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RESEARCH REPORT

Precast Concrete Hollow-Core Floor Unit Support and Continuity

J. C. Mejia-McMaster and R. Park

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Department of Civil Engineering

University of Canterbury
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PRECAST CONCRETE HOLLOW-CORE FLOOR UNIT SUPPORT AND CONTINUITY

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JUAN C. MEJIA-McMASTER

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ABSTRACT

Three types of special tie reinforcement at the end supports of hollow-core precast concrete floor units with a cast-in-place topping slab were investigated. This tie reinforcement is intended to prevent collapse of the floors in the event of inadequate seating lengths or imposed movements due to volume changes or earthquake excitations. The tie reinforcement passed over the supporting beam and was anchored in two filled voids in the ends of the precast units. Two types of test were conducted.

In one test the vertical load was applied when the support seating was zero. In this case it was found that the shear capacity at the support can be calculated from the shear friction across the cracks in the cast-in-place concrete topping slab and in the two filled voids used to anchor the ties plus any vertical component of force in the tie bars, providing anchorage failure of the tie bars did not occur. A cautious value for the shear-friction coefficient μ to be used in calculating the available shear friction capacity is recommended.

In the other test the vertical load was applied after the precast floor unit had been pulled horizontally off the supporting beam. The maximum shear force which can be transferred by kinking of the tie reinforcement at the connection if the floor unit suddenly slides off or loses its support was evaluated using conditions of energy equilibrium. The tie reinforcement was shown to be capable of supporting at least the service gravity loads in these extreme conditions if it is well designed.

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NOTATION

β_1 = ratio of equivalent concrete stress block depth to neutral axis depth

δ = free end slip of the prestressing tendon

δ_i = vertical displacement at the connection at scan i

ϵ_{si} = strain in reinforcing steel i

ϵ_{yi} = yield strain of reinforcing steel i

θ_i = angle of the shear-friction reinforcement to the normal to the shear plane

θ_k = kinking angle of the bar to the normal of the crack

θ_t = angle of the kinked reinforcement crossing the crack to the normal of the crack

μ = coefficient of friction

σ_{sp} = stress in the prestressing steel (fully developed)

ρ_p = prestressing steel ratio

Ø = diameter of reinforcing bar or prestressing tendon

ø = strength reduction factor

a = depth of rectangular stress block

A_{si} = area of steel i for flexural strength calculations

A_{ps} = area of prestressing steel

A_{vf} = area of shear-friction reinforcement

c = neutral axis depth

C = internal compressive concrete force

d_t = tie bar diameter

D = service dead load

E_s = modulus of elasticity of reinforcing steel

f_{ps} = stress in the prestressing steel at the design load

f_{pu} = ultimate strength of prestressing steel

f_{su} = tensile strength of the tie bar

f_y = yield strength of steel

f'_c = concrete cylinder compressive strength

L_d = anchorage length of the tie bar

L_i = length of clear span i

L_t = transfer length of the prestressing force

L = service live load

$M_{a,i}$ = moment due to the eccentricity of support 1 with respect to the hinge mechanism, at scan i

M_i = negative bending moment at the face of support 3 at scan i

M_n = nominal flexural strength

M_u = ultimate bending moment

N = number of tie bars

P_i = applied vertical load from the ram at scan i

R = vertical reaction at the support in the event of loss of bearing

$R_{A,i}$ = vertical reaction at support 1 at scan i

T_i = tensile force provided by the kinking of the tie bars acting across the vertical crack

$T_{i,h}$ = horizontal component of the tensile force in the tie bars

T_s = tensile strength of the reinforcing bars

T_{si} = internal force at steel i for flexural strength calculations

U = ultimate load

$V_{d,i}$ = dynamic shear capacity of the connection up to scan i

$V_{d,max}$ = maximum shear force transferred during dynamic loading

V_i = vertical reaction at the face of support 3 at scan i

V_i = vertical component of the tensile force in the tie bars at scan i

V_{max} = maximum shear resisted for the connection

V_n = shear force transferred by shear-friction

V_n = shear force transferred by kinking across a vertical crack

V_s = shear strength at the serviceability limit state

W = uniformly distributed load

W_d = weight of the Dy-core unit plus the 65 mm thick concrete topping

W_{d1} = weight of the Dy-core unit plus the 65 mm thick concrete topping

W_{d2} = weight of the concrete used to fill the voids of the Dy-core unit at the left end of span 1

W_{ext} = external work done by the vertical reaction of the floor

$W_{int,i}$ = strain energy stored by the tie connection at scan i

$W_{int,max}$ = maximum strain energy stored by the tie connection

Chapter 1

INTRODUCTION

1.1 General

The construction of multi-storey buildings incorporating precast concrete elements is now common practice in New Zealand, as in many countries, while cast-in-place concrete construction is less utilized. Precast concrete floors have been widely studied but some aspects related with floor support and continuity in building structures subjected to seismic or accidental loads have not been fully considered by New Zealand or overseas design standards, which in spite of have extensive rules of the use of cast-in-place concrete structures do not consider all aspects of precast concrete structures.

In particular, tie connections which are used to transfer tensile forces between precast concrete floor units and the supporting beams, or between two continuous precast concrete floor units at both sides of the supporting beam, are not covered by current New Zealand design standards. The role of the tie connections should be to prevent collapse of precast concrete floors in the event of end bearing being lost as a result of bearing failure or of movements causing the precast floor units to be dislodged from the supports.

Reinforcing bars or more sophisticated arrangements from steel can be used for tie connections. Directly anchored tie connections can be embedded in the concrete when the precast floor is cast and protrude from the precast concrete. Alternatively indirectly anchored ties can be placed in voids of the precast units, or in joints between the units, and fixed with grout or cast-in-place joint concrete which in turn transfers the tie forces to the components by bond, friction and/or interlocking at interfaces.

1.2 The Importance of Ductile Tie Connections

Tie connections for precast concrete floor units with their supporting elements should be designed and constructed to have adequate strength, stiffness, and ductility to resist not only the gravity loads but also imposed movements, such as due to volume changes of structural members, or due to increase in span caused by elongation associated with plastic hinging of adjacent beams in a ductile frame.

Additionally, failure of precast floor systems, leading to collapse, can occur due to seating lengths in the direction of the span being too small as a result of tolerances not being met at the construction site. In such cases tie connections may provide the required additional strength, and be used as remedial technique.

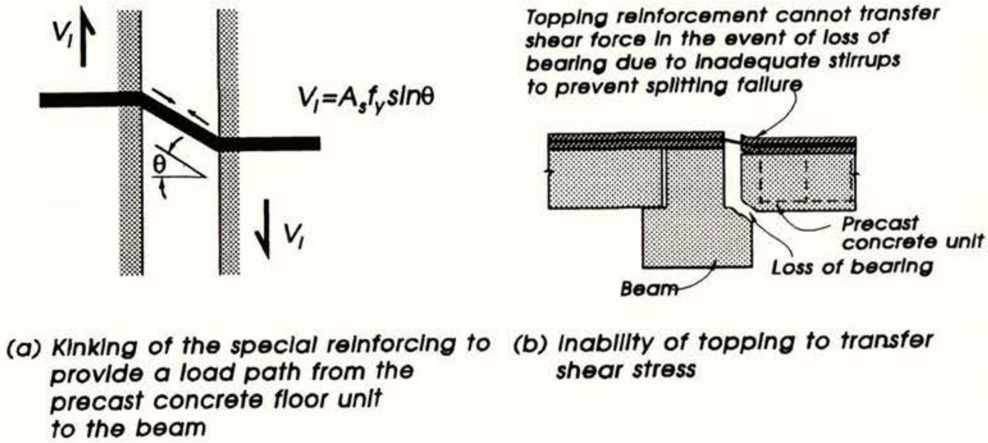


Fig. 1.1 Kinking of Reinforcement and Possible Failure of Topping if Bearing of Precast Unit is Lost According to the New Zealand Guidelines [1]

Special reinforcement for tie connections can be designed to prevent collapse as a result of loss of support of the precast floor units. The shear force carried by the kinking of reinforcement (see Fig. 1.1a) can often be utilized by the designer. However the imposed movements cannot be catered for by the kinking of conventional top reinforcement in cast-in-place concrete topping slabs since normally the connection between the topping slab and the precast unit is inadequate to prevent concrete splitting (see Fig. 1.1b)

Special reinforcement in the form of inclined hanger bars or saddle bars, as shown in Figs. 1.2 and 1.3, can be designed to carry the precast units in the event of failure of the supporting beam or lateral movement of precast units off the supporting beams [1]. Overseas practice including the use of tie or continuity reinforcement at supports as suggested by the

FIP [2] and the PCI [3] is illustrated in Figs. 1.4 and 1.5.

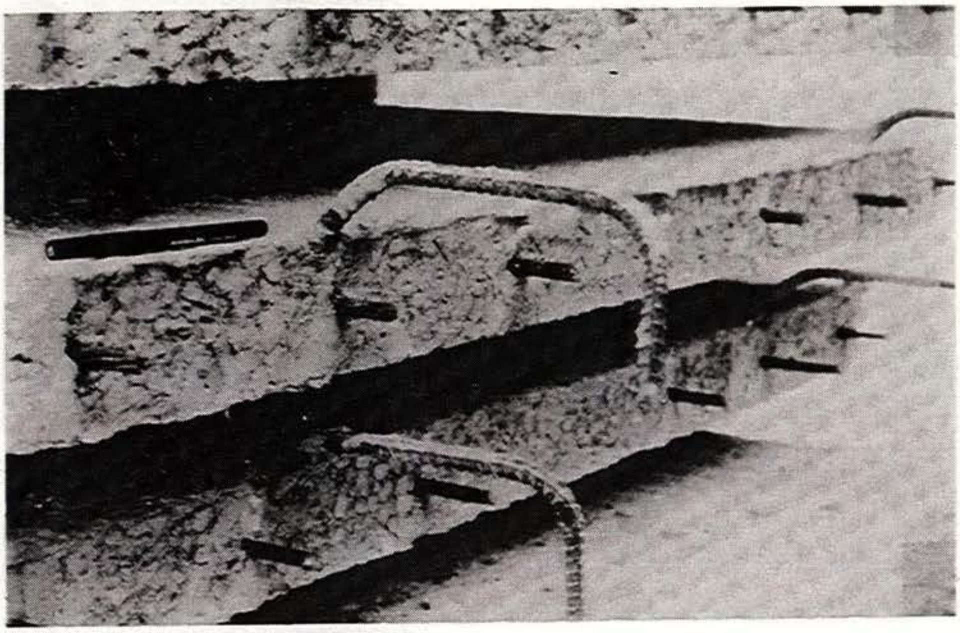


Fig. 1.2 Hanger Bars for Precast Concrete Floor Units According to the New Zealand Guidelines [1]

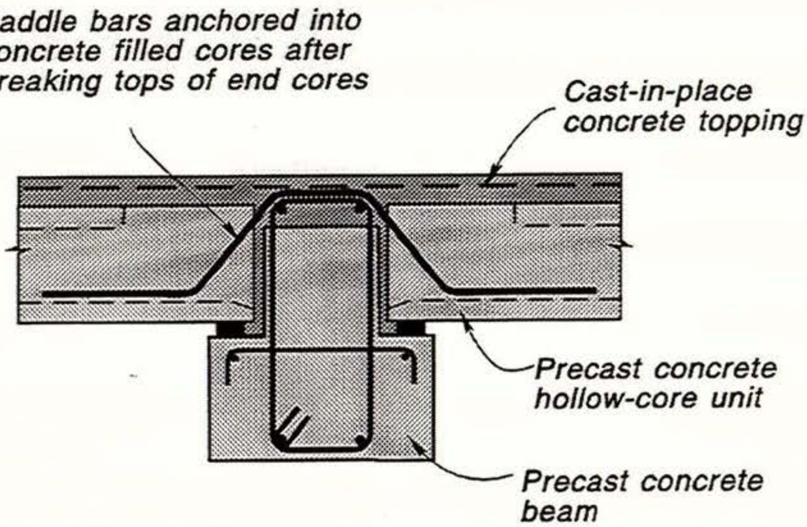


Fig. 1.3 Saddle Bars for Precast Hollow-Core Units According to the New Zealand Guidelines [1]

FIP [2] and the PCI [3] is illustrated in Figs. 1.4 and 1.5.

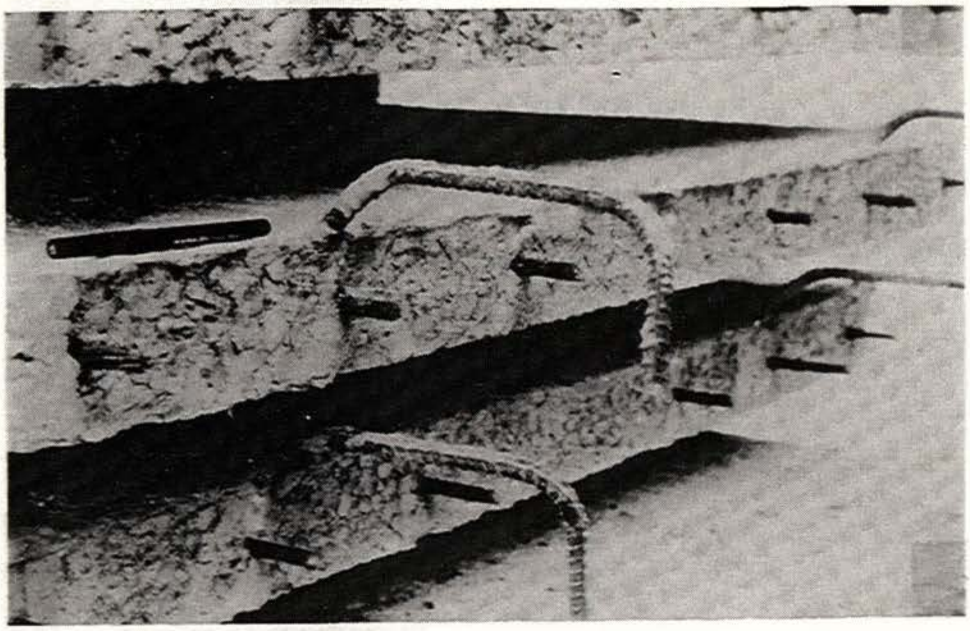


Fig. 1.2 Hanger Bars for Precast Concrete Floor Units According to the New Zealand Guidelines [1]

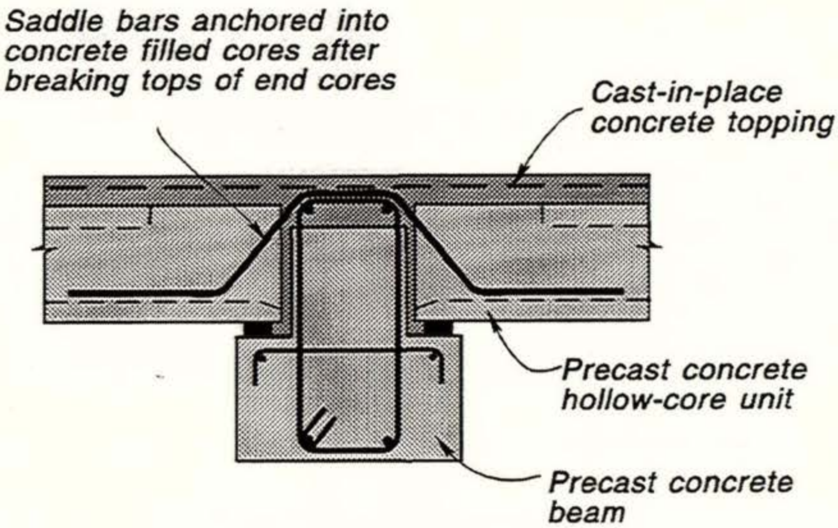


Fig. 1.3 Saddle Bars for Precast Hollow-Core Units According to the New Zealand Guidelines [1]

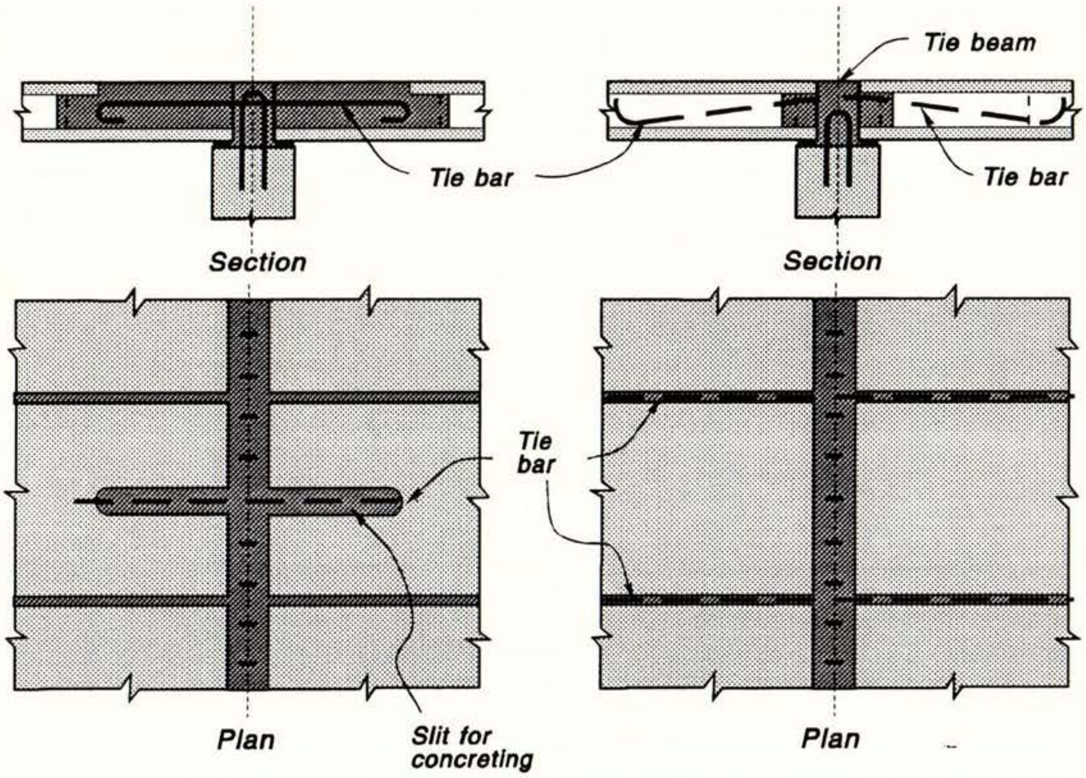


Fig. 1.4 Details of Tie or Continuity Reinforcement Suggested by the FIP [2]

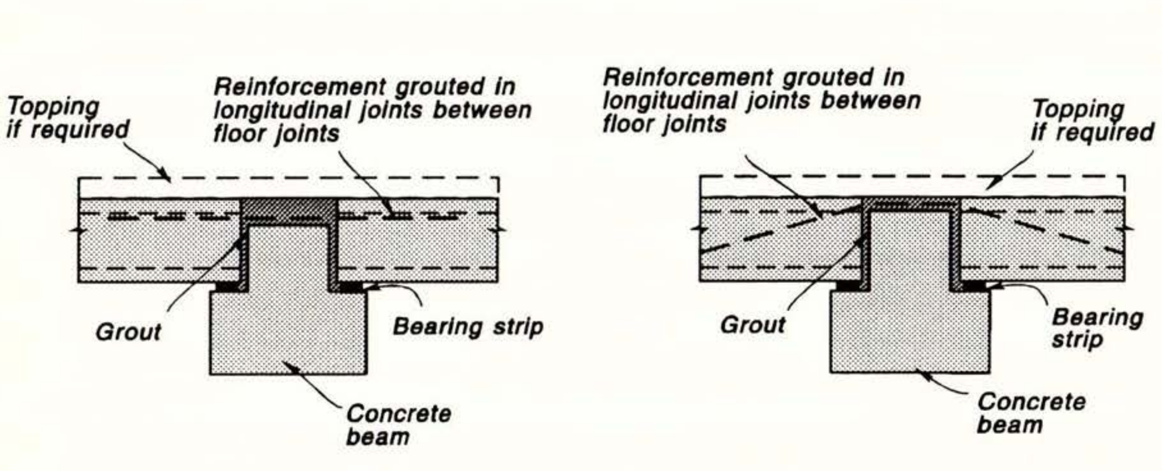


Fig. 1.5 Details of Tie or Continuity Reinforcement Suggested by the PCI [3]

It is also important that the tie connection provides certain integrity and robustness to the structure, so it can remain stable in case of accidental loading due to explosion, impact, fire or other extreme actions. If the structural system is damaged by accidental or seismic loads, the consequences should be limited to an appropriate level [4].

If the tie connections have high ductility and high energy absorption capacity, large relative displacements can be reached without loss of structural integrity, and substitutive load-carrying configurations can be developed, for example by kinking of the ties, catenary or membrane action, or cantilever action. This will permit the precast concrete floor units to remain supported upon the loss of bearing, thereby eliminating the danger of a progressive collapse caused by floor units falling onto another.

1.3 The Aim of This Research

The aim of this research is to study the behaviour of three different types of indirectly anchored tie connections in strips of precast hollow-core floors (commercially available Dy-core units), when the seating length is inadequate or when subjected to large horizontal displacements, under two different test conditions:

- Test A : The floor unit is constructed with the precast hollow-core unit having zero seating length and connected to the supporting beam by the tie connection and the cast-in-place reinforced concrete topping slab. External vertical load is applied and the shear strength and the vertical load-displacement relationship of the tie connection is measured.

- Test B : First the floor unit is pulled horizontally from its support until the support is lost. If the tie connection displays a ductile behaviour, the floor unit is held suspended by the ties. External vertical load is then applied and the vertical load-displacement relationship of its remaining shear capacity is measured.

The maximum shear force which can be transferred by the tie connection if the floor unit suddenly slides off or loses its support is evaluated using conditions of energy equilibrium. In a real situation a constant shear force acts upon the connection while the deflection occurs instead of the incrementally increased shear force used in the tests.

Important parameters such as anchorage capacity, likelihood of brittle failure, ductility and energy absorption capacity of the tie connections are observed during the tests and simple design and calculation methods based on the test results can be suggested.

1.4 Review of Previous Research

Previous research on precast concrete unit support has been conducted at Central Laboratories, Lower Hutt [5,6], and Chalmers University of Technology, Gotenburg, Sweden [7,8].

Yap [5] at Central Laboratories conducted ultimate load test on 200 mm-thick Dy-core units with two different end bearing lengths in the direction of the span (nominally 40 and 75 mm) and three support conditions (topping concrete continuous over two adjacent units and supporting beam, topping concrete over simply supported unit, and topping concrete over fixed end unit). The load tests showed that variations of bearing lengths between 37.5 and 60 mm had no influence on the shear strength of Dy-core units.

Blades et al [6] at Central Laboratories conducted ultimate load tests on Dy-core units with 65 mm thick topping concrete to investigate the effects of various seating conditions. The conclusions reached were:

(a) End bearing lengths in the direction of the span as little 5 mm are satisfactory for continuous supports providing that both 665 welded steel mesh in the cast-in-place topping concrete and saddle bars anchored in cast-in-place concrete placed in voids in the precast units obtained by breaking back the top flange of the end of the unit, both continuous over the support, are present.

(b) The shear capacity of the hollow-core unit can be improved significantly by breaking back the top flange of the end of the unit over a 300 mm length, placing D12 hair pin bars in each void and filling the broken back voids with concrete during the topping pour.

(c) Torsional rotation of an exterior beam on which the units are supported markedly reduces the shear strength if the seating length in the direction of the span is as low as 5 mm. This can be overcome to some extend by including hairpin bars in the filled voids, but it is recommended that short seating lengths be restricted to interior continuous supports.

Engstrom [7,8] in Sweden has conducted several tests to study the basic behaviour 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 behaviour and the type of failure were recorded. The conclusions reached were:

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

(b) Plain round tie bars should normally 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 normally be provided with end hooks.

(c) Large tie extensions in the plastic range can be obtained if bond failure propagates along the bar thus permitting an increase in length of bar reaching the yield strength. Hence the possible elongation of a tie bar can be increased by any method that increases the length of the region of bond failure.

(d) The tensile behaviour of the connections between precast floor units seems not to be affected by the dynamic action which follows the sudden release of the support. Thus in analysis, the behaviour of floor connections can be characterized by load-displacement relationships which were derived from tests with static loading.

Chapter 2

SELECTION AND DESIGN OF THE TIE CONNECTIONS TESTED

2.1 General

In this research project three different types of tie connections between precast hollow-core floor units of Dy-core type (200 mm thick and 1200 mm wide, with a 65 mm thick cast-in-place reinforced concrete topping slab) and their supporting precast concrete beams were investigated.

Owing to the extrusion method of manufacturing prestressed concrete hollow-core units, directly anchored tie bars protruding from the joint faces are normally avoided or not even possible. For that reason, the tie bars are placed and indirectly anchored in voids of the Dy-core units and filled with cast-in-place concrete during the topping pour. The ducts or voids used for the anchorage of the tie bars are opened by breaking back the top flange at the ends of the Dy-core units.

2.2 Selection of the Types of Support

The New Zealand Guidelines [1] identifies the three basic types of support for precast floor units shown in Fig. 2.1. Two types of support were selected to be used in combination with three different types of tie connection: beam support Type 1 with tie connection Type 1, and beam support Type 2 with tie connections Types 2 and 3 (see Fig. 2.2). These three types of tie connection, tested in this research programme, all have two 16 mm diameter bars of Grade 300 steel.

2.3 Selection of Diameter and Steel Grade of the Tie Bars

According to the FIP [2], the anchorage capacity of ties at the stage of splitting of

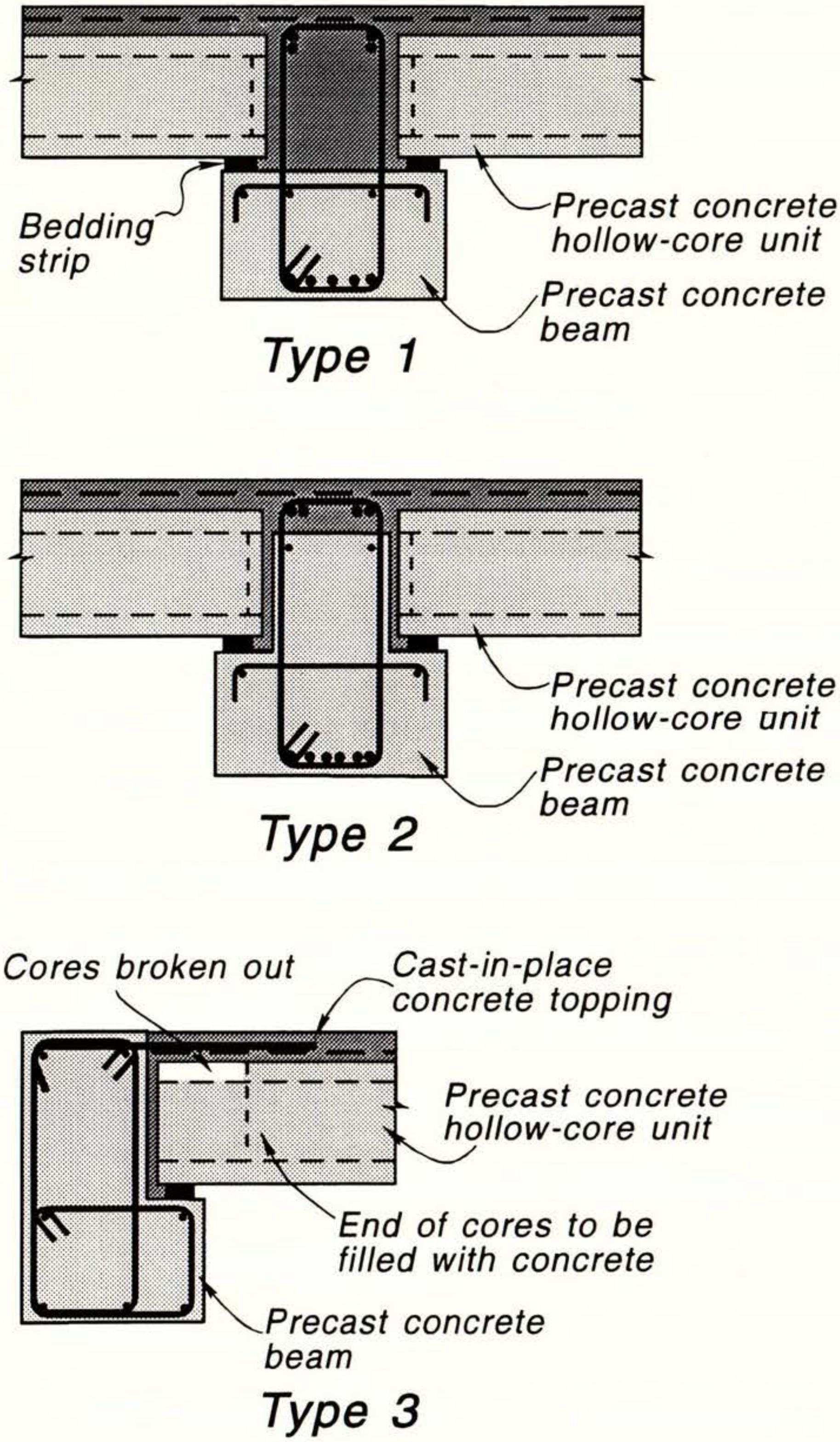


Fig. 2.1 Types of Support of Precast Concrete Hollow-Core Floor Units by Precast Concrete Beams According to the New Zealand Guidelines [1]

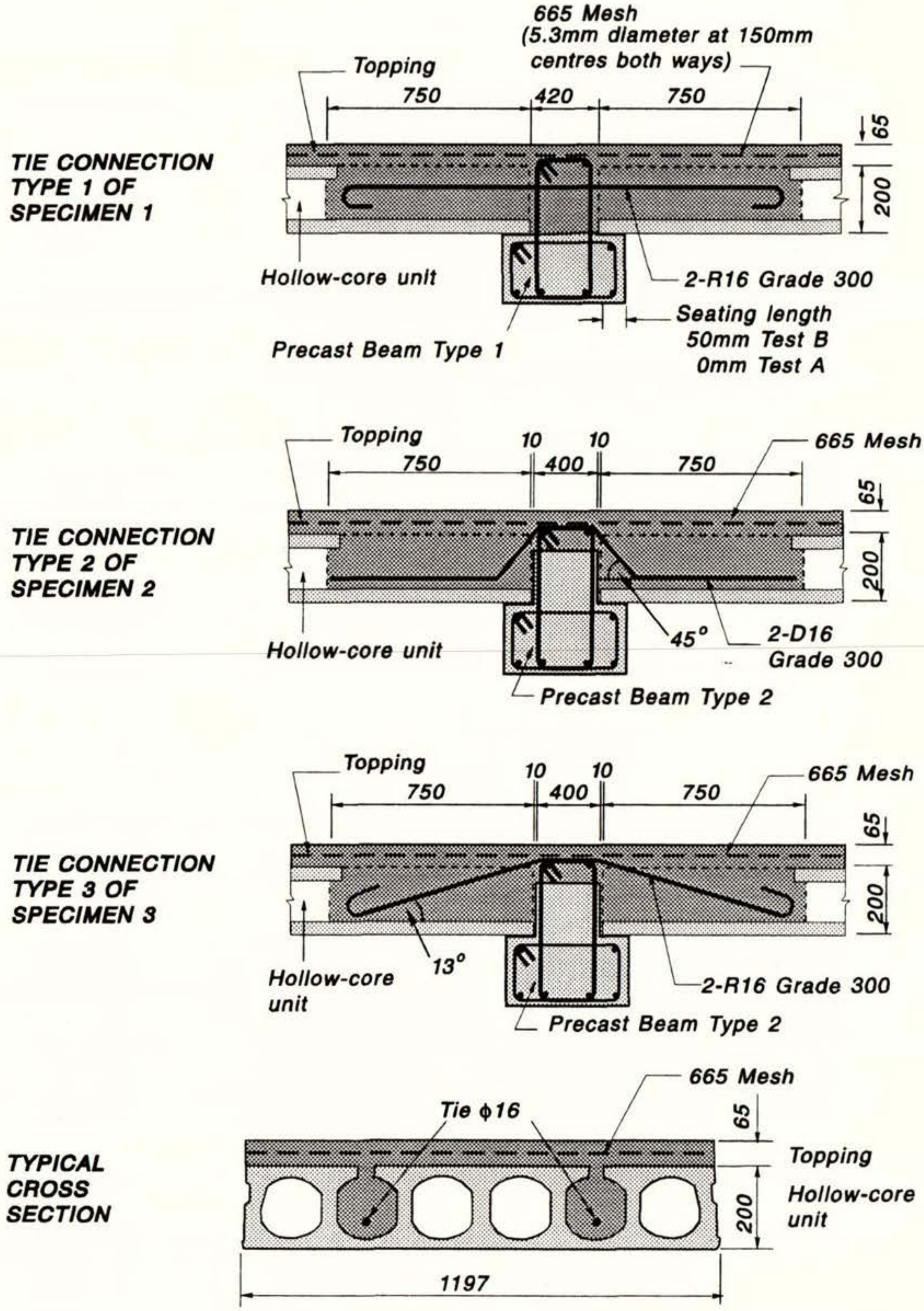


Fig. 2.2 Details of the Tie Connections Tested

the hollow-core unit has not been examined satisfactorily. For 265 mm thick hollow-core units it is recommended that the yield load of the tie bars introduced at one end should not exceed 160 kN, and that the yield load of tie bars introduced in each core should be limited to 80 kN. If this same criterion is applied to 200 mm thick hollow-core units with a reinforced concrete topping slab 65 mm thick, then:

Maximum yield load of one tie = 80,000 [N] = (Tie Area) x f_y
Maximum Tie Area [mm^2] = 80,000 [N] / f_y [MPa]
Therefore the maximum tie diameter, d_t [mm], is:

$$d_t\ [mm] = \sqrt{\frac{80{,}000\ x\ 4}{\pi\ x\ f_y\ [MPa]}} \qquad (2.1)$$

Tie number (N), area and yield strength of the ties can be related as follows:

$$N\ x\ (\text{Tie Area}\ [mm^2])\ x\ f_y\ [MPa] \leq 160{,}000\ [N] \qquad (2.2)$$

Using the above equations, the possible tie arrangements for 265 mm thick precast hollow-core units or equivalent, which meet conditions with regard to the control of splitting of the unit are presented in Table 2.1

Considering that for practical purposes and facility during construction the ideal number of ties is two, a good tie selection from Table 2.1 is two 16 mm diameter bars Grade 300 steel, which gives the maximum yield load for two tie bars.

