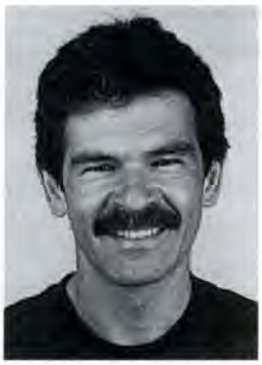
# Tests on Special Reinforcement for the End Support of Hollow-Core Slabs



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*Details of special reinforcement at the end supports of hollow-core precast concrete floor units with a cast-in-place concrete topping slab are discussed. This reinforcement is intended to prevent collapse of the floors in the event of inadequate seating lengths or imposed movements due to volume changes or earthquakes. Results from experimental tests in which a vertical load was applied to the hollow-core precast concrete floor units with a cast-in-place topping slab and special support reinforcement are also presented. Three types of special reinforcement were investigated with two types of test. In one test, the vertical load was applied when the support seating was zero; in the other test, the vertical load was applied after the precast floor unit had been pulled horizontally off the supporting beam. The special reinforcement was shown to be capable of supporting at least the service gravity loads in these extreme conditions, if well designed.*

The use of precast concrete in flooring systems has become commonplace in many countries. While design and construction aspects for precast concrete floors have generally been carefully considered, aspects of the support of precast concrete floors in building structures have not been fully covered by building codes. For example, both the current New Zealand concrete design standard, NZS 3101:1982,[1] and the building code for reinforced concrete of the American Concrete Institute, ACI 318-89,[2] contain comprehensive provisions for the design of cast-in-place concrete structures but do not contain provisions covering all aspects of structures incorporating precast concrete elements.

In regions of countries where earthquakes may occur, the design and construction of structures incorporating precast concrete elements subjected to seismic forces require particular care. Because New Zealand is in an active earthquake zone, and the use of precast concrete in buildings increased significantly in the 1980s, the New Zealand Concrete Society and the

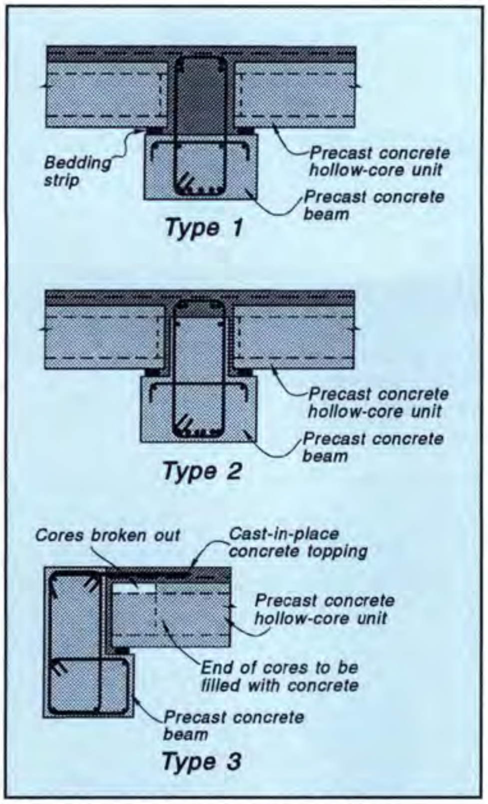


Fig. 1. Types of support used in New Zealand for precast concrete hollow-core floor units seated on precast concrete beams.

New Zealand National Society for Earthquake Engineering formed a study group in 1988 to summarize and present data on precast concrete design and construction, with emphasis on seismic aspects. The outcome of the deliberations of the study group is a manual authored by the members of the group titled "Guidelines for the Use of Structural Precast Concrete in Buildings," which was published by the Centre for Advanced Engineering of the University of Canterbury in August 1991.[3]

Currently, practically all floors in New Zealand buildings are constructed of precast concrete, spanning one-way between beams or walls. The precast concrete elements generally act compositely with a cast-in-place concrete topping slab. Typically, the topping slab is 65 mm (2.6 in.) thick and contains at least the minimum reinforcement required for slabs in order to transfer the seismic shear forces to the supporting structure through diaphragm action. In New Zealand, very limited use has been made of untopped precast concrete floor units with appropriate shear transfer provided.

The chapter in the New Zealand Guidelines[3] on floor unit support and continuity identifies three basic types of support for precast concrete hollow-core floor units using precast concrete beams (see Fig. 1). These basic types of support can also be used for other types of precast concrete floor systems. The main difference among these types is the depth of the precast concrete supporting beams.

In the case of the Type 1 support shown in Fig. 1, the use of relatively shallow precast concrete supporting beams enables the cast-in-place concrete to be placed easily between the ends of the floor units, thus ensuring reliable negative moment continuity to be developed between those units. However, the relatively shallow precast concrete supporting beams may need to be propped during construction.

The Type 2 support has deeper supporting beams that require less propping, but if the vertical gaps between the precast supporting beam and the floor units are small, there may be difficulty in placing the cast-in-place concrete in the gaps. The Type 3 support is useful at perimeter beams and at walls.

Fig. 2. Example of hanger bars for precast concrete floor units used in New Zealand.

## AVOIDANCE OF POSSIBLE FAILURE AT SUPPORTS

Failure of the support of precast concrete floor systems, leading to collapse, can occur due to two main reasons:

**1.** Inadequate seating lengths in the direction of the span, normally as a result of tolerances not being met.

**2.** Seating lengths in the direction of the span that are reduced as a result of imposed movements from volume changes of structural members resulting from creep and shrinkage, from seismic shaking dislodging the floor units, or from an increase in distance between the supporting beams caused by the elongation of adjacent beams from plastic hinging in a ductile frame during a severe earthquake.[3,4]

Special reinforcement in the form of inclined hanger bars or saddle bars, as shown in Figs. 2 and 3, can be designed to carry the precast concrete floor units in the event of bearing failure at the end of the units or lateral movement of units off the supporting beams.[3] Overseas practice, including the use of continuity or tie steel at supports as suggested by the Fédération Internationale de la Précontrainte (FIP)[5] and the Precast/Prestressed Concrete Institute (PCI),[6] is illustrated in Figs. 4 and 5.

The special reinforcement must be designed to be capable of providing an alternative load-carrying mechanism that will permit the precast concrete floor units to remain suspended in the event of loss of end bearing. This will eliminate the danger of a progressive collapse caused by floor units falling onto the floors below.

The draft revision of the New Zealand standard for the design of concrete structures[7] makes certain recommendations for precast concrete floor or roof members, with or without the presence of a cast-in-place reinforced concrete topping slab. The standard states that, unless the performance of alternative details at the supports is proven by analysis or test to be acceptable, each member and its supporting system shall be designed so that, under a reasonable combination of unfavorable construction tolerances, the distance from the edge of the support to the end of the precast member in the direction of its span is at least $^1/_{180}$ of the clear span but not less than 50 mm (2 in.) for solid or hollow-core slabs or 75 mm (3 in.) for beams or ribbed members.

Therefore, according to the draft New Zealand standard, proven alternative support details are required unless the specified end distances are provided. The above recommendation is similar to that being considered by ACI Committee 318 for the next revision of the current ACI Building Code. The above end distances are similar to those recommended by ACI-ASCE Committee 550.[8]

A research project is currently in progress at the University of Canterbury that tests the performance of some possible alternative support details in the form of special reinforcement at the ends of precast concrete hollow-core floor units in buildings designed for earthquake resistance. This paper presents the results of the first series of tests of the research project. Further details may be found in Ref. 9.

## VERTICAL SHEAR FORCE TRANSFER

When considering ways of transferring the vertical reaction at the ends of precast concrete floor units in the event of loss of seating at the supporting beams, the designer may assume the shear force is being carried by one of two actions:

**1.** Shear friction across vertical cracks in the cast-in-place concrete topping slab and at the interface at the

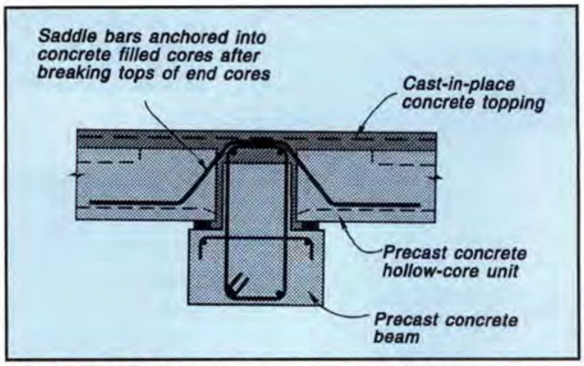


Fig. 3. Saddle bars for precast concrete hollow-core units suggested by the New Zealand Guidelines.

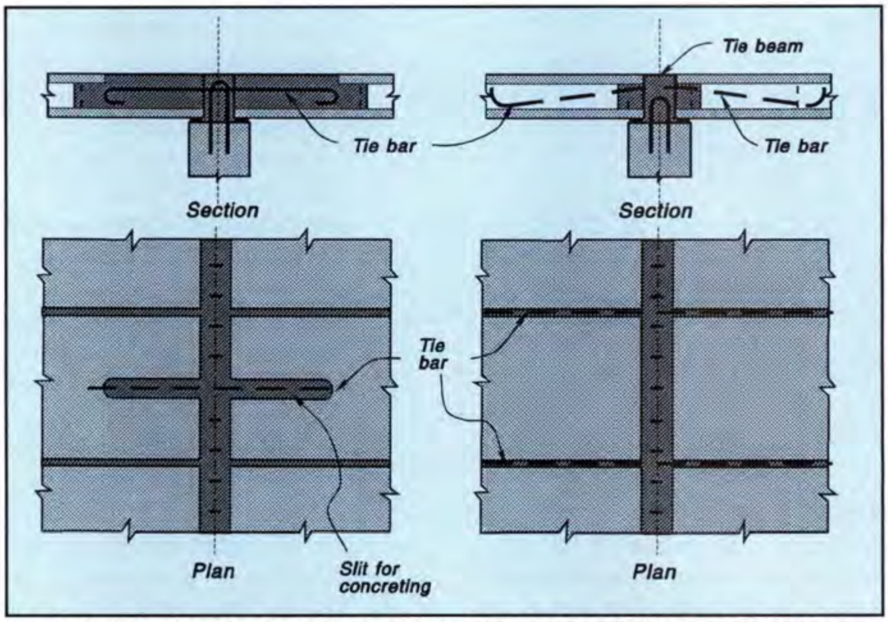


Fig. 4. Details of continuity or tie steel suggested by the Fédération Internationale de la Précontrainte (FIP).

