The Influence of Diaphragms on Strength of Beams

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ABSTRACT: In the seismic design of ductile multi-storey moment resisting frames the majority of potential plastic regions are located in the beams. Capacity design requires the maximum likely flexural strength of these zones to be determined. The remainder of the structure is then proportioned to resist the maximum actions that these regions can apply without exceeding their nominal strengths. This process is intended to ensure that inelastic deformation is confined to the potential plastic regions, which have been detailed to sustain the required deformation.

Recent research at the Universities of Auckland and Canterbury on the performance of perimeter reinforced concrete frames has shown that interaction of typical floors with the beams can lead to a substantial increase in the flexural over-strength of potential plastic regions. This increase is considerably greater than that predicted by the current Structural Concrete Standard (NZS 1995). An under-estimate of beam over-strengths may have serious consequences in that non-ductile failure mechanisms, such as a column sway mechanism, may form leading to premature collapse of the building in a major earthquake. This paper describes basic theory related to determination of over-strength actions together with some test results obtained from a recent large scale test.

1 BACKGROUND

In the last three and a half decades major advances have been made in our understanding of the behaviour of concrete structures in major earthquakes. Research in New Zealand has contributed very significantly to this knowledge. The behaviour of beams, columns, beam-column joint zones, walls and other individual components of concrete structures have been extensively studied and tested. However, the behaviour of floors, which act as diaphragms, has received relatively little attention. There would appear to be two main reasons for this, namely;

- Diaphragms are expensive to build in realistic sizes and difficult to test and there are relatively few laboratories that have the resources to undertake such testing;
- The behaviour of diaphragms is complex and it is difficult to obtain a quick return on research in terms of published papers.

However, the good performance of diaphragms is essential if a building is to have a satisfactory performance in a major earthquake as it is this component which holds the structure together, distributes forces to the different load resisting elements and gives the structure toughness by allowing structural actions to be redistributed when elements lose strength. Diaphragms also restrain columns and walls from buckling.

In the last few years four tests have been made to investigate the interaction of diaphragms with ductile concrete moment resisting frames. Three of these were made at Canterbury University and one at Auckland University. At Auckland the main concern was with the potential increase in strength of the beams in the perimeter frame due to interaction with floor slabs. Some of the results and conclusions from this test were reported in the previous conference (Fenwick et al. 2005). At Canterbury, due to the damage that was sustained by the hollowcore units at small displacements and the premature collapse of the floor at displacements less than the design value, concern was focused on developing detailing which would give acceptable behaviour (Matthews, Lindsay, MacPherson).
Some of the measurements taken during one of the tests (MacPherson), which were not examined in detail at the time, have been studied and found to throw considerable light on the interaction of beams and diaphragms containing precast prestressed units.

A major cause of interaction between diaphragms and beams in ductile moment resisting frames is the elongation that occurs in beams when flexural cracks form. This increases when plastic hinges develop (Megget and Fenwick 1989, Fenwick and Megget 1993). Tension is induced in a floor slab as it partially restrains the elongation of the beams. This can result in a very substantial increase in strength when a floor contains prestressed floor units such as hollowcore, double Tee or stem beams.

2 PERIMETER FRAME AND FLOOR SLAB TEST UNIT AND LOADING SYSTEM

Figure 1 shows one of the test sub-assemblies constructed at Canterbury (MacPherson 2005). On one
side of the sub-assembly (line 1) was a frame with two bays each of 6,100mm, while at each end were two transverse frames, on lines A and C, which also had spans of 6,100mm. The floor slab was constructed from 300 series hollowcore units, which were each prestressed with seven 12mm strands. The units supported on the transverse beams giving them an effective span between support points of 11,725mm. A gap of 750mm was left between the side of the beams on line 1 and the side of the first hollowcore unit. A 75mm insitu concrete topping was cast on top of the hollowcore units and extended over the 750mm gap to give a 75mm thick slab linking the beams to the hollowcore units, as recommended in Amendment 3 to the Structural Concrete Standard [NZS 1995]. The insitu concrete was reinforced with D12mm bars at 300mm centre to centre in both directions over the complete floor. The hollowcore units were seated on low friction bearing strips on the transverse beams. At each end of the hollowcore units two cells were broken out over a distance of 900mm and these were reinforced by a single plain 16mm bar that was located as close to the base of the cell as possible. These were filled when the insitu concrete was placed. As shown in Figure 1 the transverse beams were reinforced to keep the plastic hinge regions adjacent to the columns. With this test arrangement the prestressed hollowcore units spanned directly past the central column, B, in the frame on line 1. Critical reinforcement details are given on Figure 1 together with key material strengths. Further details may be obtained from the reference (MacPherson).

