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Tests on precast concrete resisting frame components typical from New Zealand

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ABSTRACT: Test results from an ongoing research programme at the University of Canterbury on moment resisting frames incorporating precast concrete members are presented. Results indicate that the connections between precast concrete members in frames can be designed to achieve levels of stiffness, strength and ductility similar to monolithic construction.

1 INTRODUCTION

Despite of the lack of design guidelines, moment resisting frames incorporating precast concrete members, designed to be ductile and providing the primary earthquake resistance of a building, are widely used in New Zealand. One reason for the acceptance of this alternative form of construction is that the frames are designed to behave as if of monolithic construction. The capacity design procedure (NZS 3101 1982) is followed in which the location of the regions intended to dissipate energy durring severe earthquakes are deliberately chosen and designed for adequate strength and ductility while other regions in the structure are made overstrong. A number of different systems of frame construction using precast concrete elements have been described elsewhere (Park 1990).

A common feature of the New Zealand approach is to connect the precast concrete members by cast in situ concrete joints. Furthermore, in multistorey buildings the whole of the earthquake resistance is generally allocated to stiff perimeter frames with rather short beams and columns. Typically the ratio of clear span to overall depth for beams varies between 3 and 6.

This paper summarizes recent tests involving a range of connection details between precast concrete beam elements of perimeter frames. The tests are part ot an ongoing research programme at the University of Canterbury.

2 TEST PROGRAMME

2.1 *Tests on units with precast concrete elements connected at the midspan of beams*

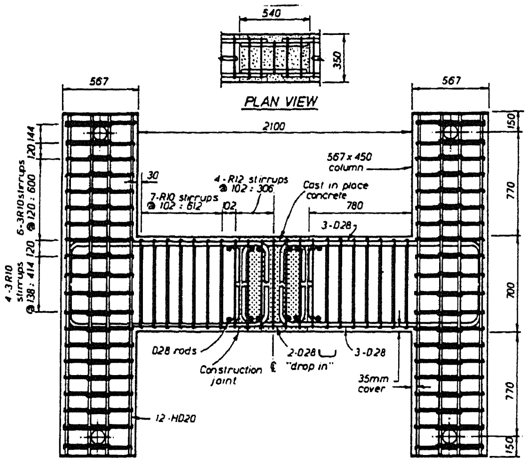
The first series of tests was conducted to evaluate the cyclic load performance of different cast in situ connection details located in the midspan region of beams.

A difficulty encountered by designers when connecting precast concrete beam elements at midspan is to achieve the connection in a small enough length to avoid lap splices of beam bars encroaching into the potential plastic hinge regions at the end of the beams.

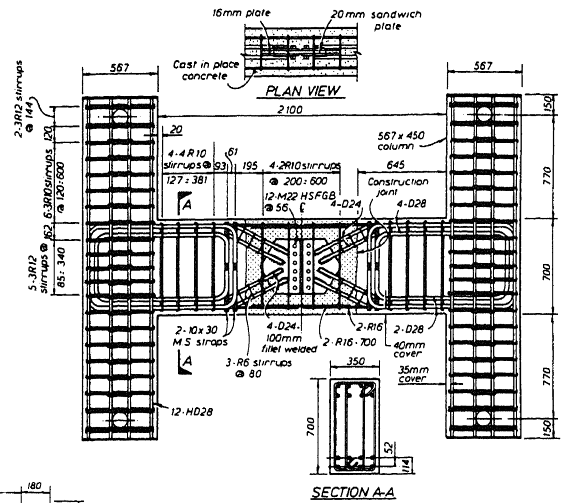
Three H-shaped subassemblages, Units 1, 2 and 3, were constructed and tested to evaluate the performance of different connection details and the effects of the proximity of the lap splices to the critical regions in the beams at the column faces. Figure 1(a) shows complete reinforcing details of one of the units tested, Unit 2. A clear span to overall depth ratio of 3 was chosen for the beams to represent the smallest span/depth ratio of a perimeter frame encountered in practice. The loading frame shown in Figure 2(a) was used to test the H-shaped units. During the tests lateral loads were applied to the column tops. The drift imposed on each of the two columns of the units was kept the same. The bending moment induced in the beam was similar to that of a beam of a perimeter frame where moments due to gravity loads are small compared with moments due to seismic actions and the point of contraflexure is then located near the midspan of the beam.

Midspan connections in perimeter frames are located in a region of low moment and the shear in the beam may be a critical factor in their design. Therefore in designing the H-shaped units the nominal shear stress in the beam was the maximum allowed by the New Zealand Concrete Design Code (NZS 3101) of $0.3\sqrt{f'_c}$ (MPa) when diagonal reinforcement is to be avoided in the potential plastic hinge regions.

