Appendix A

Estimating Seismic Demands for loading structural components in laboratory experiments

Summarised within this appendix is all the results obtained from the various time history analyses run.

For each earthquake there will be the following summarised:

The earthquake record

General performance of the structure (roof displacement vs time)

Localised storey performance (maximum interstorey drift vs time)

Note that the graphs that have been included for the localised storey performance are only for the floors in which the maximum drift is observed to occur.

For example, the results from EL40NSC saw the 3, 6, 9 and 12 storey structures have there maximum interstorey drifts in the 1st, 2nd, 3rd and 5th stories respectively.
A.1 EL40NSC results

(a) EL40NSC earthquake record

(b) General performance of the structures

(c) Localised storey performance

A-2
A.2 TAFTNW results

(a) TAFTNW earthquake record

(b) General performance of the structures

(c) Localised storey performance
A.3 SYFF943 results

(a) SYFF943 earthquake record

(b) General performance of the structures

(c) Localised storey performance
A.4 SYLM949 results

(a) SYLM949 earthquake record

(b) General performance of the structures

(c) Localised storey performance
A.5 KOBE95NS results

(a) KOBE95NS earthquake record

(b) General performance of the structures

(c) Localised storey performance
A.6 KOBE95EW results

(a) KOBE95EW earthquake record

(b) General performance of the structures

(c) Localised storey performance
A.7 4203ELC1 results

(a) 4203ELC1 earthquake record

(b) General performance of the structures

(c) Localised storey performance
A.8 4203ELC2 results

(a) 4203ELC2 earthquake record

(b) General performance of the structures

(c) Localised storey performance
A.9 4203OLY1 results

(a) 4203OLY1 earthquake record

(b) General performance of the structures

(c) Localised storey performance
A.10 4203OLY2 results

(a) 4203OLY2 earthquake record

(b) General performance of the structures

(c) Localised storey performance
A.11 4203TFT1 results

(a) 4203TFT1 earthquake record

(b) General performance of the structures

(c) Localised storey performance
A.12 4203TFT2 results

(a) 4203TFT2 earthquake record

(b) General performance of the structures

(c) Localised storey performance
A.13 SEC1 results

(a) SEC1 earthquake record

(b) General performance of the structures

(c) Localised storey performance
A.14 SSF1 results

(a) SSF1 earthquake record

(b) General performance of the structures

(c) Localised storey performance
A.15 SSF5 results

(a) SSF5 earthquake record

(b) General performance of the structures

(c) Localised storey performance
A.16 SIV1 results

(a) SIV1 earthquake record

(b) General performance of the structures

(c) Localised storey performance
A.17 SKB1 results

(a) SKB1 earthquake record

(b) General performance of the structures

(c) Localised storey performance
A.18 SKB2 results

(a) SKB2 earthquake record

(b) General performance of the structures

(c) Localised storey performance
A.19 SNOR1 results

(a) SNOR1 earthquake record

(b) General performance of the structures

(c) Localised storey performance
A.20 SNOR2 results

(a) SNOR2 earthquake record

(b) General performance of the structures

(c) Localised storey performance
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Appendix B

Design of the Experiment

B.1 Design of the test specimen

B.1.1 Beam design

The beams were designed in accordance with the New Zealand Concrete Standard (NZS3101:1995). A typical longitudinal reinforcement ratio of 0.01 was assumed and then the rest of the design was based around that ratio. In order to ensure a weak beam-strong column mechanism formed several moment curvature analyses were undertaken to determine the possible overstrength actions for the beams. The sensitivity study undertaken analysed the affect that varying effective flange width, varying reinforcement properties and varying concrete strengths has on the beams overstrength actions.

Five different flange widths were investigated. The first being the beam only (i.e. no slab interaction), the second was activating the reinforcing mesh within the cast insitu diaphragm topping, the third was activating the prestressing tendons within the precast prestressed hollow-core units. The fourth was a combination of the second and third scenarios while the fifth scenario was activating the transverse beam starter bars.

Three different concrete strengths were analysed. The first was 30 MPa, as this is the specified 28 day compressive strength for the precast units. Two other concrete strengths (45 and 50 MPa) were also investigated, as the probability of the concrete strength being 30 MPa was quite low.
Combinations of varying reinforcing strengths were investigated to see what effect these had on the overall strength of the beam. The lower fifth percentile values for the reinforcing steel were used as well as the specified maximum allowable values as stated by the NZS3402:1989.

Before the sensitivity study was commenced a preliminary beam design was undertaken to determine the longitudinal and transverse reinforcement ratios. This sensitivity study investigated all the various combinations of overstrength actions that the beams could possibly experience. The maximum overstrength actions obtained from the study were then used to determine the finalised design actions for the remainder of the test specimen and load frame.

According to the NZS3101:1995 and Cheung et al (1991), a portion of the tension flange must be accounted for when determining the negative overstrength bending moment capacity of the beam. The extra tension steel only contributes to the tension reinforcement if it has developed its full capacity at the zone of interest. This increased negative overstrength moment was taken account of when ensuring that the columns did not hinge. Another scenario was also investigated to ensure the columns do not hinge, this case was when a large crack opens at the start of the hollow-core unit. This could quite possibly happen as the discontinuity between the hollow-core unit and the perimeter bream acts as a crack instigator. The length of the crack was taken to be the same as the effective flange width used above.