An elaborate loading system was used to apply the structural actions to the perimeter and transverse frames. The shear was applied to the columns at levels of 1.75m above and below the mid-height of the perimeter and transverse beams. This loading system ensured that;
- Equal and opposite shear forces were applied to each column;
- The columns remained parallel to each other;
- There was no restraint to elongation of the beams imposed by the loading system.

With this arrangement the induced structural actions should be representative of those sustained by a perimeter frame and floor slab in mid levels of multi-storey buildings. In this region axial loads sustained by the beams and the portion of the floor slabs that act compositely with them are small.

There were three phases to the loading sequence. In the first and third phases lateral forces were applied to the three columns, A, B and C, shown on line 1 in Figure 1, in the longitudinal direction. In the second phase loading was applied in the transverse direction to the end frames on lines A and C. The relative displacement between the load points on the columns above and below the floor slab were assumed to be equivalent to the inter-storey drift. In all loading stages except the first cycle in phase three two compete cycles to ± predetermined inter-storey drifts were applied. The details of the displacement cycles are given in Table 1.

<table>
<thead>
<tr>
<th>Loading Phase</th>
<th>Loading direction</th>
<th>Percentage inter-storey drift</th>
</tr>
</thead>
<tbody>
<tr>
<td>Phase 1</td>
<td>Along line 1 to columns A, B and C</td>
<td>±0.5, ±1.0 &amp; ±2</td>
</tr>
<tr>
<td>Phase 2</td>
<td>Along lines A and C to columns A, D, C &amp; E</td>
<td>±0.5, ±1.0, ±2.0 &amp; ±3.0</td>
</tr>
<tr>
<td>Phase 3</td>
<td>Along line 1 columns to A, B and C</td>
<td>±2, ±3.0, ±4.0 &amp; ±5.0</td>
</tr>
</tbody>
</table>

3 TEST RESULTS

During the test diagonal cracks developed in the linking slab and extended into the topping on the hollowcore units. The cracks were not wide and they inclined at an angle of close to 30° to the axis of the beam following the pattern shown in Figure 3 (a). Photographs of the crack patterns can be seen in references (MacPherson, MacPherson et al. 2005). Wide cracks formed between the transverse beams and the floor slab near the supports of the hollowcore units in the vicinity of the beams in line 1. It is apparent that these cracks were due to elongation of the plastic hinges, which formed in the beams at the face of the columns A and C. From the observed crack pattern it is apparent the floor slab tended to restrain elongation associated with the plastic hinges in the beams adjacent to the central
Displacement measurements were made over a grid pattern on the slab. Summing the strains along grid lines between the transverse cracks at the supports on the transverse beams indicated the extent that this slab acted to provide restraint to elongation caused by the plastic hinges in the beams adjacent to the central column. The results of these measurements are shown in Figure 2. They indicate that the floor slab within a distance of about 3 times the beam depth, measured from the face of the beam, participated in providing restraint to elongation. This corresponds to a distance of close to 2,500mm from the beam centreline.

In this paper only the actions in the longitudinal direction, phases 1 and 3 of the loading regime, are considered. Two strength values are of interest, namely the theoretical strength, which should be developed at inter-storey drifts of the order of 0.75% and the over-strength at drifts of the order of 2 to 3%. Table 2 gives the maximum, minimum and average shear forces sustained by the columns at the peak displacements in each cycle for drift levels of 0.5 to 3 percent. The actions sustained at the 4 and 5 percent drift cycles were smaller than the corresponding values in the 3 percent cycles.