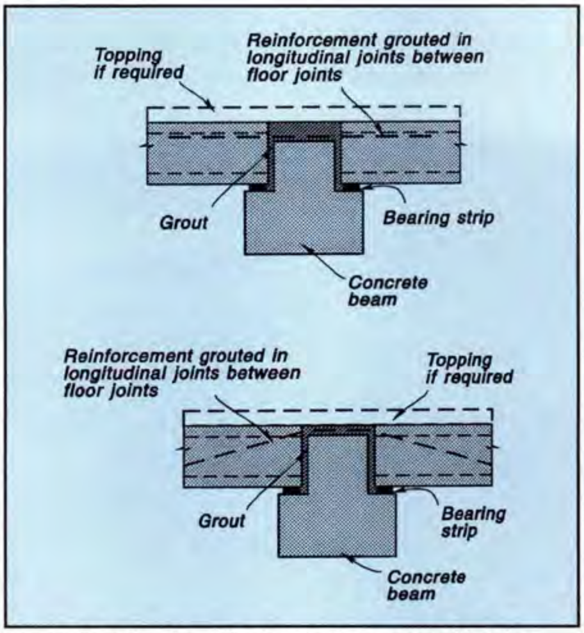


Fig. 5. Details of continuity steel suggested by the Precast/Prestressed Concrete Institute (PCI).

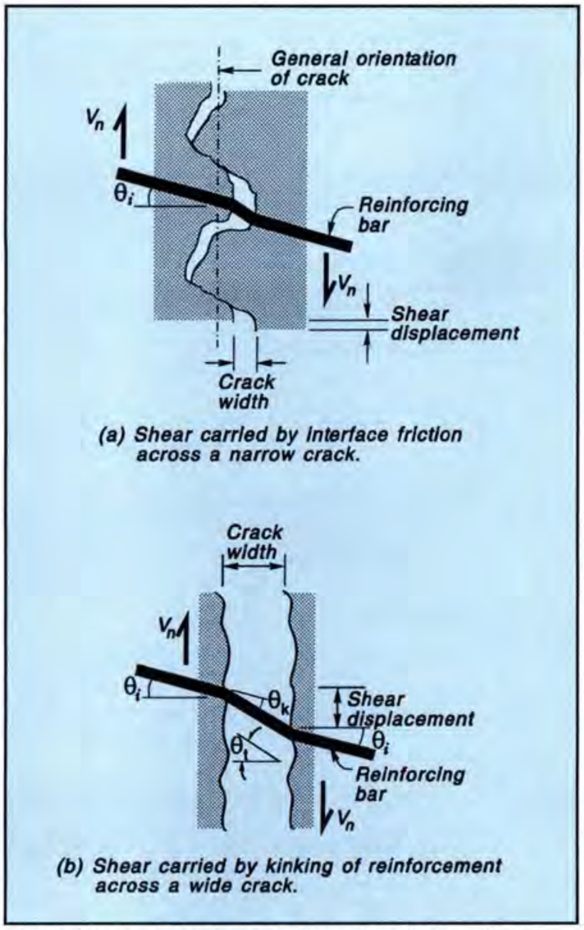


Fig. 6. Two mechanisms for transfer of shear force across cracks.

ends of the precast concrete units, if the crack widths are relatively narrow (see Fig. 6a); or

**2.** Kinking of the reinforcement crossing the cracks, if the crack widths are large (see Fig. 6b).

### Shear Friction

According to ACI 318,[2] in the general case when the shear-friction reinforcement is inclined at an angle other than 90 degrees to the shear plane and such that the shear force produces tension in the shear-friction reinforcement, the shear force $V_n$ transferred by shear friction across a crack by interface roughness is given by:

$$V_n = A_{vf} f_y (\mu \cos\theta_i + \sin\theta_i) \quad (1)$$

where

$\mu$ = coefficient of friction, which is 1.4 for a crack in concrete placed monolithically, 1.0 for a crack at the interface of concrete placed against hardened concrete that is clean, free of laitance and intentionally roughened to a full amplitude of at least 6 mm (1/4 in.), or 0.6 for concrete placed against hardened concrete that is clean and free of laitance but is not intentionally roughened to a full amplitude of at least 6 mm (1/4 in.)

$A_{vf}$ = area of shear-friction reinforcement

$f_y$ = yield strength of shear-friction reinforcement

$\theta_i$ = angle of shear-friction reinforcement to normal of shear plane (see Fig. 6a)

This dual contribution of inclined reinforcement (clamping force provided by the component of bar force normal to the shear plane and resisting force provided by the component of bar force along the shear plane) was confirmed by Mattock.[10]

Mattock et al.[11] have also reported that bending moments acting on the shear plane equal to or less than the flexural strength of the section at the shear plane do not reduce the shear that can be transferred by shear friction. Hence, Eq. (1) can be applied to a crack in a structural element transferring both shear force and bending moment.

It is assumed that the shear displacement along the crack for this mechanism is small, because the crack width is also small. Hence, the shear transferred by dowel action across the crack is negligible because the mobilization of dowel action requires a larger shear displacement.[12]

Note that the design load of the floor calculated from the vertical shear capacity at the supports due to shear friction cannot exceed that governed by the flexural strength or shear (diagonal tension) strength of the floor.

### Kinking

When the crack widths are large, and the interface roughness can no longer interlock across the crack, the shear displacement along the crack will become large. When the crack width becomes very large, the shear transfer mechanism occurs through kinking of the bars crossing the crack. The shear force, $V_n$, transferred by kinking across a vertical crack (see Fig. 6b) is given by:

$$V_n = A_{vf} f_s \sin\theta_t \quad (2)$$

where

$A_{vf}$ = area of steel reinforcement crossing the crack

$f_s$ = stress in steel reinforcement crossing the crack, normally taken as the yield strength $f_y$ but higher if the bar enters the strain hardening range after yield

$\theta_t$ = angle of kinked reinforcement crossing the crack to the normal of the crack, equaling the sum of any initial angle of inclination, $\theta_i$, of bar to the normal of the crack plus the kinking angle, $\theta_k$, of the bar (see Fig. 6b)

### Need for Special Reinforcement

The transfer of shear by shear friction or kinking of reinforcement in the cast-in-place topping slab alone may not be effective because splitting of the topping slab from the precast concrete hollow-core units may occur. This is because precast hollow-core units do not contain web shear reinforcement and hence, are not tied to the topping slab, because of the dry-cast manufacturing process. Therefore, the special reinforcement at the ends of precast concrete hollow-core units will generally need to be anchored with cast-in-place concrete in either the voids within the units or in the longitudinal joints between the units.

## LITERATURE REVIEW

Very little testing has been conducted investigating the effectiveness of the various arrangements of hanger, saddle, tie and continuity reinforcement shown in Figs. 2 to 5.

In New Zealand, Blades et al.[13] have conducted ultimate load tests to investigate the effects of various seating conditions on precast concrete hollow-core units spanning between supporting beams with a 65 mm (2.6 in.) thick cast-in-place concrete topping slab. It was concluded that end bearing lengths as small as 5 mm (0.2 in.) in the direction of the span are satisfactory for support, assuming that welded steel mesh in the topping slab and saddle bars anchored in filled voids, both continuous over the support, are present.

In Sweden, Engstrom[14,15] conducted several tests to study the basic behavior of tie connections when subjected to large displacements. Various simple types of tie connections between concrete panels were loaded to failure in pure tension, bending, or combined normal and transverse loading. The load-displacement relationship, the anchorage behavior, and the type of failure were recorded. The objective was to investigate the achievable deformation capacity for some basic types of tie connections and to suggest simple design and computational methods based on the test results. The main conclusions reached were:

**1.** To achieve a large deformation capacity, it is important to prevent anchorage failure of the tie bars. This means that the anchorage capacity should exceed the fracture strength of the ties. Therefore, the strength of the ties in the strain hardening range must be taken into account.

**2.** Plain round tie bars should be anchored by end hooks. Deformed bars can be anchored by adequate straight embedment length but, because of possible poor or incomplete filling of joints and voids, deformed bars should also be provided with end hooks.

**3.** Large tie extensions in the plastic range can be obtained if bond failure propagates along the tie bar, thus permitting yielding of steel to extend along the bar. The possible plastic elongation of a tie bar can be increased by any method that increases the length of the region of bond failure.

**4.** The tensile behavior of the connections between precast floor units does not appear to be affected by the dynamic action that follows the sudden loss of a support. In analysis, the behavior of floor connections can be characterized by load-displacement relationships that were derived from tests with static loading.