Another advantage of using Grade 300 steel instead of Grade 430 steel, is its bigger ultimate elongation capacity (20% vs 15% minimum) [9]. In the design for accidental actions and against progressive collapse, it is normally considered desirable to take advantage of large deformation capacity and large displacements, and to use increased strength values, compared with ordinary design in the ultimate state.

Table 2.1 Possible Tie Arrangements for 265 mm Thick Hollow-Core Units

Steel Grade f_y [MPa]	Max. Tie Diameter (Eq. 2.1)	Possible Tie Arrangements (Eq. 2.2)		
		Diameter [mm]	Number	Yield Load [MPa]
430	15	12	2	97,266
		12	3	145,899
		10	2	67,544
		10	3	101,317
		10	4	136,089
300	18	16	2	120,636
		12	2	67,860
		12	3	101,790
		12	4	135,720

2.4 Selection of the Shape of the Ties

Fig. 2.2 shows the three different types of tie connection tested in this research. The shape of tie bar of connection Type 1 is suggested by FIP [2], and this tie was made using two plain 16 mm diameter Grade 300 bars. The tie bar of connection Type 2 is suggested in the New Zealand Guidelines [1], and is made using two deformed 16 mm diameter Grade 300 bars. As an alternative, the tie bar of connection Type 3 using two plain 16 mm diameter Grade 300 bars was proposed. It was considered that tie connection Types 1 and 3 would perform better than tie connection Type 2, because their straighter shape and plain bar surface would permit bond failure to propagate along the tie bar, making large ultimate elongation possible and therefore increase the energy absorption capacity of the tie.

2.5 Selection of the Length of the Ties

In ordinary design providing anchorage length for the yield load of the tie bars would be sufficient. With regard to design against progressive collapse, the anchorage length should provide for the fracture load, and in this way is possible to take advantage of the possible plastic elongation of the tie.

The additional length for deformed or ribbed bars, required to anchor the difference in bar force between the ultimate and yield load, can be estimated by using Table 2.2 taken from reference [2].

Table 2.2 Additional Anchorage Length for Deformed Bars

f_y [MPa]	$f_{su}/f_y = 1.2$		$f_{su}/f_y = 1.4$	
	Concrete Grade		Concrete Grade	
	20 MPa	30 MPa	20 MPa	30 MPa
300	8Ø	6Ø	16Ø	12Ø
400	10Ø	8Ø	20Ø	16Ø
500	13Ø	10Ø	26Ø	20Ø
600	16Ø	12Ø	32Ø	24Ø

Where: Ø is the bar diameter
f_{su} is the tensile strength of the tie bar
f_y is the yield strength of the tie bar.

The additional anchorage length for a 16 mm diameter bar, obtained from Table 2.2, considering a concrete strength of 20 MPa and that the ratio f_{su}/f_y is approximately equal to 1.4 for Grade 300 steel, is

Additional anchorage length = 16Ø = 256 mm.

The anchorage length for the yield load of deformed bars in tension for Grade 300 steel is [10]:

L_d [mm] = 24Ø [mm] x 300 [Mpa]/275 [MPa], and for Ø=16 mm, L_d = 419 mm.

Therefore the total anchorage length required for a 16 mm diameter deformed tie bar of

Grade 300 steel is :

$L_d = 419 + 256 = 675$ mm.

However, sudden changes of the cross section of the hollow-core unit should be avoided within a critical zone near the support, so that significant crack inducements are not present. The length of the critical zone should be taken equal to the transfer length of the prestressing force, which is the length required to transfer the prestressing force to the concrete by bond [2].

The ending of the concrete fill in the voids will form a discontinuity in the hollow-core unit and create a potential plane of cracking. For that reason, the concrete fill and the tie bars should not be ended within the critical zone near the support. For an anchorage length no less than the critical zone, the bond at the interface between the concrete fill and the core will normally not be critical for the anchorage. In this respect the tie bar can be regarded as being directly anchored in the hollow core unit [2].

According with the ACI Code [11], the transfer length for seven wire strands can be assumed to be equal to 50 times the diameter of the strand. For strands of 12.5 mm diameter:

Transfer Length = $L_t = 50\varnothing = 50 \times 12.5$ mm $= 625$ mm.

Libby [12] suggests the transmission length of tendons released by cutting with an abrasive wheel can be expected to be about 20 to 30% greater than of tendons that are released gradually. Then:

Transfer Length = $L_t = 1.20 \times 625$ mm $= 750$ mm.

The FIP [13] also suggests the following relation between the free end slip δ and the transmission length L_t,

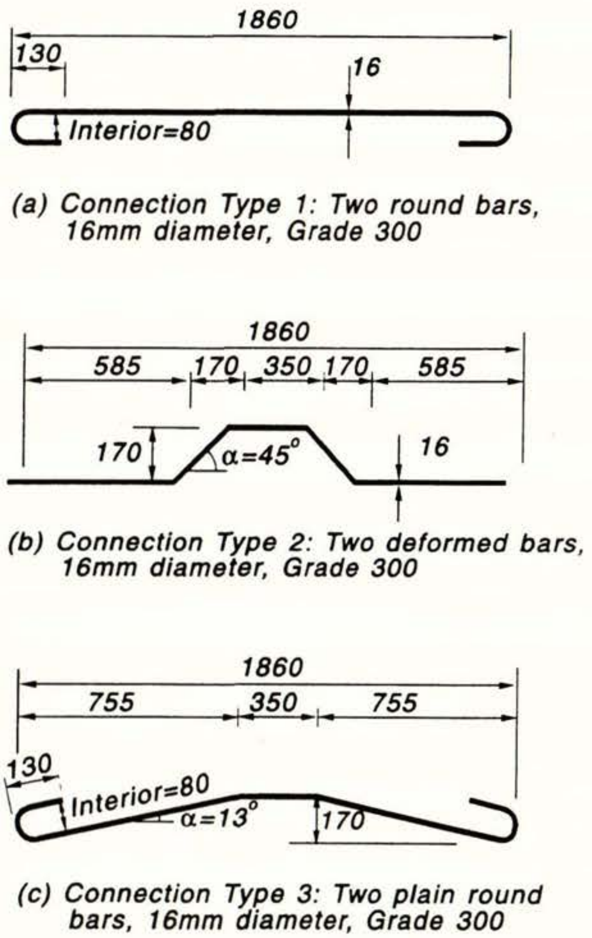
$$\delta = 1/3 \times (L_t \times \sigma_{sp}/E_s)$$

where σ_{sp} = stress in the prestressing steel (fully developed)

E_s = modulus of elasticity of prestressing steel

The Dy-core units used in this research have the following values: $\sigma_{sp}=1073$ MPa, $E_s=190.7$ MPa, and free end slip δ of 1 to 2 mm was often observed. For these values, the transfer length varies from 533 to 1066 mm. Therefore a transfer length of 750 mm appears reasonable.

The calculated required anchorage length of 625 mm is less critical than the calculated required transfer length of 750 mm. Fig. 2.3 shows the dimensions of the three types of tie bar used in the connections.



(a) Connection Type 1: Two round bars, 16mm diameter, Grade 300

(b) Connection Type 2: Two deformed bars, 16mm diameter, Grade 300

(c) Connection Type 3: Two plain round bars, 16mm diameter, Grade 300

Fig. 2.3 Dimensions of the Tie Bars Used in the Connections Tested

Chapter 3

DESCRIPTION OF THE TESTS

3.1 The Test specimens

Three test specimens were constructed from longitudinal strips of precast hollow-core floor units of Dy-core type, 1200 mm wide by 200 mm deep, with a 65 mm thick cast-in-place reinforced concrete topping slab. Each specimen tested consisted of 3 spans, the clear spans being $L_1 = 3.20$ m, $L_2 = 1.80$ m and $L_3 = 3.25$ m, which were mounted on four supports (see Fig. 3.1).

The first exterior support was a hinge mechanism mounted over rollers which was free to move horizontally. The other supports were precast concrete beams seated on the laboratory strong floor. The 65 mm thick concrete topping was reinforced with 665 welded wire mesh 665 giving 145 mm^2 of cross sectional area per metre width. The topping slab was cast in a continuous layer over the precast units and the supporting beams, deepening over the supporting beams, simulating the details typically used at interior beams of buildings.

The connections tested were located at the two interior precast concrete supporting beams, which are referred to as Support 2 and Support 3. The same type of tie connections were tested subjected to vertical load at both supports, but under different conditions of bearing length and horizontal load applied (Tests A and B).

3.2 Construction of the Test Specimens

3.2.1 End Support 1 (Hinge Mechanism)

Support 1 was a hinge mechanism mounted over rollers and free to move horizontally. It was built using two steel beams 310 UC 97 placed at top and bottom of the Dy-core unit at a distance of 441 mm from its end (see Fig. 3.2a). These two steel beams

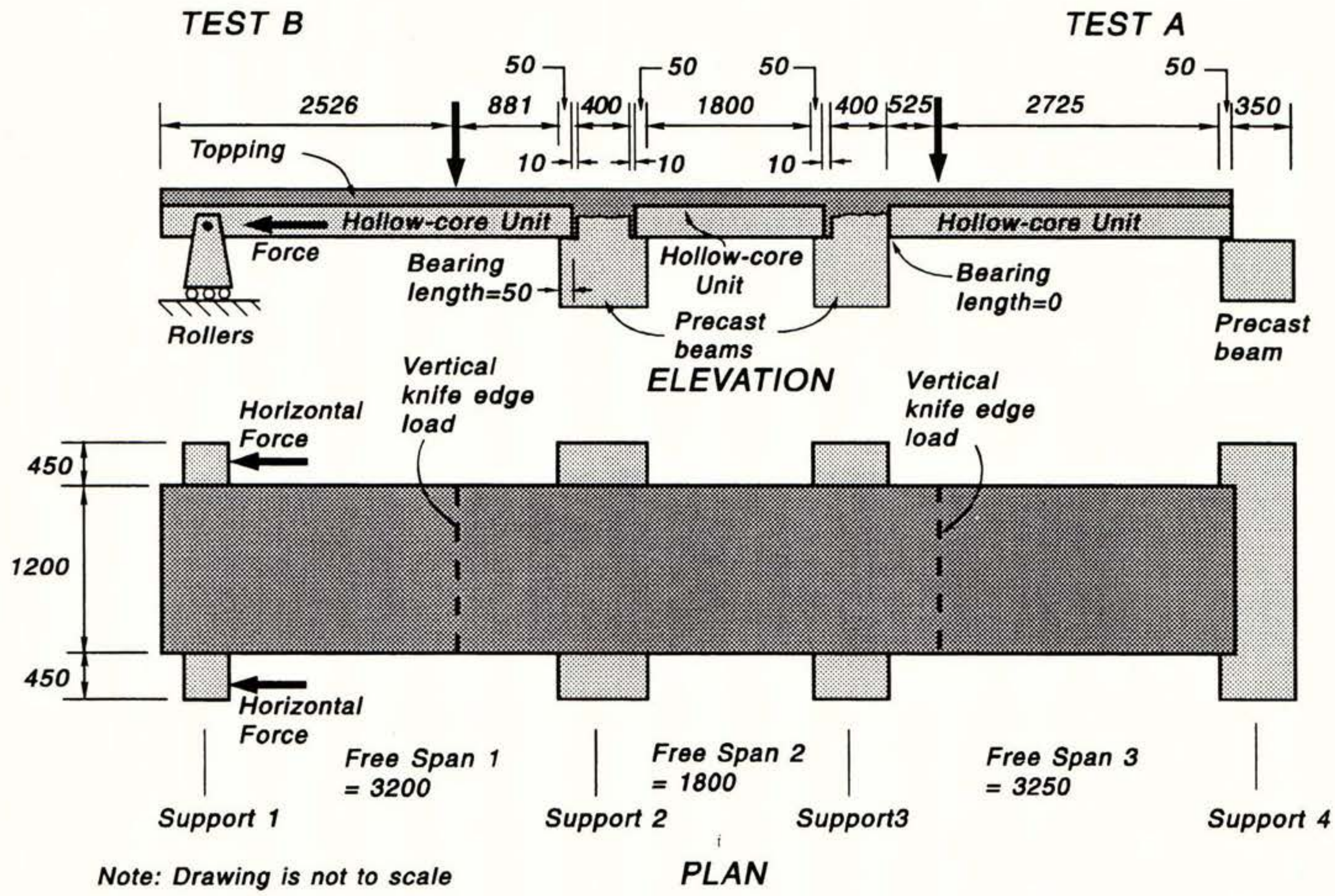


Fig. 3.1 Dimensions and Test Set-Up of Specimens.

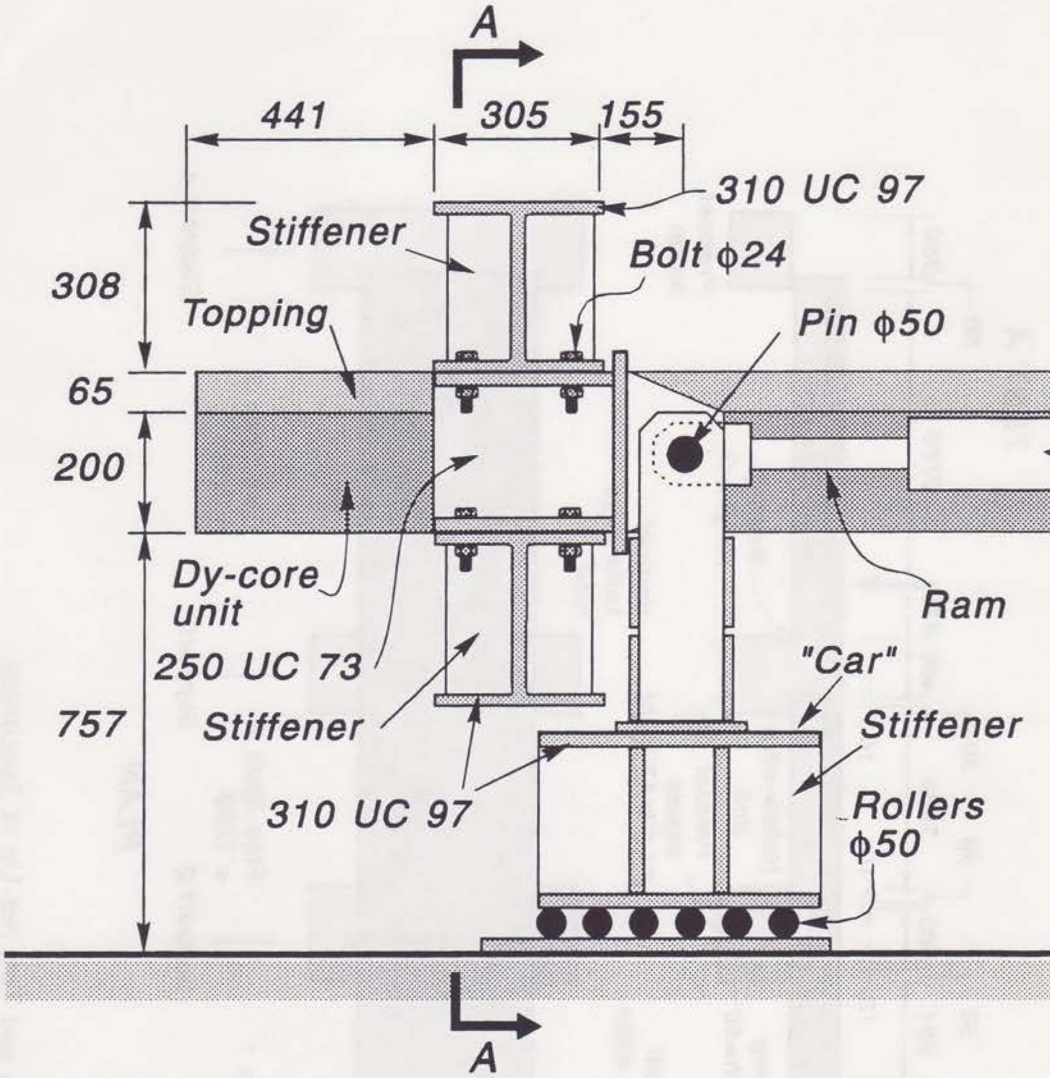


Fig. 3.2a Hinge Mechanism at Support 1. - Elevation

were connected by means of 12 screwed round bars of 24 mm diameter placed across the Dy-core unit and the concrete topping (see Fig. 3.2b). The voids of the Dy-core unit at this end were reinforced with 6-D12 Grade 430 bars, 1500 mm long, and filled with concrete along that length during the pouring of the topping (see Fig. 3.2c). At both sides of the Dy-core unit and between the two steel beams of Support 1, two pieces of steel section 250 UC 73, 330 mm long, were bolted to the steel beams. These pieces of steel had plates at one end which were designed to be pined to the head of the two horizontal rams and also to the top of the "cars". The "cars" were steel devices mounted over steel rollers which allowed horizontal movement of the specimen during the first stage of test B while at the same time

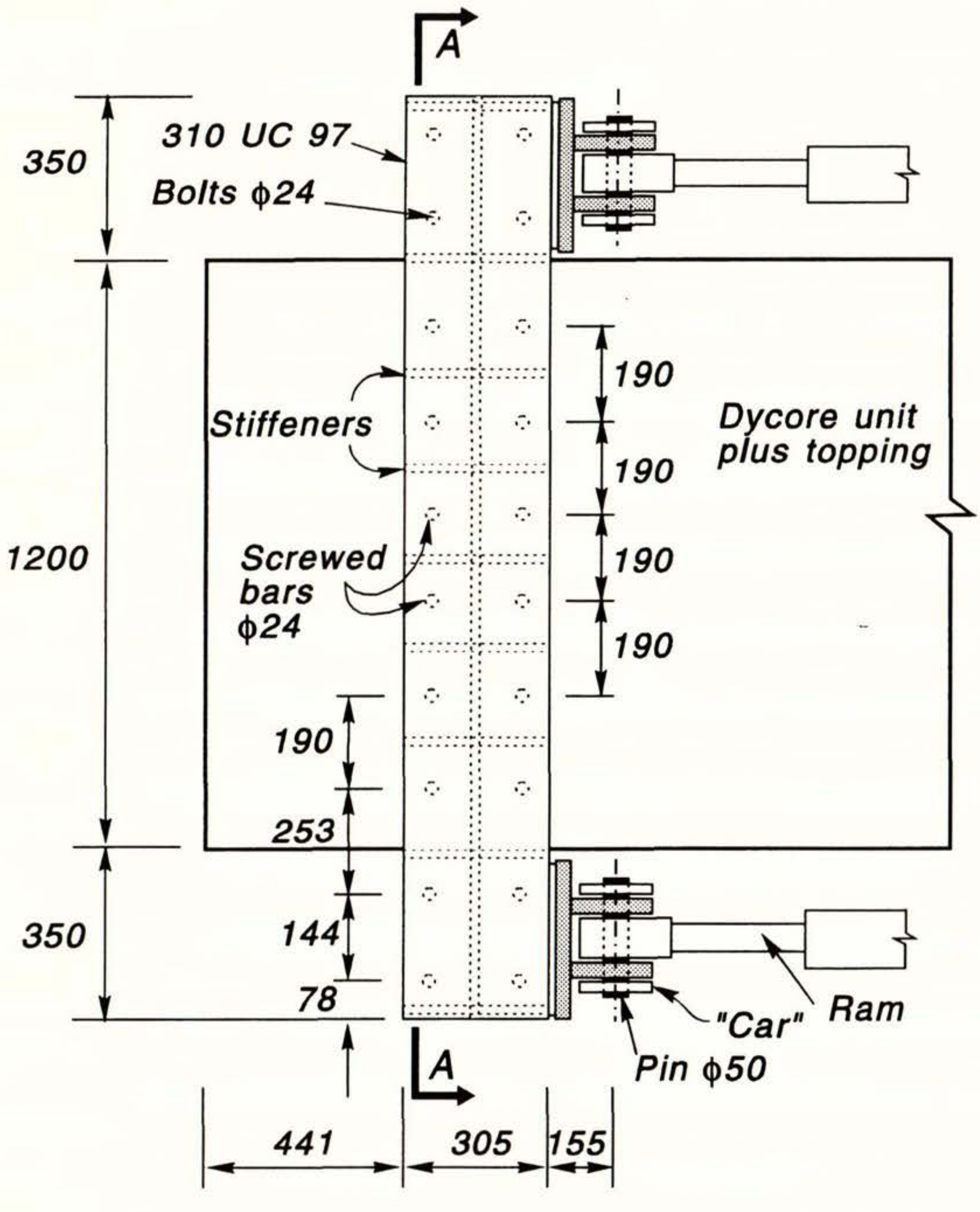


Fig. 3.2b Hinge Mechanism at Support 1. - Plan

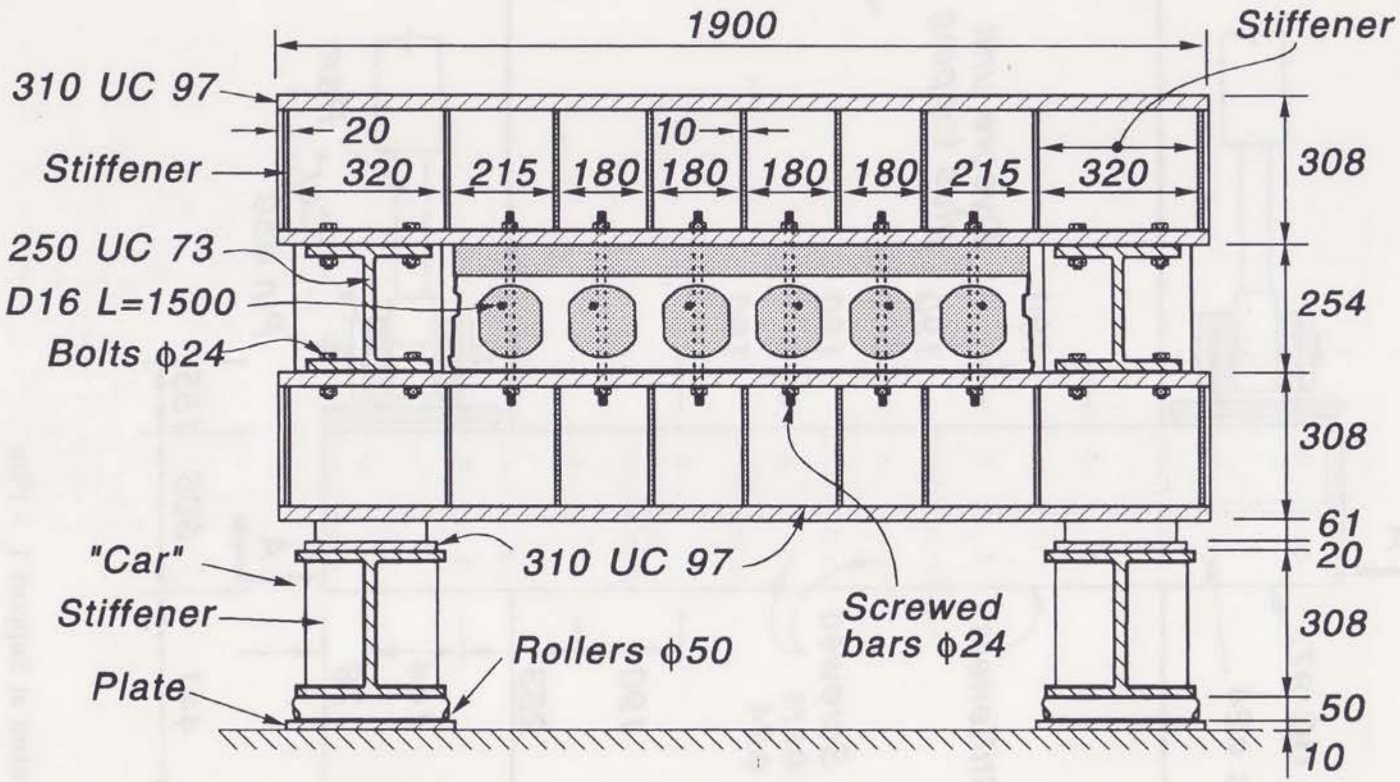


Fig. 3.2c Hinge Mechanism at Support 1. - Section A-A

transmitting the vertical reaction from the steel beams of Support 1 to the ground (Fig. 3.2c). The intended functions of Support 1 and its hinge mechanism were:

1) To support the floor unit at this end, and to transmit its vertical reaction to the two "cars" at both sides on the floor.
2) To transmit and distribute the horizontal forces applied by the two rams to the specimen.
3) To allow free horizontal movement of the specimen when the two horizontal rams extend during the first stage of test B. This is possible as the "cars" slide over steel rollers at their bases
4) To allow free rotation of the end of the specimen during the second stage of test B (vertical displacement). This is possible because of the pinned connection between the rams, the steel beams and the "cars".

3.2.2 Interior Supports 2 and 3

Supports 2 and 3 were precast concrete beams seated on two concrete blocks resting on the laboratory strong floor. For Specimen 1, Supports 2 and 3 were of beam support type 1, and for specimens 2 and 3 were of beam support type 2 (see Fig. 2.1).

Support 2 was designed to support the two horizontal rams used in the first stage of test B, and therefore steel plates of dimensions 300 x 360 x 10 mm were attached to the Support 2 to be used as bearing surfaces and for the fixing of the rams.

Support 3 did not have seating for the precast concrete floor units on one of its sides, and this side was used as an interface in the connection with the precast floor units in test A, simulating a connection without seating.

Figs. 3.3 to 3.6 shows the dimensions and reinforcement of the precast concrete beams used as Supports 2 and 3.