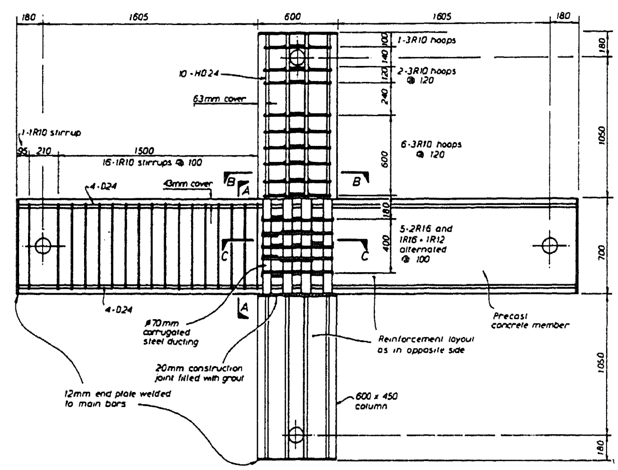
The midspan connection in Unit 2 consisted of two double 90° hooked "drop in" bars overlapping two-

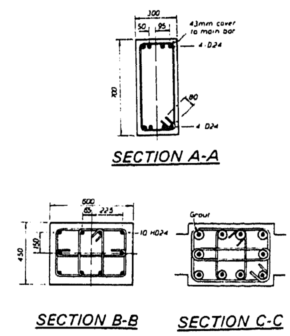


(a) Unit 2



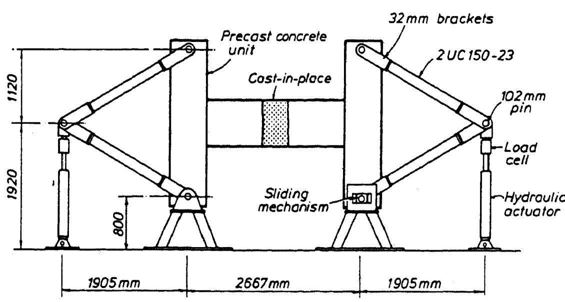
(b) Unit 4



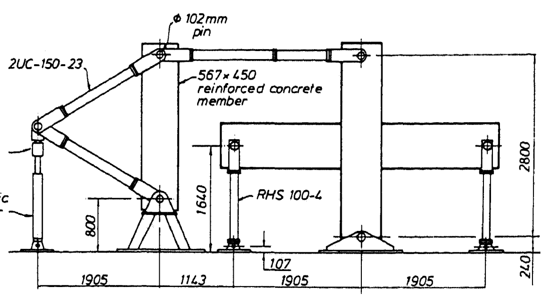


(c) Unit 6

Figure 1 Reinforcing Details of Units Tested



(a) For Midspan Connections



(b) For Beam-Column Joint Connections

Figure 2 Loading Frames used in the Test Programme thirds of the longitudinal bars in the precast beam. Transverse rods of the same bar diameter as the longitudinal bars were placed in contact with the hooks to improve the anchorage. The connection commenced at a distance of 1.23d from the column faces, where d is the effective depth of the beam. The transverse reinforcement in the connection region was capable of transferring 80% of the beam shear force when the beam attained a flexural strength corresponding to a steel overstrength of $1.25f_y$. This unit was designed to develop plastic hinges at the end of the beams; other regions in the structure were made overstrong. Units 1 and 3 incorporated other types of splice details at the midspan connection.

A further H-shaped subassemblage, Unit 4, was constructed using an arrangement which has occasionally been used in New Zealand when the nominal shear stress in the potential plastic hinges of the beams exceeds $0.3\sqrt{f'_c}$ (MPa). Unit 4 is shown in Figure 1(b). The beam is detailed with strong end regions which are designed to enforce the beam to behave inelastically only in the diagonally reinforced cast in situ central region. This arrangement has the advantage that the beam-column joint region can be designed to be less congested since the adjacent portions of the beam remain in the elastic range. The diagonally reinforced Unit 4 was designed in accordance with the simple truss model shown in Figure 3. The reinforcement lay out and dimensions were obtained from an existing design. The diagonal reinforcement was welded to 16 mm thick steel plates in the midspan region. A 20 mm thick steel sandwich plate and twelve 22 mm diameter high strength friction grip bolts were used to interconnect the precast concrete beams. Besides observing the cyclic load behaviour of such a design, during the tests attention was also given to the regions at the bends of the diagonal reinforcement, which had been artificially strain aged.

