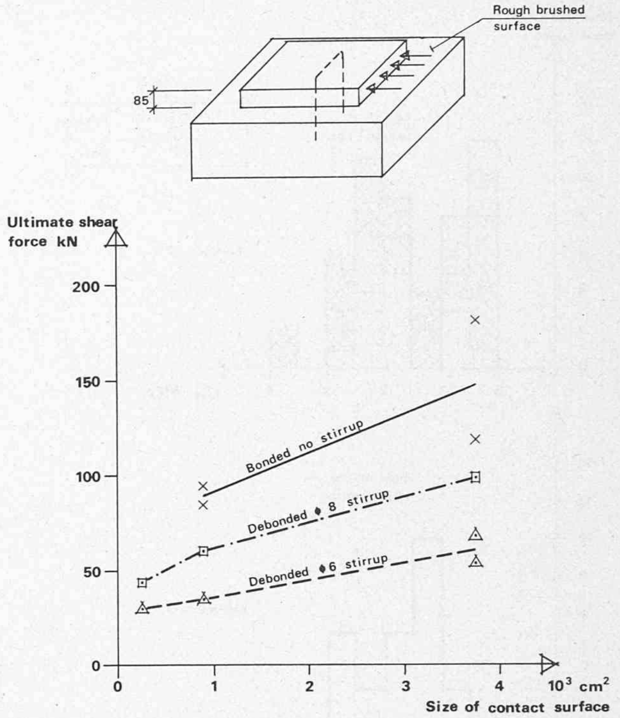


Fig. 9. Preliminary results from push off tests aiming at establishing the influence of size of interface surface and reinforcement ratio. Note especially the increase in push off force by growing debonded contact surface

at the interface. One test specimen showed ice christals and another saw dust at the interface (fig. 6). It was further noted that the bond above the TT-beams was much higher at the same sections where low bond values were measured above the slab parts. The difference is possibly due to variations in the degree of compaction at the interface.

The above test results show the necessity of establishing efficient execution and control procedures. It is well possible that a higher degree of execution safety can be gained if some of the potential bond strength deliberately is sacrificed. To achieve a safe manual a research program has been started. It will be carried out by the Swedish Cement and Concrete Institute and is financed by the Swedish Council of Building Research. This study will mainly concentrate on establishing which surface qualities are acceptable and how to achieve sufficient compaction at interface, a compaction that must overcome the lack of the concrete mixing effect between the two concretes at the interface. The research will partially be carried out as field surveys, partially in the laboratory.

A preliminary result from cylinder pull tests indicates the importance of the interface *surface quality*. Test points were selected previous to the pouring of the floor topping so that a set of interface surface qualities (fig. 7) were represented. All the test points were located on a floor area which was poured in one sequence. The roughness of each individual test surface was measured according to a statistical method (Swedish standard, SIS 81-2005).

The results (fig. 8) indicate that more important than roughness is the surface interface quality. Sediments from water curing of the TT-elements and laitance skins led to large dispersions in bond strength even in cases where the interface surface was roughened.

Fig. 10. Types of anchor rupture of stirrups cast into a thin concrete topping. Note the difference in type of rupture for smaller (15×15 cm^2) and larger (30×30 cm^2) surfaces. When size was (61.5×61.5 cm^2) the rupture was a pure tension cone around the stirrup

Low shear at rough interfaces using tie bars

A further matter of interest is to establish the lowest necessary demand of ties assuming the bond at roughened interfaces to be zero. An investigation has been started at the Division of Concrete Structures at the Chalmers University of Technology. The study will be directed to toppings on TT-slabs. Up to now, however, only some pilot investigations have been made to establish the efficiency of single ties when anchored in 65 mm thick toppings having varying size of interface contact surface.