<table>
<thead>
<tr>
<th>Drift %</th>
<th>Columns A and C, +ve</th>
<th>Columns A and C -ve</th>
<th>Column B, ± ve</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Max.</td>
<td>Min.</td>
<td>Average</td>
</tr>
<tr>
<td>0.5</td>
<td>115</td>
<td>113</td>
<td>114</td>
</tr>
<tr>
<td>1.0</td>
<td>191</td>
<td>164</td>
<td>173</td>
</tr>
<tr>
<td>2.0</td>
<td>242</td>
<td>204</td>
<td>214</td>
</tr>
<tr>
<td>3.0</td>
<td>251</td>
<td>174</td>
<td>208</td>
</tr>
</tbody>
</table>

The shear forces sustained at 3% drift were less than those at 2%. This was probably a result of the sub-assembly sustaining appreciable damage in the phase 2 loading, which occurred between these two drift levels.

4 **BASIC THEORY**

4.1 *Internal column where precast unit spans past the column (such as column B)*

When flexural cracks form in a beam it increases in length. This elongation increases very significantly when plastic hinges develop. A portion of a floor slab constructed alongside a beam will
partially restrain this elongation. The region of the floor which provides this restraint is subsequently referred to as the effective flange. As illustrated in Figure 3 (a) a horizontal truss like action develops, with the slab which links the beam to the first precast unit (linking slab) in the floor, acting like the web of a beam to transmit horizontal shear between the beam and the floor. It should be noted that the direction of horizontal shear in the linking slab opposes the elongation of the beam and consequently when the load direction reverses the direction of shear force does not reverse as the beam continues to elongate. In several tests (Lau, Lindsay, MacPherson) this linking slab has been observed to develop cracks at an angle of close to 30° to the axis of the beam, allowing diagonal compression forces to be sustained between these cracks, as shown in Figure 3. At the central column the cracks are close to right angles to the beam axis. The horizontal forces in the effective flange necessary for equilibrium in the direction normal to the beams are sustained by the reinforcement in the insitu concrete, and in
the longitudinal direction by the precast units acting together with its associated insitu concrete and longitudinal reinforcement. As shown later in this section it is this longitudinal force that increases the flexural strength of the beams.

Shear forces acting at mid height of the linking slab induce longitudinal tension in the effective flange. This force acts at mid height of the linking slab and it is eccentric to the composite hollowcore and insitu concrete section, as illustrated in Figure 4. Part (a) of this figure shows the shear forces acting on the slab with the result that an eccentric tension force is induced in the effective flange in the mid regions of the span of the precast floor units.

Part (b) of Figure 4 shows the actions that could be sustained on a composite precast unit and insitu topping if no vertical loads acted on it. In this situation there are no bending moments acting on the section and consequently the prestress compression force is concentric with the prestress force in the pretension strands. As the tension force in the effective flange acts at mid height of the insitu concrete, \( T_{\text{flange}} \), can be found by taking moments about the centroid of the pretensioning strands. On this basis \( T_{\text{flange}} \) is equal to the tension force carried by the reinforcement in the insitu concrete plus any additional force that can be resisted by the negative flexural strength of the precast unit. However, as the prestressed strands are located close to the bottom surface of the unit the internal lever-arm, which is shown as \( l_a \) in the figure, is small and for practical purposes may be taken as zero. On the basis of this assumption the value of \( T_{\text{flange}} \) may be assessed by-

\[
T_{\text{flange}} = A_{\text{slab}} f_s
\]

where \( A_{\text{slab}} \) is the area of longitudinal reinforcement in the topping of the effective flange and \( f_s \) is the stress in this reinforcement.

Part (c) of Figure 4 shows the actions associated with the bending moments arising in the effective flange due to vertical forces that act on it. It spans like a slab or beam between its support points on the transverse beams. Bending moment in the effective flange causes the centre of compression in the section to rise. In this situation the bending moment acting on the effective flange, \( M_v \), is equal to the compression force times the internal lever-arm, \( l_a \). If a tension force, \( T_{\text{flange}} \), is now applied its maximum value can be found by taking moments about the centroid of the pretension strands. It is given by-

\[
T_{\text{flange}} = \frac{M_v}{z} + A_{\text{slab}} f_s
\]

where \( z \) is the distance between the mid-height of the insitu concrete and the centroid of the prestressing tendons, as illustrated in Figure 4 (d).