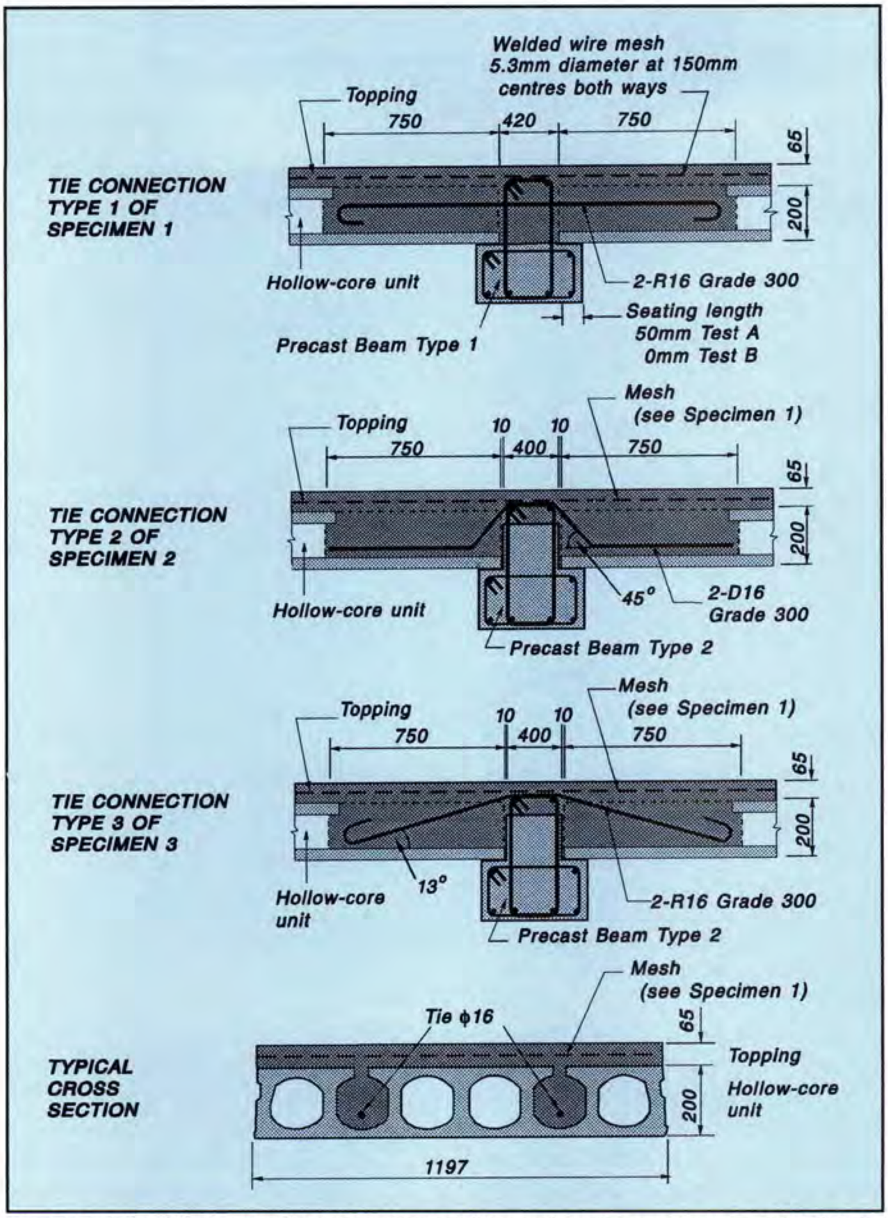

Fig. 7. Details of tie connections investigated (dimensions in mm; 1 mm = 0.0394 in.).

## THE CURRENT TEST SERIES

### Test Specimens and Loading

The behavior of three different types of tie connection between the ends of precast, pretensioned concrete hollow-core units, subjected to large displacements, has been investigated at the University of Canterbury.[9] Each hol-

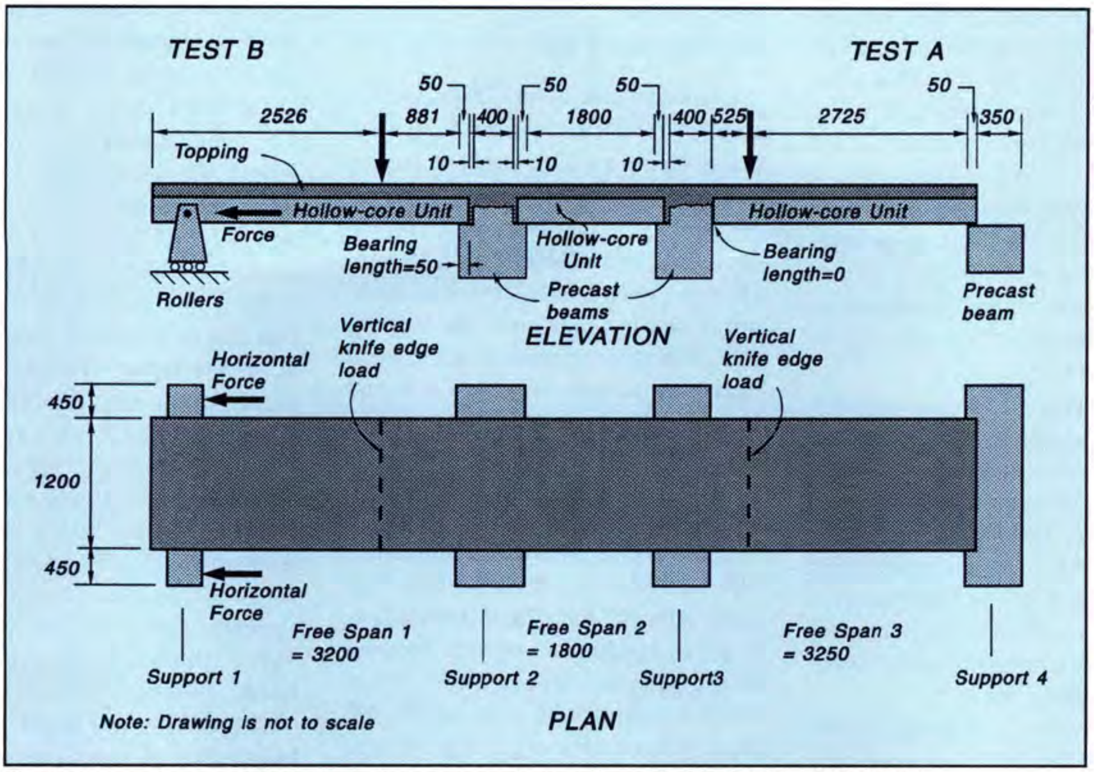

Fig. 8. Dimensions and test setup of specimens (dimensions in mm; 1 mm = 0.0394 in.).

low-core unit had a cross section 200 mm (7.9 in.) deep x 1200 mm (47.2 in.) wide and contained six voids.

The seven prestressing tendons were placed at the bottom of the webs, the five inner tendons being 12.5 mm (0.5 in.) diameter strand and the outermost strands being 11.0 mm (0.43 in.) diameter strand. A typical cross section is shown in the Appendix. All dimensions,. properties, and materials of the hollow-core units are also given in the Appendix.

Cast-in-place concrete was present over the precast units and the supporting beams. The 65 mm (2.6 in.) thick topping slab over the units was reinforced at mid-depth by a square welded wire mesh formed of 5.3 mm (0.21 in.) diameter wires at 150 mm (5.9 in.) centers. This mesh reinforcement had an area that was 0.0023 of the gross area of the topping slab and is typical of that used in New Zealand buildings.

The three types of tie connection that were investigated are shown in Fig. 7. The ties were placed in two voids of each unit, which were broken back over a length of 750 mm (29.5 in.) and filled with cast-in-place concrete during the placing of the topping concrete. The tie bars were from Grade 300 reinforcing steel, which has a characteristic yield strength of $f_y$ = 300 MPa (43.5 ksi). Plain round 16 mm (0.63 in.) diameter bars were used for tie connection Types 1 and 3 and deformed 16 mm (0.63 in.) diameter bars were used for tie connection Type 2.

Electrical resistance strain gauges of 5 mm (0.2 in.) gauge length were attached to the tie bars at intervals. Preliminary trials had shown that for the type of gauges and adhesive used, strains up to at least 6 percent could be measured before failure of the gauges.

The test specimens were constructed with the dimensions and test setup shown in Fig. 8. The hollow-core units used for each test specimen were in three separate lengths, forming three spans. Each test specimen contained the same type of tie connection between the units at the two interior supports.

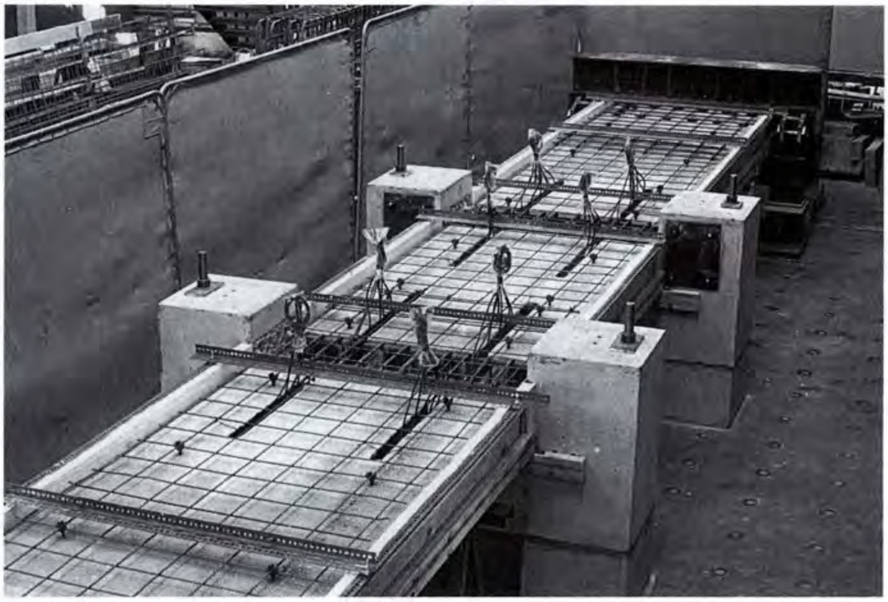
Fig. 9. Specimen 1 during construction.

One exterior support was a hinge mechanism mounted over rollers that was free to move horizontally. The other supports were precast concrete beams. All supports were seated on the laboratory floor. The vertical sides of the precast beams adjacent to the ends of the hollow-core units were relatively smooth, as in practice. Fig. 9 shows Specimen 1 during construction.

Each test specimen was subjected to two loading tests:

**Test A** — This test involved the right side span of the specimen only (see Fig. 8); the left end of this span had been constructed without bearing on the supporting beam. During construction, polystyrene pads were used to block out the voids that were not being used to anchor the two tie bars; the reason for this was to prevent concrete from penetrating inside those voids.

The polystyrene pads protruded out of the voids towards the supporting beam a distance of 20 mm (0.79 in.). In this way, only the shear-friction capacity provided by the topping slab and the two filled voids and their reinforcement was studied. In the test, the vertical load on that span was increased and the vertical load vs. vertical displacement relationship at the left end was measured.

**Test B** — This test involved the left side span of the specimen only (see Fig. 8). First, a horizontal force was applied to the left end support, forcing that span to slide horizontally until the right end of the hollow-core unit pulled off its interior support. This involved a 55 mm (2.2 in.) horizontal movement of that span resulting in a transverse crack of about that width in the topping slab at the right end of the span. This movement was imposed because it is equal to the total seating length of the floor in the direction of the span plus 5 mm (0.20 in.) additional movement to avoid contact between the end of the floor unit and the beam face during the vertical displacement to follow.