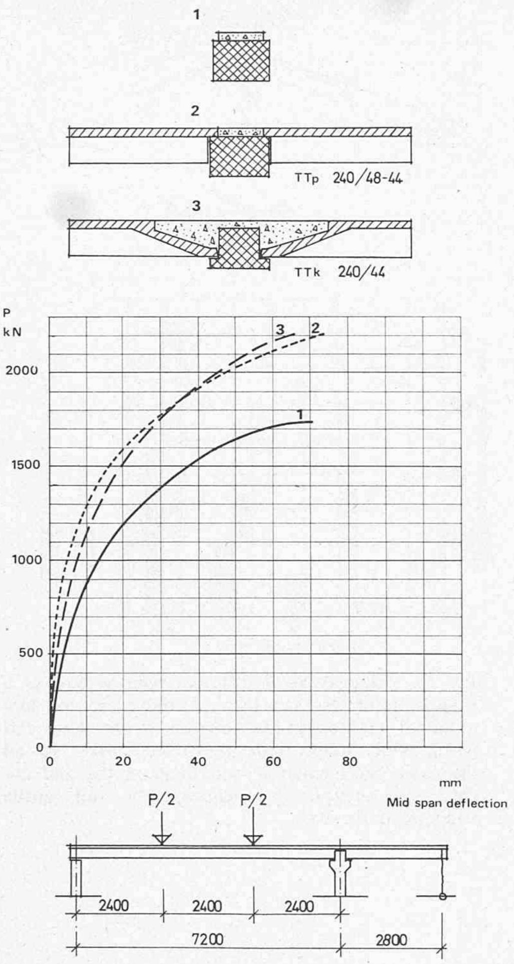


Fig. 11. Deflection at midspan (between the point loads) for three different composite sections. The one having only topping but no slabs was made for reference. The continuity support arrangement which is used since long proved full composite action. The shoulder type support developed also full composite action before the slabs were sheared off. In both cases the interface between concrete topping and slab end faces were precracked before the main test by a preliminary loading of the slab cantilevers

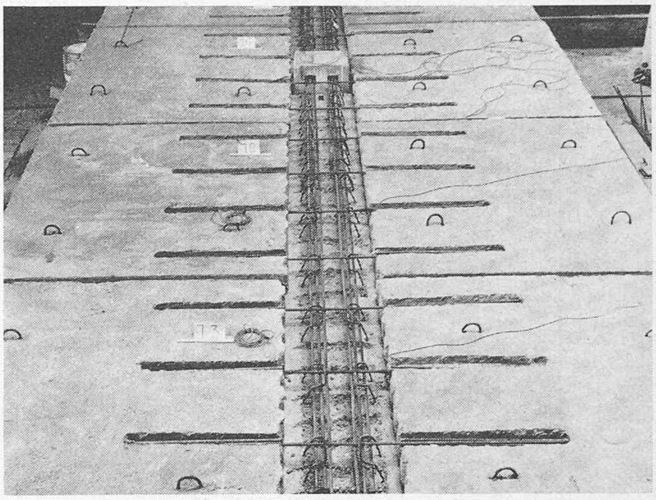


Fig. 12. Arrangement of the connection reinforcement for the TTp-shoulder support. In practice the connections above the beam between the slab units are instead made in form of projecting stirrups

Results (fig. 9) indicate that bonded surfaces without shear reinforcement have higher shear capacity than the corresponding debonded reinforced joints. However, the ultimate shear stress for bonded interfaces decreases markedly when increasing the size of the contact surface, e.g. about to half the value when the surface size is increased from 900 to 3800 cm². If, however, rough interfaces are debonded and are tied with stirrups the ultimate shear force grows when increasing the contact surface. This fact indicates that not only the strength of stirrup decides the shear capacity. In fact, applying the shear friction hypothesis the friction coefficient seems to grow when increasing the size of the contact surface.

These pilot tests also show that it is important to confine the experiments to the current dimensions of practice when studying low shear at interface.

Another observation of interest was that the larger stirrups ∅ 8 gave a higher shear force resistance than the smaller ∅ 6 stirrups, even if in the comparison the different reinforcement ratio is taken into account. The reason for this is not yet understood.