The discontinuity crack scenario gave a larger negative overstrength bending moment than the NZS3101:1995 recommendation so was used to design the columns and hence protect them against hinging ensuring that the appropriate post-elastic mechanism formed.
B.1.2 Column Design

A capacity design approach was used to design the columns. The overstrength moments and shears acting in the beam plastic hinge zones were used to determine the design actions for the columns. As explained in the beam design the amount of enhancement from the floor diaphragm to the beams flexural strength was unknown. To ensure a strong column weak beam mechanism formed it was crucial to give the central column extra protection to ensure the column did not hinge during testing. To achieve this, the central column had a relatively high percentage of longitudinal reinforcing consisting of 24-HD24 bars ($\rho=0.022$). Since the columns have 12 drossbach ducts per column, this particular column needed two bars per duct. Though this is not common practice it was considered acceptable for this test unit, as the columns performance was not a focal point. The only column performance criterion that this research required was for the columns not to form plastic hinges.

The longitudinal reinforcement at both the top and bottom of the columns was welded onto the column end plates. This ensured the endplates were not pulled off during testing due to the load being transferred from the load frame connection brackets to the columns.

B.2 Loading Frame Design

B.2.1 Primary Loading Frame

Upon completion of the test specimen design it was possible to design the load frame. Since all the overstrength actions were known, as well as the way in which the specimen was loaded, it was possible to size the load frame.
The design of the loading frame was carried out in accordance with the Steel Structures Standard (NZS 3404:1997). A key component to the design of the load frame was to keep all the components as similar as possible for ease of construction. The load frame was designed to withstand a force of 1000 kN being applied from each of the hydraulic actuators. The load frame consists of two sets of scissor arms.

Figure B-1 shows one of the scissor load frames. Each individual load arm consists of a W12-202 section. A spherical bearing was used to attach the load arm to the column connection bracket so that all the rotations that occur during testing can be accommodated.

![Diagram of scissor load frame](image)

**Figure B-1 A typical scissor load frame**

Since both ends of the scissor load frame are attached to the columns by spherical bearings a prop (Figure B-2) is required, to be attached to the load frame, to stop the frame from dropping and resting against the opposite load frame. If the load frame was to rest against the opposite load frame there is a possibility that the system could bind. The prop is placed on a load skate so that the inward and outward movement of the frame is accommodated. The load transferred through this prop is only the self-weight of the load frame, hydraulic actuator and loadcell.
Figure B-2 Location of the load frame props

The load frame has been designed so that it can be unbolted from the longitudinal frame and placed onto the transverse frame.

### B.3 Secondary loading frame

As the test specimen is taken into the inelastic range, the flexural strength in the positive and negative plastic hinges will not be equal. This could be due to either the slab reinforcement contributing to the beams flexural strength or one of the plastic hinges degrading in strength. When the beam flexural strengths become unbalanced, the diagonal loading frame is unable to maintain equilibrium. To ensure equilibrium is maintained, a secondary loading frame is required.

Due to the large number of components intersecting at the columns point of inflections, it would be extremely difficult to construct the specimen with all the centrelines meeting at the same point. A study was undertaken to see how the loads in the specimen would change if a different connection was used for both the primary and secondary loading frames. The study showed that the variance in loads was little and that at full displacement the centrelines were out of alignment by less than 3% of
the column height. This difference was insignificant allowing the specimen to be designed using separate pins for all the connections. Using separate pins allowed smaller components to be used and many of the components could be duplicated for other parts of the specimen.

**B.3.1 Design considerations**

A suite of analyses was carried out to determine the maximum expected load in each leg of the secondary loading frame (SLF). The analyses looked at the effect that a varying number of degraded hinges within the bent had on the applied loading. The number of hinges that degraded varied between zero and four. All the analyses were carried out using an elastic structural analysis programme. Since the test specimen will not remain elastic, a pushover type analysis was undertaken to represent this.

The pushover analysis was undertaken in the following way. The test specimens coordinates, end supports, properties and members were all inputted into the programme. An analysis was run and the applied loads were altered until one of the beams reached its yield moment. Once this value was obtained all the applied loads and member forces were recorded and a pin (release) was placed at the location of the expected yielding. Again, the loads were increased until the next member yielded. This process was continued until a mechanism formed. Summation of all the applied loading determined the applied base shear and loads within the SLF.

The initial loadcase consisted of every hinge reaching its full capacity. This determined the maximum load required in the primary load frame and the respective loads in the SLF. The process was then rerun with 50% strength degradation in all the plastic hinge zones. Since the hinges appeared weaker, due to the strength degradation, the load required to form the mechanism was less than if each hinge was
at full capacity. The assumption was then made that the difference in load, in the primary loading frame, between the two mentioned load cases needed to be transferred through the SLF. This is because the load that was initially resisted by the super-assembly would be resisted by the SLF when the plastic hinges degrade in strength. The initial loads in the SLF due to the first loadcase were also added.

Several load scenarios were examined to determine a design envelope of forces for the SLF. The different load scenarios focussed on varying amounts of strength degradation within the plastic hinge zones. This method of determining the SLF loads is conservative, it had the added advantage that the members designed would be relatively stiff ensuring minimal axial lengthening and shortening of the SLF occurred during the experimental programme.