As the tension force, \( T_{\text{flange}} \), is eccentric to the effective flange bending moments are induced in the composite precast insitu concrete section and consequently it deflects upwards. This movement induces rotation at the ends of the precast units and hence positive moments are developed in these locations if there is continuity reinforcement located in the bottoms of the units, as was the case in the test unit shown in Figure 1 (section C-C). As shown on this figure one 16mm bar was placed in each of the two broken out cells at both ends of each hollowcore unit. When the frame is loaded the beams deform, but due to the longer span of the precast hollowcore units these rise relative to the longitudinal beams. This causes the 75mm thick linking slab, which connects the beam to the first hollowcore unit, to distort as shown in Figure 3 (b) and to transfer vertical shear between the beam and the first precast unit and insitu concrete topping. As both the tension force in the transverse reinforcement and the compression force in the diagonal strut are of constant magnitude over the width of the linking slab, see Figure 3 (c), unmodified conventional flexural theory does not apply. In this case the bending moment is resisted by the displacement of the compression force in the strut as shown in Figure 3 (d) with the shear equalling the vertical component of the inclined force. This concept is described in reference (Fenwick) for the design of webs subjected to shear and out of plane flexure and it was developed and used for predicting the actions in a linking slab (Lau). It should be noted that the
diagonal compression forces in the linking slab are sustained in a region containing diagonal cracks and these reduce the compressive strength of the concrete [NZS, 1995]. In this case it is assumed the strength is lowered from $0.85 \sigma_c$ to $0.6 \sigma_c$. Assuming a $30^\circ$ angle for the struts corresponding stress normal to the beams is equal to $0.6 \sigma_c / \sin^2 30^\circ$, which reduces to or $0.15 \sigma_c$. Using this value the location of the compression force at each end of the linking slab can be determined. In effect the flexural strength can be found using standard theory with the modification that $\alpha_1$, which is usually 0.85, is taken as 0.15. At the central column the situation is different in that in this location the cracks are essentially normal to the column. Consequently in this location standard flexural theory can be used with a stress of $0.6 \sigma_c$ in the concrete. From the flexural strengths of the linking slab the vertical shear forces that can be transmitted from the beam to the first precast unit can be found. Only relatively small vertical displacements are required to sustain the vertical shear forces. Consequently for practical purposes in the calculation of over-strength actions the distribution of the vertical shear forces between the beams and first precast unit may be assumed to be uniform. However, the shear force transmitted by the linking slab in the strip associated with the central column is likely to be appreciably higher than for a comparable length of beam due to two reasons. Firstly due to the reinforcement required to tie the column into the diaphragm, and secondly due to the shorter span of the linking slab in this location (in the test unit this distance was 575mm instead of 750mm).
The bending moment in the effective flange, \( M_v \), corresponding to over-strength, can be found by summing the moments arising from:

- Gravity loads that act on the effective width of slab;
- Vertical shear forces from the linking slab;
- Positive moments applied to the support points of the units in the effective flange.

Figure 2 indicates that the effective flange in this test was equal to the width of floor slab that lies between the side of web of the beam and a distance of three times the beam depth from the web. Other effective widths might be appropriate in different situations. However, as only the one situation has been tested this width has been assumed in the following calculations and it has been adopted in the (draft) Structural Concrete Standard (NZS, 2006). With this assumption the bending moment in the effective flange can be determined and the over-strength longitudinal tension force carried by the associated hollowcore units and insitu concrete in the effective flange found.

4.2 **External columns (such as A and C)**

A wide crack develops at the junction between the floor slab and the transverse beams in the region close to the columns. Reinforcement in the appropriate width, defined below, which crosses this crack, has been assumed to act to increase both the theoretical and over-strength values of the beams in line 1 at the face of the external columns. The term theoretical strength in this context applies to the calculated strength using standard flexural theory and measured material properties while nominal strengths are found using lower characteristic material properties. The effectiveness of this reinforcement in increasing the strength of the beams in line 1 depends on the lateral and torsional strength of the transverse beams, which is likely to decrease when plastic hinges develop in the transverse beams. In this test plastic hinges formed in the transverse beams in the phase 2 loading cycles.

The (draft) Structural Concrete Standard (NZS, 2006) assumes that for nominal strength calculations the reinforcement lying within a distance of twice the width of the transverse beam measured from the face of the column acts to increase the strength. For over-strength values this distance is increased to three transverse beam widths. These values have been used in calculations reported in section 6.