Fig. 13. Arrangement of diaphragm reinforcement in slabs at the end of test floors built up by 27/120 hollow core slabs. In a pilot test said reinforcement 2 ∅ 16 Ks 60 was placed only along the tensile edge and for other tests also along the compressive edges. The slabs were loaded by pressing the two floors apart with jacks. The arrangement allowed different length of shear spans to be examined

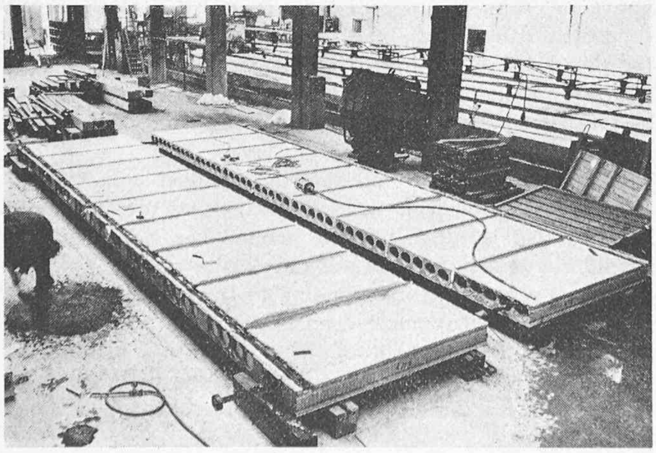
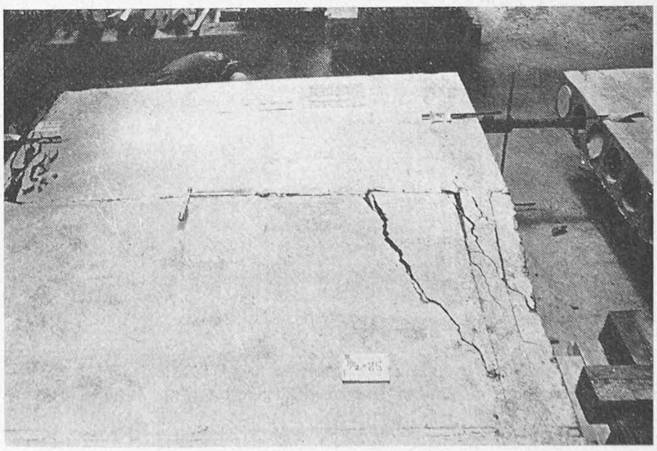


Fig. 14. After a primary shear failure of a test specimen according to fig. 13 the slip along the failed joint is large. This, however, requires an increase in the shearing load. The joint thereby satisfies high demands with regard to ductility