The diagonal arms of the SLF were designed for both compression and tension. Varying effective lengths were used to design these members. Localised buckling of the members was designed for as well as the entire frame buckling.

At the connection where the SLF attaches onto the column face, a spherical bearing was attached to allow for all the displacements expected during the test. The connection is pinned to the column to allow the column to sway during loading. The connection was also required to handle some out of plane rotation to occur when the column is displaced transversely to the main longitudinal frame, hence the reason for using a spherical bearing.

The vertical arm of the SLF was made from two different section profiles, a rigid Universal Channel section and a steel bar. The steel bar is required, as it has to pass through the centre of the perimeter beam. A steel bar was chosen, as this would keep the beams penetration as small as possible (Figure B-3). When designing this vertical member for compression it was too conservative to design it assuming the
entire member was this reduced section. Designing the member in that manner meant that the member was so large it was impossible to make it fit through the beam. Using a more realistic design philosophy made it possible to determine the reduced sections effective length. When the vertical member was split into two portions, a UC and steel bar section, the assumption was made that all the deformation occurred in the steel bar as the ratio of the two sections moment of inertias was greater than five.

![Reduced Section passing through the beam](image)

**Figure B-3 Photo of the reduced section within the vertical leg of the SLF**

Also, within the vertical member of the SLF a pin type arrangement was added to eliminate the vertical member from being subjected to torque. During testing there is a possibility that the columns will not remain parallel with each other in the out of plane context. If this did occur, and the pin was not present, then the vertical member would be subjected to a torque, causing premature yielding of this member. In order for the pin to be able to rotate the bearing stress at the pins connection with the vertical member has been limited to a stress much less than 150 MPa. The threaded pin also allowed some vertical adjustment to the assembly of the SLF.
To allow the reduced vertical member to be passed through the beam the bottom pinned connection will be bolted on after the member has been passed through the beam (Figure B-4).

![Bolted connection at the base of the vertical leg of the SLF](image)

**Figure B-4 Bolted connection at the base of the vertical leg of the SLF**

### B.4 Transverse loading

The primary and secondary loading frames are able to load the super-assemblage in both the longitudinal and the transverse directions. The central column cannot be loaded in the same way in the transverse direction as the floor diaphragm does not allow the scissor load arms to be attached, therefore a new load frame is required. To displace the central column a hydraulic actuator is attached to the top and bottom of the column (Figure B-5). The load frame consists of a Universal Channel and hydraulic actuator placed between two spherical bearings at the top of the column and just the actuator and the spherical bearings at the bottom.
B.5 Special considerations

B.5.1 Catch frames

For safety reasons catch frames were placed under the floor so that if the floor failed and dropped during testing it would not collapse to the ground. These frames were placed each side of the central column as can be seen in Figure B-6(a). At each end of the super-assembly, steel angles were bolted onto the transverse beam face to act as catch brackets (Figure B-6(b)).

(a) Large catch frame either side of the central column to catch the floor if it failed  
(b) Steel angle bolted to the supporting beam to catch the floor if it loses its seat

Figure B-6 Catch frames placed to catch the floor if it failed during the test.
**B.5.2 Universal joint connection**

To simulate the columns point of inflection and to allow the column to incline, for either direction of loading, a universal joint was placed at the bottom of each column. The universal joint comprised of a double-pinned connection detail (Figure B-7). The top half of the universal is supported on two legs, which are, seated a 60mm diameter steel pin. This steel pin allows the rotation to occur in one direction. The centre of the 60mm pin is housed within a larger 200mm diameter pin that allows rotation in the other direction. The larger pin is also supported by the bottom half of the universal joint on two legs. These legs allow the pin to rotate when the test specimen is displaced.

The design of the universal joint was governed by bi-axial loading. Although the super-assemblage is not being loaded in that manner, it was designed for those loads as future experiments may include bi-axial loading.

![Figure B-7. The universal joint set up](image)
B.5.3 Front bearing design

The bearings required at the base of the three front columns were required to be capable of allowing unrestrained movement in any direction parallel to the floor, while withstanding both compression and tension loading. This was achieved by using linear bearings. A grillage of rails, in both the x and y directions, were set up to allow the direction of movement required. The running blocks that are attached to the ball rails are able to withstand both tension and compression loading. The running block (Figure B-8(a)) units have a series of ball races within its head unit. Each of the ball bearings within the ball races has been designed so that it will have four points of contact that keeps the contact stresses low allowing large loads to be carried. Figure B-8 shows the assembly and completed column base set up.

Design actions and displacement

A design envelope for the axial loads in the bearings was obtained from the analysis undertaken when designing the primary and secondary loading frames. This envelope also included bi-axial loading encase future testing required it.