5 **COMPARISON OF EXISTING AND DRAFT STRUCTURAL CONCRETE STANDARDS**

A comparison of the requirements for the effective widths of tension flanges for the longitudinal beams shown in Figure 1 is given in Table 3. The clauses relating to the effective flange widths in the Structural Concrete Standard, NZS 3101-2006, have been developed from NZS3101-1995 and a review of the test results obtained on the four frame diaphragm tests carried out recently in New Zealand. The major difference between these is that the new Standard recognises the contribution of the prestressed units can make to the tension force that can be resisted by the effective flange while the previous Standard made no such recognition.

6 **CALCULATION OF THEORETICAL AND OVER-STRENGTH OF BEAMS**

6.1 **General**

In calculations of both the theoretical and over-strength values for the beams the measured cylinder strength of the concrete was used. For the theoretical strengths the stress in the reinforcement was limited to the measured yield stress in both tension and compression. For the over-strength calculations the stress in tension reinforcement in the beams and in the topping at the ends of the precast units was limited to the measured stress levels sustained at a tensile strain of 5%, while for compression it was limited to the yield strength. For the reinforcement in the insitu topping away from the ends the limiting stress was taken as the yield stress as the crack widths in this region were not wide and appreciable strain hardening was unlikely to have occurred.
Table 3: Comparison of effective tension flange widths in NZS3101-1995 and DZ3101-2005

<table>
<thead>
<tr>
<th>NZS3101-1995</th>
<th>All columns, A, B and C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective width = ½ span transverse beam or ¼ span of beam minus half web width.</td>
<td>For test this was 1.325mm</td>
</tr>
<tr>
<td>The possible contribution of prestressed units to strength is not mentioned and it appears to be ignored in practice.</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>NZS3101-2006</th>
<th>Corner columns, A and C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nominal strength</td>
<td>Effective flange = 2 transverse beam widths measured from face of column</td>
</tr>
<tr>
<td>For this test this was 900mm.</td>
<td></td>
</tr>
<tr>
<td>Over-strength</td>
<td>Effective width = 3 times transverse beam width measured for face of column</td>
</tr>
<tr>
<td>For this test this was 1,300mm</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>For internal column, B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nominal strength</td>
</tr>
<tr>
<td>Over-strength</td>
</tr>
<tr>
<td>The contribution of prestressed components (or parts of components) is included as detailed in section 4 of paper.</td>
</tr>
</tbody>
</table>

6.2 Summary of calculations for over-strength of beams at Column B

The effective width of flange measured from the outside edge of the beam is 3 x beam depth giving a distance of 2,250mm. Of this 750mm is linking slab and the remaining 1,500mm is hollowcore units (1.25 units).

The dead load per metre run of this effective flange, including half the weight of the linking slab, is 8.03kN/m. The calculated flexural strength of the linking slab, based on the yield strength of the reinforcement and found following the approach set out in section 4, is 3.22kNm/m alongside the beams and 10.1kNm at the central column. This corresponds to shears of 8.58kN/m along the beams and a shear force of 35kN at the column.

The positive moment capacity due to reinforcement in the cores at the ends of the units arises from two and a half 16mm bars (1,500/600 bars at a spacing was 600mm) giving an area of 503mm². Taking the lever-arm as between the slab reinforcement and the centre of the 16mm bars gives a positive moment capacity of 49kNm. Hence the positive moment due to vertical forces and the end moment of 49kNm is 412.8kNm at mid span. As the height of the prestressed strands was measured as 44mm, the distance, z, between the strands and mid depth of the insitu concrete is 0.293m, the tension force resisted by the hollowcore units is \((412.8/0.293)\) 1,409kN. To this is added the tension force that can be resisted by the reinforcement in the insitu concrete, which is based on the yield strength. As the width is 2,250mm and the bars are at a spacing of 300mm the area is 848mm² and a tension force of 260kN. Hence the total calculated tension force in the flange, \(T_{flange}\), is 1,667kN.

The calculated over-strength tension force of the reinforcement in the web of beams in the plastic hinge regions was 970kN. This was 1.23 time the corresponding value for the theoretical strength, which was based on yield strength. The calculated tension force in the effective flange was approximately 1.5 times this value. The effective compression flange width for positive moments, shown in Figure 5, was taken as equal to beam web plus 4 times the slab thickness (700mm). Based on these assumptions the negative and positive beam over-strength moments at the face of column B were calculated as 1,634kNm and 605kNm respectively.