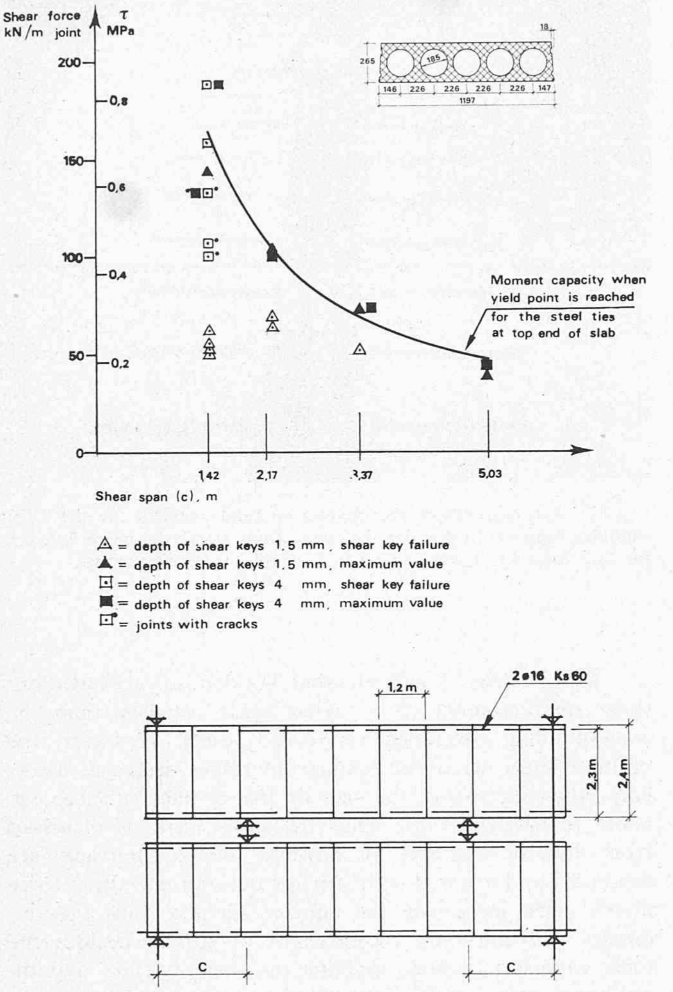


Fig. 15. The test results regarding shear capacity of specimens according to fig. 13 are summarized in the diagram. Note that the primary shear failure capacity gets more than doubled if the depth of shear key indentations are increased from 1.5 to 4 mm

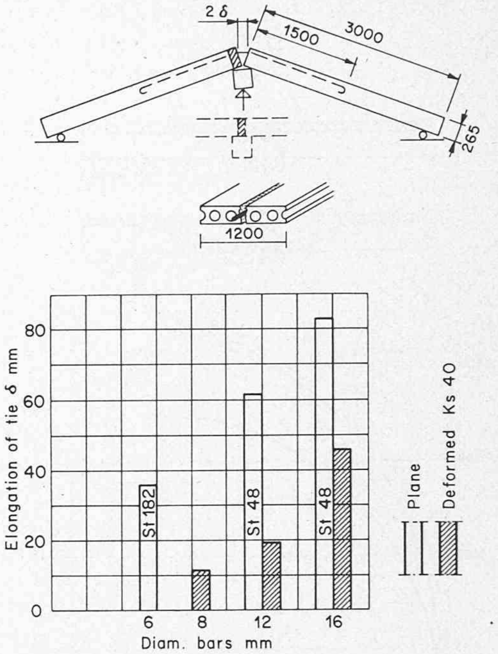


Fig. 16. Elongation of different type tie bars at a support joint, the ties being anchored in the longitudinal grouted joints between 27 cm deep and 60 cm wide hollow core prestressed slabs. In all the cases represented in this diagram the end anchor was capable of anchoring the full tensile strength of the bar

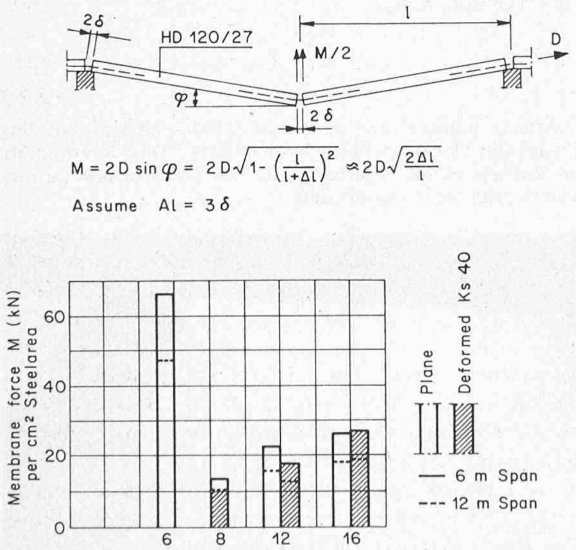


Fig. 17. Calculated maximum membrane force when loss of a central support assuming the ultimate elongation values according to fig. 16. The calculation is made for the floor spans 6 and 12 m

Fig. 10 shows the mode of failure. In all cases here presented the anchor of the stirrup in the topping became critical, however, often at displacements beyond the maximum load.