The vertical reaction of the floor at this support could only be provided by the kinking of the tie bars. Next, keeping the horizontal position of the unit constant, the vertical load on the span was successively increased and the vertical load vs. vertical displacement relationship at the right end was measured.

## Selection of Tie Size and Shape

**Diameter and Steel Grade** — According to the FIP,[5] the anchorage capacity of ties in filled voids of precast concrete hollow-core units at the stage of splitting the unit has not been satisfactorily examined. For 265 mm (10.4 in.) thick hollow-core units without a cast-in-place concrete topping slab, it is recommended that the yield load of the tie bars introduced at an end should not exceed 160 kN (36 kips) and that the yield load of tie bars introduced in each core at an end should be limited to 80 kN (18 kips).[5]

If the above criterion is applied to 200 mm (7.9 in.) thick hollow-core units with a 65 mm (2.6 in.) thick cast-in-place reinforced concrete topping slab, a good tie selection is two 16 mm (0.63 in.) diameter bars of Grade 300 steel [with characteristic yield strength of 300 MPa (43.5 ksi)]. This gives the maximum yield load for two tie bars.

An advantage of using Grade 300 steel instead of Grade 430 steel [with characteristic yield strength of 430 MPa (62.4 ksi)] in New Zealand is the larger ultimate elongation available from Grade 300 steel (20 percent for Grade 300 and 15 percent for Grade 430 are the New Zealand specified minimums[16]).

**Shape and Surface** — The shape of tie connection Type 1 (Fig. 7) is that suggested by the FIP and was constructed using plain round bars and beam support Type 1 (Fig. 1). The shape of tie connection Type 2 is that suggested in the New Zealand Guidelines,[3] and was constructed using deformed bars and beam support Type 2.

As an alternative, a proposed tie connection Type 3 was also constructed using plain round bars and beam support Type 2. The use of plain round bars enables bond failure to propagate along the tie bar, thus making large ultimate elongation possible and, therefore, increasing the energy absorption capacity of tie connection Types 1 and 3.

**Length** — For the selection of the tie length, two approaches were followed:

**1.** With regard to design against progressive collapse, the anchorage of the tie should be sufficient to resist the fracture load (rather than the yield load used in ordinary design). In this way, it is possible to take advantage of the force carried by the tie connection in the entire plastic range of the bar. The conventional straight anchorage length, $L_d$, of deformed tie bars as found in design standards, i.e., NZS 3101[1] or ACI 318,[2] should be increased to make this possible.

The additional length for deformed bars can be estimated using Table 2.2 of the FIP report.[5] This table indicates that for a 16 mm (0.63 in.) diameter Grade 300 bar [with characteristic yield strength of 300 MPa (43.5 ksi)], the additional length required is 256 mm (10 in.), considering a concrete strength of 36 MPa (5200 psi) and a ratio of tensile strength to yield strength, $f_{su}/f_y$, of 1.5 (the maximum value for that ratio permitted for Grade 300 steel in New Zealand[16]).

Therefore, the total anchorage length for a 16 mm (0.63 in.) diameter deformed tie bar of Grade 300 steel is the conventional length[1] plus the additional length required, or:

$$\begin{aligned} L_d &= 419 \text{ mm } (16.5 \text{ in.}) \\ &\quad + 256 \text{ mm } (10 \text{ in.}) \\ &= 675 \text{ mm } (26.5 \text{ in.}) \text{ or } 42d_b \end{aligned}$$

where $d_b$ is the bar diameter.

**2.** The end of the cast-in-place concrete within the filled voids will form a discontinuity in the hollow-core unit and create a potential plane of cracking. For that reason, the cast-in-place concrete fill and the tie bars should extend at least a distance equal to the transfer length of the prestressing strand from the end of the hollow-core unit.[5]

According to NZS 3101[1] and ACI 318,[2] the transfer length for 12.5 mm (0.5 in.) diameter seven-wire strand can be assumed to be 50 x 12.5 = 625 mm (24.6 in.). Libby[17] suggests that the transfer length of tendons released by cutting with an abrasive wheel can be expected to be 20 to 30 percent greater than that of tendons that are released gradually. Then the transfer length becomes 1.20 x 625 = 750 mm (29.5 in.).

In the test specimens, the tie connections were anchored over a length of 750 mm (29.5 in.) inside the filled voids of the hollow-core units (see Fig. 7).

## RESULTS FROM TESTS ON SPECIMEN 1 WITH TIE CONNECTION TYPE 1

### Test A of Specimen 1 With Tie Connection Type 1

**Test Results** — In Test A, the applied vertical load on the right side span was increased incrementally (see Test A in Fig. 8). The measured relationship between the applied vertical load and the vertical displacement at the left end of the hollow-core unit, which was the end without bearing length, is shown in Fig. 10.

The first crack (induced by negative moment) was observed on the top surface of the topping slab above the end of the hollow-core unit at an applied vertical load of 165 kN (37.1 kips) and a vertical displacement of 0.1 mm (0.004 in.). The maximum applied vertical load reached was 374 kN (84.1 kips) at a vertical displacement of 2.2 mm (0.087 in.). At this stage, the end of the topping slab separated from the precast unit (see Fig. 11).

The vertical reaction at the face of the support can be calculated by statics from the applied vertical load and the geometry of the loading shown in Fig. 8. When the applied load was a maximum, the end reaction had a value of:

$$374 \times 2725/3250 = 313.6 \text{ kN } (70.5 \text{ kips})$$

from the applied load plus 7.5 kN (1.7 kips) from the dead load of the hollow-core unit with topping. This gives a total of 321 kN (72.2 kips) if the floor is considered to be simply supported, or a slightly higher value (3 percent greater) if the negative moment flexural strength at the support due to the mesh and ties is considered.

**Theoretical Ultimate Shear Force** — The theoretical ultimate vertical shear strength of this connection can be estimated using the shear-friction concept. The critical vertical crack will cross monolithic concrete in the cast-in-place topping slab and in the filled voids at the end of the precast unit. In Eq. (1), $\mu = 1.4$ and the clamping force across the crack is given by the sum of the forces provided by the steel mesh and the ties acting normal to the vertical crack.

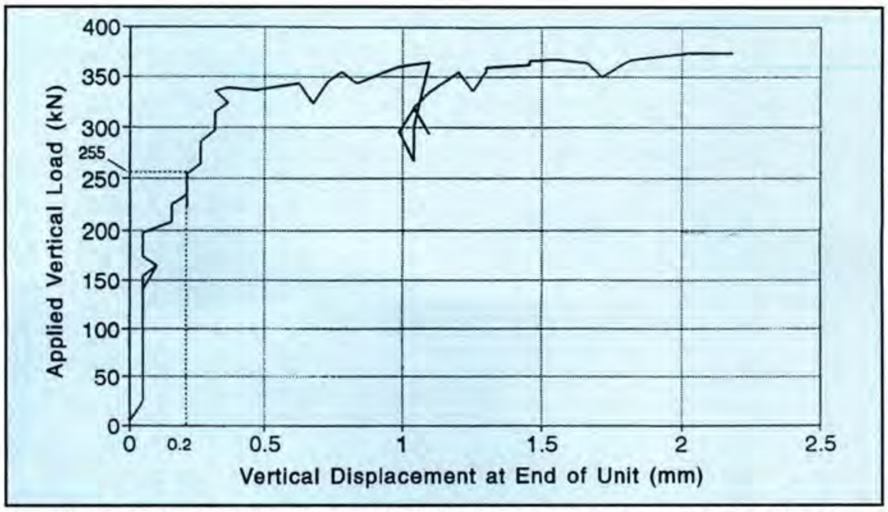


Fig. 10. Applied vertical load vs. vertical displacement measured at end of unit during Test A on connection Type 1 (1 kN = 0.225 kips; 1 mm = 0.0394 in.).

For the mesh:

$A_{vf}$ = 176 mm² (0.27 sq in.)

$f_y$ = 551 MPa (79.9 ksi), but 415 MPa (60.0 ksi) is the maximum allowed in shear-friction calculations[2]

$\theta_i$ = 0 degrees

For the ties:

$A_{vf}$ = 402 mm² (0.62 sq in.)

$f_y$ = 317 MPa (46.0 ksi)

$\theta_i$ = 0 degrees

Therefore, from Eq. (1):

$$V_n = (176 \times 415 + 402 \times 317) \times 1.4 = 281 \text{ kN } (63.2 \text{ kips})$$

This theoretical value can be compared with the maximum vertical end reaction of 321 kN (72.2 kips) obtained from the test.


Fig. 11. Connection Type 1 at end of Test A (side view of end of unit).

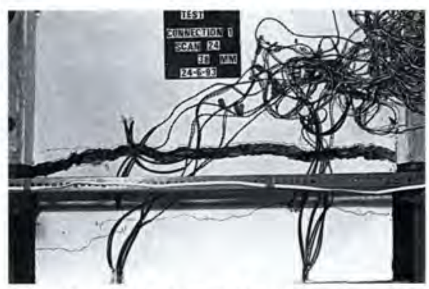

Fig. 12. Crack formed in the topping slab during the horizontal movement of Test B of tie connection Type 1 (top view of slab).

### Test B of Specimen 1 With Tie Connection Type 1

**Test Results** — In the first stage of the test, the left side span was pulled longitudinally by a horizontal tensile load (Test B in Fig. 8). During this loading, the vertical joint between the faces of the hollow-core unit and the supporting beam at the right end of the span opened up as the hollow-core unit slid horizontally. The total horizontal tensile load applied reached a maximum value of 380 kN (85.4 kips) when the width of the vertical crack there was 1.5 mm (0.06 in.). By the end of the horizontal movement of 55 mm (2.2 in.), all the wires of the welded wire mesh in the topping slab that crossed the crack had fractured and the applied tensile load had diminished to 164 kN (36.9 kips). The opening of the crack in the topping slab at the face of the hollow-core unit is shown in Fig. 12.