The expected movement that the linear bearings had to accommodate could be slit up into two components. The first is movement due to the test specimen being displaced. The bearing rails must be long enough to accommodate the movement associated with an interstorey drift displacement of 5%. The second form of movement expected is due to beam elongation. Previous investigations (refer to Chapter 3) found that the magnitude of elongation, per plastic hinge, ranges between 2-5% of the beam depth. This corresponds to a value of 17-38mm of elongation per plastic hinge zone. The value allowed for in the bearing design was 50mm elongation per hinge.
(a) Greasing the running block

(b) Lowering the top section of the double acting rollers into place

(c) Completed bearing

Figure B-8 Photos showing both the individual components of the base connection as well as the completed unit

B.5.4 Rear bearing design

Since the back frame of the test specimen is supposed to represent the remainder of a building these columns would be fixed against displacement in the y direction but free to displace in the x direction. Rather than using the ball rail bearings, it was possible to use a standard set of steel pins between two plates (Figure B-9), as the back columns would remain in compression throughout the duration of the testing programme.
Figure B-9 Steel pin roller connection
The back columns were also placed on universal joints.

B.5.5 Fixed central back column

The central back column was fixed to the ground to ensure the super-assemblage was tied to the strong floor since the remainder of the columns were on rollers. This was
particularly essential since the back hydraulic actuators were not self-equilibrating, thus this fixity was required to transfer the applied shear from the back two hydraulic actuators to the ground.

B.6 Specimen construction and erection

As far as was possible the test specimen was constructed as if it was on a building site. The only difference was that some of the longitudinal reinforcement had strain gauges or other instrumentation stubs attached. No extra care was taken to ensure the workmanship was of a higher standard than what would normally take place on the construction site.

B.6.1 Concrete test Units

The precast components required to build the test specimen were constructed in three stages. The columns and beams were cast on a casting bed in the laboratory while the hollow-core units were cast at Stresscrete.

Column units.

Both the top and bottom column units were cast with drossbach ducts placed full height of the column. Typically, drossbach ducts are only placed in the tops of columns, not both, but since this research is only focusing on the beam performance the ducts were cast full height to ease construction.

Beam units

The precast beam units were cast as half units. The bottom half of the beams were cast along with the full beam column joint. Figure B-10(a) shows a typical beam unit. The central regions of the beams were not cast to allow a mid-span lap splice to be cast when the beam units were erected. The mid-span splices, the top half of the beams
and the floor diaphragm topping were all cast as one after the specimen had been erected (Figure B-10(b)).

(a) Precast components have been erected  
(b) After the topping and lap splices have been poured

Figure B-10 Photographs of the super-assembly

**B.6.2 Hollow-core units**

Care was taken to ensure the hollow-core units used in project were in a condition similar to what would be expected on a typical building site. The units were not rejected if they have any of the following: incorrect surface roughness; split webs in the units (Figure B-11a); holes in the flanges of the units (Figure B-11b); or significant strand pull in (Figure B-11c).
Figure B-11 Photographs showing the defects seen on the hollow-core units used

B.7 Photographs of the construction of the precast components

(a) Close up showing ducts within the columns
(b) Column ready to be placed in the formwork

(c) Columns ready to be poured

(d) Corner beam unit being placed in the formwork
(e) Typical corner beam column joint ready for pouring

(f) Pouring one of the corner units

(g) The hollow-core bed before the units is poured. Note: The strands are in place and stressed
(h) Pouring the hollow-core unit

(i) Close up showing the difference between the brushed and un-brushed top of the hollow-core unit

B.8 Photographs assembling the super-assembly

(a) The precast components being set out on the floor
(b) Erected columns ready for the beams to be attached

(c) Lowering a beam into place

(d) Lowering the top of the column into place

(e) All the precast components are in place
(f) A completed lap splice before the formwork is attached

(g) Preparing for grouting of the drossbach ducts
(h) Grouting the ducts

(i) Lifting the hollow-core units into place

(j) Positioning the last hollow-core unit
(k) Close up showing the hollow-cores seat length

(l) Formwork in place, ready for pouring

(m) Tepping ready to pour
(n) Reinforcing mesh lap splice

(o) Close up showing the starter bars connection detail

(p) Pouring the topping
B.9 Construction of the Loading and Secondary frames.

The fabrication of the frames was carried out in the Civil Engineering machine shop. This fabrication process was very comprehensive and took several months to complete. All the components that required profile cutting were cut by an outside contractor and then assembled and welded in the Laboratory.
B.10 Photographs showing the erection of the loading frames

(a) Positioning the bottom of the secondary loading frame

(b) Bottom portion of the frame assembled

(c) Most of the secondary loading frame attached
(d) Assembled secondary loading frame

(e) Load frame ready to be attached

(f) Lifting of the load frame into position
(g) Large load frame attached

(h) Large load frames attached, hydraulic actuators are removed to allow the small load frame to be attached

(i) Complete assembly
(j) Positioning the back hydraulic actuator

(k) Back hydraulic actuator in position
B.11 Materials

B.11.1 Reinforcing Steel

The tensile properties of the reinforcement used were obtained by monotonic loading in an Avery Universal testing machine. Table B.1 shows the experimental results. Each of the results shown is the average value obtained from the testing of three samples.

Table B.1 Measured properties of the reinforcing tests used in the test specimens.