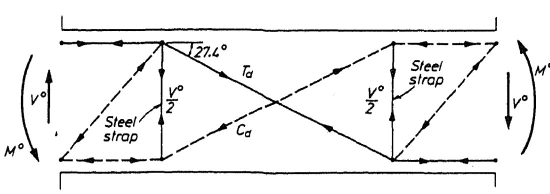


Figure 3 Assumed Simple Truss Model for Unit 4

2.2 *Tests on units with precast concrete elements connected at the beam-column joint regions*

The second series of tests was conducted to evaluate the performance of different connection details located at the beam-column joint. Two cruciform shaped subassemblages, Units 5 and 6, were tested.

Figure 1(c) shows the reinforcing details of Unit 6, in which the beam and the beam-column joint core are part of the precast beam element. The precast beam elements of this method of construction are generally connected at midspan using a connection detail similar to Units 1 to 4. Vertical corrugated ducts in the precast member allow the longitudinal bars of the column below to pass through the joint. The precast concrete member is seated on shims on the column below so as to leave a 20-30 mm gap. This gap is grouted in the same operation as the grouting of the vertical column bars in the corrugated ducting in the precast member.

Unit 6 was designed to develop plastic hinges in the beam at the column faces. As shown in the test set up in Figure 2(b), no axial force was applied to the column. In accordance with the New Zealand Concrete Design Code (NZS 3101) all the horizontal shear in the beam-column joint was allocated to the joint core reinforcement. The main point of interest to be investigated was the effectiveness of the grout. The grout needs to provide adequate bond and to permit adequate transfer of the transverse forces from the joint hoops to the vertical column bars. The performance of the construction joints in the column at the face of the precast concrete member was also of interest, especially the lower joint where the precast concrete member has a rather smooth face.

3 MATERIALS

In conformity with the New Zealand practice, the longitudinal reinforcement was deformed reinforcing steel with a characteristic yield strength of 300 MPa for the beams and 430 MPa for the columns. Plain round reinforcement with a characteristic yield strength of 300 MPa was used for transverse reinforcement. Table 1 summarizes the measured properties of the bars used as reinforcement in the beams of the units described. Table 2 presents the mean concrete and grout compressive strengths measured at the time of beginning each test and based on 100 mm diameter x 200 mm cylinders for concrete and 50 mm diameter x 100 mm cylinders for grout.

4 TEST PROCEDURE

Quasi-static lateral loading which simulated severe seismic loading was applied to the units. The first two load cycles were to $\pm 0.75\ H_a$, where H_a = the theoretical lateral capacity of the specimen calculated

Table 1. Mean measured tensile yield strength of reinforcing steel

Description	Location	f_y (MPa)
R10	All Units	356
D28	Units 2 and 4	321
D24	Unit 4	320
D24	Unit 6	285
30x10 straps	Unit 4	315

Table 2. Mean compressive strength of concrete and grout

Unit	Location	f'_c at Test (MPa)
2	Precast Concrete Members	33
	Cast In Place Joint	32
4	Precast Concrete Members	35
	Cast in Place Joint	36
	Repaired Region	62
6	Precast Concrete Member and Lower Column	44
	Top Column	35
	Grout	64

using the measured properties of the materials. The first yield displacement Δ_y was determined as 4/3 times the average positive and negative lateral displacements measured at the peak of the first two cycles to $0.75H_a$. Then, displacement controlled load cycles were applied as follows: 2 cycles to $\mu=\pm2$, 2 cycles to $\mu=\pm4$, 4 cycles to $\mu=\pm6$ and, if possible, cycles to $\mu=\pm8$, where $\mu=\Delta/\Delta_y$ and $\Delta=$ maximum lateral displacement imposed.

5 TEST RESULTS

5.1 *Unit 2*

Figure 4(a) shows the lateral load-lateral displacement hysteresis loops measured for Unit 2. The interstorey drift at first yield displacement Δ_y was 0.35%. A stable hysteretic response was attained to $\mu=\pm4$. In the cycles to $\mu=\pm6$ the hysteresis loops became very pinched and the lateral load capacity gradually decreased. This pinching occurred because of yielding of the stirrups in the plastic hinges of the beams. In fact, shear distortion in those regions became the dominant mode of deformation at the end of the test, as would be expected for beams with a small span/depth ratio. The connection detail in the midspan region and its proximity to the critical end regions of the beam did not affect the performance of the test unit. The test results of Units 1 and 3 showed similar trends (Restrepo et al 1990). A recent amendment to the current Concrete Design Code (NZS 3101) has incorporated these test results. Lap splices of the beam longitudinal bars are now permitted to commence at a distance d from the faces of the columns.