The readings of the strain gauges on the tie bars at this stage showed that bond failure had propagated along the whole straight length of the tie bars. The measured strains were larger than 2.5 percent, which meant that the bars were yielding and were in the strain hardening region. Hence, the anchor-

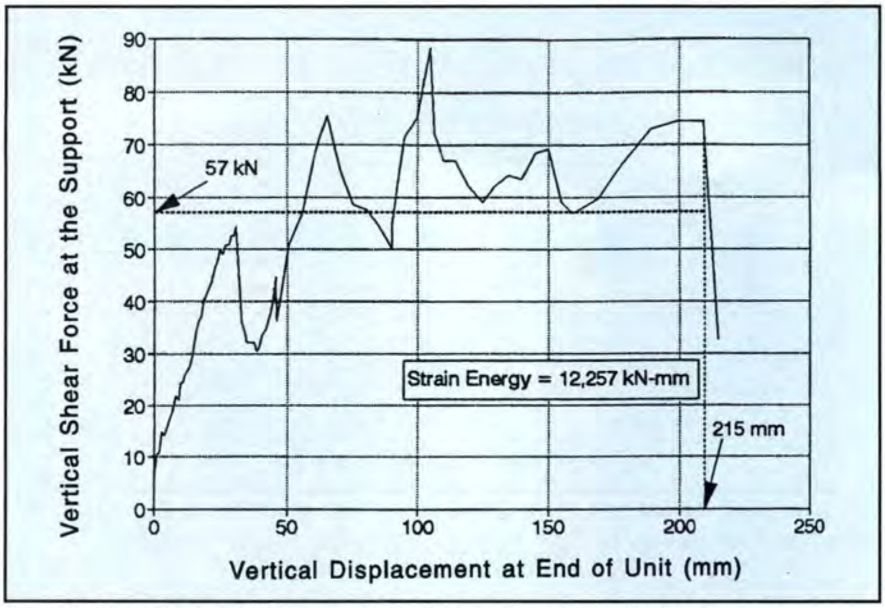


Fig. 13. Vertical shear force vs. vertical displacement at end of unit measured during Test B on connection Type 1 (1 kN = 0.225 kips; 1 mm = 0.0394 in.).


Fig. 14a. The end of Test B on connection Type 1; failure region.

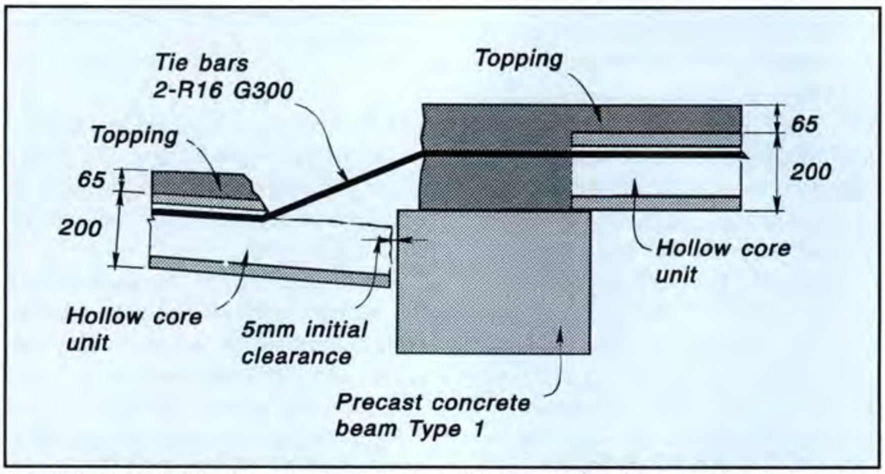


Fig. 14b. The end of Test B on connection Type 1; mode of failure (dimensions in mm; 1 mm = 0.0394 in.).

age of the tie bars was mainly provided by the end hooks.

In the second stage of the test, the right end of the left side span was forced downward by an applied vertical load that was increased incrementally (see Test B in Fig. 8). The applied vertical load was equilibrated by the vertical component of the tensile force in the kinked tie bars. That is, the hollow-core unit was hanging from the ties, without friction at the supporting beam, because the end of the hollow-core unit was 5 mm (0.2 in.) clear of the supporting beam due to the horizontal displacement applied in the first stage of the test. Fig. 13 shows a plot of the vertical shear force, calculated by statics from the applied vertical load and the geometry of the loading vs. the vertical displacement measured during the test.

Failure eventually occurred due to horizontal splitting cracks in the units, and the ties tore out as illustrated in Fig. 14. However, as can be seen in Fig. 13, the behavior of the connection up to that stage was quite ductile.

The area under the curve of Fig. 13, up to the point at which the ties fractured at a vertical displacement of 215 mm (8.5 in.), is 12257 kN-mm (108.5 kip-in.). Therefore, the average vertical shear force resisted over this range of vertical displacement is 12257/215 = 57 kN (12.8 kips).

Hence, during dynamic loading a vertical shear force of 57 kN (12.8 kips) can be transferred at the support by the tie connection up to a vertical displacement of 215 mm (8.5 in.). If the shear force is applied slowly (statically) to the ties, it could be argued that a vertical shear force of almost 90 kN (20.2 kips) could be transferred (see Fig. 13). It is evident that in a dynamic situation, the input work done by the dropping weight of a floor should not exceed the strain energy capacity of the ties if the floor is to be supported.

**Theoretical Ultimate Shear Force** — The magnitude of the vertical shear force carried by the tie connection in this test implies considerable kinking of the tie bars. At the peak static shear force carried of 88 kN (19.8 kips), the strain measured on the ties by the electrical resistance strain gauges was

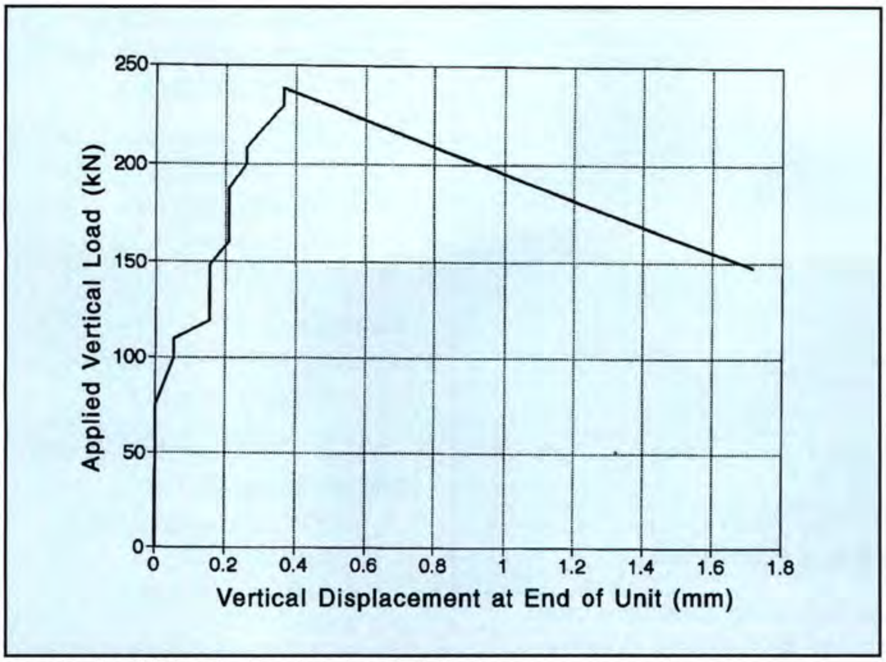


Fig. 15. Applied vertical load vs. vertical displacement at end of unit measured during Test A on connection Type 2 (1 kN = 0.225 kips; 1 mm = 0.0394 in.).

Fig. 16. The end of Test A on connection Type 2 (side view of end of unit).

0.05, which corresponded to a stress of $f_s$ = 440 MPa (63.8 ksi). This was greater than the measured yield strength of 317 MPa (46.0 ksi) due to the steel entering the strain hardening range. Also, $A_{vf}$ = 402 mm² (0.62 sq in.). Therefore, Eq. (2) gives:

$$\sin \theta_t = \frac{88,000}{402 \times 440} = 0.498$$

resulting in $\theta_t$ = 30 degrees.

Note that $\theta_t = \theta_k$ in Fig. 6b because $\theta_i = 0$. On this basis, the estimated angle of kinking of the tie bar at the maximum shear force was $\theta_k$ = 30 degrees.

## RESULTS FROM TESTS ON SPECIMEN 2 WITH TIE CONNECTION TYPE 2

### Test A of Specimen 2 With Tie Connection Type 2

**Test Results** — Fig. 15 shows the measured relationship between the applied vertical load and the vertical displacement at the end of the unit. The maximum applied vertical load reached was 238 kN (53.5 kips) at a vertical displacement of 0.4 mm (0.016 in.) when the crack width at the top of the topping slab above the end of the unit was 0.4 mm (0.016 in.).

This maximum applied load and the dead load of the floor unit plus topping resulted in a vertical reaction of 207 kN (46.5 kips) at the face of the support if the span is considered to be simply supported. At this load the electrical resistance strain gauges showed that the tie bars had not reached the yield strength. Fig. 16 shows that, at the end of the test, the splitting cracks had propagated along the filled voids that anchored the tie bars and the adjacent webs of the hollow-core unit. These splitting cracks were due to the transverse forces induced by the 45 degree bends in the tie bars.