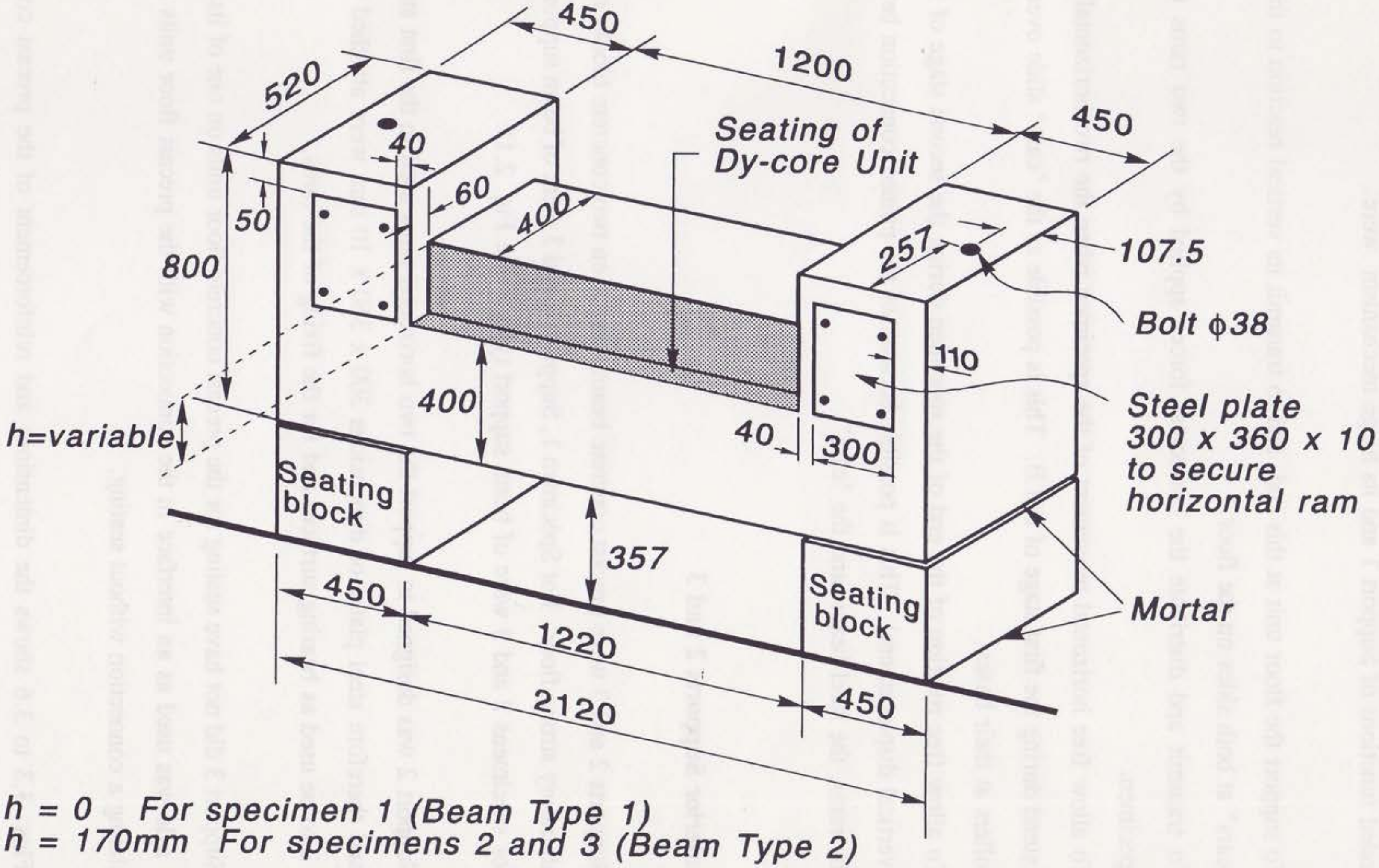


Fig. 3.3 Dimensions of Precast Concrete Beam Used as Support 2.

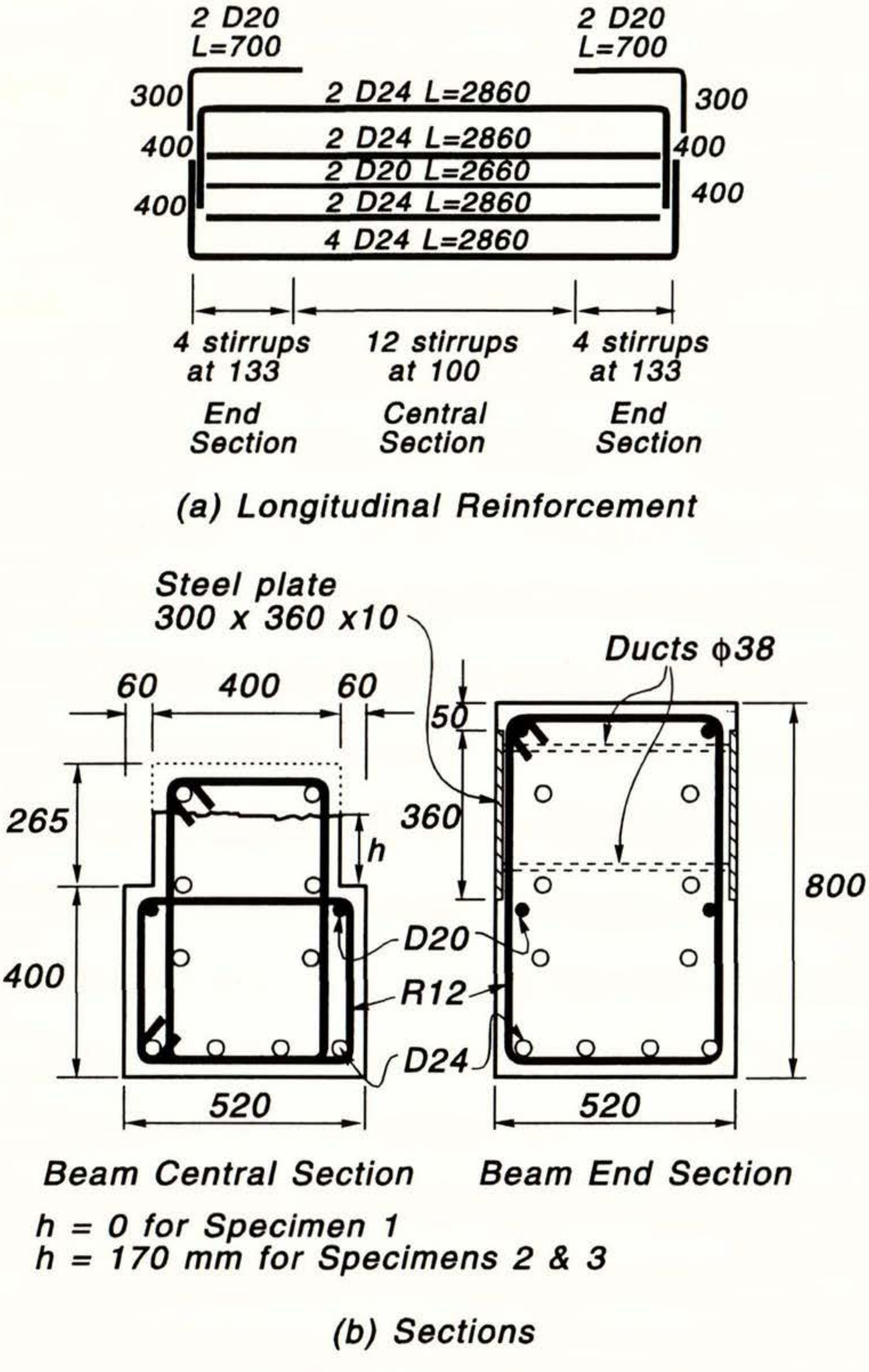


Fig. 3.4 Reinforcement of Support 2.

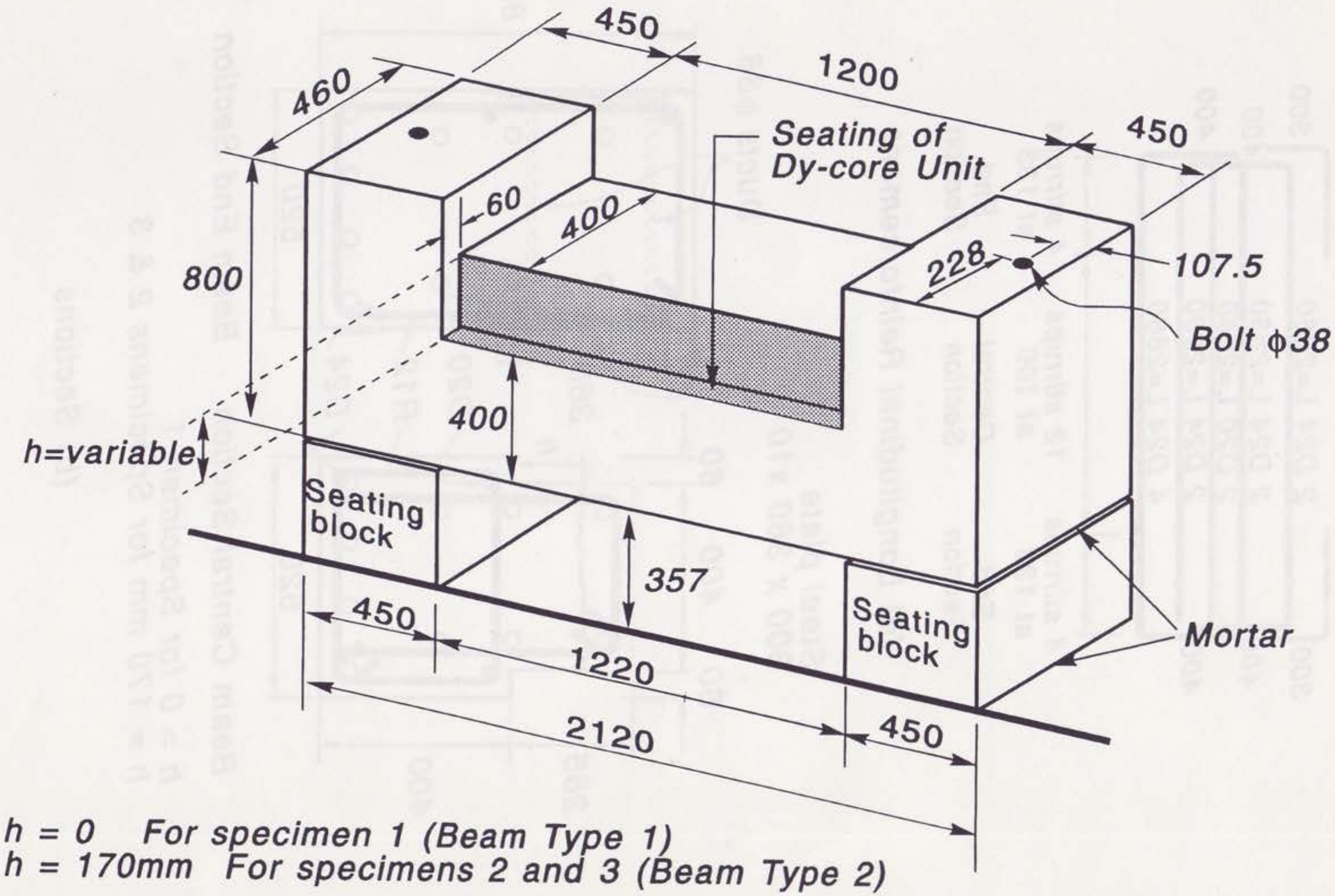
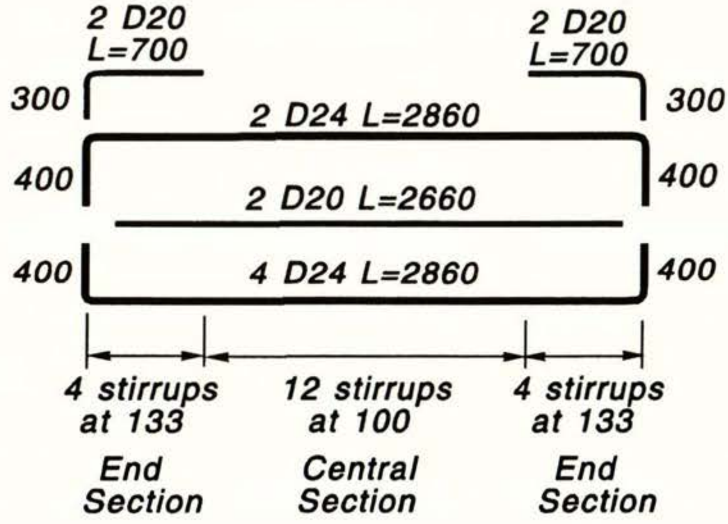
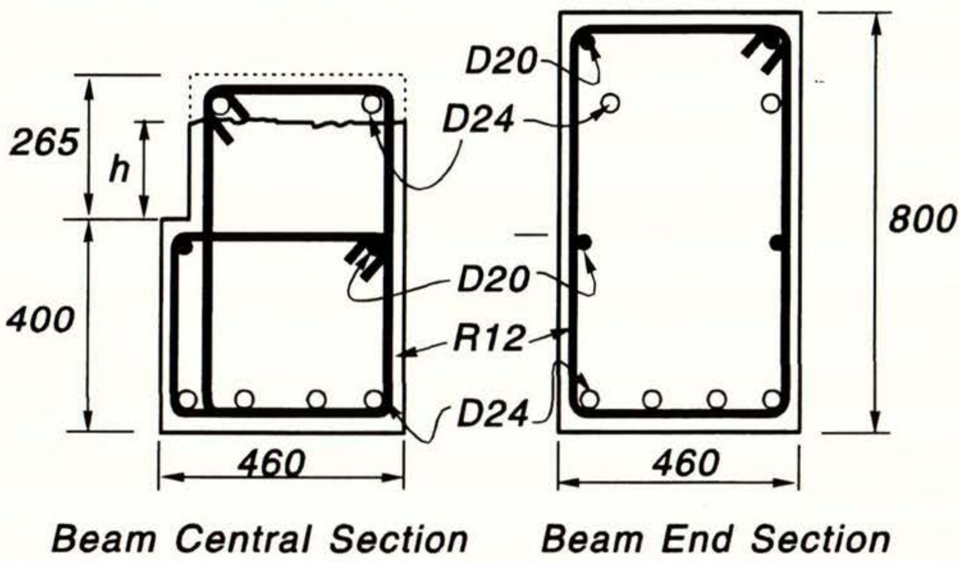


Fig. 3.5 Dimensions of Precast Concrete Beam Used as Support 3.



(a) Longitudinal Reinforcement and Stirrups



h = 0 for Specimen 1
h = 170 mm for Specimens 2 & 3

(b) Sections

Fig. 3.6 Reinforcement of Support 3.

3.2.3 End Support 4

Support 4, shown in Fig. 3.7, was also a precast concrete beam but without tie bars or poured concrete connecting the precast floor units to the supporting beam. The floor was simply supported at Support 4. The Support 4 beam was directly plastered to the strong floor.

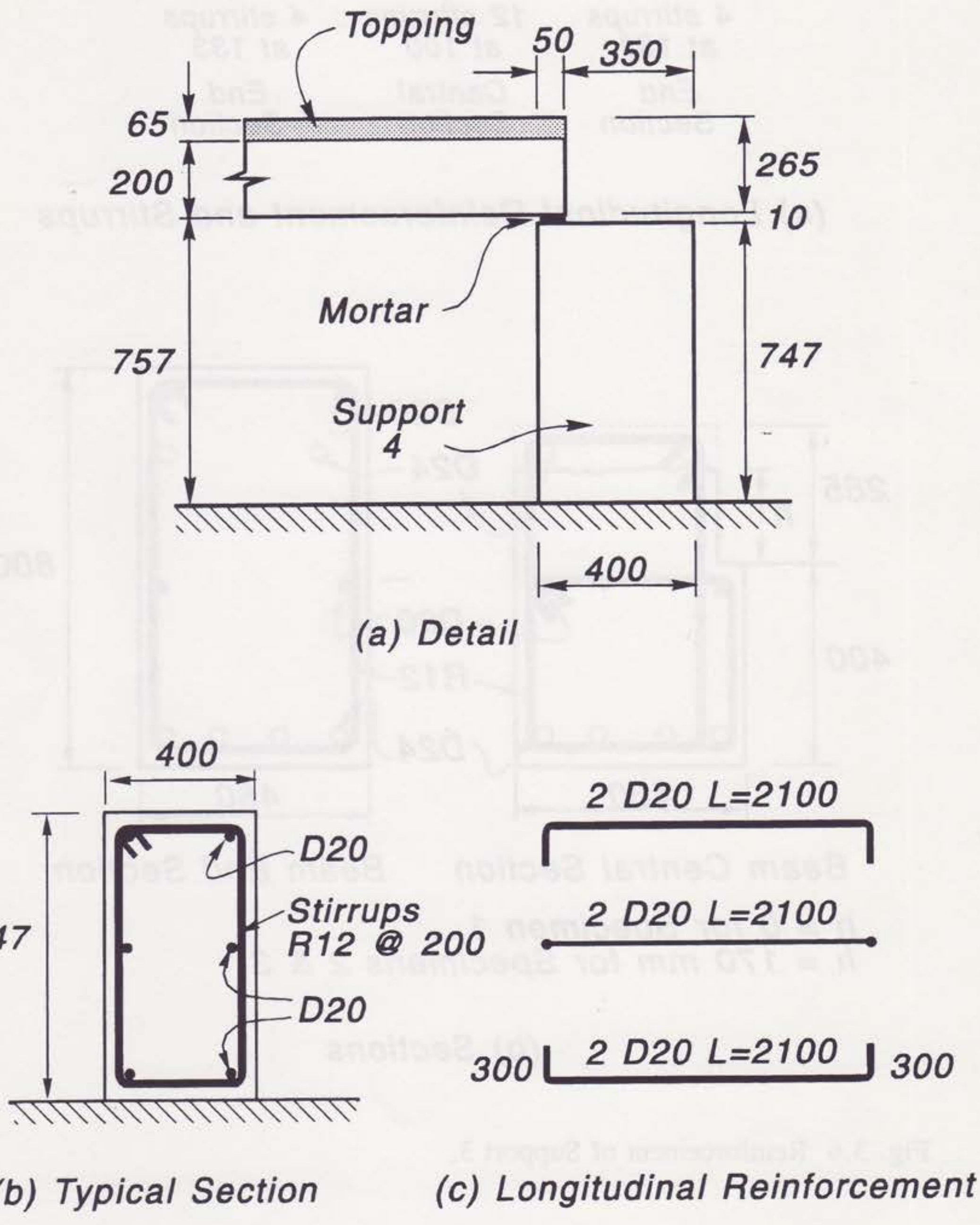


Fig. 3.7 Dimensions and Reinforcement of Support 4

3.2.4 Hollow-core Units

The hollow-core units, of the commercially available Dy-core type, were produced by Precision Precasting Ltd, Christchurch, using the technique of extrusion on a long casting line. The units were saw cut to the required length.

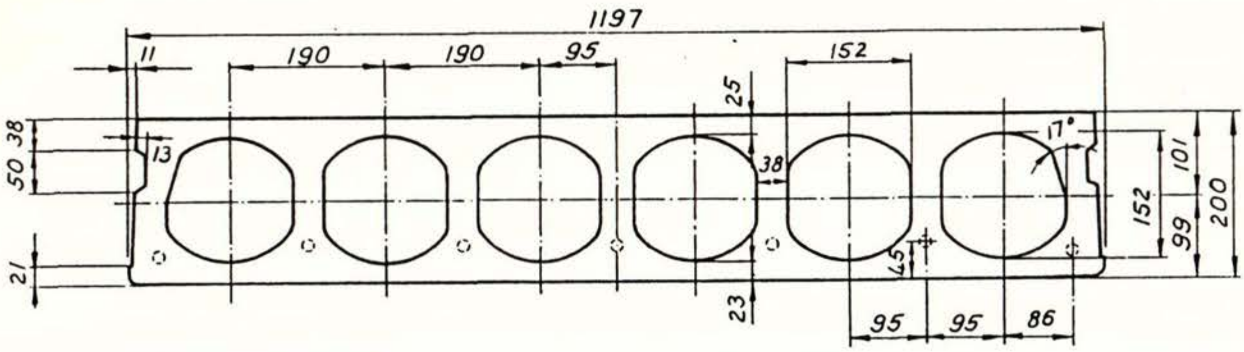


Fig. 3.8 Dimensions of Dy-core Used in the Tests

Each unit had a cross section 200 mm thick and 1200 mm wide, and contained seven prestressing tendons, the five inner tendons being 12.5 mm diameter strand and the outermost ones being 11.0 mm diameter strand. A typical cross section is shown in Fig. 3.8. All the dimensions, properties and materials of the Dy-core units are given in Appendix A.

3.2.5 Setting Up the Specimens

Initially the three reinforced concrete support beams (Supports 2, 3 and 4) were cast in their moulds (see Fig. 3.9a). Then after a minimum period of one week to allow the concrete to gain strength, the beams were moved and plastered into their respective position (see Fig. 3.9b). The bottom steel beam of Support 1 was then propped temporary in its location and the Dy-core units were seated on their supports or were temporary propped. Note that the interior end of span 3 was propped because this end had zero seating width.

Fig. 3.9a Pouring of Supports

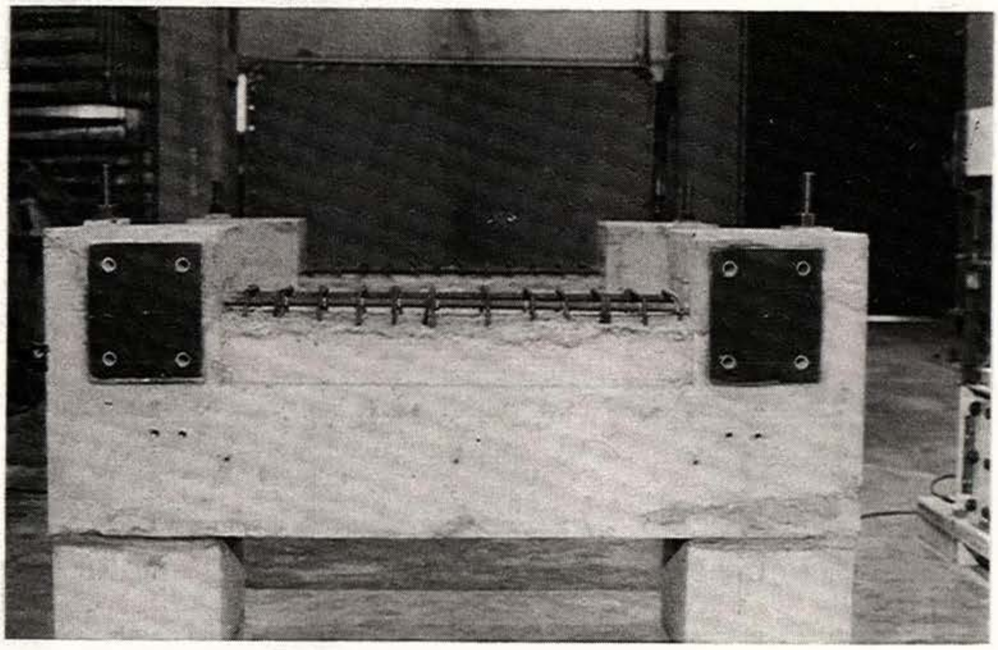


Fig. 3.9b Supports Plastered in Their Position

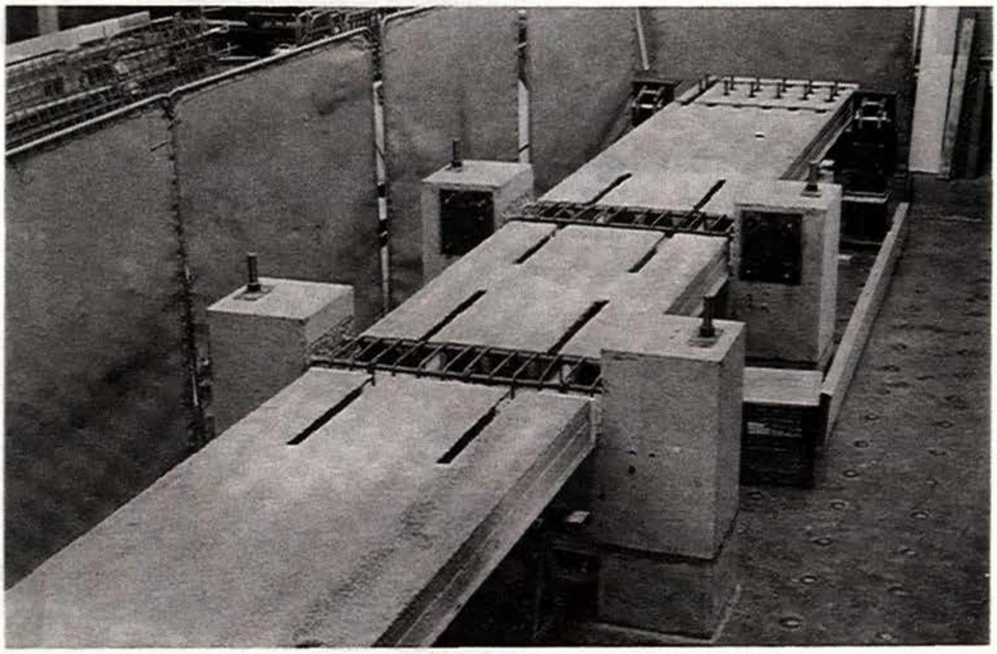


Fig. 3.9c Dy-core Units with Top Flange of the Voids Broken Back to Anchor Tie Bars

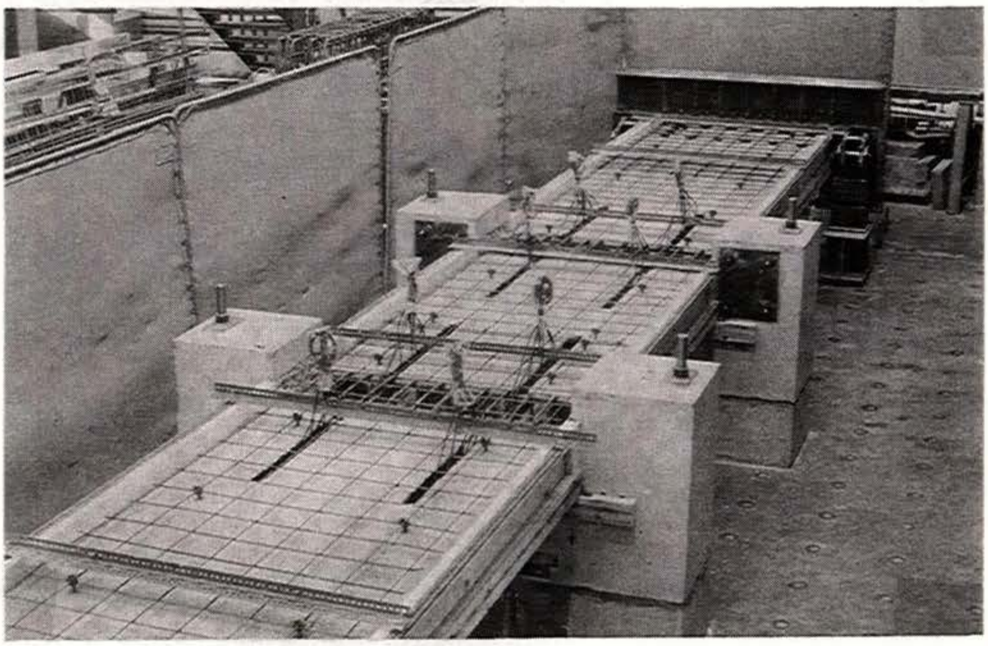


Fig. 3.9d Wire Mesh Placed Above the Units

Subsequently, the top flange of the voids which anchored the tie connection bars at the interior Supports 2 and 3, or the bolts of the Support 1, were broken back (see Fig. 3.9c). The units were then cleaned and the tie connection bars were placed in their respective position. Also, the bolts and reinforcement inside the voids of Support 1 were fixed in position. Paper stops were inserted inside the voids used to anchor reinforcement at 30 mm from the end of the bars to prevent the passage of wet concrete further along the voids during the pouring of the cast-in-place topping.

Finally the 665 welded wire mesh was placed 30 mm above the units (see Fig. 3.9d). Timber formwork was constructed and the 65 mm thick topping slab was poured in a continuous layer over the units and the supporting beams (Fig. 3.9 e).