5.2 *Unit 4*

The lateral load-lateral displacement hysterisis loops of Unit 4 displayed less ductility, as shown in Figure 4(b). The first yield displacement in terms of interstorey drift was 0.39%. At the first cycle to $\mu=\pm2$ large splitting cracks formed between the inner diagonal bars and the outer D28 bars around the bends of the diagonal bars. These cracks eventually propagated and inhibited the development of the simple truss mechanism illustrated in Figure 3. It is likely that a combination of transverse forces required for equilibrium to balance the node at this point, plus high radial bearing forces in the concrete around the bend of the diagonal D24 bars, contributed to the splitting of the concrete and therefore the early reduction of load carrying capacity in this test. An inspection of the damaged regions showed that the concrete in contact with the diagonal bars had been crushed, as shown in Figure 5(a).

A repair was undertaken to verify the above hypothesis. The detailing of the reinforcing steel in the bend region was modified as shown in Figure 5(b). Transverse rods were placed in contact with the bent bars and extra ties surrounding the outer D28 bars were also added to resist the three-dimensional force components required for equilibrium. Also, 60 MPa strength concrete was cast in the repaired region.

The lateral load-lateral displacement hysteresis loops measured for the repaired unit, Unit 4r, are illustrated in Figure 4(c). A satisfactory ductile response was attained in this test. The high lateral load overstrength capacity of 1.4 times the ideal or nominal strength is due to strain hardening from the previous test and perhaps also to strain ageing of the steel in the previously yielded region which occurred during the lapse of time between the test of the original and repaired units. This overstrength caused plastic hinges to appear in the beams at the column faces because the strong end regions had been designed using an overstrength factor of 1.25 the ideal or nominal strength, as normally recommended (NZS 3101). The

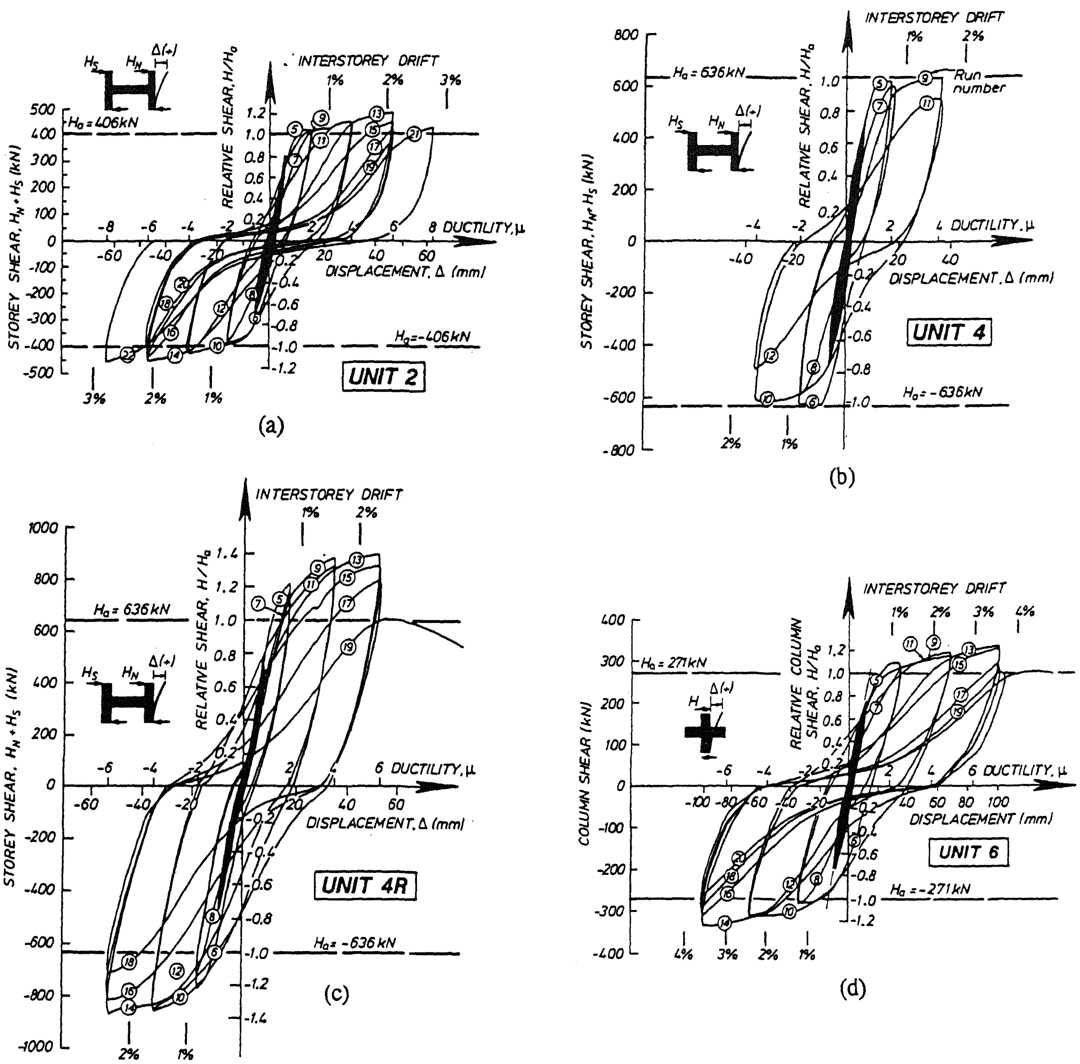
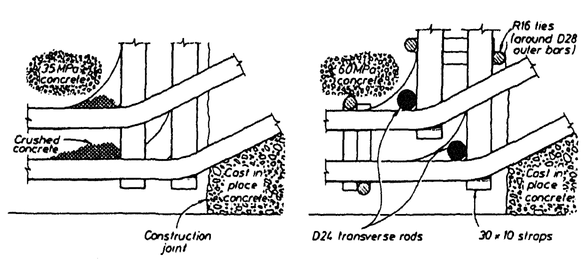