**Theoretical Ultimate Shear Force** — The magnitude of the vertical shear force carried by the tie connection in

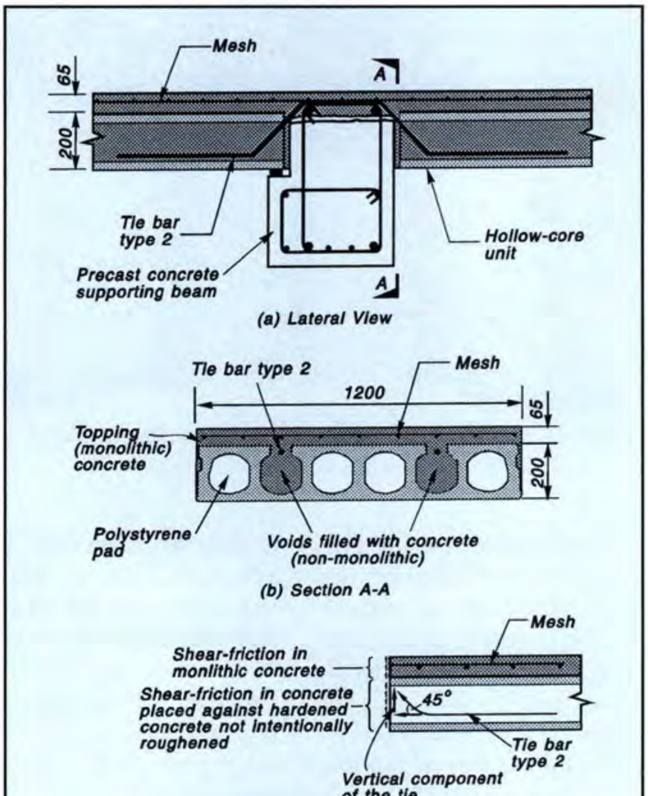


Fig. 17. Transfer of end reaction by shear friction during Test A of connection Type 2 (dimensions in mm; 1 mm = 0.0394 in.).

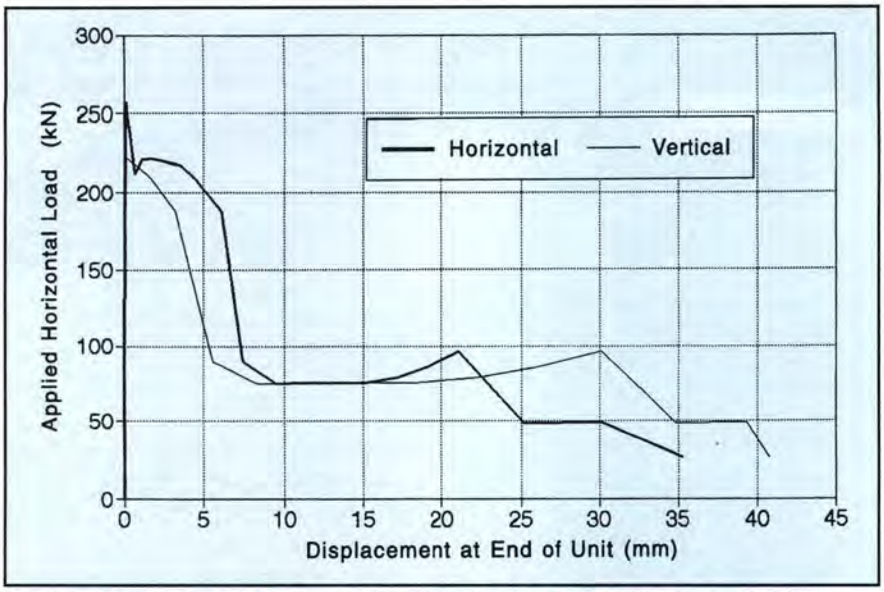


Fig. 18. Applied horizontal load vs. horizontal and vertical displacements at end of unit measured during the first stage of Test B on connection Type 2 (1 kN = 0.225 kips; 1 mm = 0.0394 in.).


Fig. 19. The end of Test B on connection Type 2 (top view of end of unit).

this test can be calculated using Eq. (1), which sums the vertical forces resisted by shear friction and the vertical component of the forces in the ties crossing the critical crack at the end of the hollow-core unit.

The value of the shear-friction coefficient, $\mu$, to be used in this calculation for Specimen 2 is debatable. In Specimen 1, the cast-in-place concrete was placed in the topping slab and over the supporting beam across the full depth of the end of the hollow-core units, and the critical vertical crack crossed monolithic concrete. However, in Specimen 2, the cast-in-place concrete in the ends was placed in the topping slab and in the two broken back voids of the hollow-core units (see Fig. 17).

For the cast-in-place topping slab (concrete placed monolithically), $\mu$ = 1.4 is appropriate.[1,2] The cast-in-place concrete below the topping slab in Specimen 2 was placed against the hardened concrete of the supporting beam, which was not intentionally roughened, and, therefore, $\mu$ = 0.6 is appropriate[2] for that concrete. On average, a value of $\mu$ = 1.0 might be reasonable and is used in the following calculation for Specimen 2.

For the mesh:

$A_{vf}$ = 176 mm² (0.27 sq in.)

$f_y$ = 415 MPa (60.0 ksi) — the maximum allowed[2] in Eq. (1)

$\theta_i$ = 0 degrees

For the ties:

$A_{vf}$ = 402 mm² (0.62 sq in.)

$f_y$ = 310 MPa (45.0 ksi)

$\theta_i$ = 45 degrees

Therefore, from Eq. (1):

$$V_n = (176 \times 415 + 402 \times 310 \times \cos 45^\circ) \times 1.0 + (402 \times 310 \times \sin 45^\circ)$$
$$= 161.1 + 88.1$$
$$= 249 \text{ kN } (56.0 \text{ kips})$$

The above theoretical value was not attained during the test. This was because splitting failure of the concrete occurred for this connection type before the tie reinforcement reached the yield strength, as shown by the measured strains on the ties.

## Test B of Specimen 2 With Tie Connection Type 2

**Test Results** — During the imposed horizontal movement of the first stage of Test B, the unit lifted due to the vertical component of force in the ties. Fig. 18 shows the measured relationships between the applied horizontal load and both the horizontal and vertical displacements. The welded wire mesh in the topping concrete fractured when the horizontal movement was about 5 mm (0.2 in.). The tie bars continued to carry horizontal load but the unit failed before the preselected horizontal movement of 55 mm (2.2 in.) was reached.

The maximum horizontal load resisted was 260 kN (58.5 kips). Again, the failure was due to splitting cracks in the concrete as a result of the transverse forces induced by the 45 degree bends in the tie bars (see Fig. 19). The second stage of Test B, with vertical load applied, was not conducted.

**Theoretical Ultimate Shear Force** — This calculation was not made because the vertical load test was not conducted.

## RESULTS FROM TESTS ON SPECIMEN 3 WITH TIE CONNECTION TYPE 3

### Test A of Specimen 3 With Tie Connection Type 3

**Test Results** — In this test, the vertical load was applied at a distance of 906 mm (35.7 in.) from the face of the supporting beam, instead of the 525 mm (20.7 in.) used for the two previous specimens, in order to apply the vertical force outside of the anchorage region of the tie bars.

Fig. 20 shows the measured relationship between the applied vertical load and the vertical displacement at the end of the unit. The maximum applied load reached was 309 kN (69.5 kips) at a vertical displacement of 0.4 mm (0.016 in.), when the crack width at the top of the topping slab above the end of the unit exceeded 0.4 mm (0.016 in.). This maximum applied load and the dead load of the floor unit plus topping resulted in a vertical reaction of 230 kN (51.7 kips), if the span is considered to be simply supported. The connection failed when diagonal tension cracks propagated, as shown in Fig. 21.

**Theoretical Ultimate Shear Force** — The shear-friction strength for this test can be calculated using Eq. (1) as for connection Type 2, the only difference being that the ties were inclined at 13 degrees to the horizontal in connection Type 3 and the yield strength of the ties was 317 MPa (46.0 ksi). Again, $\mu = 1.0$ is assumed, as for connection Type 2.

Therefore, from Eq. (1):

$$\begin{aligned} V_n &= (176 \times 415 + 402 \times 317 \times \cos 13°) \times 1.0 + (402 \times 317 \times \sin 13°) \\ &= 197.2 + 28.7 \\ &= 226 \text{ kN } (50.8 \text{ kips}) \end{aligned}$$

This predicted value compares well with the maximum vertical end reaction of 230 kN (51.7 kips) obtained from the test.

### Test B of Specimen 3 With Tie Connection Type 3

**Test Results** — The measured relationship between the applied horizontal load and the horizontal displacement at the end of the unit is shown in Fig. 22. As for the other specimens, the crack in the topping slab developed over the end of the unit. The maximum horizontal load resisted was 235 kN (52.8 kips) at a 5 mm (0.2 in.) horizontal movement just before the welded wire mesh fractured. The horizontal load capacity was maintained quite well with further horizontal movement. Some vertical lifting of the end of the unit was observed.

In the second stage of the test, with

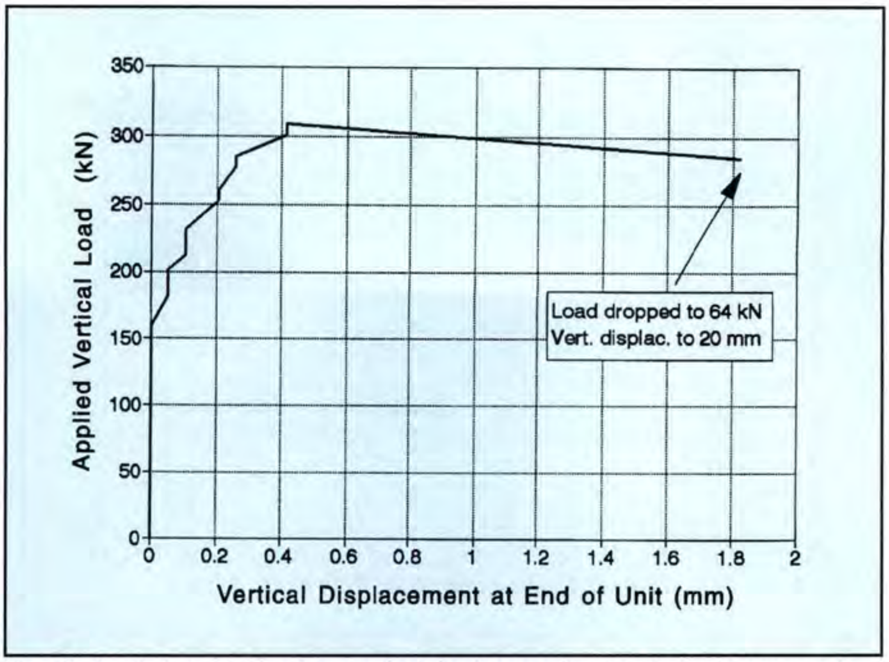


Fig. 20. Applied vertical load vs. vertical displacements at end of unit measured during Test A on connection Type 3 (1 kN = 0.225 kips; 1 mm = 0.0394 in.).


Fig. 21. The end of Test A on connection Type 3 (view of side of end of unit).