<table>
<thead>
<tr>
<th>Type</th>
<th>( f_y ) (MPa)</th>
<th>( E_s )</th>
<th>( E_{sh} )</th>
<th>( \varepsilon_{sh} )</th>
<th>( \varepsilon_{su} )</th>
<th>( f_u ) (MPa)</th>
<th>Location</th>
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<tr>
<td>D24</td>
<td>294</td>
<td>238</td>
<td>5.6</td>
<td>0.012</td>
<td>0.11</td>
<td>451</td>
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<td>231</td>
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<td>0.12</td>
<td>582</td>
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<td>0.009</td>
<td>0.17</td>
<td>438</td>
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<td>665</td>
<td>-</td>
<td>195</td>
<td>-</td>
<td>-</td>
<td>0.019</td>
<td>654</td>
<td>Diaphragm reinforcement</td>
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</table>

Figure B-12 shows the individual stress versus strain plots for the various bars tested.
(a) Longitudinal beam reinforcement  
(b) Column reinforcement  
(c) Column reinforcement  
(d) Starter bars  
(e) Transverse reinforcement  
(f) Topping reinforcement

Figure B-12 Individual steel test results
B.11.2 Concrete

The concrete was supplied by Firth Industries Limited. The concretes target 28 day strength was 30 MPa and the maximum aggregate size was 19 mm. Table B.2 shows a summary of the properties for the different portions of the test specimens. Each of the results shown is the average value obtained from the testing of three samples.

Table B.2 Measured concrete properties.

<table>
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<th>Pour Number</th>
<th>Pour Location</th>
<th>Mix Code</th>
<th>28 day strength</th>
<th>Start of test strength</th>
<th>Age at start of test</th>
<th>End of test strength</th>
<th>Age at end of test</th>
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<td>602</td>
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<td>708</td>
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<td>48.5</td>
<td>599</td>
<td>51.9</td>
<td>705</td>
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<tr>
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<td>27.8</td>
<td>34.7</td>
<td>594</td>
<td>33.6</td>
<td>700</td>
</tr>
<tr>
<td>4</td>
<td>Columns</td>
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<td>697</td>
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<tr>
<td>5</td>
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<td>663</td>
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<td>437</td>
<td>40.2</td>
<td>543</td>
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</table>
B.12 Electronic valve controller

The remainder of the flow charts used to are shown below.

Figure B-13 Initial set up flow chart
Figure B-14 Flow chart required to impose a negative drift

Note: Correction is made bay by bay so if one bay is correct only the incorrect bay is adjusted
Basic Operation Task List

Imposing a Positive Inclination

Displacement controlled rams, B & E → Extend

Semi-Load controlled rams, A & D → Retract

Imposing a Negative Inclination

Displacement controlled rams, A & D → Extend

Semi-Load controlled rams, B & E → Retract

Figure B-15 Basic operation of the controller

Error Message P1
INITIAL COLUMN FACE ERROR TO GREAT

Error Message P2
COL FACE ERR TO GREAT >20kN

Error Message P3
UNEXPECTED CFE AFTER MOVING DISPL RAM

Error Message N1
INITIAL COLUMN FACE ERROR TO GREAT

Error Message N2
COL FACE ERR TO GREAT >20kN

Error Message N3
UNEXPECTED CFE AFTER MOVING DISPL RAM

Figure B-16 Additional flowcharts to allow any loading errors to be corrected

B-36
B.13 References


Appendix C

Construction Drawings

Precast Concrete Components Drawings                  C3-2 - C3-24
Connection Drawings                                    C3-26 - C3-35
Primary Loading Frame Drawings                         C3-36 - C3-43
Secondary Loading Frame Drawings                       C3-44 - C3-50

Note: Drawings are no longer to scale as they have been reduced to fit within this document.
Notes:
Construction tolerance
= +/-45mm
fy=300MPa
f'c=33MPa
Min cover=35mm

Note:
Refer to lap splice details for information regarding stirrup and lapped bar location within the in-situ lap splice.

Department of Civil Engineering
University of Canterbury
Christchurch, New Zealand

FACTORATION DRAWINGS
Plan of corner unit B

Drawn: JGM
Date: 9/3/00
Revisions: P Scales: 1:20

Drawing No. P4
Note:
Refer to lap splice details for information regarding stirrup and lapped bar location within the in-situ lap splice.

Notes:
Construction tolerance ±15mm
f' = 300MPa
f'c = 30MPa
Min. cover = 35mm

Department of Civil Engineering
University of Canterbury
Christchurch, New Zealand

FABRICATION DRAWINGS
Plan and Elevation of Straight unit B

Drawn: JCM
Date: 9/3/00
Revision: F
Scales: 1:20

Drawing No. P7
NOTE:
Refer to lap splice details for information regarding stirrup and lapped bar location within the in-situ lap splice.

Notes:
Construction tolerance = ±15mm
fy=300N/mm²
f'c=30MPa
Min cover=35mm

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FABRICATION DRAWINGS
Beam cross section details

Drawn: JGM
Dates: 9/3/00
Revisions: F
Scale: 1:20

Drawing No. P8
Notes:
Construction tolerance
= +/-45mm
f_y = 300MPa
f_c' = 30MPa
Min cover = 35mm

Refer to lap splice details for information regarding stirrup and looped bar location within the in-situ lap splice.
Note:
Refer to lap splice details for information
regarding stirrup and lapped bar location
within the in-situ lap splice

Notes:
Construction tolerance
± +/- 15mm
ty = 300 MPa
t'c = 30 MPa
Min cover = 35 mm
Notes:
Construction tolerances
0 to -3mm
All dimensions mm
f'c=30MPa
f=430MPa
f'c=30MPa
Min cover= 35mm