Figure 4 Measured Lateral Load-Lateral Displacement Hysteresis Responses of Test Units



(a) Unit 4 (b) Repaired Unit, Unit 4r

Figure 5 Reinforcing Details of the Bends of the Diagonal Bars test finally showed a vertical sliding shear failure in one of the end plastic hinge regions.

5.3 *Unit 6*

The hysteretic response of the cruciform specimen, Unit 6, is illustrated in Figure 4(d). A very satisfactory ductile response was obtained from this system. Its interstorey drift at the first yield displacement was 0.49%. A predominant flexure response was observed in the test of this unit. The hysteretic loops were very stable and only at the end of the test did some pinching of the loops occur. This pinching was the result of the opening and closing of the inclined cracks in the plastic hinge regions of the beams, as expected. Recorded strains in the longitudinal column bars indicate that the corrugated ducting provided a good degree of confinement. No important displacements were recorded along the construction joints in this unit.

6 DISCUSSION OF TEST RESULTS

The subassemblages demonstrated that levels of stiffness, strength and ductility similar to equivalent monolithic construction can be attained by well connected precast elements. All tests attained at least 2 cycles to $\mu = \pm 6$ and interstorey drifts in excess of 2% without reducing their capacity by more than 20% of the maximum measured.

However, the measured stiffnesses of Units 2 and 6 in the first load cycle to 75% of the theoretical load capacity were only 31 and 49%, respectively, of those calculated using an elastic analysis. The elastic analysis assumed section properties for beams and columns of 0.5 of the gross section values and took into account only flexural and shear deformations of the members. This reduced stiffness was due to deformations in the joint core caused by bond slip and cracking, and to the so called tension shift effect in the members, becoming important in subassemblages with members with relatively low span to depth ratios (Park and Paulay 1975). This reduced stiffness may affect the predicted seismic response of buildings incorporating perimeter frames as the main earthquake resistant system.

7 CONCLUSIONS

The simulated seismic load tests on the subassemblages showed that :

(1) Properly designed cast in situ connections between precast concrete members in laterally loaded frames will result in the same behaviour as monolithic construction.

(2) The splice details used for the longitudinal beam bars at the midspan connection performed very satisfactorily. The tests showed that the splice can commence at a distance of one effective depth from the column face.

(3) Diagonally reinforced midspan connections between precast concrete members with relocated plastic hinges such as Unit 4 can display a ductile response. However, careful detailing of the bend region is required where transverse forces are expected to occur.

(4) Unit 6 was a typical subcomponent from a precast concrete system in which the precast concrete member forms the beam and the beam-column joint. The precast concrete member is integrated to the structure by grouting vertical corrugated ducts acting as sleeves for the longitudinal column reinforcement. This unit exhibited an excellent behaviour, the same as expected in monolithic construction.

(5) All units tested had short beams, typical of perimeter frames. Their measured stiffnesses indicate that the cracked stiffness may be significantly less than that calculated using an elastic analysis incorporating the gross section properties for beams and columns and taking into account only flexural and shear deformations. Other sources of deformation caused this reduction in stiffness.

8 ACKNOWLEDGEMENTS

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