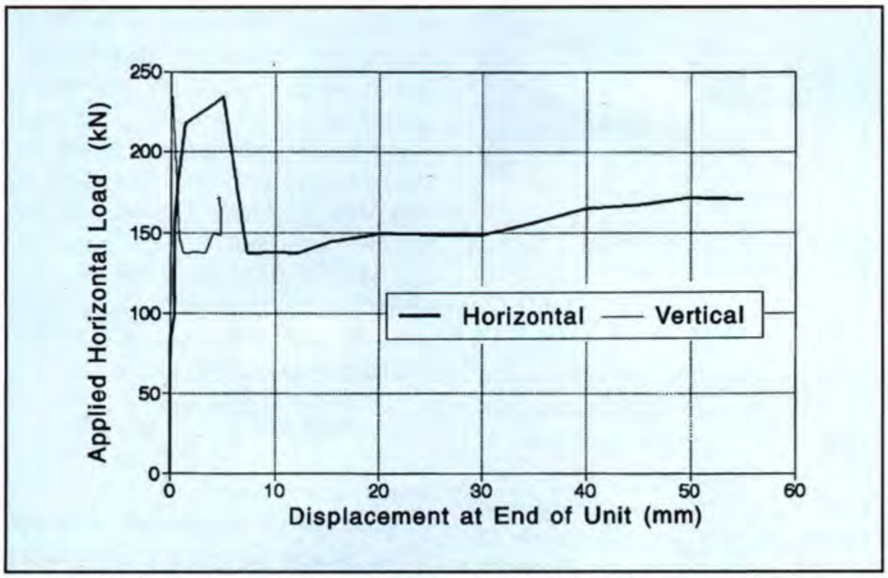


Fig. 22. Applied horizontal load vs. horizontal and vertical displacements at end of unit measured during the first stage of Test B on connection Type 3 (1 kN = 0.225 kips; 1 mm = 0.0394 in.).

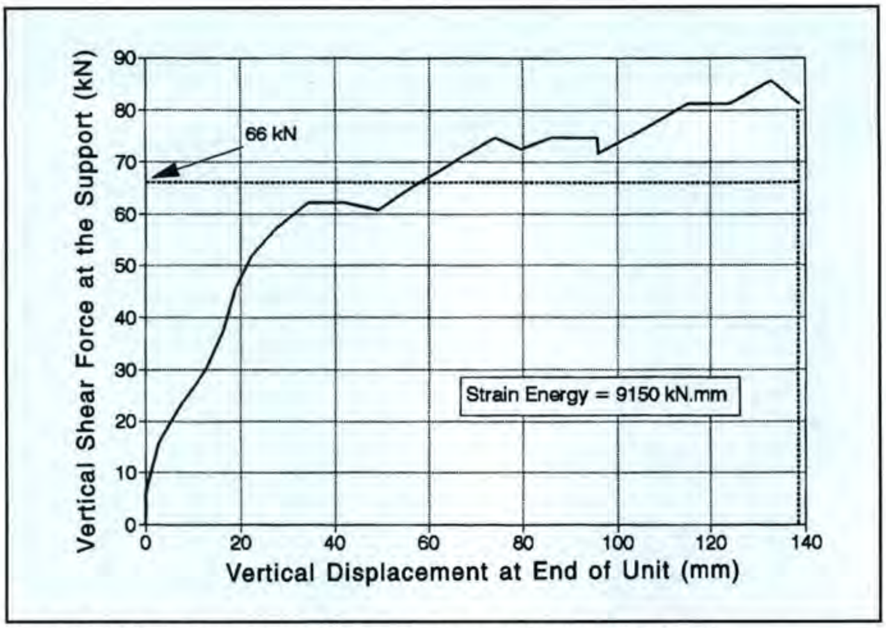


Fig. 23. Vertical shear force at the support vs. vertical displacement at the end of unit measured during Test B on connection Type 3 (1 kN = 0.225 kips; 1 mm = 0.0394 in.).

(a) Top view of end of unit


(b) Side view of end of unit

Fig. 24. The end of Test B on connection Type 3.

vertical load applied, the behavior of the connection was similar to the connection Type 1, and the same mode of failure as depicted in Fig. 14 was observed. The maximum applied load corresponded to a vertical reaction of 85 kN (19.1 kips) if the span is considered to be simply supported. The test ended when one of the ties fractured at an applied vertical load of 106 kN (23.8 kips) and at a vertical displacement of 139 mm (5.5 in.). Fig. 23 shows the relationship between the calculated vertical shear force at the support of the unit vs. the vertical displacement at the end of the unit during the test. Fig. 24 shows the specimen at the end of the test.

The mean vertical shear force resisted over the range of vertical displacement in Fig. 23 is 66 kN (14.8 kips), compared with 57 kN (12.8 kips) for connection Type 1 (see Fig. 13).

**Theoretical Ultimate Shear Force** — Again, the magnitude of the shear force carried implied considerable kinking of the tie bars. At the peak static shear force carried of 85 kN (19.1 kips) (see Fig. 23), the strain measured on the tie bars corresponded to a stress of 464 MPa (67.3 ksi). This was greater than the measured yield strength of 317 MPa (46.0 ksi) due to the steel entering the strain hardening range. Also, $A_s = 402$ mm$^2$ (0.62 sq in.).

Therefore, Eq. (2) gives:

$$\sin\theta_t = \frac{85{,}000}{402 \times 464} = 0.456$$

Hence, $\theta_t = 27$ degrees.

Since the tie bar initially was inclined at $\theta_i = 13$ degrees to the horizontal, the kinking of the bar caused a further deviation angle of:

$$\theta_k = 27 - 13 = 14 \text{ degrees}$$

# DISCUSSION OF TEST RESULTS

## Capacity of the Three Types of Tie Connection

**Test A** — In Test A, the vertical load was applied to the connection without horizontal displacement but with zero seating under the end of the precast concrete unit. Table 1 summarizes the results of Test A for the three types of connection at the serviceability and ultimate limit states.

The serviceability limit state is reached when the vertical displacement of the precast unit at the support is 0.2 mm (0.008 in.), a criterion that has also been suggested by Tsoukantas and Tassios.[18] At a vertical shear slip of 0.2 mm (0.008 in.), no harmful cracks will appear at the connection. The ultimate limit state is reached when the vertical shear force resisted is a maximum. The shear forces in Table 1 are the vertical shear forces calculated at the face of support from the applied vertical load, assuming the floor units to be simply supported.

The maximum shear forces resisted in the tests for connection Types 2 and 3 were smaller than for connection Type 1, evidently because of the smaller depth of cast-in-place concrete over the precast supporting beams for those two connections (see Fig. 7) and because of the shape of the tie bars. In particular, the 45 degree bends in the tie bars of connection Type 2 resulted in splitting cracks in the concrete that caused anchorage failure of the tie bars.

Table 1 also lists the maximum shear forces predicted using the shear-friction concept. For the greater depth of cast-in-place concrete over the supporting beam (connection Type 1), a shear friction coefficient of $\mu = 1.4$ appears reasonable. But for the shallower depth of cast-in-place concrete over the supporting beam, use of a more cautious value of $\mu = 1.0$ is appropriate.

Those values of $\mu$ were used for calculating the maximum shear forces predicted that are listed in Table 1. The predicted values are in reasonable agreement with the test values for maximum shear force resisted except for connection Type 2, in which the tie bars had an anchorage failure.

Table 1. Summary of the vertical shears at the support in Test A (zero seating length and vertical load applied without horizontal displacement).

| Description | Connection | | |
|---|---|---|---|
| | Type 1 | Type 2 | Type 3 |
| **At serviceability limit state (at vertical displacement at connection of 0.2 mm)** | | | |
| Shear force resisted in test | 221 kN | 167 kN | 188 kN |
| **At ultimate limit state** | | | |
| Maximum shear force resisted in test | 321 kN | 207 kN | 230 kN |
| Maximum shear force predicted* | 281 kN | 249 kN | 226 kN |
| Maximum shear force resisted in test/ maximum shear force predicted | 1.14 | 0.83† | 1.02 |
| Vertical displacement at end of test | 2.2 mm | 1.7 mm | 1.8 mm |

**Note:** 1 mm = 0.0394 in.; 1 kN = 0.225 kips.

* Calculated using Eq. (1).

† Anchorage failure of the tie bars occurred due to splitting of the hollow-core unit.

Table 2. Summary of the vertical shears at the support in Test B (zero seating length and vertical load applied after 55 mm horizontal movement imposed).

| Description | Connection | | |
|---|---|---|---|
| | Type 1 | Type 2 | Type 3 |
| Maximum shear force resisted in test | 88 kN | – | 85 kN |
| Calculated angle of bar axis to horizontal $\theta_t$ due to initial deviation and kinking to obtain maximum shear force resisted in test* | 30 degrees | – | 27 degrees |
| Vertical displacement at end of test | 215 mm | – | 139 mm |

**Note:** 1 mm = 0.0394 in.; 1 kN = 0.225 kips.

* Calculated using Eq. (2).

**Test B —** In Test B, the vertical load was applied after the precast unit had been displaced horizontally by 55 mm (2.2 in.), so that the seating of the ends of the precast units was lost. Table 2 summarizes the results of Test B for the three types of connection. Connection Type 2 failed by splitting of the concrete of the hollow-core unit in the region that anchored the tie bars before the 55 mm (2.2 in.) horizontal displacement was reached, and hence vertical load was not applied.

The precast units with connection Types 1 and 3 were displaced horizontally by 55 mm (2.2 in.) without failure of ties, due to bond failure propagating along the plain round tie bars and the capacity of the hooked anchorages of the tie bars exceeding the fracture load of the ties. When connection Types 1 and 3 were subjected to vertical load, they resisted the vertical shear by kinking of the tie bars after being displaced horizontally. The maximum vertical shear forces resisted in the tests for connection Types 1 and 3 were equal to the vertical component of tie bar force for the tie bars inclined at 30 and 27 degrees to the horizontal, respectively, as shown in Table 2.