Where small contact surface the anchor failure was due to splitting of the topping. Where large contact surface the failure in the topping formed a tensile rupture cone around the stirrup.

The above results indicate that there is still much to learn about the tieing of thin toppings.

Composite action between supporting beams and precast TT-floors

To prove the efficiency of two kinds of TT-support arrangements some full scale tests were performed at AB Strängbetong (fig. 11).

The structural connection was in both cases a topping above the supporting beam between the end faces of the supported slabs. In the case of continuity arrangement of the slab supports the horizontal tie bars can be regarded as indirect. In the case of shoulder-support the horizontal tie bars can be regarded as direct with regard to the critical shear surfaces.

It is interesting to note that the shoulder-support solution produced the same level of stiffness as the continuity support, that is full composite action. The failure of the shoulder type arrangement was due to shear at the slab end interfaces at a load corresponding to the ultimate moment capacity. The behaviour indeed is the same as that of an ordinary monolithic T-beam. The supporting rectangular beam being wide enough it is thus possible to increase its load bearing capacity by using the preceast floor as compression zone.

The shoulder support alternative (fig. 12) is of special interest in cases where large paraziting tensile stresses due to creep of prestressed slabs shall be avoided. It also fits well in connection with the camber compensated TTp-slab. Such a component has a growing slab thickness towards its ends which results in sufficient slab thickness for the supporting shoulder. The TTp-slab is normally used without structural concrete topping.

Shear transfer between hollow core floor slabs

The shear transfer along longitudinal joints is often transferred by a groutfill between shear keyed slab side faces. A simple way to secure the restraint transversally to the joints is to use the diaphragm moment reinforcement for the purpose. Such reinforcement is preferably placed at the slab ends at or near the supports. The structural model is actually a segmental one.

To get a better understanding of the shear capacity of longitudinal joints in pretensioned hollow core floors, some large size tests were carried out. Each test floor was composed from 9 slabs having the width 120 cm, all according to fig. 13. The diaphragm reinforcement was placed in concreted slots at the ends of each floor. When testing, two floors were simultaneously loaded against each other in a horizontal position. Each test specimen was loaded several times up to the yield stress of the tensile reinforcement or up to shear failure. At each new loading the shear span was decreased whereby several measurements were achieved from one single test specimen.

The primary shear failure (fig. 14) at a joint was indicated by the failure in dowel action of the diaphragm reinforcement. A first pilot test with tensile reinforcement only, showed this kind of rupture to be brittle and final. In the other test specimens, however, the diaphragm was provided also with equally much compression reinforcement. This being the case the load could be increased quite much above the primary rupture level. The load increase led to large relative displacements along the failing joints of the order up to 10 cm. The reason of the increased load capacity at large deformations was that also the compression reinforcement was exposed to tension thereby increasing the frictional capacity of the joints.

The test results are best seen in the diagram fig. 15 from which the measured shear capacity at various shear spans may be read. Two different type of shear keys were tested, one having only 1.5 mm average depth and the other about 4 mm. The primary shear failure level was markedly increased using the deeper shear keys by enlargement of the shear transfer zone (compression zone) also to parts of the joints exposed to bending cracks.

The fact that the diaphragm capacity of a prefabricated floor increases also after an initial shear failure (second level shear capacity) and allows large displacements is of great interest. Design with regard to seismic forces and progressive collapse can profit from such qualities. The matter thus seems to be worth further consideration.

Indirect ties, anchored in longitudinal slab joints

The anchoring of ties in longitudinal joints between floor slabs is an economically attractive and simple method and is therefore often applied in practice. The soundness can be doubted due to shrinkage cracking and the risk of inadequate execution of mortar joints. Further, does it function when large deformations of the floor?

Using test arrangements composed from hollow core slabs an investigation was carried out at the Division of Concrete Structures at the Chalmers University of Technology.