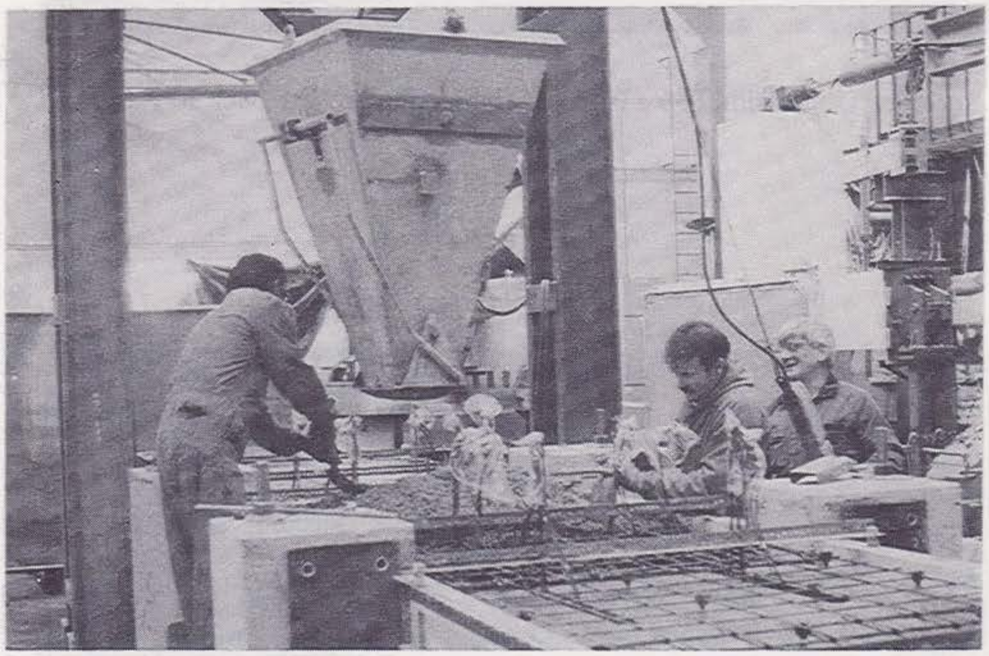
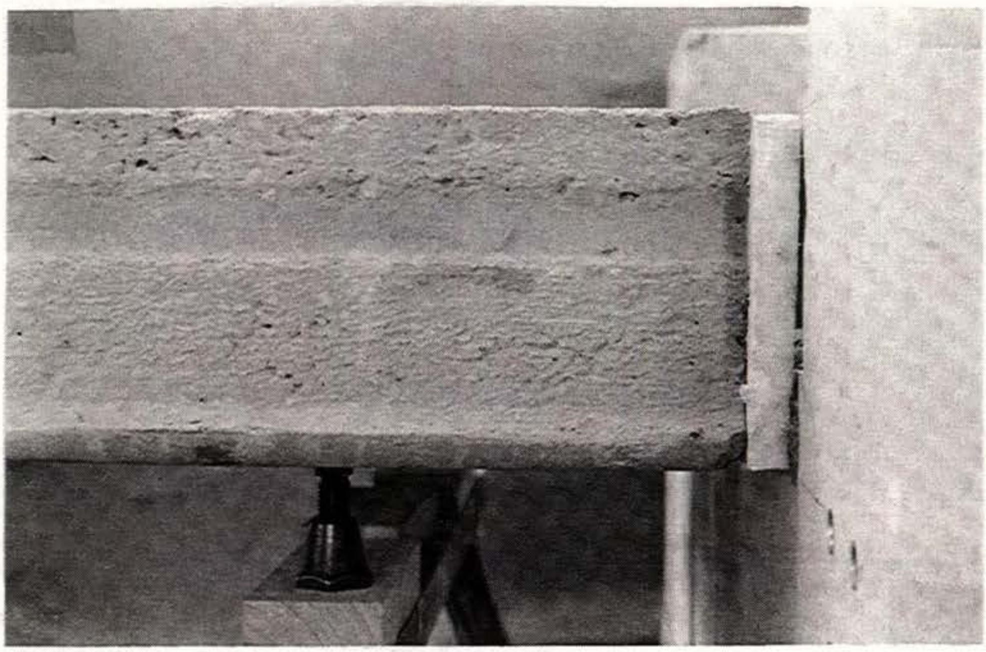
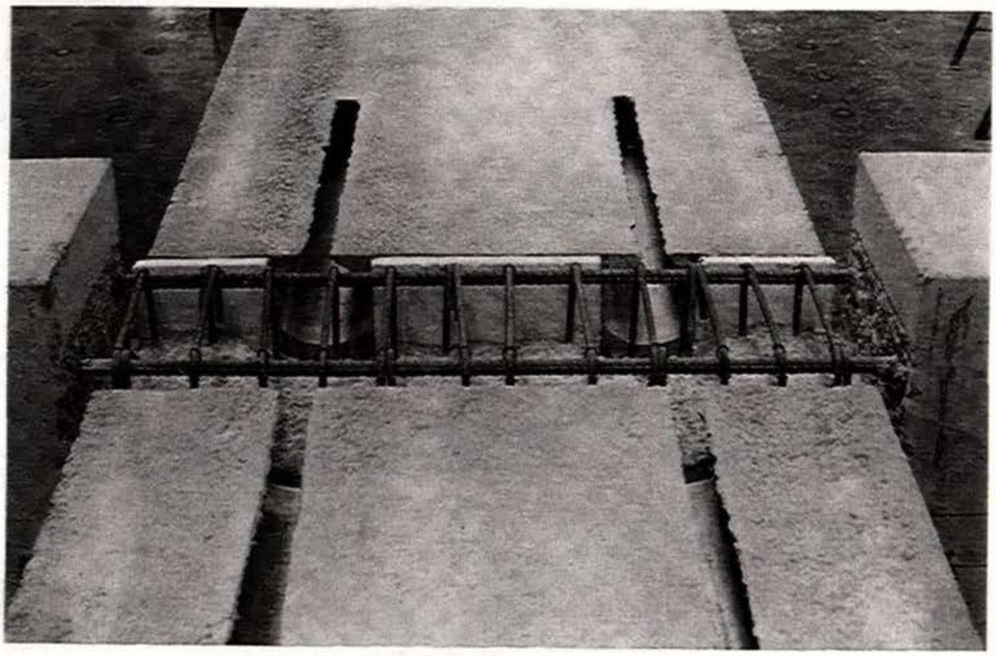


Fig. 3.9e Pouring of Cast-in-Place Topping Concrete Specimen 1

During the construction of the tie connection of span 3 with Support 3, polystyrene pads were placed to block out the voids of the Dy-core unit which were not being used to anchor the two tie bars of the connection, with the object of preventing concrete from penetrating inside those voids (see Fig. 3.10). The polystyrene pads protruded out of the voids towards the supporting beam a distance of 20 mm. In this way only the shear capacity



a) Lateral View



b) Top View

Fig. 3.10 Polystyrene Pads Blocking Out the Voids provided by the reinforced topping slab and the tie bars in the two filled voids was studied. It is to be noted that when supporting beam Type 2 (see Fig. 2.1) is used the filling of the narrow gap between the faces of the supporting beam and the Dy-core unit during the pouring of the concrete topping can be very difficult or even impossible.

3.3 Test Apparatus

3.3.1 Test A

In this test vertical load was applied without horizontal load. The vertical loading system was designed to apply a transverse line load (knife edge load) to the top surface of the right hand span of the specimens. The load was applied at a distance of 552 mm from the face of the supporting beam (Support 3) for tie connections Types 1 and 2 and 906 mm from the face of the supporting beam for tie connection type 3. A 130 ton jack with a system of spreader beams was used to apply the uniformly distributed vertical knife load acting across the specimen (see Fig. 3.11). Even contact between the first spreader beam and the topping was achieved using dental plaster. The jack and steel beams reacted against a structural steel frame, fixed to the laboratory strong floor on either side of the test unit. Figs. 3.11 and 3.13 show the dimensions and construction details of the vertical loading system.

A horizontal restraining system (see Fig. 3.12) was used at both sides of the specimen to prevent any movement or rocking of the supporting beam (Support 3) during the test.

3.3.2 Test B

In this test horizontal load was first applied and then vertical load.

The horizontal load was applied by two 43 ton jacks which acted simultaneously at both sides of the specimen to force the precast unit to slide away from its support during

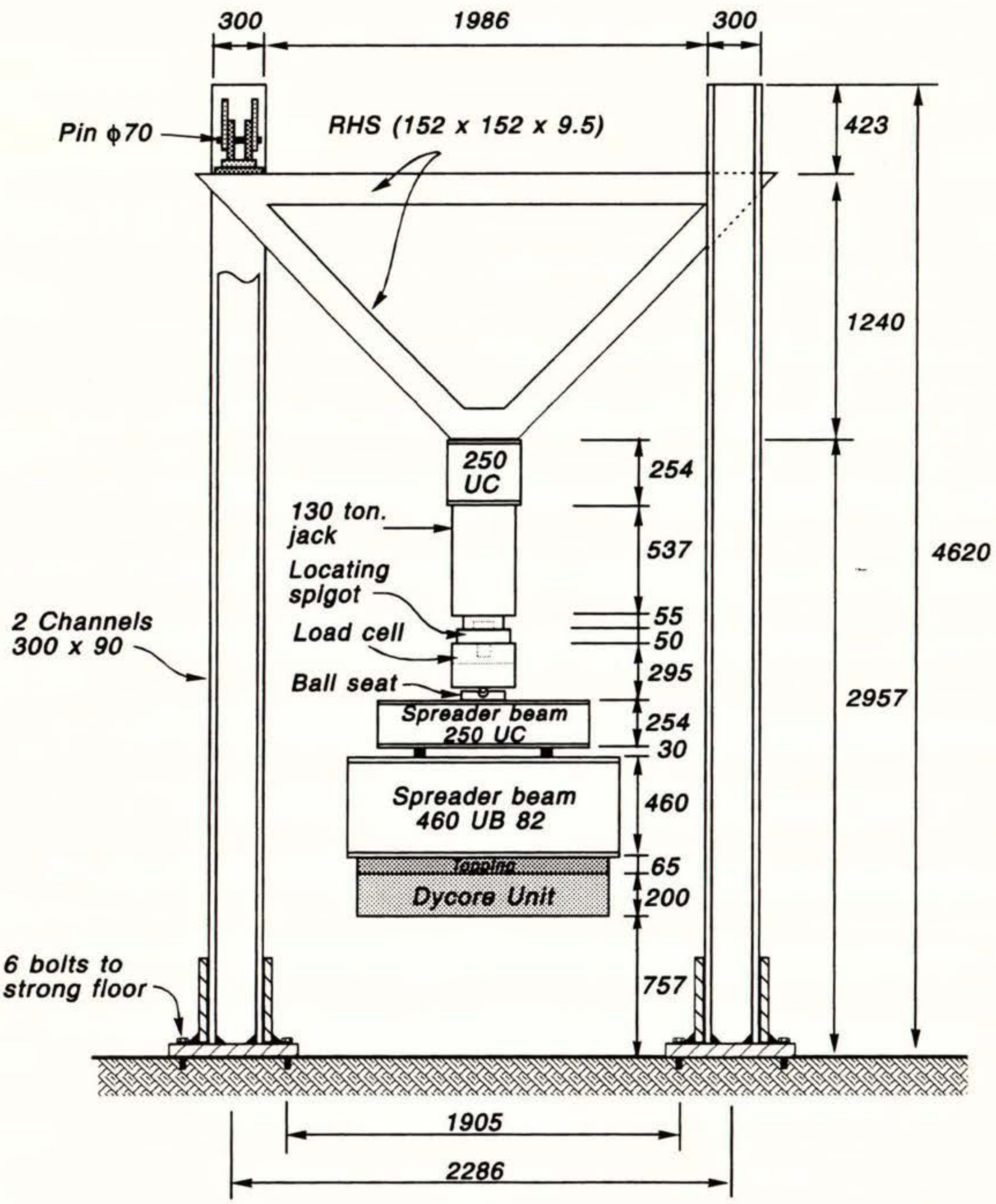


Fig. 3.11 Vertical Loading System of Test A and B

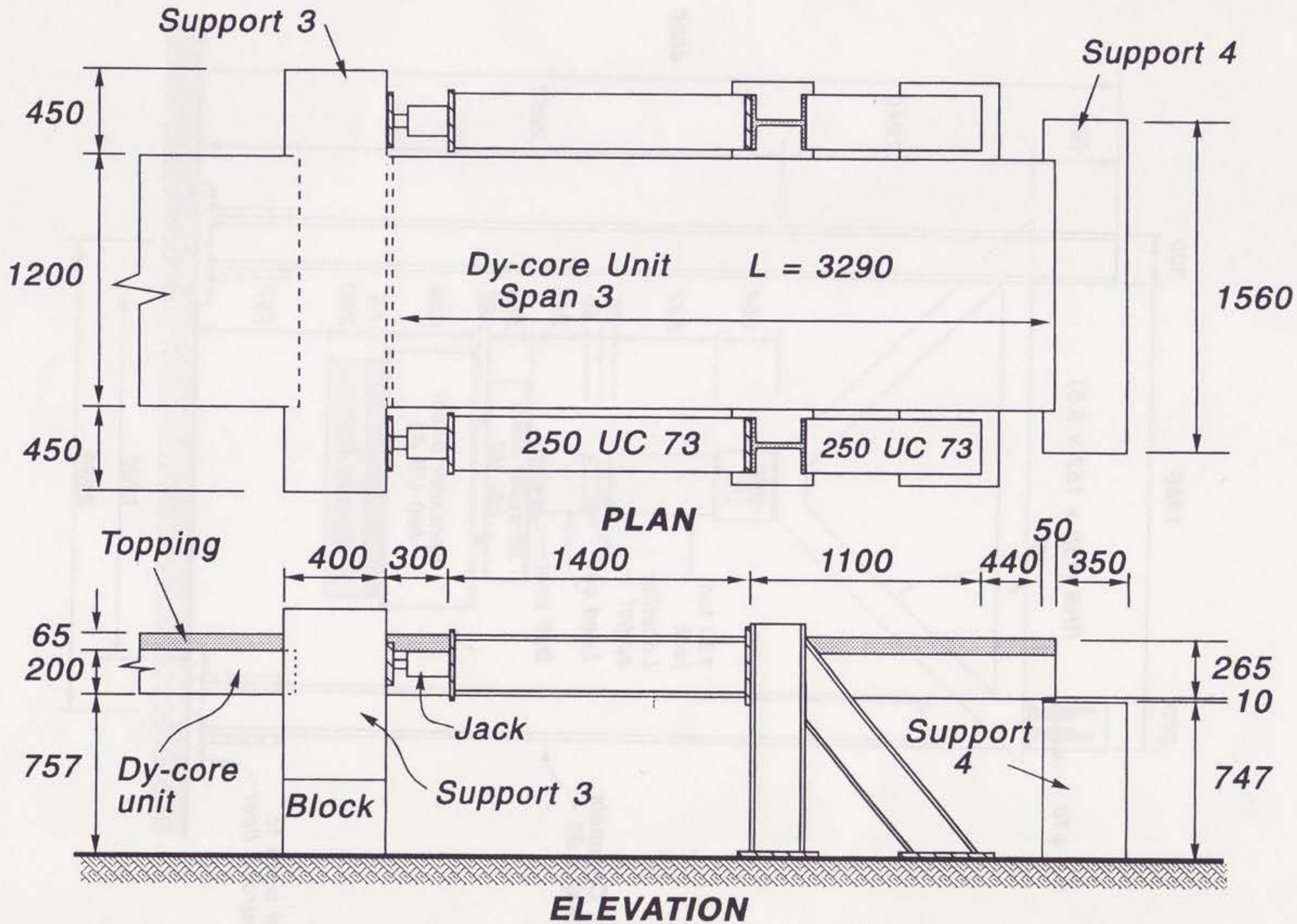


Fig. 3.12 Horizontal Restraint System of Support 3

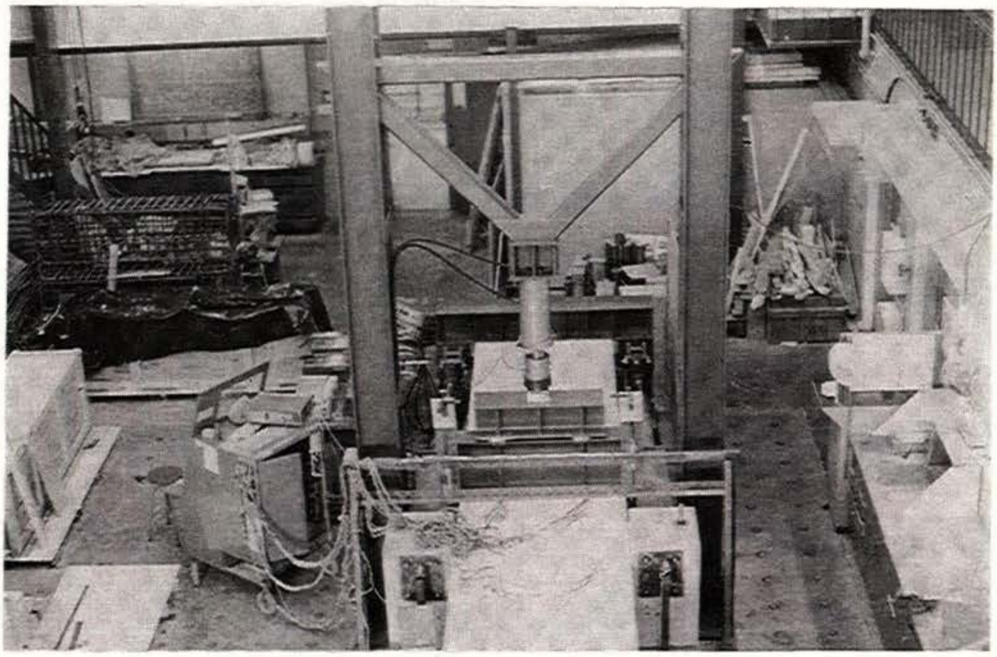


Fig. 3.13 Vertical Loading System in Position for Test B

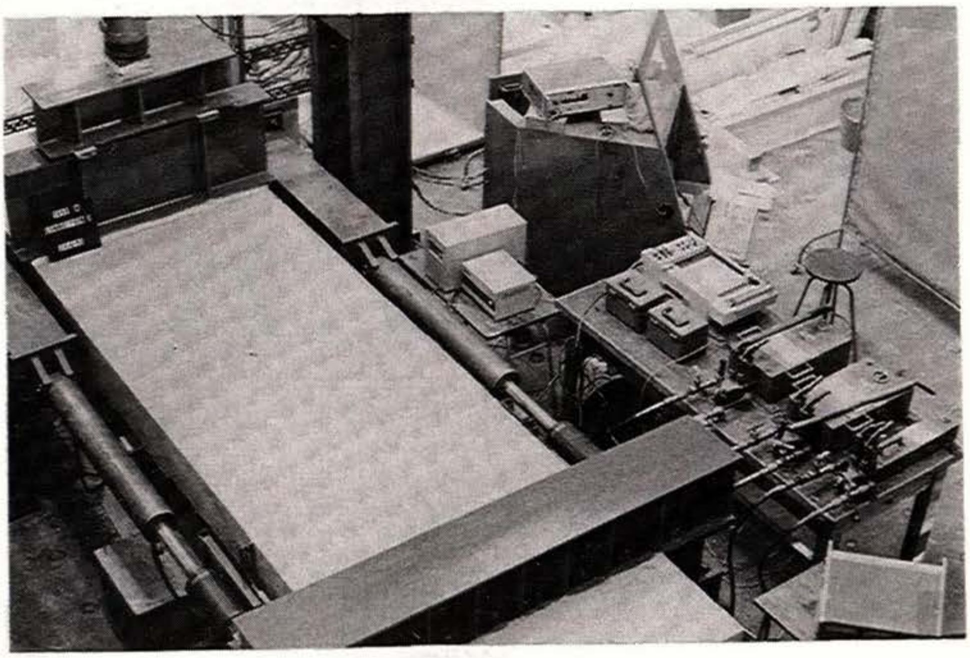


Fig. 3.14 Top View of Horizontal Loading System of Test B

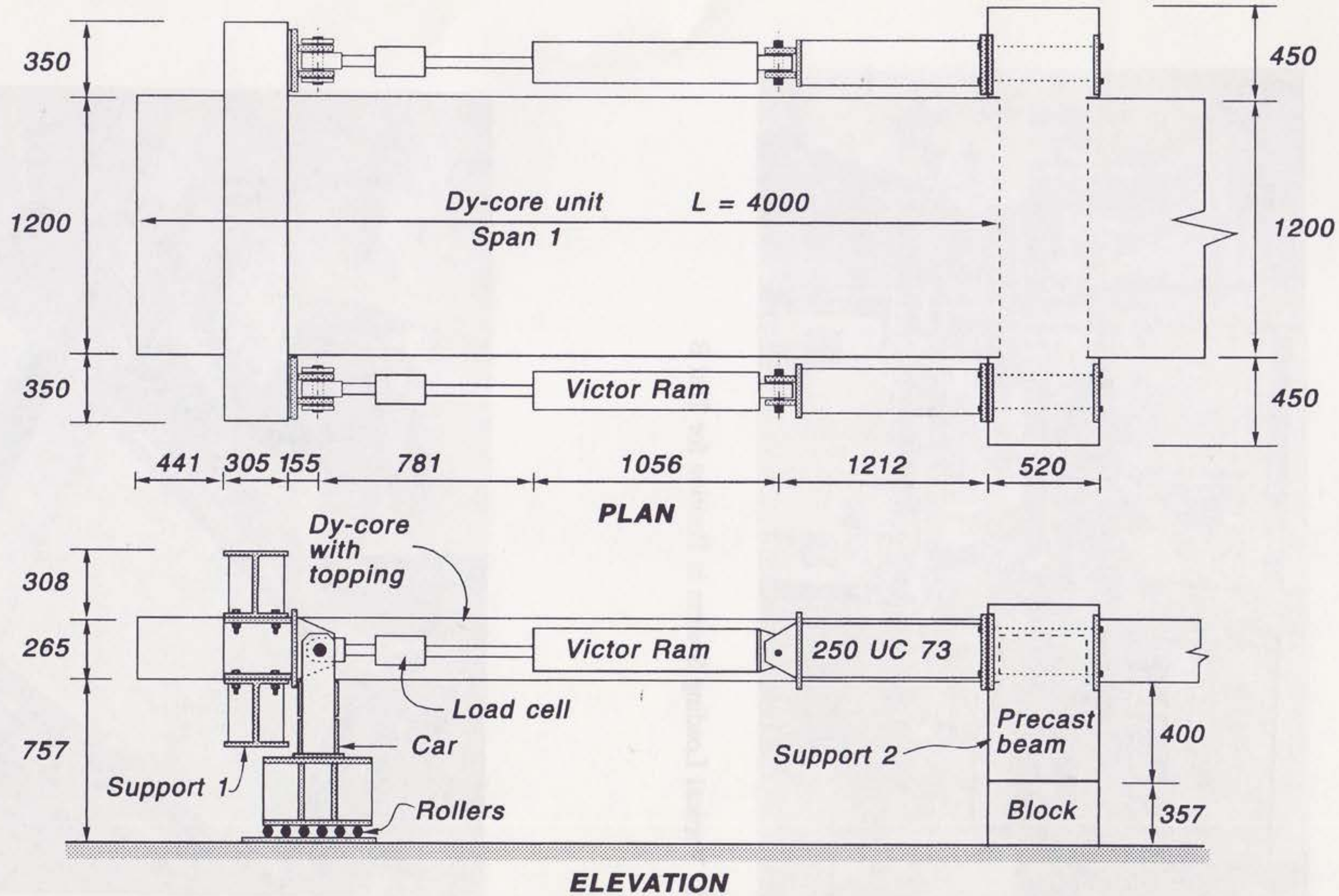


Fig. 3.15 Horizontal Loading System of Test B

the first stage of the test. Each horizontal jack was fixed at one end to a horizontal steel column (250 UC 73) which in turn was fixed to Support 2. At the other end the rams were pinned to a hinge mechanism which transmitted the force from the ram to the specimen through the steel beams of Support 1 (Figs. 3.14 and 3.15).

The vertical loading system utilized in test B was the same as that utilized in Test A (see Fig. 3.11 and 3.13), but the load was applied a distance of 881 mm from the face of the supporting beam (Support 2).

3.4 Instrumentation

3.4.1 Preliminary Tests for the Selection of the Strain Gauges and Adhesive

It was anticipated that in Test B it would be necessary to measure very large tensile strains (15% or more) on the tie reinforcement in the connection regions, as this test required loading to large displacements. Under such a large strain, ordinary strain gauges may not function, because they may peel off the gauge base or fracture of the grid wire may occur.

High elongation strain measurements place severe demands on the gauge installation, and necessitate special gauge and adhesive selection and surface preparation procedures [14]. As special strain gauges with such high elongations had not been used before in the laboratory of the Department of Civil Engineering, two different commercially available strain gauges were tested in order to select the best one. These were:

1) Strain gauge type EP-08-250BG-120, 120 Ω and 5 mm length, from Measurements Group Inc., USA.

2) Strain gauge type YL-5, 120 Ω and 5 mm length, from Tokyo Sokki Kenkyujo Co., Japan.

Also two types of adhesives were used in combination with the strain gauges:

1) Armstrong A-12 epoxy adhesive, which Measurements Group Inc. suggest should be used with high elongation strain gauges. This epoxy adhesive was used either cured for 2 hours at 165° F in an oven, or cured for 2 weeks at room temperature, as recommended.

2) Loctite 401, an ethyl cyanocrilate adhesive ("CN" adhesive), which Tokyo Sokki Kenkyujo Co suggest should be used. This adhesive is cured at room temperature.

The surface preparation of the bars before attaching the strain gauges followed the standard procedures recommended in the Departmental guidelines [15], plus these additional final steps:

- Abrade the specimen surface in a direction at 45° to the intended axis of strain measurement.

- Lightly abrade in a direction at 90° to the first abrasion

- Repeat the degreasing step.

The bonding of the gauges followed the standard procedures outlined in the appropriate instruction bulletin for the adhesive selected and in the Departmental guidelines.

The behaviour of these strain gauges, in combination with the two types of adhesives, is summarized in Table 3.1.

Table 3.1 Maximum Strains Measured by the Strain Gauges Tested with Different Adhesives.

ADHESIVE USED	MAXIMUM STRAIN MEASURED	
	EP-08-250BG-120	YL-5
A-12 cured in oven	19 %	19 %
A-12 cured at room temp.	8 %	7 %
Cyanocrilate "CN"	4 %	6 %

These test results showed that either of the two types of strain gauges investigated gave very good results if they are used with A-12 adhesive cured for two hours at 165° F in an oven. Strain gauges type YL-5 were finally selected as their delivery time was shorter.

3.4.2 Measurement of Strains on the Tie Bars

Six pairs of electrical resistance strain gauges were attached to each tie bar on opposite sides of the steel surface, and the readings of each pair of strain gauges were averaged to obtain the bar axial strain at this position. The position of the strain gauges on the tie bars is shown in Fig. 3.16.

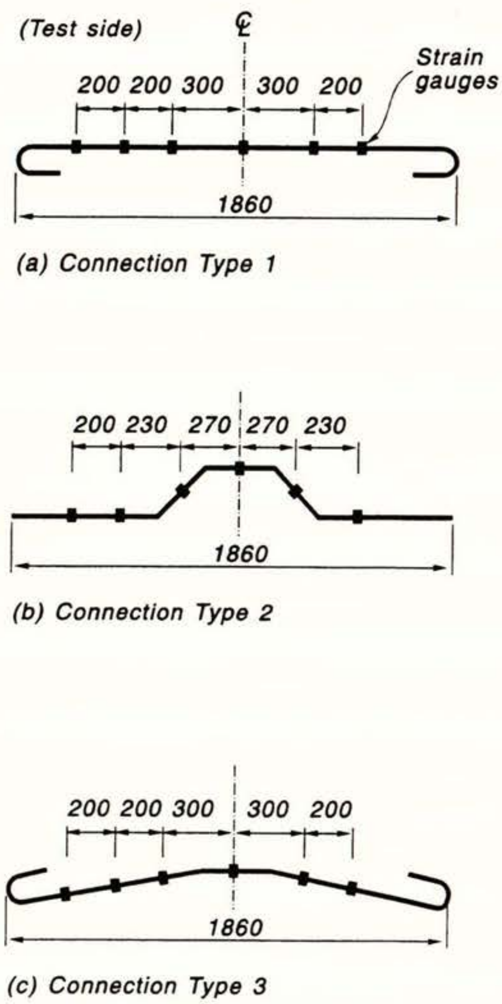


Fig. 3.16 Position of the Electrical Resistance Strain Gauges on the Tie Bars

3.4.3 Measurement of Applied Loads

During Tests A and B, the vertical applied load was measured using a 100 ton Phillips load cell. X-Y plotters were used to record continuously the applied load and measured vertical displacement of the connection relative to its supporting beam.

The horizontal loads applied at both sides of the specimens in the first stage of test B, were measured with two 44 ton load cells built in the Department of Civil Engineering.

Each load cell was connected both to the data logger unit and to a strain indicator in order to have two different sources of information to be used in case of any problem during the tests. All the load cells used were calibrated for each test, using an Avery Universal Testing Machine.

3.4.4 Measurement of Displacements.

In Tests A and B, the vertical displacement of the floor slab relative to the supporting beam at the connection was measured at both sides of the floor slab using two linear potentiometers of 300 mm travel. Additionally any possible rocking of the supporting beam at the connection tested was monitored using two linear potentiometers of 100 mm travel, placed at the top of the ends of the beam.

During the first stage of Test B, the imposed horizontal movement of the floor slab relative to its supporting beam was measured using two linear potentiometers of 100 mm travel, placed at both sides of the floor slab.

In Test A, the measurement of the crack width at the top of the specimen at the interface between the supporting beam and the floor slab, was made manually using three two inch demountable mechanical strain gauges (DEMEC) and a crack magnifying lens.

3.4.5 Data Logger Unit

All the circuits from the strain guages, load cells and linear potentiometers were connected to a Burr Brown data logger unit, which stored electronically all the scans and converted voltage changes into digital values.

3.5 Test Procedures

3.5.1 Preliminary Work

Before commencing each test, all the load cells and linear potentiometers used were calibrated. The strain gauges were checked for continuity and resistance to earth. A coat of white paint was applied to the specimens in the area surrounding the connections to be tested, to permit better crack observation.

3.5.2 Test A

This test involved applying the vertical knife edge load to the right hand span (span 3) of the specimen only (see Sec. 3.3.1). The left hand end of the span 3 at the interface with the support 3, was constructed with zero bearing on the supporting beam (see Sec. 3.2.5).

The vertical load was applied in small increments and the complete set of readings of load cells, potentiometers and strain gauges were recorded by the data logger at each increment. The vertical load versus vertical displacement measured at the connection was recorded continuously. Additional manual readings of crack widths in the topping slab at the joint between the faces of Dy-core unit and the supporting beam were taken using DEMEC gauges and the crack magnifier. The test finished when the load carried reduced significantly, due to splitting of the concrete.

3.5.3 Test B

(a) First Stage (Horizontal movement)

The connection at support 2 (see Fig. 3.1) was loaded horizontally at both sides of the floor unit by means of the horizontal loading system applied to the left hand span (see Sec. 3.3.2). During this loading the transverse joint between the faces of the floor slab and the support 2 cracked and the crack widened as the floor slab slid on that support. From the readings of the two horizontal potentiometers placed at both sides of the floor slab (span 1), an even displacement (uniform transverse crack width) was obtained by the forces applied by the two rams of the horizontal loading system. The span 1 was forced to slide horizontally until its right hand end pulled off its interior support (support 2). This involved a 55 mm horizontal movement of that span resulting in a 55 mm wide crack in the topping slab at the face of support 2. This horizontal movement was imposed since it was equal to the total seating length of the floor in the direction of the span (50 mm, being the manufacturer's recommendation) plus 5 mm additional movement to avoid friction between the end of the floor unit and the face of the supporting beam during the vertical displacement to follow (second stage of test B). Hence the vertical force carrying the floor unit at this support could only be provided by the vertical component of the ties bars by kinking.

(b) Second Stage (Vertical movement)

After the preselected horizontal movement was reached (55 mm), keeping the horizontal position of the unit constant, the applied vertical knife edge load on the span was successively increased using the vertical loading system (see Sec. 3.3.2). The test finished when one of the tie bars of the connection fractured. During the vertical displacement the applied horizontal forces were still acting to maintain the horizontal position of the unit.

At each increment of load, the readings of all the load cells, potentiometers and strain gauges were recorded in the data logger unit. The vertical load versus vertical displacement measured at the connection was recorded continuously.

Chapter 4
THE MECHANICAL PROPERTIES OF THE MATERIALS USED

4.1 Dy-core Units

The Dy-core units were manufactured by Precision Precasting Ltd, of Christchurch. A very dry concrete mixture is used in the manufacturing of Dy-core units, as the production technique of extrusion requires the use of such a concrete. Several attempts were made to take concrete cylinders at the casting line of the Dy-core units, but it was impossible to achieve a similar compaction of the concrete as was achieved by the production machine of the precast units. As a consequence, the results of the compression tests of the concrete cylinders did not represent the quality of the concrete of the Dy-core units. According to the manufacturer, the minimum 28 days compression strength f_c' of the concrete used in the production of the Dy-core units was 40 MPa.

Table 4.1. Tests Results of the 12.5 mm Diameter Strand Used for Pretensioning the Dy-core Units.