NE and NW columns

East Face

South Face

<table>
<thead>
<tr>
<th>Department of Civil Engineering</th>
<th>FABRICATION DRAWINGS</th>
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<tbody>
<tr>
<td>University of Canterbury</td>
<td>Plan and Elevation of</td>
</tr>
<tr>
<td>Christchurch, New Zealand</td>
<td>Column unit 1 (top)</td>
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</tbody>
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Drawn: JCM
Date: 9/3/00
Revisions: E
Scale: 1/20
Drawing No. P16
Notes:
- Construction tolerances: 0 to -3mm
- All dimensions in mm
- $f_y = 300$ MPa
- $f_t = 430$ MPa
- $f_c = 30$ MPa
- Min cover = 35 mm

FABRICATION DRAWINGS
Plan and elevation of column unit 1 (bottom)

Number req'd = 2 off

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Drawn: JGM
Date: 9/3/00
Revisions: F Scales: 1:20

Drawing No.
P17
SE and SW columns  South Face  East Face

Notes:
Construction tolerances
0 to +3mm
All dimensions mm
fyh = 300MPa
fy = 430MPa
f'c = 30MPa
Min. cover = 35mm

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Fabrication Drawings
Plan and elevation for
column unit 2 (top)

Drawn: JGM
Date: 9/3/00
Revisions: F Scales 1:20
Drawing No.
P18
Notes:
Construction tolerances
0 to +3mm
All dimensions mm
f_y = 300MPa
f_t = 420MPa
f_p = 30MPa
Min cover = 35mm

H026, R2, R2, R2

PLAN

75mm DUCT

South Face

Number req'd = 2 off

FACTORATION DRAWINGS
Plan and Elevation of column unit 2 (bottom)

SE and SW columns

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Christchurch, New Zealand

Date: 9/3/00
Revisions. F Scales: k20

Drawing No.

P19

C-21
North Central Column

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Christchurch, New Zealand

FABRICATION DRAWINGS
Plan and Elevation for
column unit 3 (top).

Notes:
Construction tolerances
0 to -3mm
All dimensions mm
fyh=300MPa
fty=630MPa
f'cu=30MPa
Min cover= 35mm

South Face

East Face

Drawing No. P20

Drawn: JGM
Date: 5/3/00
Revisions: F Scales: 1:20
Notes:
- Construction tolerances
  - 0 to -3mm
  - All dimensions mm
- $f_{y,h}=300\text{MPa}$
- $f_{y}=430\text{MPa}$
- $f_{c}=30\text{MPa}$
- Min cover 35mm

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**FABRICATION DRAWINGS**
Plan and Elevation for column unit 4 (bottom)

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*Drawn: JCM*  
*Date: 9/3/00*  
*Revisions F Scales b20*
Notes:
Construction tolerances 0 to +3mm
All dimensions mm unless stated otherwise
fy=300MPa unless stated otherwise
All welds are grade SP
No. required = 2
All bolts are M30 High strength

Fabrication Drawings
Secondary Frame Connection
Top detail

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Christchurch, New Zealand

Drawn: JDM
Date: 16/3/00
Revision: F Scale: 1:20

Drawing No. C1
Notes:
Construction tolerances 0 to -3mm
All dimensions mm unless stated otherwise
Fy=300MPa unless stated otherwise
All welds are grade 5P
No. required = 2
All bolts are M30 High strength

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FABRICATION DRAWINGS
Secondary Frame Connection
Central detail

Drawn: JGM
Date: 3/4/00
Revisions F Scale: 1:20

Drawing No.
C2
Notes:
Construction tolerances 0 to -3mm
All dimensions mm unless stated otherwise
fy=300MPa unless stated otherwise
All welds are grade SP
No. required = 2
All bolts are M30

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FABRICATION DRAWINGS
Secondary Frame Connection
Top corner detail

Drawn: JGM
Date: 3/4/00
Revisions: E Scales: 1:20

Drawing No. C3
Notes:
Construction tolerances ±0.3 mm
All dimensions mm unless stated otherwise
Fy = 300 MPa unless stated otherwise
All welds are grade SP
No. required = 4
Angle of connection = 30 deg

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FABRICATION DRAWINGS
Top loadarm
connection detail

Drawn: JGM
Date: 20/6/00
Revision: F, Scale: 1:5

Drawing No. C4
Notes:
- Construction tolerances 0 to -3mm
- All dimensions mm unless stated otherwise
- fy=300MPa unless stated otherwise
- All welds are grade SP
- No. required = 2
- Angle of connection=30deg
Notes:
Construction tolerances 0 to -3mm
All dimensions mm unless stated otherwise
All welds are grade SP
No. required = 2
Angle of connection = 30 deg
Notes:
Construction tolerances 0 to ±3mm
All dimensions mm unless stated otherwise
ty=300MPa unless stated otherwise
All welds are grade SP
No. required = 2
Angle of connection=30 deg
Notes:
Construction tolerances 0 to -3mm
All dimensions mm unless stated otherwise
fy=300MPa unless stated otherwise
All welds are grade 5P
Drawing No. L3

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Fabrication Drawings
Left component for long load arm