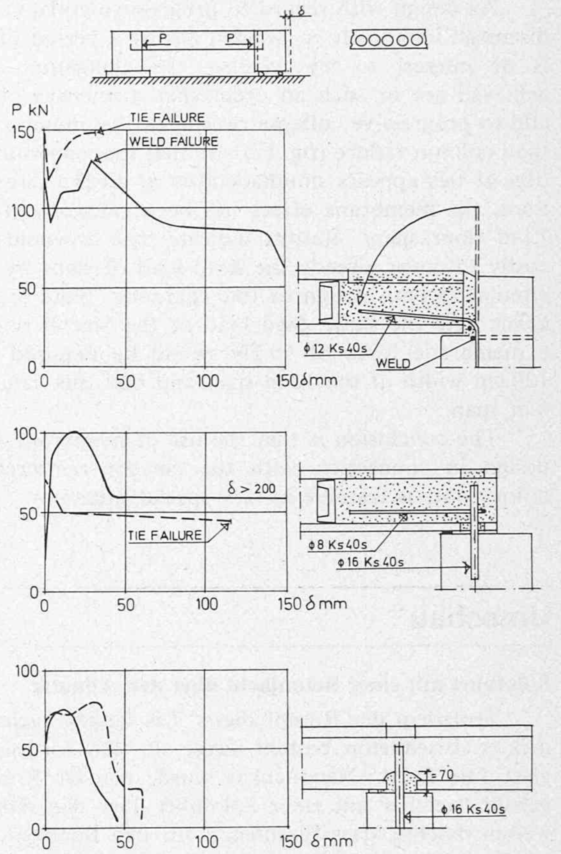


Fig. 18. Horizontal displacements at various type of supports versus an applied horizontal load. The tests are to be regarded as pilot investigations in order to give an idea of the ductility of the tie arrangement. Tests: AB Strängbetong

The test specimens (fig. 16) consisted of an underlaying supporting beam from which two pair of slab units cantilevered in opposite directions, their ends resting on movable simple supports. The ties were reinforcing bars placed in the transverse concreted joint between the end faces of the slabs above the beam support and in the longitudinal grouted joints between the slabs. By elevating the beam support the tie bar in the longitudinal joints was exposed to tension. Bars of various types, sizes and placement were tested – first with relatively short anchor lengthes, 70 cm, then with longer anchor lengthes, 300 cm. These tests revealed that the ultimate strength of bars could not be anchored efficiently when considering also the effects of cracked joints and the uncertainty to get the joint fill grout well executed.

The conclusion was that tie bars which shall be anchored in narrow grouted joints should be provided with end anchors. The tests were continued using end anchored ties extending 150 cm into each joint.

It is of interest to study the maximum elongation capacity of the tie bars investigated at the bar tensile failure level. The tests revealed, as expected (fig. 16), that plane bars gave larger elongations than deformed bars and that the elongation capacity increased with the bar size. The elongation capacity measured for these indirect ties is certainly larger than is the case with direct ties.

As design with regard to progressive collapse has been discussed intensively in Sweden during a period of years, it is of interest to try whether the elongation capacities achieved are of such an order that membrane effects can add to progressive collapse resistance. Assuming e. g. a mid span column failure (fig. 17) and that the maximum elongation of ties appears simultaneously at all the three connections, the membrane effect has been calculated for 6 and 12 m floor spans. Results indicate that it would be quite costly to cover already the dead load of slabs by using an alternative path design of this character. Note e. g. that to cover only the static dead load of the Spiroll slabs tested, a membrane force of 50 kN would be required for each 120 cm width of the 12 m span and half this value for the 6 m span.

The conclusion is that the use of membrane effects in design in connection with the current reinforcing technology only is feasable in very special situations.

Ductility of tie arrangements at slab supports

In a Swedish draft for revision of the specifications with regard to progressive collapse it has been suggested that a tie connection should have a ductile capacity of at least 15 mm.