Item	Units	Specification	Results	
			Min.	Max.
Strand Diameter	mm	12.5 +0.4, -0.2	12.40	12.45
Steel Area	mm^2 (%)	93 (+4, -2)	93.15	93.41
Stranding Pitch	x D	12-18	13.4	13.6
Weight per 1000 m	kg (%)	730 (+4, -2)	734	737
Breaking Load	kN	Charact. 164	167.7	170.6
Proof Load at 0.1%	kN	Charact. 139	153.0	157.4
Elongation over 600 mm at Fracture	%	Min. 3.5	6.5	7.6
Modulus of Elasticity	kN/mm^2	185-205	190.2	191.2

Uncoated 7 wire stress-relieved strands were used as prestressing strands, which comply with BS 5896: 1980 [17]. Test results provided by the producer (Shinko Wire Company, Ltd., Japan) of the 12.5 mm diameter strand used in the construction of the Dy-core units are presented in Table 4.1.

4.2 Concrete Cast in the Laboratory

The concrete used in the construction of the specimens was provided by a commercial supplier. A compressive cylinder strength of 25 MPa at 28 days was specified. The maximum specified aggregate size was 19 mm for the supporting beams and 13 mm for the cast-in-place topping slab.

Table 4.2 Concrete Properties

Specimen	Test	Location	Slump [mm]	f'_c [MPa]		Age at Test [Days]
				At 28 Days	At Test	
1	A	Supporting Beams	10	29	31	81
		Topping	18	36	36	58
	B	Supporting Beams	10	29	31	74
		Topping	18	36	36	51
2	A	Supporting Beams	15	36	40	52
		Topping	17	26	26	28
	B	Supporting Beams	15	36	39	72
		Topping	17	26	28	48
3	A	Supporting Beams	11	27	29	69
		Topping	20	28	30	35
	B	Supporting Beams	11	27	29	69
		Topping	20	28	28	28

At each pour of concrete, nine cylinders of 100 mm diameter by 200 mm high were cast to determine the mechanical properties of the concrete, which are summarized in Table 4.2. The compressive strength f'_c is the average of three tests.

4.3 Steel Bars

The reinforcement used in connections Types 1 and 3 (Specimens 1 and 3), were 16 mm diameter plain round bars from Grade 300 steel. All were from the same production batch. Six samples were randomly taken for tensile tests.

The reinforcement used in connection Type 2 (Specimen 2), were 16 mm diameter deformed bars from Grade 300 steel. All were from the same production batch. Three samples were randomly taken for tensile tests.

Table 4.3 summarizes the results of the tensile tests of the steel bars.

Table 4.3 Tensile Properties of the Steel Bars

DESCRIPTION	Yield Strength [MPa]			Tensile Strength [MPa]			TS/YS* Ratio		Elongation** [%]	
	Min.	Aver	Max.	Min.	Aver	Max.	Min.	Max.	Min.	Max.
R16 Grade 300	301	317	328	468	476	480	1.46	1.53	20	23
D16 Grade 300	308	310	313	463	463	463	1.48	1.50	21	23
Specified [9]	275	-	380	-	-	-	1.15	1.50	20	-

Note: The specified [9] lower and upper characteristic yield strengths of Grade 300 steel are 300 and 355 MPa, respectively.

* TS/YS = (Tensile Strength)/(Yield Strength) Ratio

** Percentage of elongation at fracture measured outside the necking region over a gauge length of 100 mm.

The average modulus of elasticity (E_s) found was 204 GPa.

4.4 Steel Mesh Fabric

The steel mesh (electrically welded fabric) used in the construction of the specimens was produced by Wiremakers Limited, Auckland, using plain hard drawn steel wire, complying with NZS 3422 : 1975 [16]. The dimensions of the square mesh used are summarized in Table 4.4.

Table 4.4 Dimensions of the Square Mesh Used

Mesh Type	Nominal Pitch* [mm]	Main and Cross Wire		Square Mesh	
		Diameter [mm]	Cross-Sectional Area [mm^2]	Cross-Sectional Area per metre [mm^2/m]	Mass/m^2 [kg/m^2]
665	150	5.3	22.06	145	2.273

* Distance from centre to centre of two adjacent wires.

The mechanical properties of the wires used for the manufacture of the mesh fabric were found from tensile tests, as specified in NZS 3422 : 1975 [16].

Six test samples cut from longitudinal wires of the square mesh, containing one or more welds in its length, were randomly taken and tested. The results of these tests are presented in Table 4.5.

Table 4.5 Mechanical Properties of Wire from the Welded Square Mesh

Description	Diameter of Wire [mm]	Minimum 0.2% Proof Stress [MPa]	Tensile Strength [MPa]		Elongation* [%]		
			Min.	Max.	Min.	Aver	Max.
Sample Tests	5.3	551	585	639	1.3	2.7	4.3
Specified [16]	Over 3.15	485	575	775	-	-	-

* Percentage elongation at fracture measured outside of the necking region over a gauge length of 50 mm.

Table 4.5 shows that the wire of the welded square mesh complies with NZS 3422 : 1975 [16]. Nevertheless, the standard does not have any requirement for elongation at fracture. The tests exhibited a very poor elongation capacity, which suggests that this reinforcement should not be considered effective for preventing progressive collapse of structures after an initial failure, or for use in structures which eventually may be subjected to large displacements.

Chapter 5

RESULTS AND ANALYSIS OF RESULTS FROM TEST A

5.1 Basis for the Analysis of the Results from Test A

5.1.1 Test Loading

In Test A no horizontal displacement is imposed but the left hand end of the Dy-core unit is positioned without bearing on the supporting beam. The vertical load on the span is successively increased and the shear strength and the vertical load versus vertical displacement relationship at the left hand end is measured (see Test A in Fig. 3.1). The vertical reaction of the floor unit at this end is mainly provided by a shear-friction mechanism.

5.1.2 Shear Friction

According to ACI 318 [11], the shear force V_n transferred by shear friction across a crack by interface roughness, in the general case when the shear-friction reinforcement is inclined at an angle of other than 90° to the shear plane and such that the shear force produces tension in the shear-friction reinforcement, is given by

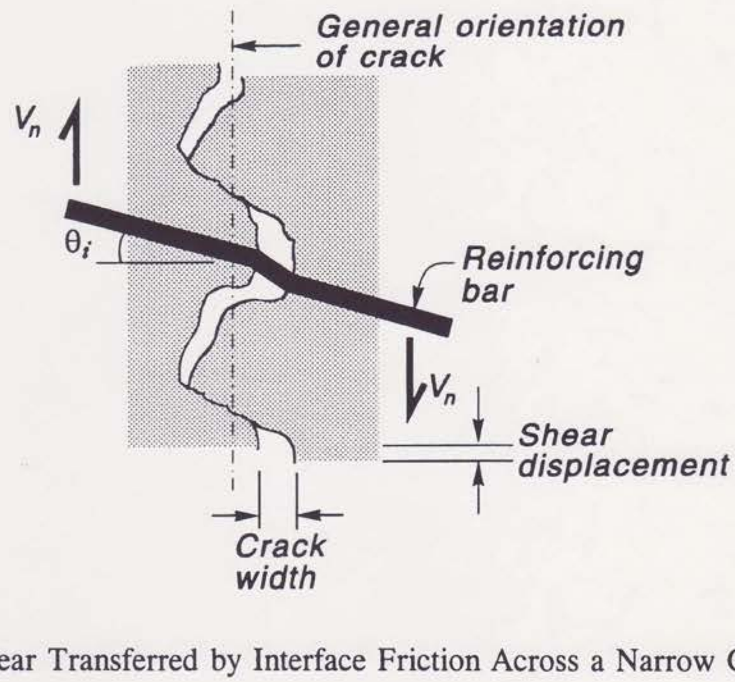


Fig. 5.1 Shear Transferred by Interface Friction Across a Narrow Crack

$$V_n = A_{vf} f_y (\mu \cos \theta_i + \sin \theta_i) \tag{5.1}$$

where

μ = the coefficient of friction

= 1.4 for a crack in concrete placed monolithically, or

= 1.0 for a crack at the interface of concrete placed against hardened concrete which is clean, free of laitance and intentionally roughened to a full amplitude of at least approximately 6 mm, or

= 0.6 for concrete placed against hardened concrete which is clean and free of laitance but is not intentionally roughened to a full amplitude of at least approximately 6 mm

A_{vf} = area of shear-friction reinforcement

f_y = yield strength of the shear-friction reinforcement

θ_i = angle of the shear-friction reinforcement to the normal to the shear plane (see Fig. 5.1)

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 [20]. Mattock [21] has also reported that bending moments acting on the shear plane equal to or less than the flexural strength of the shear plane do not reduce the shear which can be transferred by shear friction. Hence Eq. 5.1 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, since the crack width is also small. Hence the shear transferred by dowel action across the crack is negligible, since the mobilisation of dowel action requires a larger shear displacement [22]

5.1.3 Forces Acting on the Floor Unit During Test A

Fig. 5.2a shows the forces acting on span 3 at any instant (scan i) during Test A, and

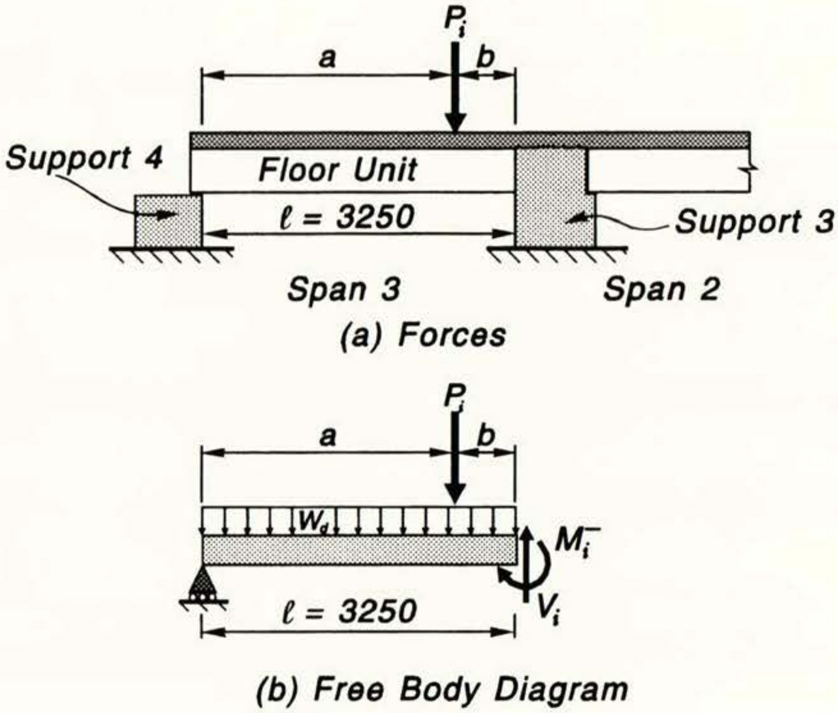


Fig. 5.2 Forces Acting in Span 3 During Test A

Fig. 5.2b shows the corresponding free body diagram, where

P_i = Applied vertical load by the ram

W_d = Weight of the Dy-core unit plus 65 mm thick concrete topping

= 2.8 kN/m (Dy-core unit) + 23.5 kN/m^3 x 1.2 m x 0.065 m (topping)

= 4.6 kN/m

V_i = Vertical reaction at the face of support 3

M_i = Negative bending moment at the face of support 3 due to the mesh and ties.

The vertical reaction at the face of support 3 can be calculated by statics from the applied vertical load and the geometry of the loading shown in Fig. 5.2b:

$$V_i = \frac{a \cdot P_i}{3.25} + \frac{3.25\ W_d}{2} + \frac{M_i}{3.25}$$

Replacing and simplifying:

$$V_i = \frac{a \cdot P_i}{3.25} + \frac{M_i}{3.25} + 7.5 \text{ [kN]} \tag{5.2}$$

For connections type 1 and 2 $a = 2.725$ m,

$$\therefore\ V_i = 0.839\ P_i + \frac{M_i}{3.25} + 7.5 \text{ [kN]} \tag{5.3}$$

For connection type 3 $a = 2.344$ m,

$$\therefore\ V_i = 0.721\ P_i + \frac{M_i}{3.25} + 7.5 \text{ [kN]} \tag{5.4}$$

5.2 Connection Type 1

5.2.1 Results for Test A

The relationship between the applied vertical load and the vertical displacement at the end of the unit measured during Test A on connection Type 1 is shown in Fig. 5.3. At 26 kN the left end of the unit, which was without bearing, reached a vertical displacement of 0.1 mm. This displacement remained practically constant up to a vertical load of 197 kN after which the vertical displacement increased to 0.2 mm. The first crack (induced by negative moment) was observed on the top surface of the topping slab at the joint above the end of the hollow-core unit at an applied load of 165 kN and a vertical displacement of 0.1 mm. When the applied vertical load was 365 kN, at a vertical displacement of 1.1 mm, cracks between the topping slab and the unit propagated near the support and the load carrying capacity reduced. With further increase in displacement the applied vertical load increased again and reached 374 kN at a vertical displacement of 2.2 mm at which stage the topping slab lifted and a crack opened on the underside of the hollow-core unit (due to positive moment). The unit finally failed due to loss of anchorage of the prestressing tendons. Fig. 5.4 shows the critical crack at the end of the test.

5.2.2 Analysis of Test Results

To find the maximum shear resisted by the connection using Eq. 5.3, it is necessary

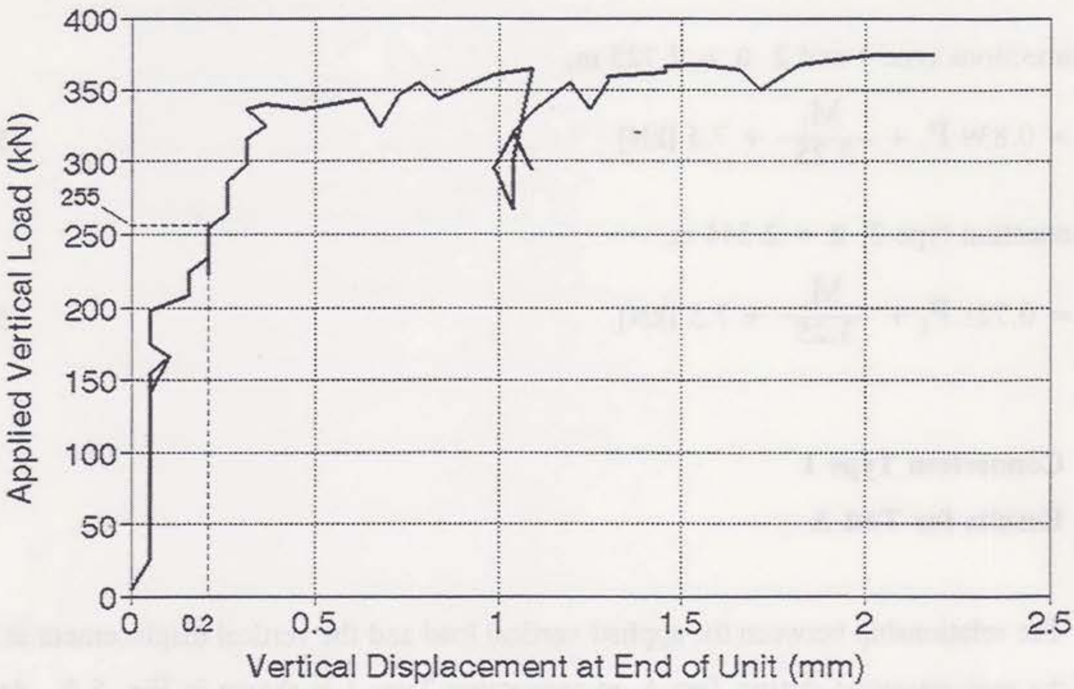


Fig. 5.3 Applied Vertical Load Versus Vertical Displacement at the End of Unit Measured During Test A on Connection Type 1

Fig. 5.4 Connection Type 1 at End of Test A

first to calculate the negative bending moment acting at the support (due to the mesh and ties) when the connection resisted the maximum applied vertical load. This moment is maximum when the connection reaches its flexural strength, which can be calculated from Fig. 5.5 and the following data

Mesh: $A_{s1} = 176 \text{ mm}^2$
$f_{y1} = 551$ MPa (actual measured)
$E_{s1} = 200000$ MPa
$\varepsilon_{y1} = 551/200000 = 0.0026$

Ties: $A_{s2} = 402 \text{ mm}^2$
$f_{y2} = 317$ MPa (actual measured)
$E_{s2} = 204000$ MPa
$\varepsilon_{y2} = 317/200000 = 0.0016$

Concrete $f'_c = 36$ MPa (actual measured)

For strain compatibility, and considering that the New Zealand and ACI codes [10,11] assume for the case with flexure that the strength of the member is reached when the extreme fibre concrete compression strain is 0.003, the strain diagram (see Fig. 5.5) gives

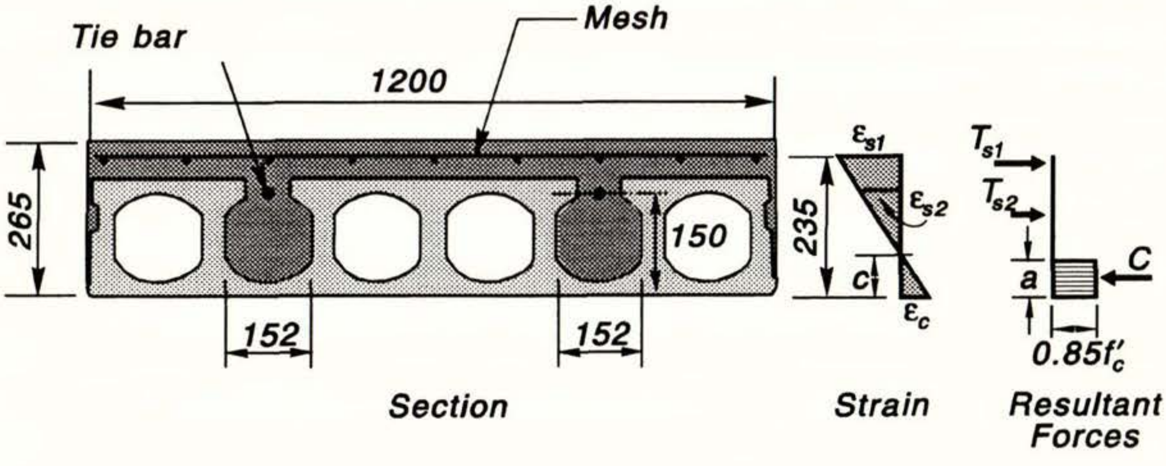


Fig. 5.5 Section at Flexural Strength on Test A of Connection Type 1

$$\frac{0.003}{c} = \frac{\varepsilon_{s1}}{(235 - c)} = \frac{\varepsilon_{s2}}{(150 - c)}$$

$$\therefore\ \varepsilon_{s1} = 0.003\,\frac{235 - c}{c} \tag{5.5}$$

$$\varepsilon_{s2} = 0.003\,\frac{150 - c}{c} \tag{5.6}$$

In common with reinforced concrete theory, the actual curved distribution of concrete compressive stress is replaced by an equivalent rectangular stress block, which has a mean stress of 0.85 f'_c and a depth of **a**. The depth of the rectangular stress block **a** may be related to the neutral axis depth **c** through a coefficient β_1 = **a/c**. In the New Zealand code [10], β_1 is taken as 0.85 when $f'_c \leq 30$ MPa. For $f'_c > 30$ MPa, β_1 is reduced continuously at a rate of 0.04 for each 5 MPa of strength in excess of 30 MPa, but β_1 is not taken as less than 0.65.

At flexural strength the internal concrete force is $C = 0.85 f'_c ab$, where b is the section width = 2 x 152 mm = 304 mm (see Fig. 5.5), and $a = \beta_1 c$. For $f'_c = 36$ MPa, $\beta_1 = 0.85 - (36\text{-}30)\text{x}0.04/5 = 0.802$. From Fig. 5.5, the internal steel forces are $T_{s1} + T_{s2} = A_{s1}f_{s1} + A_{s2}f_{s2}$, where f_{s1} and f_{s2} are the stresses in the steel (mesh and ties) at flexural strength. For equilibrium, we have

$$C = T_{s1} + T_{s2}$$

$$\therefore\ 0.85\, f'_c\, \beta_1 c\, b = A_{s1}f_{s1} + A_{s2}f_{s2} \tag{5.7}$$

Let c = 30.1 mm, from Eqs. 5.5 and 5.6 we have

$$\varepsilon_{s1} = 0.003\,\frac{235 - 30.1}{30.1} = 0.0204 > 0.0026 \quad \text{then } f_{s1} = f_{y1} = 551 \text{ MPa}^*$$

$$\varepsilon_{s2} = 0.003\,\frac{150 - 30.1}{30.1} = 0.0120 > 0.0016 \quad \text{then } f_{s2} = f_{y2} = 317 \text{ MPa}^*$$

* neglecting strain hardening

Check equilibrium using Eq. 5.7

$0.85 \times 36 \times (0.802 \times 30.1) \times 304 = 176 \times 551 + 402 \times 317$

224.6 kN = 224.4 kN the equilibrium balance is satisfactory.

The flexural strength is given by

$$M_u = A_{s1} f_{s1} (d_1 - 0.5a) + A_{s2} f_{s2} (d_2 - 0.5a) \tag{5.8}$$

$$\therefore M_u = 176 \times 551 (235 - 0.5 (0.802 \times 30.1)) + 402 \times 317 (150 - 0.5 (0.802 \times 30.1))$$

$$M_u = 39.2 \text{ kN-m}$$

The maximum shear resisted by the connection calculated using Eq. 5.3 and considering P_i = 374 kN (maximum applied vertical load) and $M_i = M_u$ = 39.2 kN-m (negative bending moment equal to the flexural strength of the connection) is

$$V_{max} = 0.839 \times 374 + 39.2/3.25 + 7.5 = 333 \text{ kN}$$

If for simplicity the floor unit is considered simply supported ($M_i = 0$), then

$$V_{max} = 0.839 \times 374 + 7.5 = 321 \text{ kN}$$

This value is slightly smaller (less than 4%) than the calculated considering the maximum negative moment, and indicates that in the calculation of the reaction at the face of the support the floor unit can be considered to be simply supported as the values found are safe and almost equal. It is to be noted that the actual bending moment acting at the connection during the test can not be calculated since the strain in the mesh is unknown.

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. Hence in Eq. 5.1, μ = 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.

For the mesh: A_{vf} = 176 mm^2

f_y = 551 MPa (actual measured), but 415 MPa is the maximum allowed in shear friction calculations [11]

θ_i = 0°

For the ties: A_{vf} = 402 mm^2

f_y = 317 MPa (actual measured)

θ_i = 0°

From Eq. 5.1: V_n = (176 x 415 + 402 x 317) x 1.4
= 281 kN

This theoretical value can be compared with the 321 kN obtained from the test.

5.3 Connection Type 2

5.3.1 Results for Test A

Fig. 5.6 shows the relationship between the applied vertical load and the vertical displacement at the end of the unit measured during Test A on connection Type 2. The maximum applied vertical load reached was 238 kN at a vertical displacement of 0.4 mm when the crack width at the top of the topping slab above the end of the unit was 0.4 mm. Fig. 5.7 shows that at the end of the test the diagonal tension cracks propagated along the filled voids which anchored the tie bars in the interior of the hollow-core units.

5.3.2 Analysis of Test Results

The maximum shear resisted by the connection if the floor unit is considered to be simply supported is according to Eq. 5.3 equal to V_{max} = 0.839 x 238 + 7.5 = 207 kN at the face of the support. At this load the electrical resistance strain gauges showed that the tie bars had not reached the yield strength.

The magnitude of the vertical shear force carried by the tie connection in this test can be calculated using Eq. 5.1 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

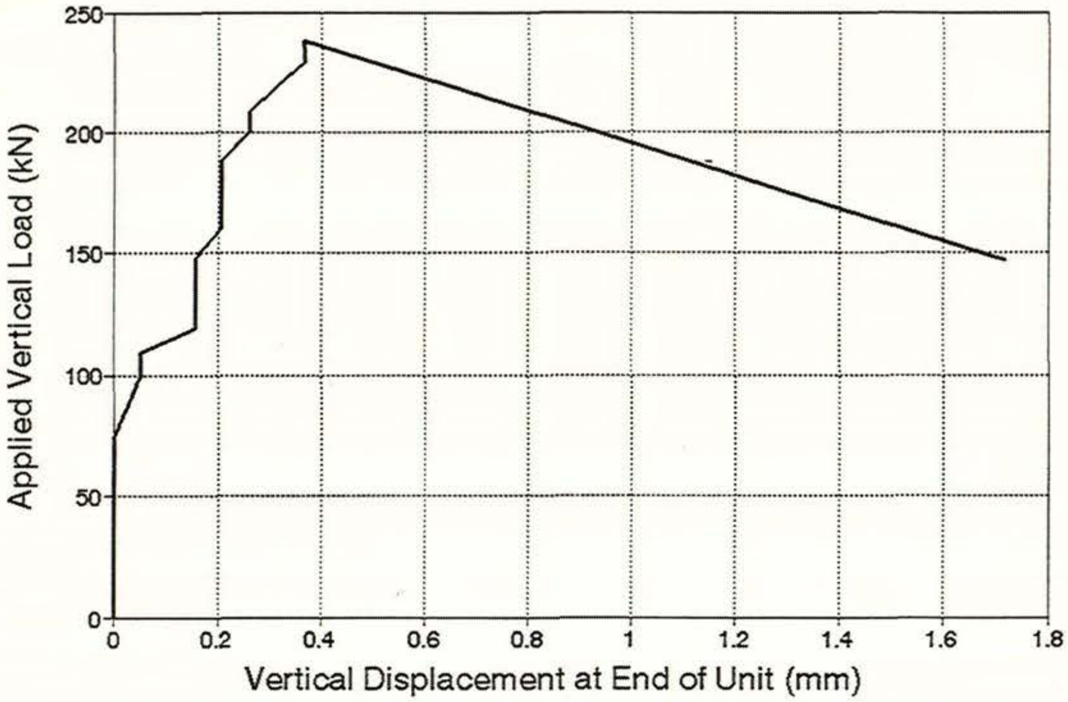


Fig. 5.6 Applied Vertical Load Versus Vertical Displacement at the End of Unit Measured During Test A on Connection Type 2

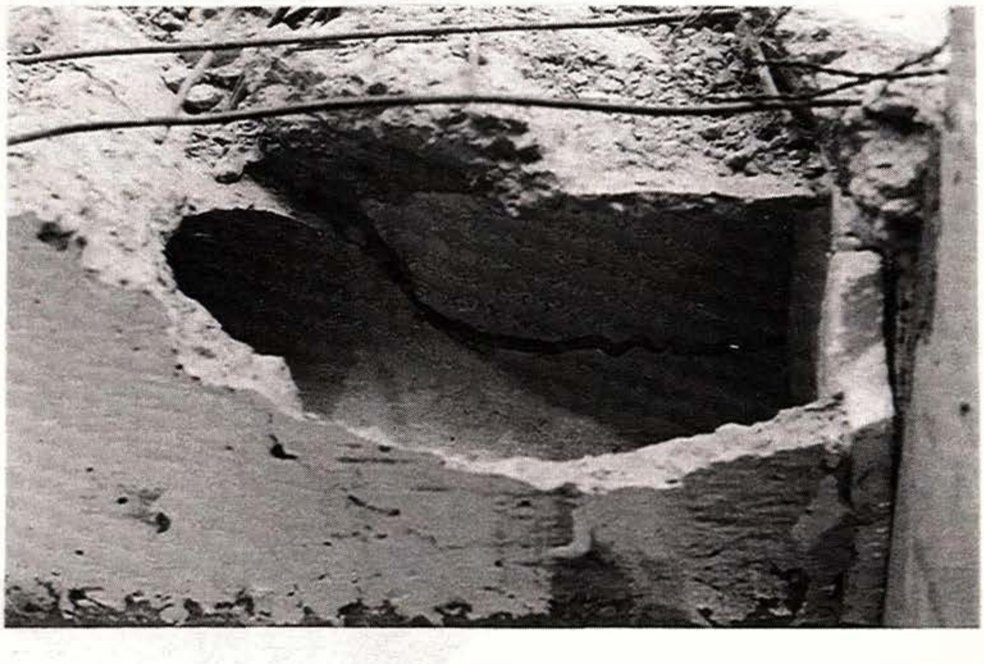


Fig. 5.7 At the End of Test A on Connection Type 2

hollow-core unit.

The value of the shear-friction coefficient μ 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 over the full depth of the end of the hollow-core units, and therefore 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 unit (see Fig. 5.8). For the cast-in-place topping slab

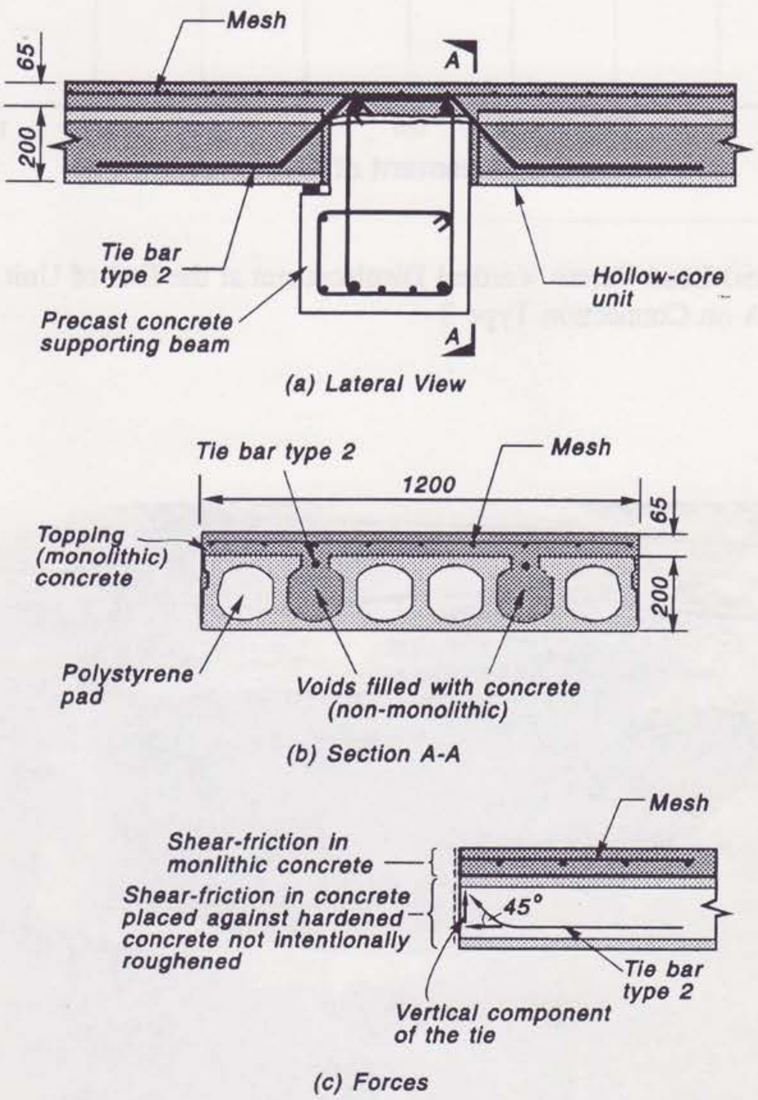


Fig. 5.8 Transfer of End Reaction by Shear Friction During Test A of Connection Type 2

(concrete placed monolithically) μ = 1.4 is appropriate [11]. 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 hence μ = 0.6 is appropriate [11] for that concrete. On average a value of μ = 1.0 might be appropriate and is used in the following calculation for Specimen 2.