Connection Type 1 exhibited a greater vertical displacement at the end of the test than connection Type 3. This was because, for connection Type 1, the entire length of tie bar between the end hooks was straight; for connection Type 3 the tie bars had been bent over the supporting beam. The initially created regions of plastic strain in the tie bars where they were bent over the supporting beam eventually resulted in fracture of the tie bars there at a smaller vertical displacement than reached by connection Type 1.

## Example of Application of the Test Results

As an example of the application of the test results, consider a precast concrete hollow-core floor unit with a cast-in-place concrete topping slab and service loads as follows:

Service dead load:

- Hollow-core unit [1.2 m (47.2 in.) wide x 200 mm (7.9 in.) thick] plus topping slab [65 mm (2.6 in.) thick]
  = 4.6 kN/m (315 lb/ft)
- Partitions plus additional service dead loads:
  1.25 kPa x 1.2 m
  = 1.5 kN/m (103 lb/ft)

Therefore, $D$ = 6.1 kN/m (418 lb/ft)

Service live load:

- Office occupancy: 2.5 kPa ($52\ lb/ft^2$) x 1.2 m
- $L$ = 3.0 kN/m (206 lb/ft)

Total service load on a hollow-core unit: $D + L$ = 9.1 kN/m (624 lb/ft) Note: This total service load is 9.1/1.2 = 7.58 kPa ($158\ lb/ft^2$).

For the above service loads, the required ultimate load[2] of the hollow-core unit acting compositely is:

$$U = 1.4D + 1.7L$$
$$= (1.4 \times 6.1) + (1.7 \times 3.0)\ kN/m$$
$$= 13.6\ kN/m\ (934\ kips/ft)$$

For a one-way slab floor of span, $l$, in meters with these service and ultimate loads, in the event of loss of bearing at the supports, the tie connections at the ends of each hollow-core unit need to resist a vertical reaction of:

At service load:

$$R = \frac{9.1l}{2} = 4.55l\ \text{kN}$$

At ultimate load:

$$R = \frac{13.6l}{2} = 6.80l\ \text{kN}$$

For tie connection Types 1 and 3, the maximum static shear force resisted in Test A was at least 230 kN (51.7 kips). In Test B the maximum equivalent dynamic shear force that connection Types 1 and 3 resisted was at least 57 kN (12.8 kips). Note that the static load capacity is appropriate for the shear-friction strength because the vertical displacements are very small. However, the dynamic load capacity is more appropriate when the load is carried at large displacements by kinking of tie bars because dynamic loading is involved, due to the floor dropping. Applying a strength reduction factor $\phi$ = 0.85 to these values gives available dependable vertical reactions of:

For Test A:

$$R = 0.85 \times 230 = 195.5\ \text{kN}\ (44.0\ \text{kips})$$

For Test B:

$$R = 0.85 \times 57 = 48.5\ \text{kN}\ (10.9\ \text{kips})$$

Therefore, the maximum spans for which the ultimate load could be supported by the connection in the event of inadequate or even absent seating at the support (conditions of Test A, with the reaction provided by the shear-friction mechanism) is:

$$l = 195.5/6.8 = 28.8 \text{ m } (94.3 \text{ ft})$$

In the event of loss of bearing as a result of imposed severe horizontal movements caused by extreme conditions, the maximum span for which the service load could be supported by the connection by kinking of the tie reinforcement (Test B, with a large crack width) is:

$$l = 48.5/4.55 = 10.7 \text{ m } (35.0 \text{ ft})$$

The nominal flexural strength of this hollow-core section with a 65 mm (2.6 in.) thick cast-in-place concrete topping slab acting compositely is $M_n$ = 206 kN-m (1830 kip-in.)(see Appendix). Hence, with a strength reduction factor $\phi$ = 0.9, the maximum simply supported span for this ultimate load is given by $Ul^2/8 = \phi M_n$.

Therefore:

$$l = \sqrt{\frac{0.9 \times 206 \times 8}{13.6}}$$
$$= 10.4 \text{ m } (34.3 \text{ ft})$$

This value is less than the span for which the tie connection Types 1 and 3 can support the ultimate load in Test A and the service load in Test B. That is, if the end of the precast concrete unit undergoes loss of bearing without horizontal displacement, the floor would fail in flexure at midspan before the shear-friction strength was reached at the supports. Additionally, these tie connections can support the service load by kinking if the end of the precast concrete unit undergoes loss of bearing as a result of large horizontal displacements relative to the supporting beam. Therefore, the tie connections are safe.

## CONCLUSIONS

**1.** Special reinforcement, placed at the end supports of pretensioned precast concrete hollow-core floor units, can be used to prevent collapse of the floor units in the event of inadequate seating lengths or imposed movements due to volume changes or earthquakes.

**2.** For hollow-core precast concrete floor units of 1.2 m (47.2 in.) width, the special reinforcement can consist of two tie bars placed across the supporting beam and anchored in cast-in-place concrete, placed in broken back voids at the ends of the hollow-core units.

**3.** If the end seating of the precast hollow-core units is inadequate or lost, without horizontal movement of the units occurring, the vertical shear capacity of the floor at the support can be calculated from the shear friction transferred across the cracks in the cast-in-place concrete topping slab and in the two filled voids used to anchor the tie bars plus any vertical component of force in the tie bars. The slab mesh and the tie bars provide the clamping force for the shear-friction mechanism.

**4.** If the hollow-core units are pulled horizontally off their supports, losing their seating and opening up very large cracks in the cast-in-place topping slab, properly designed tie bars could still support the load of the floor by the vertical shear force transferred by kinking of the tie bars across the cracks.

**5.** Tie bars placed in filled voids of hollow-core units should be straight lengths of plain round bar of adequate length, terminated by standard end hooks. The lengths of tie bar in the filled voids should be straight because transverse forces acting on the concrete as a result of bends in tie bars can lead to longitudinal splitting of the webs of hollow-core units that lack vertical shear reinforcement. Plain round tie bars should be used rather than deformed bars because bond failure permits yielding to propagate along them, making large plastic elongations of the bars possible.

## RECOMMENDATIONS

The tests were conducted on pretensioned precast concrete hollow-core units with a cross section 200 mm deep and 1200 mm wide (7.9 and 47.2 in.). A 65 mm (2.6 in.) thick cast-in-place concrete topping slab, reinforced by welded wire mesh, was present over the units and continuous over the supporting beams at the ends. For such floors, it is recommended that:

**1.** A cautious value for the shear-friction coefficient, $\mu$, should be used when calculating the vertical shear capacity that can be transferred by shear friction across cracks in the topping slab and at the vertical face at the end of the precast unit in the event of loss of end bearing. A value of $\mu$ = 1.4 may be used if the top of the supporting beam is level with the bottom of the hollow-core unit (see support Type 1 in Fig. 1) so the depth of the cast-in-place concrete over the supporting beam is the full depth of the topping slab and the hollow-core unit.

However, the use of an average value of $\mu$ = 1.0 would appear to be more appropriate if the top of the supporting beam is level with the top of the hollow-core unit (see support Type 2 in Fig. 1) so the cast-in-place concrete is placed over the depth of the topping slab and against the sides of the hardened concrete of the supporting beam, which is not intentionally roughened. Note that the design load of the floor calculated from the vertical shear capacity at the supports due to shear friction cannot exceed that governed by the flexural strength or the shear (diagonal tension) strength of the floor.

**2.** Two plain round 16 mm (0.63 in.) diameter Grade 300 steel bars [characteristic yield strength of 300 MPa (43.5 ksi)] extending straight about 750 mm (29.5 in.) in cast-in-place concrete placed in two broken back voids at each end of the hollow-core unit, and terminated with standard end hooks, could be used to support the floor in the event of large displacements. The tests showed that these two tie bars (either Type 1 or Type 3 in Fig. 7) can support a vertical shear force of at least 85 kN (19.1 kips) when the unit is pulled horizontally about 55 mm (2.2 in.) and is then displaced vertically by 140 mm (5.5 in.). The tie bars should extend into the filled voids at least a distance equal to the transfer length of the pretensioned tendons in the precast concrete units.

## ACKNOWLEDGMENT

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## APPENDIX: PROPERTIES OF THE HOLLOW-CORE UNITS

Transformed section properties of hollow-core unit alone (see cross section in Fig. A):

- Moment of inertia of cross section = $6.07 \times 10^8$ mm$^4$ (1458 in.$^4$)
- Area of cross section = $1.179 \times 10^5$ mm$^2$ (183 sq in.)
- Distance of centroid from top fiber = 100 mm (3.94 in.)
- Concrete compressive cylinder strength = 40 MPa (5800 psi)

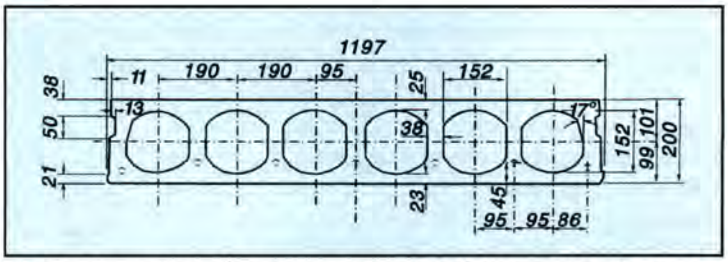


Fig. A. Dimensions of the precast concrete hollow-core units used in the tests (1 mm = 0.0394 in.).

Transformed section properties of hollow-core unit plus 65 mm (2.6 in.) thick topping slab:

- Moment of inertia of composite cross section = $1.321 \times 10^9$ mm$^4$ (3174 in.$^4$)
- Area of a cross section = $1.767 \times 10^5$ mm$^2$ (274 sq in.)
- Distance of centroid from top fiber = 125 mm (4.92 in.)
- Topping concrete compressive cylinder strength = 36 MPa (5.22 ksi), 27 MPa (3.92 ksi) and 29 MPa (4.21 ksi) for Specimens 1, 2 and 3, respectively

Prestressing steel: Five 12.5 mm (0.5 in.) diameter plus two 11 mm (0.43 in.) diameter pretensioned tendons:

- Area of prestressing steel = 607 mm$^2$ (0.941 sq in.)
- Elastic modulus, minimum = $190.2 \times 10^3$ MPa ($27.6 \times 10^6$ psi)
- Ultimate tensile strength = 1070 kN (241 kips)
- Prestressing force immediately after transfer = 805 kN (181 kips)