To find out whether this was possible some pilot tests were carried out using prestressed hollow core slabs with a depth of 27 cm. The results (fig. 18) show that the suggested displacement can be achieved for the type of connections tested.

The tests revealed also a relatively high dispersion in the test results which was due to the mode of failure. Tests of this character are of interest when developing new type joints as their weaknesses show up at large deformations.

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Umschau

Solofahrt mit einer Betonjacht über den Atlantik

Trotzdem der Rumpf dieser 7 m langen Jacht aus 1/2″ dickem Eisenbeton besteht, zeigt sie sich formschön und glatt. Die Jacht «Nanouchka» wurde von Dr. Robert Levy gebaut, welcher mit einer Solofahrt über den Atlantik beweisen möchte, dass Eisenbeton für den Bootsbau geeignet ist. Der Mann am Steuer, Dr. Robert Levy, wird seinen Posten an einem Krankenhaus in Mittelengland aufgeben, um die Überfahrt nach New York anzutreten.

«Nanouchka» wurde nach mehrjähriger Forschung von Dr. Levy entwickelt und wird das kleinste Betonboot sein, das den Atlantik bisher überquert hat. Der Rumpf ist nur 12,7 mm dick und wiegt im vollbeladenen Zustand 2270 kg – etwa ebensoviel wie ein vergleichbares Holzboot. Eisenbeton lässt sich wirtschaftlich verarbeiten, ist Temperaturextremen gegenüber beständig und wird im Seewasser immer härter.

Dr. Levy erklärte: «Dies ist eine Technologie, welche in die Hinterräume von Universitäten und in die Hintergärten von Bootsbauamateuren verdrängt wurde; doch dieser Werkstoff bietet grosse Möglichkeiten». DK 629.125.12:691.32

Netzkommandostelle für die Stadtwerke Frankfurt

Die Stadtwerke Frankfurt erteilten kürzlich AEG-Telefunken den Auftrag zum Aufbau einer neuen Netzkommandostelle, die aufgrund ihres Systemkonzeptes hohen Anforderungen an eine sichere Betriebsführung gerecht wird. Der Gesamtwert dieses Projektes, das Ende 1978 in Betrieb gehen wird, beläuft sich auf über 6,5 Mio DM.

Prozessrechnersystem AEG 80-40

Kernstück des neuen Netzautomatisierungssystems ist ein Prozessrechner AEG 80-40 mit der für die gestellten Aufgaben erforderlichen Peripherie. Er verfügt über 192 kByte Kernspeicherkapazität und ist mit zwei Wechselplatten mit je 60 MByte ausgestattet. Für den Dialogverkehr zwischen dem Bedienungspersonal und dem Rechnersystem, für die Wiedergabe von Netzdarstellungen, Anlagenbildern sowie für die Eingabe von Steuerbefehlen und automatischen Schaltprogrammen stehen zwei Farbsichtgeräte zur Verfügung. Zwei Schwarz/Weiss-Sichtgeräte sind für die Ausgabe der Betriebsprotokolle vorgesehen. Die wichtigsten Aufgaben des Prozessrechners sind die Aufbereitung und Verarbeitung der über Fernwirkanlagen übertragenen Betriebsinformationen, der Dialogverkehr mit dem Bedienungspersonal, das Protokollieren von Betriebsdaten sowie das Durchführen von übergeordneten rechenintensiven on-line-Programmen wie Netzsicherheitsrechnungen und Lastprognosen. Für die Netzplanung sollen mit dem Rechnersystem Lastfluss- und Kurzschlussberechnungen durchgeführt werden.

Netzwarte

In der neuen Netzwarte wird das Bedienungspersonal in einem nach heutigen Erkenntnissen gestalteten Wartenraum unter ergonomisch günstigen Umgebungsbedingungen arbeiten, um den vielfältigen Aufgaben gerecht zu werden. Die Farbsichtgeräte geben eine schnelle Auskunft über die momentane Situation im Hochspannungsnetz. Ein raum-