For the mesh: A_{vf} = 176 mm^2
f_y = 551 MPa (actual measured), but 415 MPa is the maximum allowed in shear friction calculations [11].
θ_i = 0°

For the ties: A_{vf} = 402 mm^2
f_y = 310 MPa (actual measured)
θ_i = 45°

From Eq. 6.1: V_n = (176 x 415 + 402 x 310 x cos 45°) x 1.0 + (402 x 310 x sin 45°)
= 161.1 + 88.1 = 249 kN

This shear strength was not attained during the test. This was because splitting 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.

5.4 Connection Type 3

5.4.1 Results for Test A

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

Fig. 5.9 shows the relationship between the applied vertical load and the vertical displacement at the end of the unit measured during Test A on connection Type 3. The maximum applied load reached was 309 kN at a vertical displacement of 0.4 mm and when the crack width at the top of the topping slab above the end of the unit exceeded 0.4 mm.

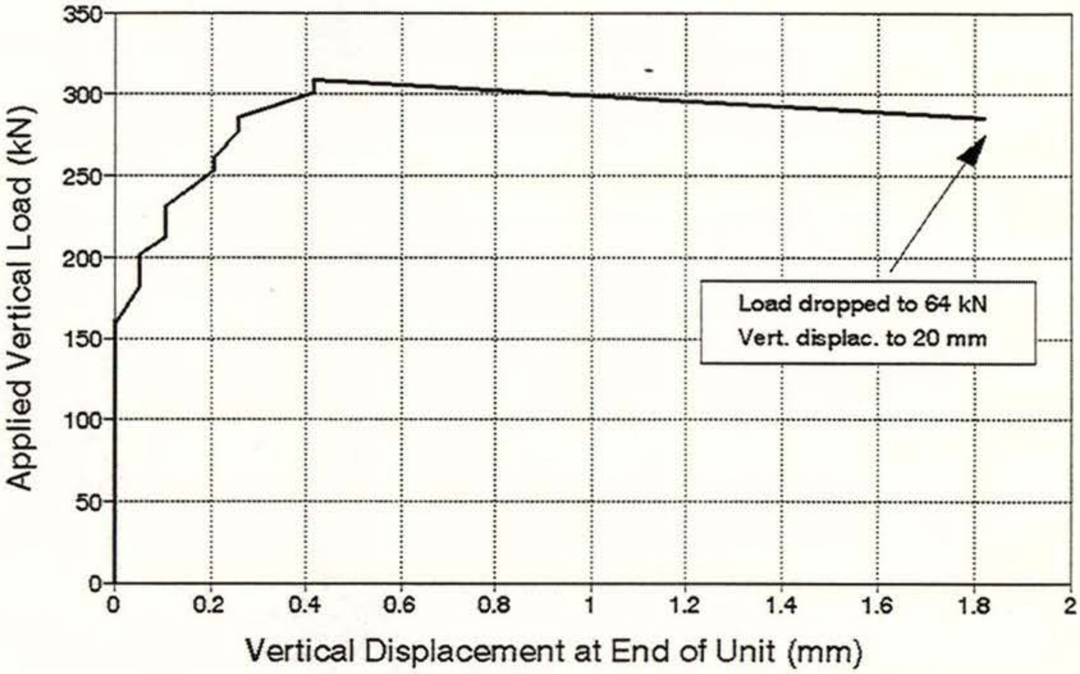


Fig. 5.9 Applied Vertical Load Versus Vertical Displacement at the End of Unit Measured During Test A on Connection Type 3

Fig. 5.10 At the End of Test A on Connection Type 3

The connection failed when diagonal tension cracks propagated as shown in Fig. 5.10.

5.4.2 Analysis of Test Results

The maximum shear resisted by the connection if the floor unit is considered to be simply supported is estimated using Eq. 5.4: $V_{max} = 0.721 \times 309 + 7.5 = 230$ kN at the face of the support.

The shear-friction strength for this test can be calculated using Eq. 5.1 as for connection Type 2, the only difference being that the ties were inclined at 13° to the horizontal in connection Type 3 and the yield strength (actual measured) of the ties was 317 MPa. Again $\mu = 1.0$ is assumed, as for connection Type 2.

$$\therefore\ V_n = (176 \times 415 + 402 \times 317 \times \cos 13^{\circ}) \times 1.0 + (402 \times 317 \times \sin 13^{\circ})$$
$$= 197.2 + 28.7 = 226 \text{ kN}$$

This predicted value compares well with the 230 kN obtained from the test.

Chapter 6

TEST RESULTS AND ANALYSIS OF RESULTS FROM TEST B

6.1 Basis for the Analysis of the Results from Test B

6.1.1 Test Loading

In Test B the Dy-core unit was first pulled horizontally off its supporting beam and then, keeping the horizontal position of the unit constant, the vertical knife edge load on the span is successively increased. The resulting vertical reaction of the floor unit versus the measured vertical displacement relationship at this support is obtained. The vertical reaction of the floor at this support in Test B can only be provided by the kinking of the tie bars.

6.1.2 Kinking

When the cracks 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 becomes kinking of the bars crossing the crack. The shear force V_n transferred by kinking across a vertical crack (see Fig. 6.1) is given by:

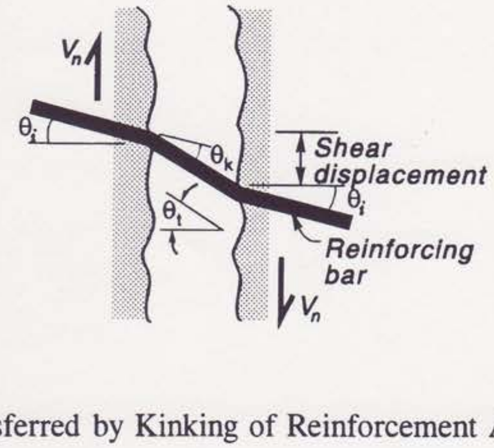


Fig. 6.1 Shear Transferred by Kinking of Reinforcement Across a Wide Crack

$$V_n = A_{vf}\, f_s \sin\theta_t \tag{6.1}$$

where

A_{vf} = area of reinforcement crossing the crack

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

θ_t = angle of the kinked reinforcement crossing the crack to the normal of the crack, being the sum $\theta_i + \theta$k, where θ_i is the initial angle of inclination of the bar to the normal of the crack and θ_k is the kinking angle of the bar (see Fig. 6.1b)

6.1.3 Equivalent Dynamic Shear Strength of the Connection

In a real situation when the floor unit slides horizontally off the support a constant shear force (equal to the static vertical reaction of the floor unit at this end) will instantaneously act on the connection, and the floor unit will begin to accelerate downwards. This real situation can be contrasted with the second stage of Test B where the connection is forced downwards by a vertical applied load which is measured during the test as the downward displacement of the floor is increased incrementally. In the real situation, depending on the resistance developed by the tie connection as the floor unit falls, the acceleration may decrease and the floor unit may even slow down and stop before fracture of the tie bars occurs. This is only possible if the strain energy $W_{int,i}$ absorbed and stored by the tie connection during a vertical displacement δ_i (which is represented by the shaded area under the curve in Fig. 6.2) equals the external work done W_{ext} by the vertical reaction of the floor unit at this end V_d, acting throughout the vertical movement considered, δ_i. Therefore to find the maximum shear force which can be transferred during dynamic loading $V_{d,max}$, the following condition has to be satisfied:

$$W_{ext} = W_{int,max}$$

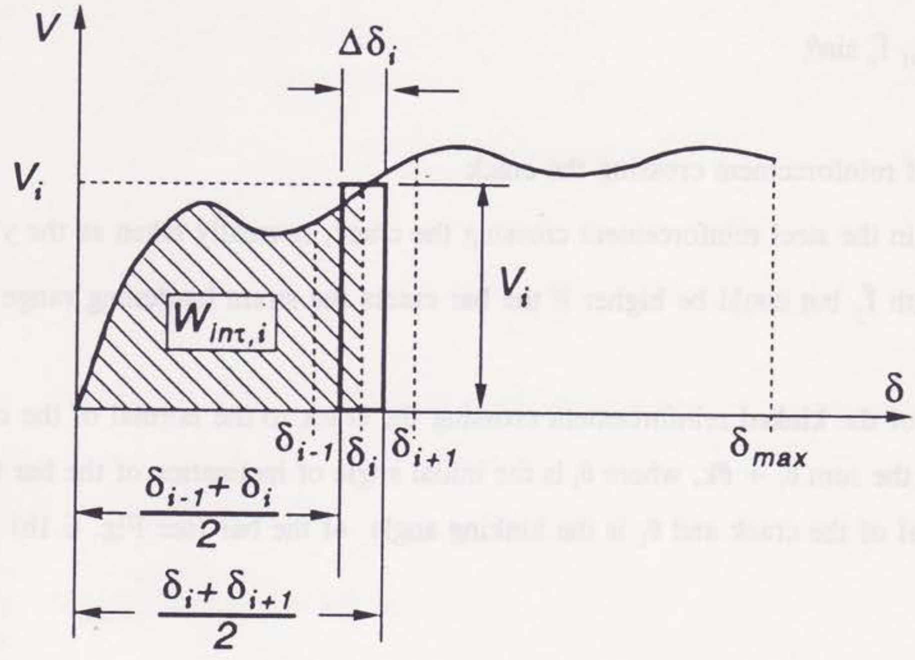


Fig. 6.2 Typical Relationship Between Vertical Shear Force at the Support and the Vertical Displacement at End of Unit

Now $W_{ext} = V_{d,max} \cdot \delta_{max}$

$$\therefore \quad V_{d,max} = \frac{W_{int,max}}{\delta_{max}} \tag{6.2}$$

where:

$W_{int,max}$ = maximum strain energy which can be stored by the connection, which is equal to the total area under the curve in Fig. 6.2

W_{ext} = external work done by $V_{d,max}$ acting a distance δ_{max}

δ_{max} = maximum vertical displacement reached by the connection before failure.

The analysis of the dynamic response of floor units subjected to the sudden loss of a support using load-displacement relationships measured during static load tests has been verified by Engstrom [8]. Experiments with ductile reinforcing steel show that with a high strain rate the yield point is higher and the amount of work necessary to produce fracture is greater than in a static test [18,19], which also indicates that the use of load-displacement relationships derived from static tests produce safe results when applied to the analysis of dynamic responses.

6.1.4 Forces Acting on the Floor Unit During Test B

Fig. 6.3a shows the forces acting on span 1 at any instant (scan i) of the second stage of Test B, and Fig. 6.3b shows the corresponding free body diagram, where:

T_i = Tensile force provided by the kinking of the tie bars acting across the vertical crack.

T_i = $A_{vf}\ f_{s,i}$, where A_{vf} is the area of the tie bars and $f_{s,i}$ is the tensile stress in the tie bars.

$T_{i,h}$ = Horizontal component of the tensile force in the tie bars, which is equilibrated by the force exerted by the two horizontal rams acting on the hinge mechanisms.

$T_{i,h}$ = $T_i \cos\theta_{t,i}$ = $A_{vf}\ f_{s,i} \cos\theta_{t,i}$, where $\theta_{t,i}$ is the angle of the kinked reinforcement (tie bars) crossing the crack to the normal of the crack.

V_i = Vertical component of the tensile force in the tie bars, which is equal to the shear force transferred by kinking of the tie bars across the open vertical crack. This mechanism provides the vertical reaction which supports the right end of span 1.

V_i = $T_i \sin\theta_{t,i}$ = $A_{vf}\ f_{s,i} \sin\theta_{t,i}$

P_i = Applied vertical load from the ram.

W_{d1} = Weight of the Dy-core unit plus the 65 mm thick concrete topping

= 2.8 kN/m (Dy-core unit) + 23.5 kN/m^3 x 1.2 m x 0.065 m (topping)

= 4.6 kN/m

W_{d2} = Weight of the concrete used to fill all the voids of the Dy-core unit at the left end of span 1

= 23.5 kN/m^3 x 6 voids x (0.15 m x 0.15 m) = 3.2 kN/m

$R_{A,i}$ = Vertical reaction at support 1.

$M_{A,i}$ = Moment due to the eccentricity of support 1 (point A) with respect to the hinge mechanism (point B)

= 0.31 x $R_{A,i}$ [kN-m]

The vertical force V_i which supports the floor unit at the right end during scan i, can be calculated by statics from the applied vertical load and the geometry of the loading shown

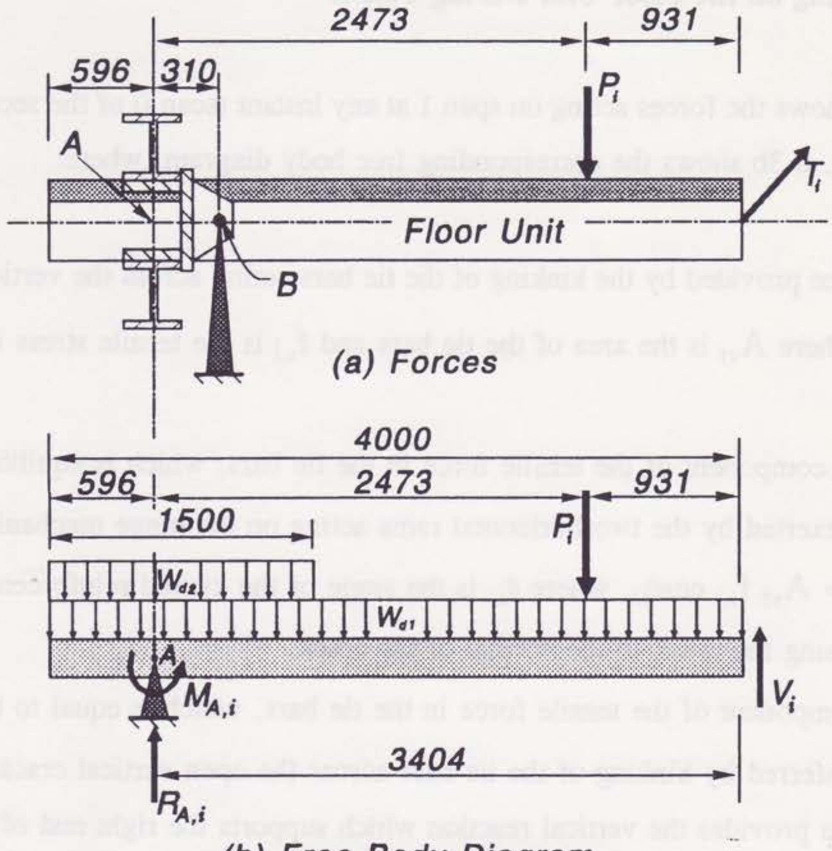


Fig. 6.3 Forces Acting in Span 1 During Second Stage of Test B

in Fig. 6.3b:

$$\Sigma M_A = 0$$

$$2.473P_i + \tfrac{1}{2}(4.0 - 0.596)^2 W_{d1} + \tfrac{1}{2}(1.50 - 0.596)^2 W_{d2} - M_{A,i} -$$

$$\tfrac{1}{2}(0.596)^2 (W_{d1} + W_{d2}) - 3.404V_i = 0$$

Replacing and simplifying:

$$2.473P_i - 0.31R_{A,i} - 3.404V_i + 26.573 = 0 \qquad (6.3)$$

$$\Sigma F_{vert.} = 0$$

$$4.0W_{d1} + 1.50W_{d2} + P_i - R_{A,i} - V_i = 0$$

Replacing and simplifying:

$$P_i - R_{A,i} - V_i + 23.20 = 0 \qquad (6.4)$$

From Eqs. 6.3 and 6.4 and simplifying:

$$V_i = 0.7P_i + 6.3 \quad [\text{kN}] \tag{6.5}$$

Therefore, the vertical reaction provided by the kinking of the tie bars at the right hand end of the floor unit during scan i (V_i), can be calculated using Eq. 6.5 by substituting the corresponding vertical force applied by the ram (P_i) at that instant.

6.1.5 Evaluation of the Equivalent Dynamic Shear Strength of the Connection

Initially it is necessary to evaluate the maximum strain energy stored by the connection ($W_{int,max}$), which as was mentioned before is equal to the total area under the curve of the plot of vertical reaction versus vertical displacement for the connection (see Fig. 6.2). If V_i represents the value of the vertical reaction at the connection at the scan i, and δ_i its corresponding vertical displacement, the work done by V_i when the vertical displacement changes to δ_{i+1} (next scan), is represented in Fig. 6.2 by the small rectangle whose dimensions and area are:

$$\text{Base} = \Delta\delta_i = \frac{\delta_{i+1} + \delta_i}{2} - \frac{\delta_i + \delta_{i-1}}{2} = \frac{\delta_{i+1} - \delta_{i-1}}{2}$$

$$\text{Height} = V_i$$

$$\text{Area}_i = \Delta\delta_i \cdot V_i = \frac{\delta_{i+1} - \delta_{i-1}}{2} \cdot V_i \tag{6.6}$$

The maximum strain energy stored by the connection ($W_{int,max}$) in the process of loading when the vertical reaction at the connection is increased from 0 to V_n, is the summation of such elemental areas:

$$W_{int,max} = \sum_{i=1}^{n} \text{Area}_i = \sum_{i=1}^{n} \frac{\delta_{i+1} - \delta_{i-1}}{2} \cdot V_i \tag{6.7}$$

where n is the number of scans taken during the test, $\delta_0 = 0$ and $\delta_{n+1} = \delta_n$.

Finally using Eqs. 6.2, 6.5 and 6.7 and considering that $\delta_{max} = \delta_n$, the maximum shear force which is possible to transfer in dynamic loading is:

$$V_{d,max} = \frac{\sum_{i=1}^{n} (\delta_{i+1} - \delta_{i-1}) \cdot (0.7P_i + 6.3)}{2\delta_n} \tag{6.8}$$

Eqs. 6.7 and 6.8 can be modified to find the strain energy stored by the connection $W_{int,j}$ and the equivalent dynamic shear capacity of the connection $V_{d,j}$, up to scan j (vertical displacement δ_j and an applied load P_j), considering $\delta_{j+1} = \delta_j$. Therefore:

$$W_{int,j} = \sum_{i=1}^{j} \frac{\delta_{i+1} - \delta_{i-1}}{2} \cdot V_i \tag{6.9}$$

$$V_{d,j} = \frac{\sum_{i=1}^{j} (\delta_{i+1} - \delta_{i-1}) \cdot (0.7P_i + 6.3)}{2\delta_j} \tag{6.10}$$

6.2 Connection Type 1

6.2.1 Results for Test B

(a) <u>First Stage:</u>

In the first stage of Test B the left hand span was loaded horizontally by a tensile force (Test B in Fig. 3.1). During this loading the transverse joint between the face of the Dy-core unit and the supporting beam at the right hand end of the span, cracked and opened up as the Dy-core unit slid horizontally on that support. The total horizontal tensile force reached a maximum value of 380 kN when the transverse crack width was 1.5 mm. By the end of the horizontal movement of 55 mm, this force was 164 kN and all the wires of the 665 welded wire mesh in the topping slab which crossed that crack had fractured.

Fig. 6.4 shows a plot of the applied horizontal load versus the horizontal displacement at the end of the unit during the first stage of Test B. The horizontal displacement is the average of the readings of two horizontal potentiometers placed at both sides of the end of

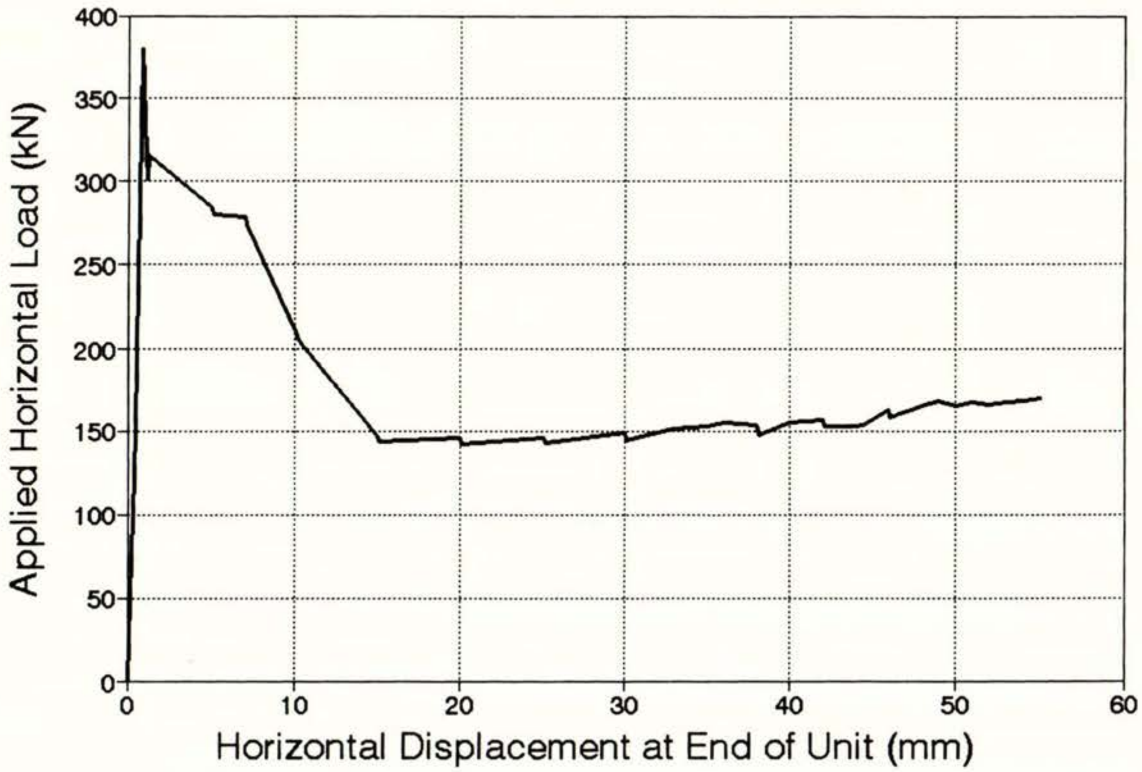


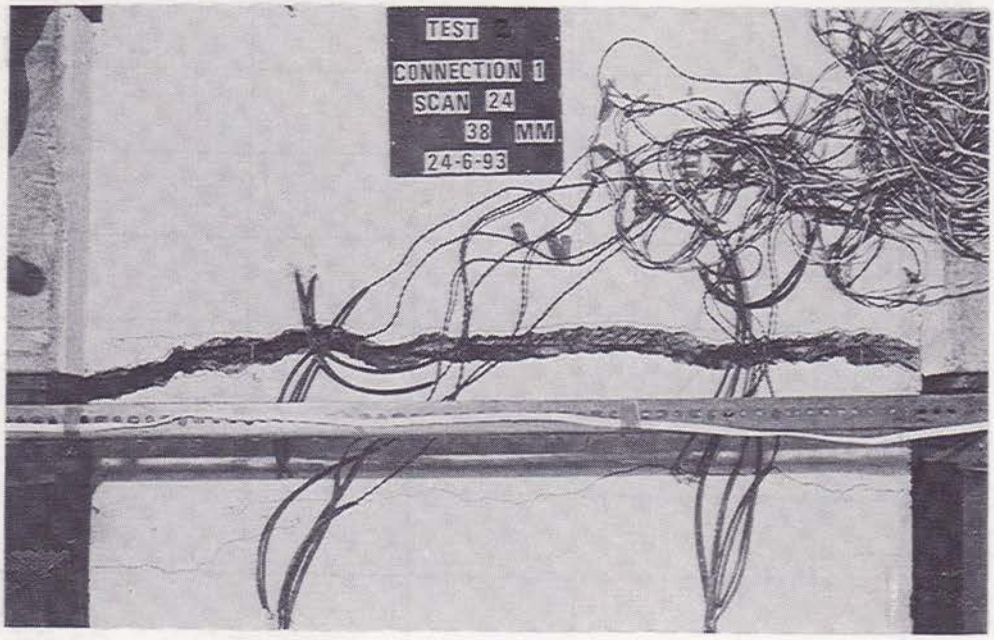
Fig. 6.4 Applied Horizontal Load Versus Horizontal Displacement at the End of Unit Measured During First Stage of Test B on Connection Type 1

the floor unit. Fig. 6.5a shows the opening of the transverse crack in the topping slab at the face of the Dy-core unit during the first stage of Test B.

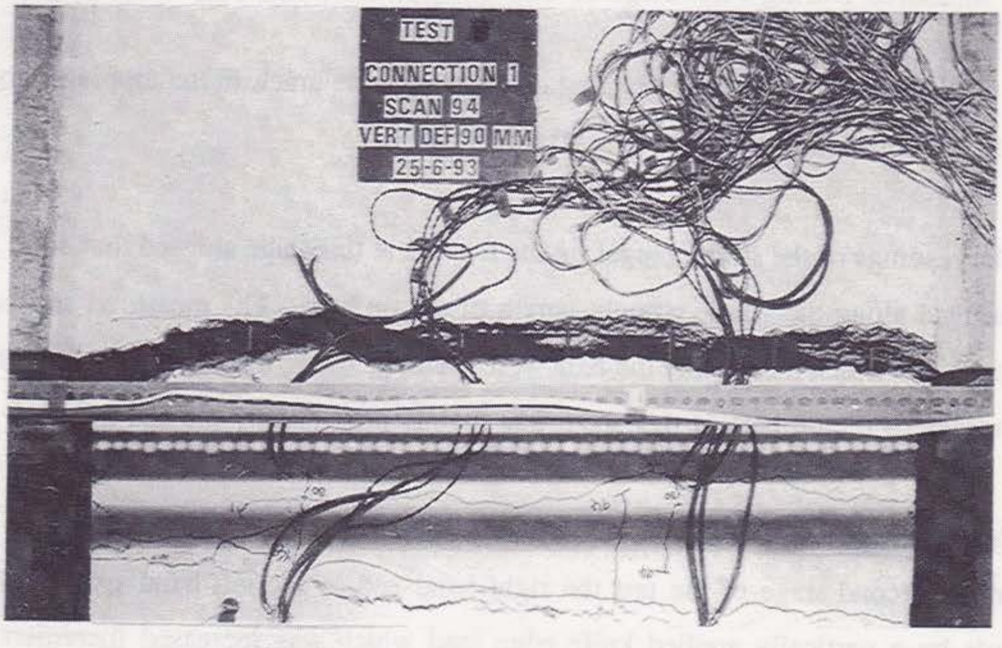
The readings of the strain gauges on the tie bars at this stage showed that bond failure had propagated along the whole straight length of the tie bars. The measured strains were larger than 2.5% which means that the bars were yielding and in the strain hardening region. The anchorage of the tie bars at this stage was mainly provided by the end hooks.

(b) Second Stage:

In the second stage of the test the right hand end of the left hand span was forced downwards by a vertically applied knife edge load which was increased incrementally as vertical displacement was forced on the connection. The horizontal displacement at the end of the unit was maintained at 55 mm during this stage of loading. The resulting cracks in the topping slab are shown in Fig. 6.5b. The applied vertical load was equilibrated totally by the vertical component of the tensile force in the ties. That is, the Dy-core unit was



a) Top Surface: Horizontal Movement = 38 mm
Vertical Movement = 0 mm



b) Top Surface: Horizontal Movement = 55 mm
Vertical Movement = 94 mm

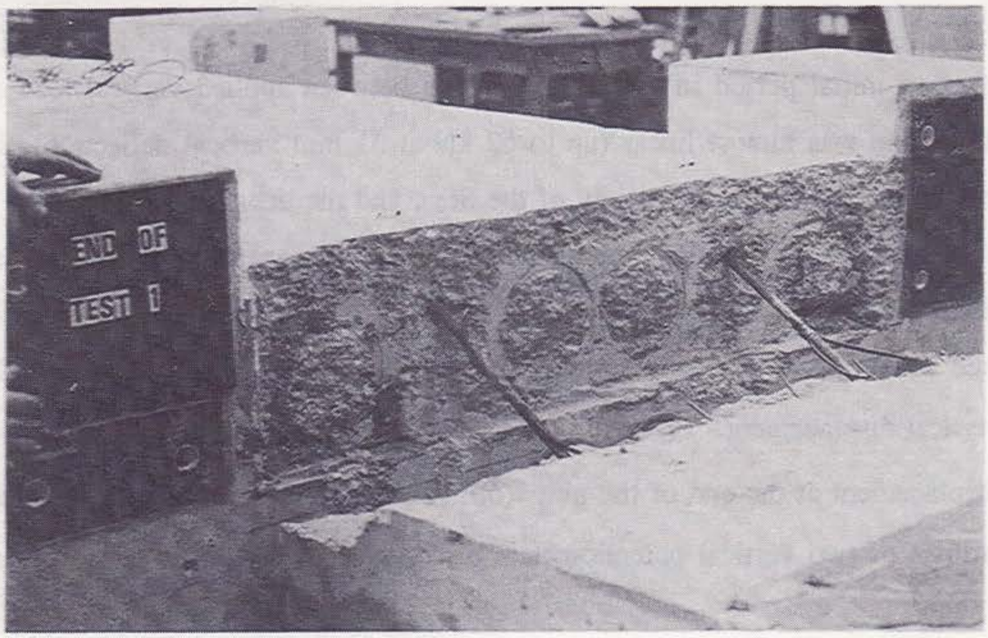
Fig. 6.5 Cracks Formed in the Topping Slab of Specimen 1 During Test B of Tie Connection Type 1 hanging from the ties, without friction at the supporting beam, since the end of the Dy-core unit was 5 mm clear of the supporting beam due to the horizontal displacement applied in the first stage of the test.