Drawn: JGM
Date: 7/11/99
Revision: D Scales: A20

Notes:
Construction tolerances 0 to -3mm
All dimensions mm unless stated otherwise
Fy=300MPa unless stated otherwise
All welds are grade 5F
No. req'd=2
Component diagram

Component diagram

Notes:
Construction tolerances 0 to -3mm
All dimensions mm unless stated otherwise
fy=300MPa unless stated otherwise
All welds are grade 5F
No. rec'd=2

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FABRICATION DRAWINGS
Right component for load arm

Drawn: JGM
Date: 7/11/99
Revision: D Scale: k20

Drawing No. L4
Notes:
Construction tolerances 0 to -3mm
All dimensions mm unless stated otherwise
fy=300MPa unless stated otherwise
All welds are grade SP
No. req'd=4

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FABRICATION DRAWINGS
Section A details
Secondary frame

Drawn: JGM
Date: 16/3/00
Revisions: 0
Scale: 1:25
Drawing No. SI
Notes:
Construction tolerances 0 to -3mm
All dimensions mm unless stated otherwise
fy=300MPa unless stated otherwise
All welds are grade 3P
No. req'd=4
Notes:
Construction tolerances 0 to -3mm
All dimensions mm unless stated otherwise
Fy=300MPa unless stated otherwise
All welds are grade 5P
No. req'd=4

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FABRICATION DRAWINGS
Section C details
Secondary frame

Drawn: JCH
Date: 16/3/00
Revisions: E
Scale: 1:25

Drawing No. S3
Notes:
Construction tolerances 0 to -3mm
All dimensions mm unless stated otherwise
fy=300MPa unless stated otherwise
All welds are grade SP
No. rev'd=4

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FABRICATION DRAWINGS
Section D details
Secondary frame

Drawn: JGM
Date: 12/5/00
Revisions: F
Scale: 1:25

Drawing No. S4
Notes:
Construction tolerances 0 to -3mm
All dimensions mm unless stated otherwise
Fy=300MPa unless stated otherwise
All welds are grade SP
No. req’d=2

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FABRICATION DRAWINGS
Section E details
Secondary frame

Drawn: JGM
Date: 13/3/00
Revisions: C Scale: h20
Drawing No. S5
Notes:
Construction tolerances 0 to -3mm
All dimensions mm unless stated otherwise
fσ = 300MPa unless stated otherwise
All welds are grade SP
No. req'd = 2
Also require 2-M80 locknuts

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FABRICATION DRAWINGS
Section F details
Secondary frame

Drawn: JGM
Date: 16/3/00
Revision: E Scale: 1:20
Drawing No. S6
Appendix D

Extended Experimental Results

D.1 General behaviour and observations during the test

Throughout this chapter reference will be made to particular hollow-core units (1st, 2nd, 3rd, 4th or 5th) within the super-assemblage or to the perimeter beams (North, South, East and West), refer to Figure D-1 for their location.

Figure D-1 Plan and elevation of the super-assemblage

The key indicator in determining the performance of the hollow-core unit connection detail is the relative rotation between the hollow-core unit and the supporting beam. Within this document this relative rotation has been defined as interstorey drift as the building investigated was considered to be a generic New Zealand moment resisting concrete frame building in which the interstorey drift closely relates to the relative rotation. In terms of predicting the amount of reinforcement slab activated as flange steel within a structure, the relative rotation between the hollow-core unit and the supporting beam should be used. As torsion of
the beams supporting the hollowcore units reduces this relative rotation, it is considered conservative to assume that the relative rotation and interstorey drift are one and the same. A designer is reminded that if a true assessment of risk or damage to the floor system is required then the designer should focus on the relative rotation between the hollow-core unit and the supporting beam as it is less conservative that using the interstorey drift as the indicator.

D.1.1 Phase I-Longitudinal loading

Initial elastic cycles of ±0.1% drift were used to ensure all the loading equipment and instrumentation was working correctly.

First cracks appeared in the super-assemblage at a drift of 0.25% drift. These cracks appeared simultaneously in several locations; the main frames plastic hinge zones, beam column joints and in the central column. Further cracks appeared during the −0.25% drift load cycle. The most significant crack was one that formed longitudinally along the soffit of the first hollow-core unit parallel to the direction of loading. The longitudinal crack that formed in the bottom of the hollow-core unit started at the stress concentration created by removing a section of the hollow-core unit around the central column to allow the unit to rest against the main perimeter beam. The crack propagated 850mm towards the east end and 600mm towards the west end and can be seen in Figure D-2. Figure D-3(a) shows the floor slab crack patterns at the end of the ±0.25% load cycle. The blue lines refer to cracks that formed during a positive inclination (the top of the column displaces in an Easterly direction while the base of the column displaces in a Westerly direction) cycle while the red cracks refer to a negative inclination (the top of the column displaces in an Westerly direction while the base of the column displaces in an Easterly direction) cycle.

D-2
Figure D-2. Initial hollow-core crack that formed in the soffit of the first unit.