It may be seen from Figure 5 that the tension force carried by the flange, \(T_{flange}\), results in a small decrease in the positive moment flexural strength due to the increased magnitude of the compression force, and hence a reduction in the effective internal level arm. However, the force in the flange makes a major contribution to the negative moment strength. Hence it is the increase in the negative moment capacity, which leads to the increase in the structural actions acting in the column.
From the calculated theoretical strength and over-strength values the moment input into the columns were calculated and hence the shear force resisted by the columns were be determined. These values are compared with the experimentally measured values in Table 4. It was anticipated that the theoretical strengths would be sustained at a drift of the order of 0.75 percent. As there was no load cycle to this displacement the experimental theoretical strength values, for the purpose of this comparison, have been taken as the average of the values sustained at the 0.5 and 1.0 percent drift cycles.

Figure 5: Flexural strength of beams at central column
Table 4: Comparison of analytical and experimental column shears

<table>
<thead>
<tr>
<th>Standard or test result</th>
<th>Action</th>
<th>Columns A and C</th>
<th>Column B</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>-ve</td>
<td>+ve</td>
</tr>
<tr>
<td>NZS 3101-1995</td>
<td>Theoretical</td>
<td>202</td>
<td>172</td>
</tr>
<tr>
<td></td>
<td>Over-strength</td>
<td>246</td>
<td>209</td>
</tr>
<tr>
<td>DZ3101-2005</td>
<td>Theoretical</td>
<td>202</td>
<td>172</td>
</tr>
<tr>
<td></td>
<td>Over-strength</td>
<td>250</td>
<td>228</td>
</tr>
<tr>
<td>Test results</td>
<td>Average of 0.5 &amp; 1%</td>
<td>208</td>
<td>143</td>
</tr>
<tr>
<td></td>
<td>Average at 2%</td>
<td>282</td>
<td>214</td>
</tr>
<tr>
<td></td>
<td>Average at 3%</td>
<td>263</td>
<td>208</td>
</tr>
</tbody>
</table>

7 DISCUSSION AND CONCLUSIONS

1 Tests of a level of a perimeter frame together with a floor slab containing precast prestressed units show that the presence of the slab resulted in a very significant increase in strength of the beams.

2 The test results indicate that the current design criteria in the Structural Concrete Standard, NZS 3101-1995, give reasonable estimates of the flexural strength and over-strength of plastic hinge regions in beams at the faces of corner columns in perimeter frames. However, the application of these criteria to the columns in perimeter frames results in major under-estimates of flexural over-strength actions if floor slabs contain precast prestressed components. For the central column in the test, which is described in the paper, the measured over-strength actions in the column were approximately 60 percent in excess of the corresponding values predicted from the Structural Concrete Standard, NZS 3101-1995.

3 The basic theory relating to the interaction of precast prestressed units in a floor slab to the flexural strength of beams is outlined. To apply the theory the effective width of floor slab, which acts with the beam (effective flange), needs to be determined. In this paper an effective flange width was found from strain measurements made on the slab in one test. However, in different structural situations different effective widths may be appropriate. Further work on this aspect is required.

4 Application of the basic theory together with the experimentally determined effective flange width gave over-strength values that were in reasonable agreement with the experimental values. It is shown that the interaction of the floor slab with the beams greatly increased the negative flexural strength of the beams and it led to a small decrease in the positive moment strength.

5 The basic theory outlined in this paper together with the experimentally observed effective flange width have been adopted in the Structural Concrete Standard, NZS3101-2006 (NZS 2006) for calculating flexural over-strengths. The theoretical strength values calculated for the central perimeter frame column appear to be on the low side and it is likely that further research will enable better estimates of this value to be determined.

6 Further research is required so that reliable estimates may be made of flexural over-strength of beams in perimeter frames at columns where precast units are supported on both sides of a transverse beam framing into a column. This is the situation of column C in Figure 3. This aspect was examined in a previous paper (Fenwick et al 2005). Test results showed that very significant strength enhancement occurred in this situation. However, practical methods of assessing over-strength values for this situation have yet to be developed.
ACKNOWLEDGEMENTS

The support provided by the CCANZ is gratefully acknowledged

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