After an initial period in which the relation between applied vertical load and the vertical deflection was almost linear (up to 62 kN at 31 mm vertical deflection), splitting cracks occurred in the unit at the height of the ties , and the ties tore out as shown in Fig. 6.6. Up to this type of splitting failure the behaviour was reasonably ductile, since the deformation capacity and energy absorption were large, due to the design of the ties. The test finished when one of the ties fractured, at an applied vertical load of 96 kN and at 215 mm of vertical displacement. Fig. 6.7 shows a plot of the applied vertical load (P_i) versus vertical displacement at the end of the unit (δ_i). The vertical displacement is the average of the readings of two vertical potentiometers placed at both sides of the end of the floor unit.

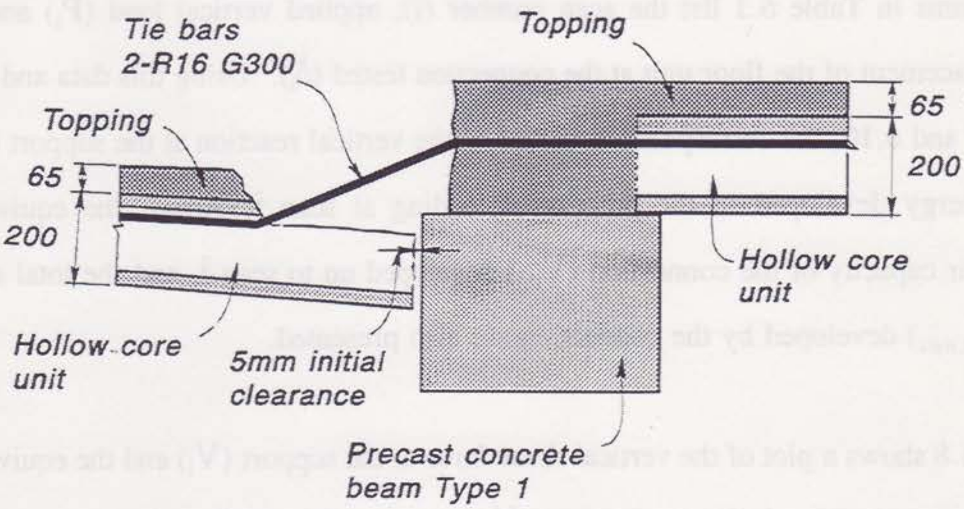
6.2.2 Analysis of Test Results

Table 6.1 presents some of the data recorded during the second stage of Test B. The vertical columns in Table 6.1 list the scan number (i), applied vertical load (P_i) and the vertical displacement of the floor unit at the connection tested (δ_i). Using this data and Eqs. 6.5, 6.6, 6.7 and 6.10, the corresponding values of the vertical reaction at the support (V_i), the strain energy developed in the process of loading at scan i ($Area_i$), the equivalent dynamic shear capacity of the connection ($V_{d,i}$) developed up to scan i, and the total strain energy ($W_{int,max}$) developed by the connection are also presented.

Fig. 6.8 shows a plot of the vertical shear force at the support (V_i) and the equivalent dynamic shear capacity of the connection ($V_{d,i}$) versus the vertical displacement (δ_i) measured during the second stage of the test. As can be seen in Fig. 6.8, the behaviour of the connection was quite ductile. The area under the curve of vertical shear force at the support (V_i) versus the vertical displacement (δ_i) of Fig. 6.8 up to when the tie fractured at



a) Failure Region



b) Mode of Failure

Fig. 6.6 End of Test B on Connection Type 1

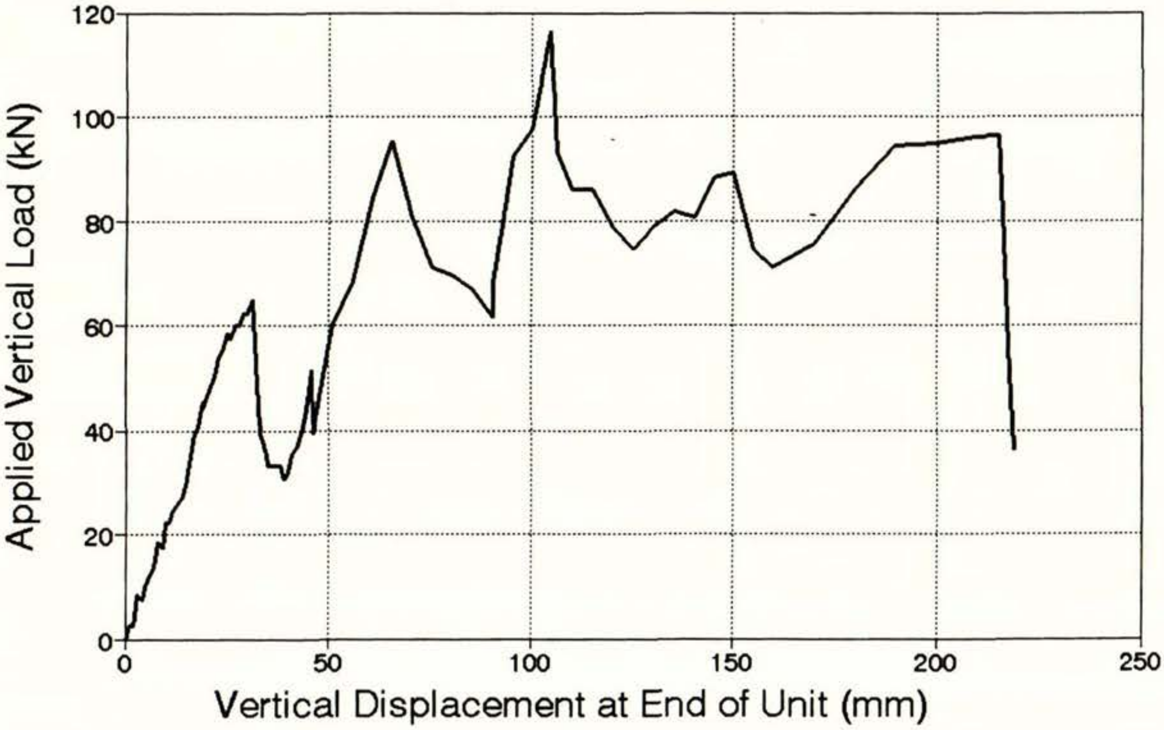


Fig. 6.7 Applied Vertical Load Versus Vertical Displacement at End of Unit Measured During the Second Stage of Test B on Connection Type 1

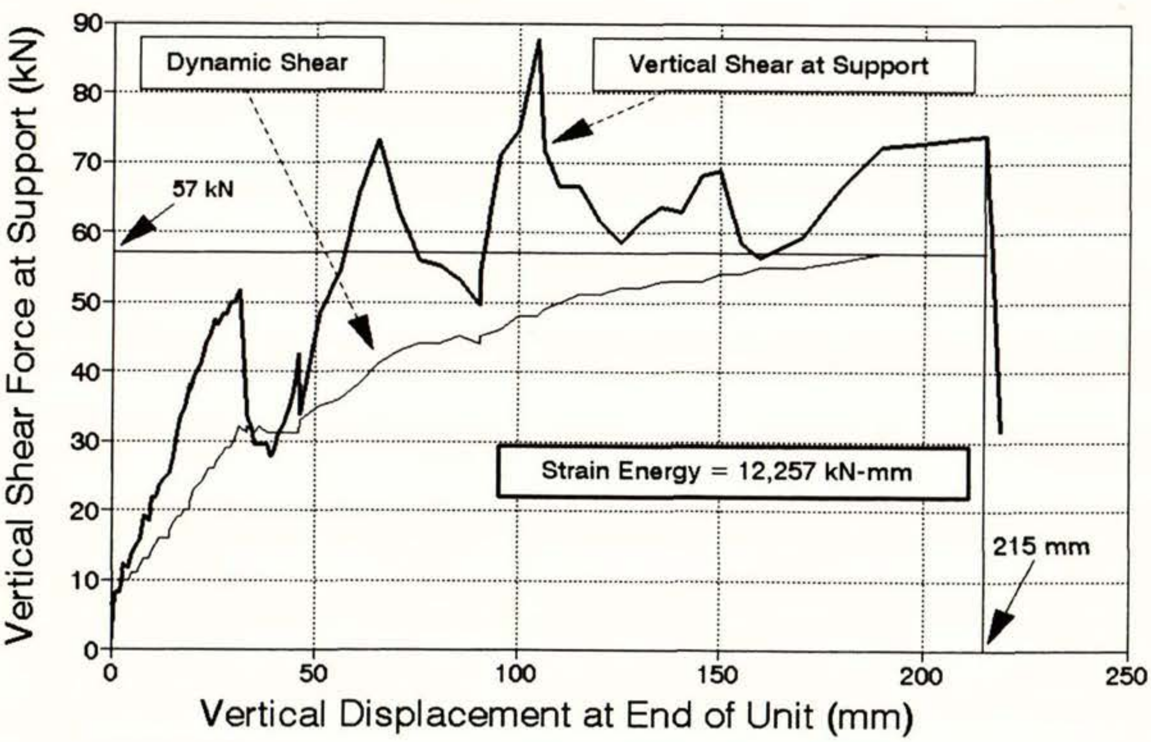


Fig. 6.8 Vertical Shear Force at the Support and Equivalent Dynamic Shear Capacity of the Connection Versus Vertical Displacement at the End of Unit Measured During the Second Stage of Test B on Connection Type 1

TABLE 6.1 CALCULATIONS FROM TEST A CONNECTION TYPE 1

i	P_i	V_i	δ_i	$Area_i$	$V_{d,i}$	i	P_i	V_i	δ_i	$Area_i$	$V_{d,i}$
	[kN]	[kN]	[mm]	[kNmm]	[kN]		[kN]	[kN]	[mm]	[kNmm]	[kN]
0	0.0	6.3	0.0	0.0	0	39	30.6	27.7	38.9	26.3	31
1	2.6	8.1	1.0	7.7	8	40	31.9	28.6	39.8	27.2	31
2	2.6	8.1	1.9	8.1	8	41	35.5	31.2	40.8	34.3	31
3	8.6	12.3	3.0	13.5	10	42	36.8	32.1	42.0	49.8	31
4	7.5	11.6	4.1	11.6	10	43	41.6	35.4	43.9	49.6	31
5	9.9	13.2	5.0	12.5	11	44	45.4	38.1	44.8	34.3	31
6	12.4	15.0	6.0	15.0	11	45	51.5	42.4	45.7	25.4	31
7	13.6	15.8	7.0	15.8	12	46	39.2	33.7	46.0	85.9	33
8	18.5	19.3	8.0	21.2	13	47	59.9	48.2	50.8	241.0	35
9	17.2	18.3	9.2	18.3	13	48	68.5	54.3	56.0	263.4	36
10	22.1	21.8	10.0	19.6	14	49	84.4	65.4	60.5	307.4	38
11	22.1	21.8	11.0	20.7	15	50	95.4	73.1	65.4	365.5	41
12	24.6	23.5	11.9	22.3	16	51	80.8	62.9	70.5	314.5	43
13	25.8	24.4	12.9	24.4	16	52	70.9	55.9	75.4	279.5	44
14	27.1	25.3	13.9	12.7	16	53	69.8	55.2	80.5	278.8	44
15	27.1	25.3	13.9	12.7	17	54	66.9	53.1	85.5	262.8	45
16	30.6	27.7	14.9	31.9	18	55	61.5	49.4	90.4	123.5	44
17	36.8	32.1	16.2	32.1	19	56	68.2	54.0	90.5	126.9	45
18	39.2	33.7	16.9	28.6	19	57	92.3	70.9	95.1	336.8	46
19	40.5	34.7	17.9	34.7	20	58	97.5	74.6	100.0	361.8	48
20	45.4	38.1	18.9	19.1	20	59	116.3	87.7	104.8	280.6	48
21	44.1	37.2	18.9	18.6	21	60	93.3	71.6	106.4	193.3	49
22	46.5	38.9	19.9	58.4	23	61	86.1	66.6	110.2	286.4	50
23	50.2	41.4	21.9	62.1	24	62	86.1	66.6	115.0	326.3	51
24	53.9	44.0	22.9	41.8	25	63	78.6	61.3	120.0	315.7	51
25	55.1	44.9	23.8	44.9	26	64	74.5	58.5	125.3	292.5	52
26	58.8	47.5	24.9	47.5	26	65	78.6	61.3	130.0	300.4	52
27	57.5	46.6	25.8	46.6	27	66	81.8	63.6	135.1	324.4	53
28	59.9	48.2	26.9	53.0	28	67	80.8	62.9	140.2	317.6	53
29	59.9	48.2	28.0	48.2	29	68	88.1	68.0	145.2	336.6	53
30	62.4	50.0	28.9	47.5	29	69	89.2	68.7	150.1	336.6	54
31	62.4	50.0	29.9	50.0	30	70	74.5	58.5	155.0	283.7	54
32	64.9	51.7	30.9	80.1	32	71	71.3	56.2	159.8	418.7	55
33	40.5	34.7	33.0	36.4	31	72	75.5	59.2	169.9	592.0	55
34	39.2	33.7	33.0	23.6	32	73	86.1	66.6	179.8	662.7	56
35	35.5	31.2	34.4	28.1	31	74	94.3	72.3	189.8	715.8	57
36	33.1	29.5	34.8	22.1	31	75	95.0	72.8	199.6	720.7	57
37	33.1	29.5	35.9	45.7	32	76	96.0	73.5	209.6	569.6	57
38	33.1	29.5	37.9	44.3	31	77	96.5	73.9	215.1	203.2	57

$W_{int,\,max}$ = 12,257 kN mm

a vertical displacement of 215 mm is equal to 12,257 kN mm. This area is equivalent to the total strain energy ($W_{int,max}$) developed by the connection. The maximum shear force which is possible to transfer in dynamic loading $V_{d,max}$ can be found using Eq. 6.2 :

$$V_{d,max} = \frac{W_{int,max}}{\delta_{max}} = \frac{12{,}257 \text{ kN mm}}{215 \text{ mm}} = 57 \text{ kN}$$

This value is equal to the one found in Table 6.1 using Eq. 6.10.

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 (see Fig. 6.8 or Table 6.1), the strains measured on the tie bars by the electrical resistance strain gauges was 0.05 which corresponded to a stress of f_s = 440 MPa, which was greater than the measured yield strength of 317 MPa due to the steel entering the strain hardening range. Also, A_{vf} = 402 mm^2. Therefore equation 6.1 gives

$$\text{Sin } \theta_t = \frac{88{,}000}{402 \text{ x } 440} = 0.498$$

$$\therefore \theta_t = 30^\circ$$

Note that $\theta_t = \theta_k$ in Fig. 6.1 since $\theta_i = 0$. On this basis the estimated angle of kinking of the tie bar at the maximum shear force was $\theta_k = 30^\circ$

6.3 Connection Type 2

6.3.1 Results for Test B

(a) First Stage:

Connection Type 2 failed in the first stage of the test while being pulled off the support before the pre-selected horizontal movement of 55 mm was reached. The maximum horizontal load resisted was 260 kN. During the horizontal movement the unit lifted due to the vertical component of force in the ties. Fig. 6.9 shows the measured relationships between the applied horizontal load and both the horizontal and vertical displacements.

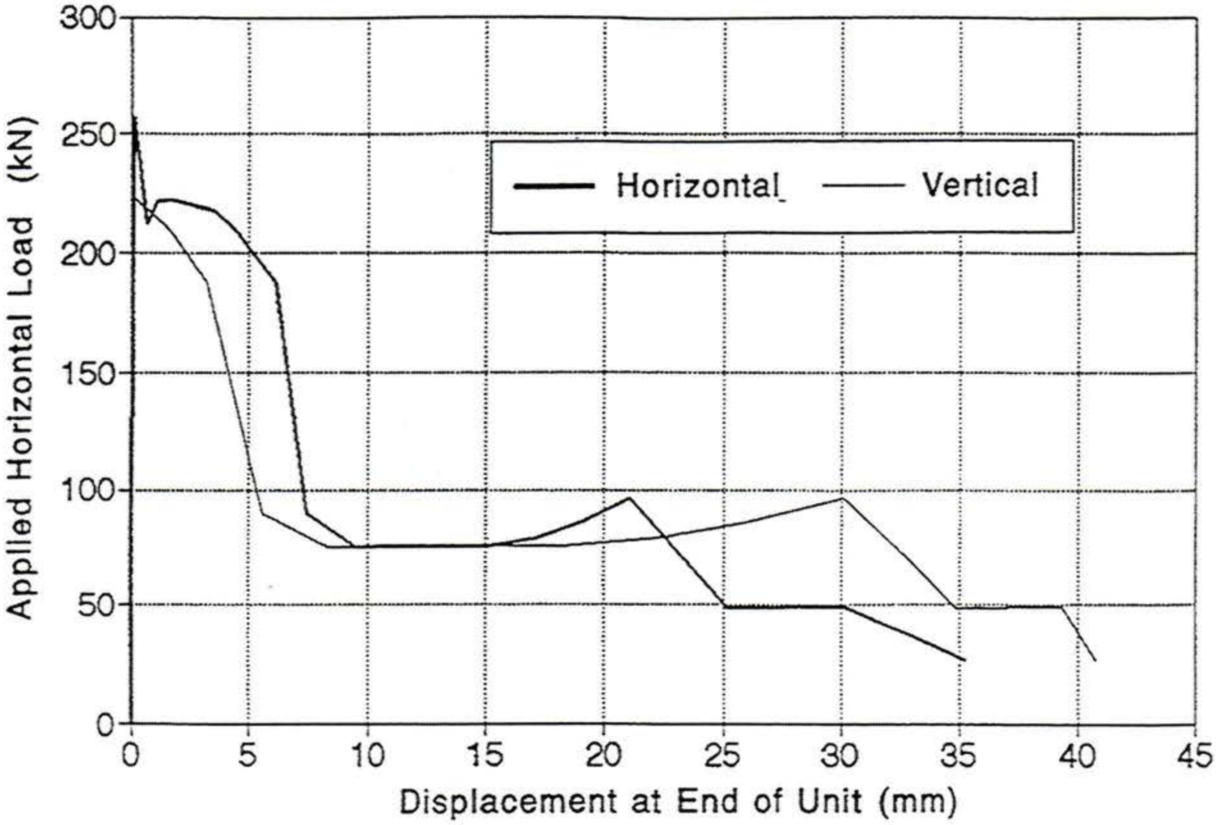


Fig. 6.9 Applied Horizontal Load Versus Horizontal and Vertical Displacements at End of Unit Measured During the First Stage of Test B on Connection Type 2

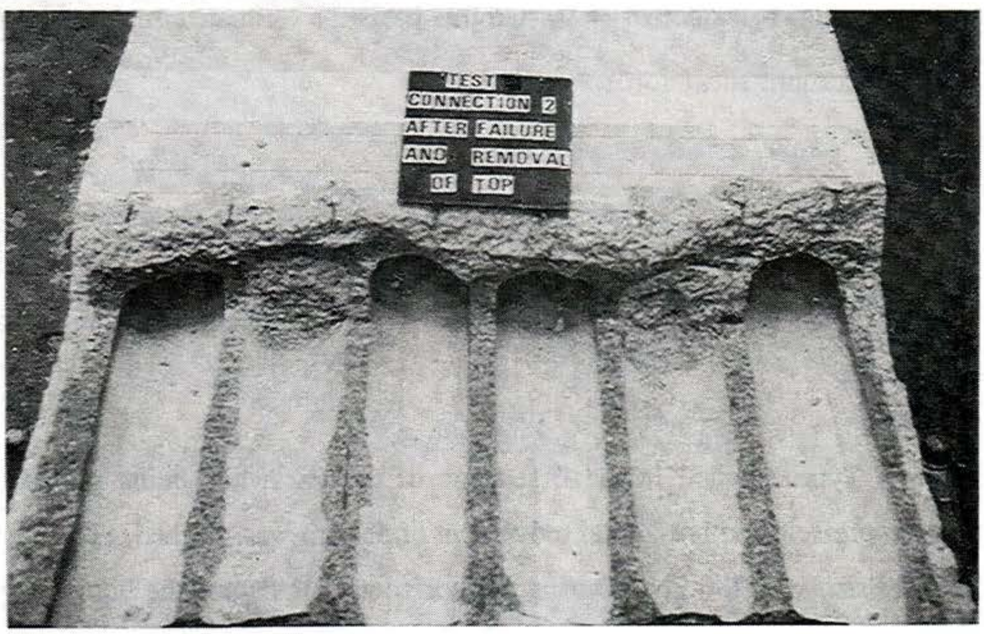


Fig. 6.10 At the End of Test B on Connection Type 2

The welded wire mesh in the topping concrete fractured when the horizontal movement was about 5 mm, and as a result the horizontal force dropped from 220 kN to 75 kN. This force is smaller than the horizontal component of the ties at yield (90 kN), because the inclined part of the ties straightened when the ties tore the concrete above them, decreasing the strain in the ties. This also caused progressive cracking of the unit that extended under the ties. Eventually failure of the connection occurred when the base of the webs of the Dy-core unit fractured at a horizontal movement of 35 mm (see Fig. 6.10). The second stage of Test B, with vertical load applied, was not conducted.

6.3.2 Analysis of Test Results

The behaviour of this connection under horizontal load shows that the sharp change in direction and the steep slope of the ties (45°) in the end region of the hollow-core unit resulted in horizontal cracks at the base of the webs of the hollow-core unit which eventually caused failure (see Fig. 6.10). Hence connection type 2 demonstrated poor deformation and anchorage capacity during the imposed horizontal deformation.

6.4 Connection Type 3

6.4.1 Results for Test B

(a) <u>First Stage:</u>

The measured relationships between the applied horizontal load and both the horizontal and vertical displacements at the end of the unit are shown in Fig. 6.11. A crack in the topping slab developed at the end of the unit, as in the other specimens. The maximum horizontal load resisted was 235 kN at a 5 mm 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.

(b) <u>Second Stage:</u>

In the second stage of the test, with vertical load applied, the behaviour of the

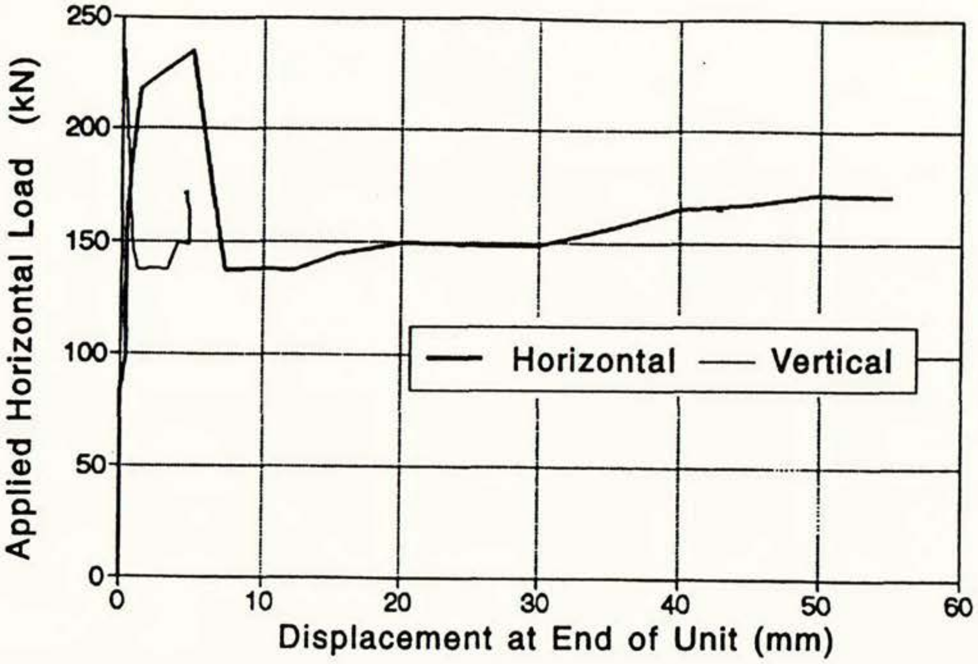


Fig. 6.11 Applied Horizontal Load Versus Horizontal and Vertical Displacements at End of Unit Measured During the First Stage of Test B on Connection Type 3

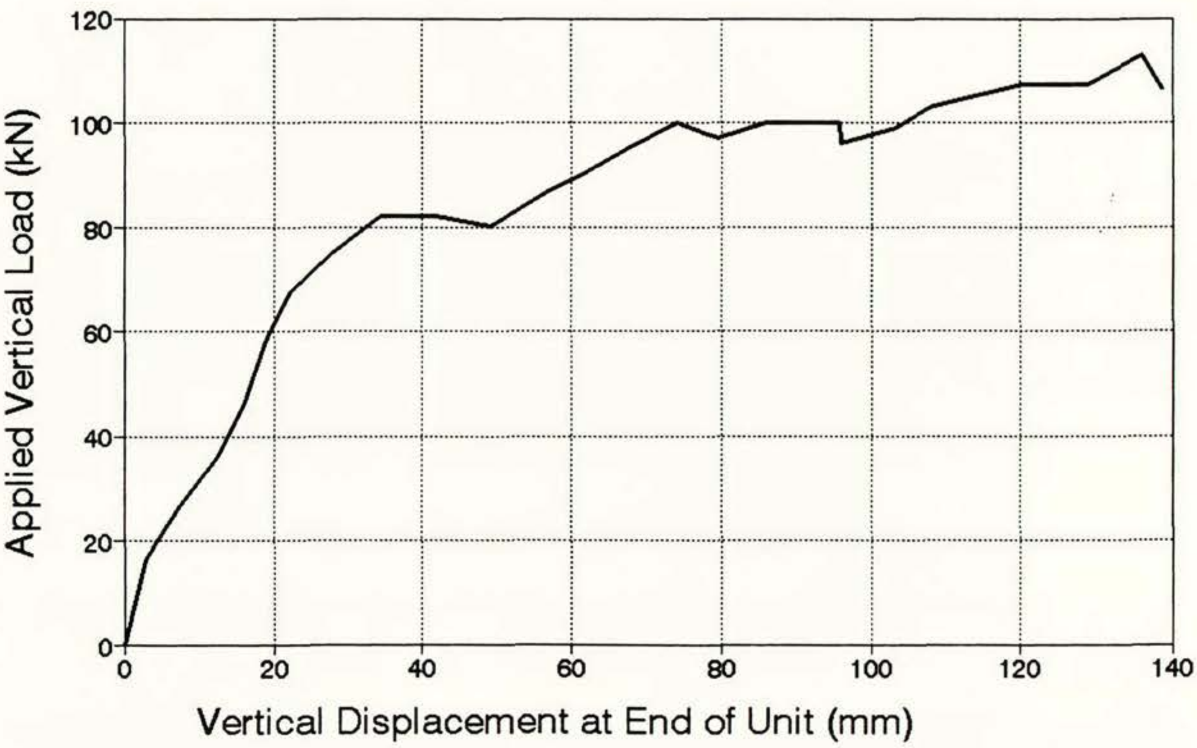


Fig. 6.12 Applied Vertical Load Versus Vertical Displacement at End of Unit Measured During the Second Stage of Test B on Connection Type 3

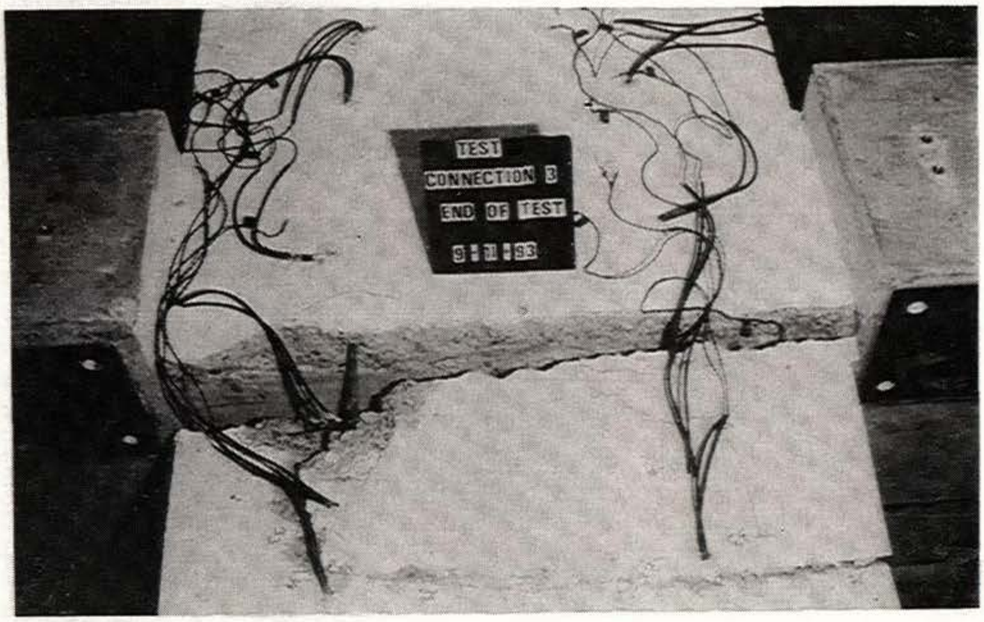
connection was similar to that of connection Type 1, and the same mode of failure as shown in Fig. 6.6 was observed. The maximum applied load was 113 kN. The test finished when one of the ties fractured at an applied vertical load of 106 kN and at a vertical displacement of 139 mm. Fig. 6.12 shows the relationship between the applied vertical load versus the vertical displacement at the end of the unit during the test. Fig. 6.13 shows the specimen at the end of the test.

6.4.2 Analysis of Test Results

Table 6.2 summarizes the results of the second stage of Test B. The calculations follow the same procedure as used in the analysis of data from connection Type 1 (Sec. 6.2.2). The data used in the calculations is: scan number (i), applied vertical load (P_i) and the vertical displacement of the floor unit at the connection tested (δ_i). Using this data and Eqs. 6.5, 6.6, 6.7 and 6.10, the corresponding values of the vertical reaction at the support (V_i), the strain energy developed in the process of loading at scan i ($Area_i$), the equivalent dynamic shear capacity of the connection ($V_{d,i}$) developed up to scan i, and the total strain energy ($W_{int,max}$) developed by the connection are also presented.

Fig. 6.14 shows a plot of the vertical shear force at the support (V_i) and the equivalent dynamic shear capacity of the connection ($V_{d,i}$) versus the vertical displacement (δ_i) measured during the second stage of the test. Like in connection type 1, the behaviour of the connection was quite ductile. The area under the curve of the vertical shear force at the support (V_i) versus the vertical displacement (δ_i) of Fig. 6.14 up to when the tie fractured at a vertical displacement of 139 mm is equal to 92,150 kN mm. This area is equivalent to the total strain energy ($W_{int,max}$) developed by the connection. The maximum shear force which is possible to transfer in dynamic loading $V_{d,max}$ can be found using Eq. 6.2 :

$$V_{d,max} = \frac{W_{int,max}}{\delta_{max}} = \frac{9,150 \text{ kN mm}}{139 \text{ mm}} = 66 \text{ kN}$$



a) Plan View

b) Elevation

Fig. 6.13 At the End of Test B on Connection Type 3

Table 6.2 Calculations from Test A Connection Type 3

i	P_i	V_i	δ_i	$Area_i$	$V_{d,i}$
	[kN]	[kN]	[mm]	[kNmm]	[kN]
0	0.0	6.3	0.0	0.0	0
1	16.1	17.6	2.8	66.0	15
2	26.6	24.9	7.5	120.8	25
3	36.0	31.5	12.5	135.5	26
4	46.6	38.9	16.1	122.5	28
5	58.1	47.0	18.8	145.7	31
6	67.6	53.6	22.3	235.8	37
7	74.9	58.7	27.6	349.3	43
8	82.3	63.9	34.2	453.7	48
9	82.3	63.9	41.8	479.3	50
10	80.1	62.4	49.2	461.8	52
11	86.4	66.8	56.6	434.2	53
12	90.6	69.7	62.2	362.4	54
13	94.9	72.7	67.0	428.9	57
14	100.1	76.4	74.0	473.7	58
15	97.0	74.2	79.4	445.2	59
16	100.1	76.4	86.0	626.5	62
17	100.1	76.4	95.8	378.2	60
18	95.9	73.4	95.9	278.9	63
19	98.9	75.5	103.4	456.8	63
20	103.0	78.4	108.0	654.6	66
21	107.3	81.4	120.1	854.7	66
22	107.3	81.4	129.0	647.1	66
23	113.0	85.4	136.0	414.2	66
24	106.4	80.8	138.7	109.1	66

$W_{int,max}$ = 9,150 kN mm

This value is equal to that found in Table 6.2 using Eq. 6.10.

Again 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 85 kN (see Fig. 6.14 or Table 6.2), the strains measured on the ties bars by the strain gauges corresponded to a stress of f_s = 464 MPa, which was greater than the measured yield strength of 317 MPa due to the steel entering the strain hardening range. Also, A_{vf} = 402 mm^2. Therefore equation 6.1 gives

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

$$\therefore \theta_t = 27^{\circ}$$

Since the tie bar initially was inclined at θ_i = 13° to the horizontal, the kinking of the bar caused a further deviation angle of θ_k = 27° - 13° = 14°.

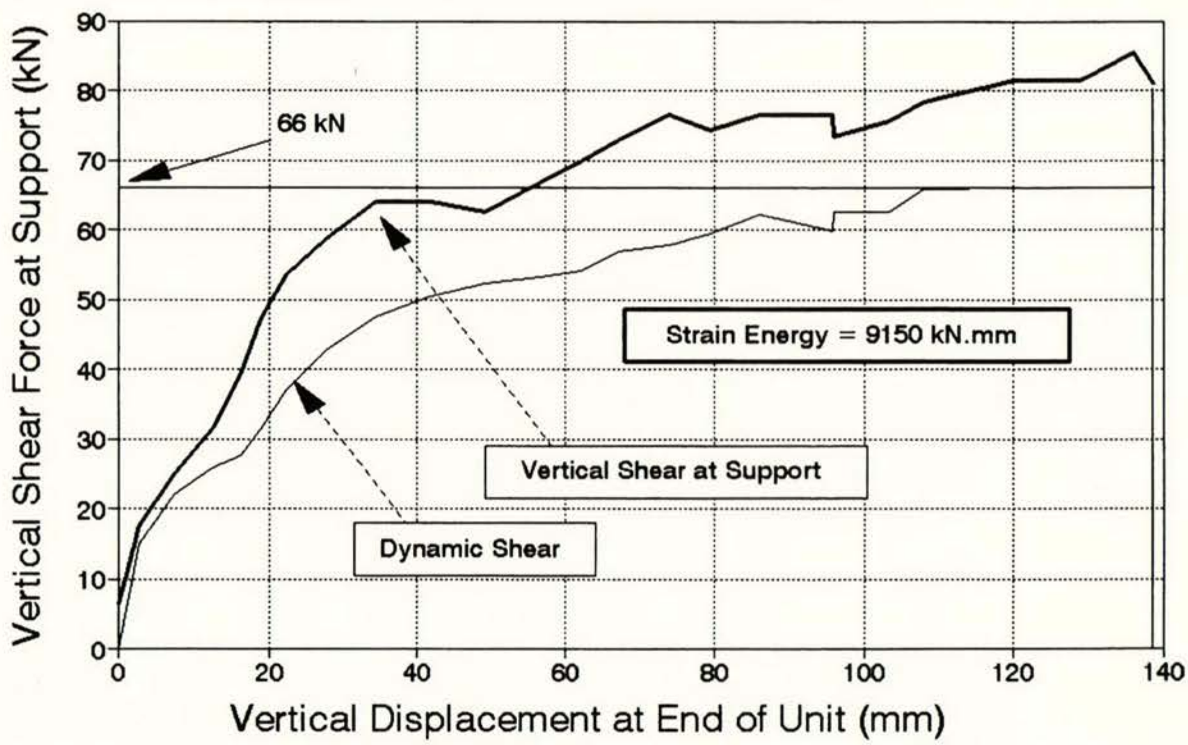


Fig. 6.14 Vertical Shear Force at the Support and Equivalent Dynamic Shear Capacity of the Connection Versus Vertical Displacement at the End of Unit Measured During the Second Stage of Test B on Connection Type 3

Chapter 7

DISCUSSION OF THE TEST RESULTS

7.1 Capacity of The Three Types of Tie Connection

7.1.1 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 7.1 summarizes the results of Test A for the three types of connections at the serviceability and ultimate limit states. The serviceability limit state is defined as when the vertical displacement at the support was 0.2 mm, which is a criterion which has also been suggested by Tsoukantas and Tassios [23]. At a shear slip of 0.2 mm no harmful cracks will appear at the connection. The ultimate limit state is defined as when the shear resisted is maximum. The shears in Table 7.1 are the shear forces calculated at the face of support from the applied vertical load assuming the floor units to be simply supported.

Table 7.1 Summary of the Vertical Shears at the Support in Test A

DESCRIPTION	CONNECTION		
	Type 1	Type 2	Type 3
1) At Serviceability Limit State (at vertical displacement at connection of 0.2 mm)			
- Shear Resisted	221 kN	167 kN	188 kN
- Shear Resisted/Maximum Shear Resisted	0.69	0.81	0.82
- Shear Resisted/Maximum Shear Predicted	0.79	0.67	0.83
2) At Ultimate Limit State			
- Maximum Shear Resisted	321 kN	207 kN	230 kN
- Maximum Shear Predicted	281 kN	249 kN	226 kN
- Maximum Shear Resisted/Maximum Shear Predicted	1.14	0.83*	1.02
- Vertical Displacement at End of Test	2.2 mm	1.7 mm	1.8 mm

* Anchorage failure of the tie bars occurred

The maximum shear forces resisted for connection Types 2 and 3 were smaller than for connection Type 1, evidently because of smaller depth of cast-in-place concrete over the precast supporting beams for those two connections (see Fig. 2.1) and because of the shape of the tie bars. In particular, the 45° bends in connection Type 2 resulted in splitting cracks which caused anchorage failure of the tie bars.

Table 7.1 also lists the shear force resisted by the connections calculated 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 the use of a more cautious value of $\mu = 1.0$ appears more appropriate. Those values of μ were used in the predictions of the maximum shear forces listed in Table 7.1, and resulted in reasonable agreement with the test values except for connection Type 2 in which the tie bars had an anchorage failure.

To examine the relevance of the shear strength of connections Type 1 and 3 in practical design, consider a simply supported one-way floor of span L in metres with a uniformly distributed load w in kN/m. If the vertical reaction at each support is limited to the minimum shear force resisted by these connections at the serviceability limit state (V_s = 188 kN, see Table 7.1), and considering a strength reduction factor $\phi = 0.85$, the following equation expresses the relationship between the maximum load w for a given span L:

$$\phi V_s = \frac{w\,L}{2}$$

$$\therefore\ w = \frac{376}{L} \tag{7.1}$$

For the same floor, the maximum uniformly distributed load w for a given span L as limited by the flexural strength M_n of the section, is given by :

$$\phi M_n = \frac{w\,L^2}{8}$$

$$\therefore\ w = \frac{8\,\phi M_n}{L^2} \tag{7.2}$$

where $\varnothing = 0.90$ is the strength reduction factor and M_n is the nominal flexural strength of this hollow-core section with a 65 mm thick cast-in-place concrete topping slab acting compositely.

Using Eqs. 7.1 and 7.2 it is possible to compare for a simply supported one-way floor of span L, the maximum uniformly distributed loads for when the connection reaches its shear strength at the serviceability state and when the floor reaches its flexural strength. For the evaluation of Eq. 7.2 it is necessary to know the nominal flexural strength of the composite floor section. Assuming a concrete compressive strength $f'_c = 30$ MPa for the 65 mm thick cast-in-place concrete topping slab, the nominal flexural strength of this composite hollow-core section is calculated using the following data (see Appendix A):

Dy-core: $f'_c = 40$ MPa

A_{ps} = Area of prestressing steel

$A_{ps} = 607\ mm^2$

$d = 200$ mm (Dy-core) + 65 mm (topping) - 45 mm = 220 mm

ρ_p = Prestressing steel ratio

$$\rho_p = \frac{607}{1200 \times 220} = 0.0023$$

f_{pu} = Ultimate strength of prestressing steel

$f_{pu} = 1763$ MPa

Topping: $f'_c = 30$ MPa

For bonded prestressing steel, NZS 3101 [10] section 13.3.6.1, gives an empirical equation for the calculation of the stress in the prestressing steel at the design load (f_{ps}), which may be used instead of a more accurate calculation which involves a trial method based on reaching compatibility between stresses and strains. That empirical equation is:

$$f_{ps} = f_{pu}\,(1 - 0.5\rho_p \frac{f_{pu}}{f'_c})$$

$$\therefore\ f_{ps} = 1763\ (1 - 0.5 \times 0{,}0023 \times \frac{1763}{40}) = 1674 \text{ MPa}$$

Then the ideal flexural strength of the composite section (M_n) is

$M_n = A_{ps} f_{ps} (d - a/2)$ where a = depth of the concrete rectangular compressive stress block

$$a = \frac{A_{ps} f_{ps}}{\beta_1 f'_c b} = \frac{607 \times 1674}{0.85 \times 30 \times 1200} = 33.2 \text{ mm}$$

$$\therefore M_n = 607 \times 1674 \, (220 - 33.2/2) = 206 \text{ kN m}$$

Substituting this value of M_n in Eq. 7.2:

$$w = \frac{8 \times 0.90 \times 206}{L^2} = \frac{1483.2}{L^2} \qquad (7.3)$$

Fig. 7.1 shows a plot of Eqs. 7.1 and 7.3 for a wide range of values of the span commonly used in the design of floors (4 m to 12 m). From Fig. 7.1 it is clear that for spans bigger than 4.6 m the shear strength of connection Types 1 and 3 at the serviceability stage will be larger than the reaction induced at the support when a uniformly loaded slab reaches its flexural strength.

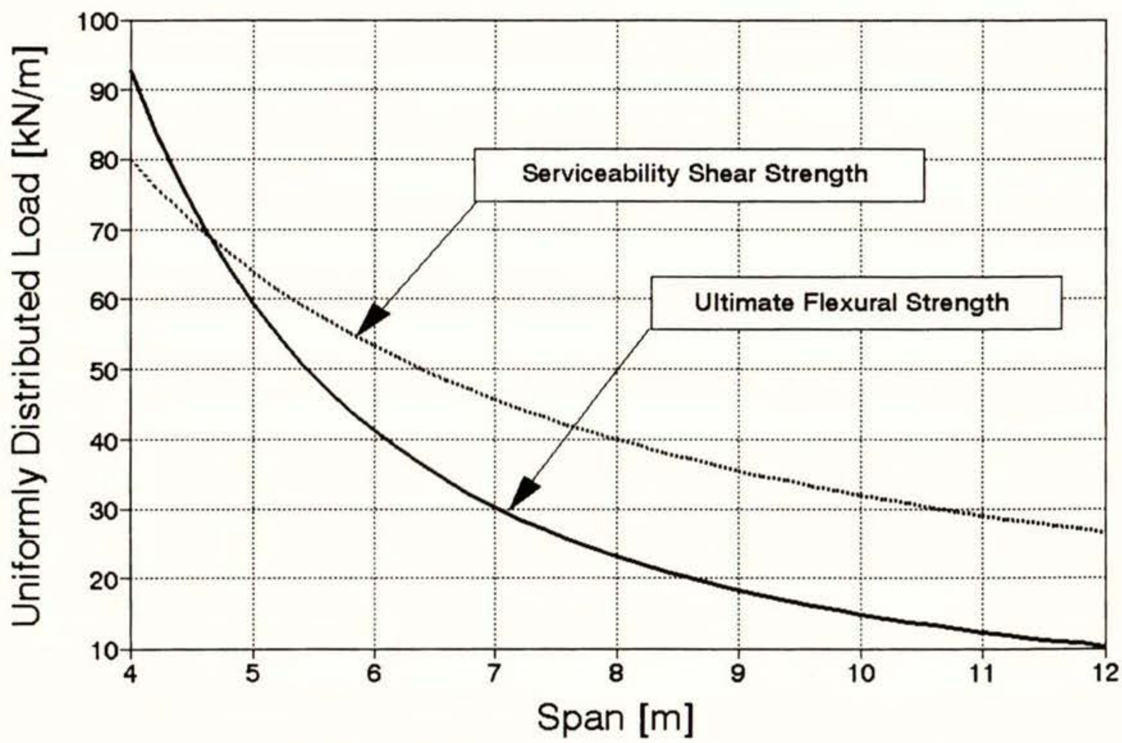


Fig. 7.1 Uniformly Distributed Load Versus Span at Serviceability Shear Strength and Ultimate Flexural Strength

For spans smaller than 4.6 m the uniformly distributed load necessary to induce a reaction at the support equal to the serviceability shear strength of the connection is very large (70 kN/m or more) compared with that used in common designs. Therefore, it can be concluded that no harmful or unsightly cracks induced by the shear-friction mechanism will be present at the connection when the floor unit is uniformly loaded. It is to be noted that for Test A these connections were constructed with the hollow-core unit without seating on the supporting beam.

7.1.2 Test B

In Test B the vertical load was applied after the connection had been displaced horizontally by 55 mm, so that the seating of the ends of the precast units was lost. Table 7.2 summarises the results of Test B for the three types of connection. Connection Type 2 failed by fracture of the concrete of the hollow-core unit in the region which anchored the tie bars before the 55 mm horizontal displacement was reached. The sharp changes of direction of the ties in the end regions of this connection evidently cause this splitting. Connection Types 1 and 3 were able to be displaced horizontally by 55 mm without failure of the ties, due to bond failure propagating along the plain round tie bars and the capacity of 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, after being displaced horizontally, they resisted the vertical shear by kinking of the tie bars. 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 almost the entire length of tie bar between the end hooks was in the plastic range at the ultimate limit state, whereas for connection Type 3 the initially created regions of plastic strain where the tie bars were bent over the supporting beam eventually resulted in fracture of the bars there at a smaller vertical displacement than was reached by connection Type 1.

The measured dynamic shear capacity of connection Types 1 and 3, when the floor unit was loaded vertically after losing its support as a result of imposed horizontal movements, was 57 and 66 kN, respectively. Applying a strength reduction factor $\phi = 0.85$

Table 7.2 Summary of Results of Test B			
DESCRIPTION	CONNECTION		
	Type 1	Type 2	Type 3
1) First Stage (horizontal movement)			
- Maximum horizontal movement reached	55 mm	35 mm	55 mm
- Maximum tensile force resisted	380 kN	260 kN	235 kN
- Tensile force at end of movement	164 kN	0	171 kN
- Maximum elevation reached	0	40 mm	5 mm
2) Second Stage (vertical movement)			
- Maximum shear resisted	88 kN	-	85 kN
- Vertical displacement at end of test	215 mm	-	139 mm
- Equivalent dynamic shear resisted	57 kN	-	66 kN
- Calculated angle of bar axis to horizontal θ_t due to initial deviation and kinking to obtain maximum shear resisted	30°	-	27°

to these values gives the dependable dynamic shear capacity of the connections. If again a simply supported one-way floor slab of span L in metres with a uniformly distributed load w in kN/m is considered, in the event of loss of bearing at the supports the tie connections at the end of the units need to resist a vertical reaction of :

For connection Type 1: $$0.85 \times 57 = \frac{w\,L}{2} \qquad \therefore\ w = \frac{96.9}{L} \tag{7.4}$$

For connection Type 2: $$0.85 \times 66 = \frac{w\,L}{2} \qquad \therefore\ w = \frac{112.2}{L} \tag{7.5}$$

Fig. 7.2 shows a plot of Eqs. 7.4 and 7.5 for a wide range of values of spans traditionally used in the construction of floors (4 m to 12 m). The calculated values of w are bigger in most cases than the service loads commonly used in the design of floors. This indicates that the ends of the floor units can undergo loss of bearing or large displacements relative to the supporting beams without loss of structural integrity, therefore permitting the precast floor units to remain suspended. It is to be noted that under such extreme conditions

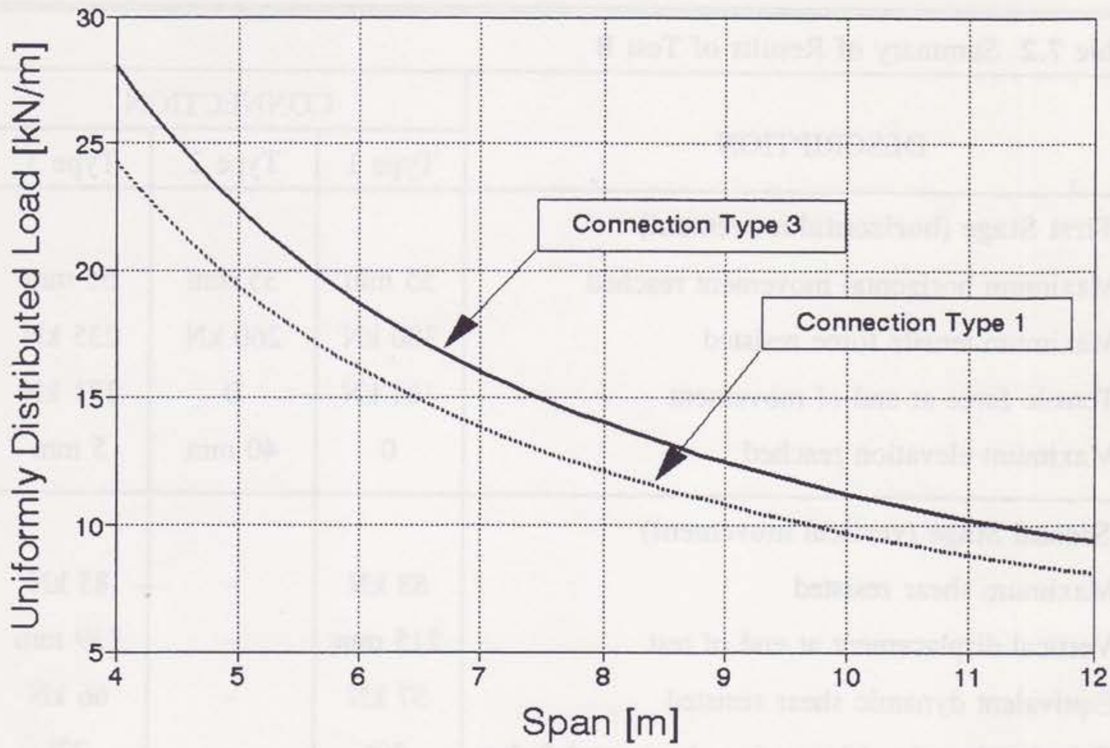


Fig. 7.2 Maximum Uniformly Distributed Load that Connections Type 1 and 3 Can Sustain if the Floor Unit Slides Off its Support Versus Span

it is reasonable to consider whether the service loads can be supported, rather than the ultimate loads.

7.2 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 and service loads as follows:

Service Dead Load:

Hollow-core unit (1.2 m wide x 200 m thick) plus topping slab (65 mm thick)	4.6 kN/m
Partitions + additional service dead loads:	1.5 kN/m
	D = 6.1 kN/m

Service Live Load:

Office occupancy: 2.5 kPa x 1.2 m	L = 3.0 kN/m

Total Service Load $\quad D + L = 9.1$ kN/m

For the above service loads the required ultimate load [10] of the floor is

$$\begin{aligned} U &= 1.4D + 1.7L \\ &= (1.4 \times 6.1) + (1.7 \times 3.0) \text{ kN/m} \\ &= 13.6 \text{ kN/m} \end{aligned}$$

For a one-way slab floor of span L in metres with these service and ultimate loads, in the event of loss of bearing at the supports the tie connections at the ends of the units need to resist a vertical reaction of

At service load $R = \dfrac{9.1\,L}{2} = 4.55\,L$ kN

At ultimate load $R = \dfrac{13.6\,L}{2} = 6.80\,L$ kN

For tie connection Types 1 and 3 the maximum static shear force resisted in Test A was at least 230 kN, and the maximum equivalent dynamic shear force resisted in Test B was at least 57 kN. Note that the static load capacity is appropriate for the shear-friction strength since 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 since dynamic loading is involved due to the floor dropping. Applying a strength reduction factor $\phi = 0.85$ to these values gives for the tie connection Types 1 and 3 available dependable vertical reactions of

For Test A $R = 0.85 \times 230 = 195.5$ kN

For Test B $R = 0.85 \times 57 = 48.5$ kN

Therefore the maximum spans for which the ultimate load could be supported by tie connections Types 1 and 3 in the event of inadequate or even absent seating at the support (conditions of Test A, where the reaction was provided by the shear-friction mechanism) is

$$L = 195.5/6.8 = 28.8 \text{ m}$$

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, where the crack width is very large) is

$$L = 48.5/4.55 = 10.7$$

The nominal flexural strength of this hollow-core section with a 65 mm thick cast-in-place concrete topping slab acting compositely is (see Sec. 7.1.1) $M_n = 206$ kN m. Hence, with a strength reduction factor $\phi = 0.90$, the maximum simply supported span for this ultimate load is given by $UL^2/8 = \phi M_n$

$$\therefore \quad L = \sqrt{\frac{0.90 \times 206 \times 8}{13.6}} = 10.4\ m$$

which is less than the span for which 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 tie connections Types 1 and 3 are safe.

Chapter 8

SUMMARY, CONCLUSIONS, AND SUGGESTIONS FOR FUTURE RESEARCH

8.1 Summary

Three types of special reinforcement for tie connections at the ends of hollow-core precast concrete floor units were tested. This special reinforcement is intended to prevent collapse of the floor units in the event of seating lengths being inadequate or movements being imposed due to volume changes, earthquakes or accidental loads, which may cause the units to be dislodged from their seating. The special reinforcement consisted of two tie bars of various shapes placed across the supporting beam and anchored in cast-in-place concrete placed in two broken back voids at the ends of the hollow-core units. The tie bars were of 16 mm diameter Grade 300 steel, which has a characteristic yield strength of 300 MPa, and extended about 750 mm into each end of the hollow-core units. Other details of the three types of tie bar were as follows (see Fig. 2.2):

Tie Connection Type 1 : Two straight plain round bars with 180° end hooks
Tie Connection Type 2 : Two deformed bars with 45° bends and no end hooks
Tie Connection Type 3 : Two plain round bars inclined 13° to the horizontal and with 180° end hooks

Three test specimens were constructed from longitudinal strips of pre-tensioned precast concrete hollow-core floor units of Dy-core type, 1200 mm wide by 200 mm deep, with a 65 mm thick cast-in-place reinforced concrete topping, continuous over the supporting beams. The connections tested were located at two interior precast concrete supporting beams. Each type of tie connections was tested subjected to applied vertical load at both supports, but under different conditions of bearing length and applied horizontal load. The two tests performed on each type of connection were:

Test A: A vertical load was imposed without any seating being present under the end of the hollow-core unit. The vertical load was increased and the shear strength of the connection determined. The vertical reaction at the connection was mainly transferred by a shear-friction mechanism.

Test B: The hollow-core unit was first pulled horizontally a distance of 55 mm so as to lose its seating and to open up a very large crack in the cast-in-place topping slab across which the slab mesh fractured. Next, keeping the horizontal position of the unit constant, the vertical load on the span was increased until the tie bars fractured. The vertical reaction at the connection was mainly transferred by the kinking of the tie bars.

8.2 Conclusions

8.2.1 Conclusions from Test A

In spite of the zero seating length, all of the tie connections were able to transfer the ultimate gravity load (dead plus typical office live load) of the floor. It was found that the shear capacity at the support can be calculated from the shear friction across the crack in the cast-in-place concrete topping slab and in the two filled voids used to anchor the ties plus any vertical component of force in the tie bars, providing that anchorage failure of the tie bars did not occur. The slab mesh and the tie bars were considered to provide the clamping force for the shear-friction mechanism. The test results showed that a cautious value for the shear-friction coefficient μ to be used in calculating the available shear friction capacity is recommended. 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 and hence the depth of 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 and hence 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.

8.2.2 Conclusions from Test B

Connection Type 2 failed by splitting of the concrete at the end of the hollow-core unit during the horizontal movement as a result of the large angle changes (45°) in the tie bars. Hence connection Type 2 is not recommended for structures which may be subjected to large horizontal movements such as due to earthquakes. Connection Types 1 and 3 behaved in a ductile manner during the horizontal movement since the use of plain round reinforcement for the tie bars enabled bond failure to propagate along them, making large plastic elongations possible, and also the anchorage capacity of the connections exceeded the fracture load of the tie bars.

Electrically welded fabric used as reinforcement of the topping exhibited a very poor elongation capacity (normally less than 4%), proving to be ineffective to prevent progressive collapse of structures after an initial failure, or in structures which eventually may be subjected to large displacements.

In second stage of Test B, connection Types 1 and 3 were able to support significant vertical load, adequate to support the service gravity load (dead plus typical office live load) of the floors, by kinking of the tie bars during this very severe test. Vertical displacements of at least 140 mm were reached at the end of the test.

The use of the tie bars of connection Types 1 and 3 is recommended for floors if large deformations are expected such as due to major earthquake shaking.

8.3 Suggestions for Future Research

Further research is needed to confirm the suggested values for the shear-friction coefficient μ to be used when calculating the available shear-friction capacity in composite floors with different types of interface between the precast floor units and the supporting beams.

The anchorage capacity of tie bars placed in cast-in-place concrete in the cores of hollow-core units has not been thoroughly examined. The positive influence of the reinforced concrete topping may allow the use of bigger tie bars which would increase the shear capacity of the connections.

In this research the dynamic shear capacity of the tie connections of hollow-core floor units was evaluated from the results of quasi-static tests (Test B) and principles of conservation of energy. Using the same test rig as for Test B, but replacing the action of the ram that applied the vertical load by a dead load equivalent, the dynamic shear capacity of the tie connections could be measured directly to prove the validity of these findings.

For tie connections in composite precast floors which may be subjected to large elongations (Test B), the influence of the cast-in-place concrete strength and the size, strength and type of tie bars need further study.

Good bond between the cast-in-place concrete topping slab and the precast hollow-core unit was observed during the tests, suggesting that the topping reinforcement could transfer some vertical shear force in the event of loss of bearing before splitting of the concrete topping away from the precast unit occurred. It is suggested that some tests be conducted without tie connections to investigate the failure mode in this case.

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APPENDIX: PROPERTIES OF THE HOLLOW-CORE UNITS

(A) Transformed Section Properties of Hollow-core Unit Alone (see cross-section in Fig. A):

Moment of inertia of a cross-section	$= 6.070 \times 10^8$ mm^4
Area of a cross-section	$= 1.179 \times 10^5$ mm^2
Distance of centroid from top fibre	= 100 mm
Concrete compressive cylinder strength	= 40 MPa

(b) Transformed Section Properties of Hollow-core Unit Plus 65 mm Thick Topping Slab:

Moment of inertia of composite cross section	$= 1.321 \times 10^9$ mm^4
Area of a cross section	$= 1.767 \times 10^5$ mm^2
Distance of centroid from top fibre	= 125 mm
Topping concrete compressive cylinder strength	= 36 MPa, 27 MPa and 29 MPa

for specimens 1, 2 and 3, respectively.

(c) Prestressing Steel:

Five 12.5 mm diameter plus two 11.0 mm diameter pretensioned tendons

Area of prestressing steel	= 607 mm^2
Elastic modulus, minimum	$= 190.2 \times 10^3$ MPa
Ultimate tensile strength	= 1070 kN
Prestressing force immediately after transfer	= 805 kN

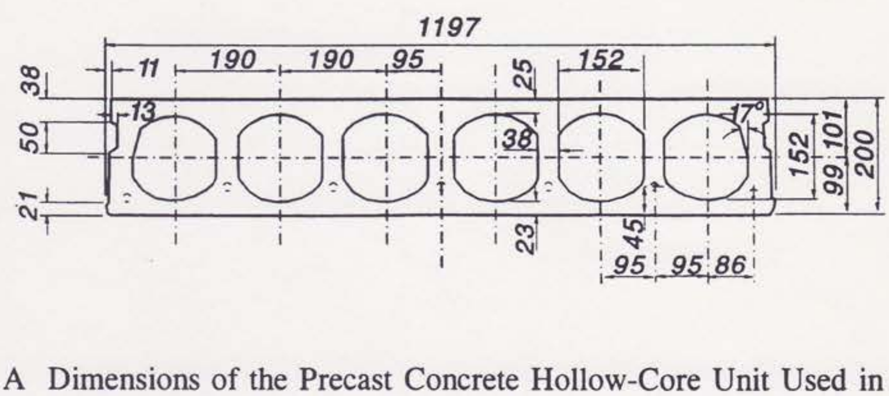

Fig. A Dimensions of the Precast Concrete Hollow-Core Unit Used in the Tests

Classn:

PRECAST CONCRETE FLOOR UNIT SUPPORT AND CONTINUITY

Juan C. Mejia-McMaster

ABSTRACT: Three types of special tie reinforcement at the end supports of hollow-core precast concrete units with a cast-in-place topping slab were investigated subjected to two types of test. 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.

Department of Civil Engineering, University of Canterbury
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