The next load cycle was up to 0.5% drift, the approximate first yield as determined by initial calculations. At 0.32% drift the first crack in the end of the hollow-core unit was observed. This crack formed at the west end of the first hollow-core unit, starting at the stress concentration created by the removed section of hollow-core around the corner column and effects the hollow-core units seat connection. See Figure D-4 to see the location and direction of this hollow-core crack. This crack effects the hollow-core units seat connection. This was the onset of potential economic loss as this damage to the hollow-core unit is irreparable.

At 0.5%, drift several cracks formed within the floor diaphragm. A crack 4.5m long formed across the floor diaphragm close to the termination of the floor starter bars. This crack ran perpendicular to the direction of loading (Figure D-5). Several other cracks also formed within the floor slab. Most started at the central column and propagated out towards the centre of the floor slab. A longitudinal crack formed in the topping along the joint between the first and second hollow-core units. Torsion cracks started to form in the east beam (transverse to the loading direction) as it was being exposed to a negative moment (causing tension in the top fibres of the floor topping).
(a) Topping cracks after the ±0.25% drift cycle

(b) Topping cracks after the ±0.5% drift cycle

(c) Topping cracks after the ±1.0% drift cycle

(d) Topping cracks after the ±2.5% and ±2.0% drift cycles

Figure D-3 Mapping of the topping cracks during the first phase of loading. Blue cracks are due to a positive inclination while the red cracks are due to a negative inclination.
These cracks formed due to the restraint being provided by the starter bars in the central region of the beam while the inclined columns tried to rotate the beam ends. This torsion can be seen in Figure D-6.

Torsion cracks started to form in the west beam during the −0.5% drift cycle. The longitudinal crack in the soffit (shown in Figure D-2) had extended nearly the full length of the super-assemblage at the completion of this load cycle. At this drift level a discontinuity crack formed within the floor topping at the west beam. The discontinuity crack is a crack that forms in the topping at the end of the hollow-core unit and is created as the beam rotates relative to the floor slab. This crack only forms in zones of negative moment. This crack formation is depicted in Figure D-7. Figure D-3(b) shows the floor slab crack patterns at the end of the ±0.5% load cycle (the blue lines refer to cracks that formed during a positive inclination cycle while the red cracks refer to a negative).

The 1.0% drift load cycle saw the discontinuity crack form at the east end of the super-assemblage. The crack in the soffit of the first hollow-core unit had now
Figure D-5. The topping crack that formed perpendicular to the direction of loading (SE Corner)

Large rotation at the end of the beam

Small rotation at the centre of the beam

Figure D-6 Torsion generated within the transverse beam due to restraint to from starter bars within the diaphragm

Activated starter bar

Supporting beam

Crack at end of hollow-core unit (Continuity crack)

Figure D-7 Continuity crack that formed between the end of the hollow-core unit and the perimeter beam

D-6
propagated along the entire length of the hollow-core unit. Both, on the floor topping and on the bottom of the first hollow-core unit, shear lag cracks formed. Shear lag cracks form when the axial tension within a floor slab is transferred to the perimeter beam by means of a compression strut. The angle of the strut was approximately 45° and can be seen in Figure D-8.

![Figure D-8. Shear lag cracks that have formed on the floor diaphragm.](image)

The crack that formed in the bottom of the first hollow-core unit at the west end is now 3-4mm wide. This crack width agrees equates to the interstorey drift multiplied by the thickness of the hollow-core unit and the topping (refer to Figure D-9). Sections along all the hollow-core seats started to show signs of distress. The beam cover concrete spalled in regions where the hollow-core unit had appeared to try and slide off the support. A 6mm crack formed at the column face of the cast beam caused by the rotation between the beam and the column. A crack formed in the floor slab from the north edge of the specimen and extended across the floor towards the front perimeter beam. This crack coincided with the termination of the starter bars that were being loaded in tension.
Figure D-9 Determination of the crack width at the end of the hollow-core unit

Delamination was observed around both the central and southeast columns. The delamination map can be seen in Figure D-10(a). Delamination was determined by tapping the surface of the topping with the ball from a ball pein hammer and listening for a hollow sound. The zones of delamination made a distinctive hollow sound.

The longitudinal crack (Figure D-8) that formed between the first and second hollow-core unit extended along half the length of the super-assemblage and had an average crack width of 1mm.

The -1.0% drift cycle saw, what appeared to be, the first and second hollow-core unit being pulled off the east beam, thus resulting in the spalling of the cover concrete of the beam. In fact, the edge of the hollow-core unit had fractured, rather than sliding. Shear lag cracks formed in the opposite direction to the previous load cycle on both the top of the floor slab and the bottom of the first hollow-core unit around the central column. The plastic hinge zones of the beams became increasingly cracked with two or three major cracks appearing in each of the plastic hinge zones. The remainder of the plastic hinge zone cracks were hairline in appearance.

At this point it was observed that the inclination of the front three columns (in the plane of the frame) were not exactly the same. The difference between the columns was due to the slop in all the linkages connecting the secondary loading frame. It was decided that the column drift that was being reported would be that of the central column. The level by which the columns were out of parallel was not
Figure D-10. Mapping of the delamination of the topping from the hollow-core unit during the experimental programme

centering but did seem to vary linearly with increasing drift. This can be seen in Figure D-11. Since the front columns were not remaining exactly parallel it was decided that the back columns inclination should match the inclination of its partner on the front frame i.e. the inclination of the NE and NW columns match the SE and
SW columns respectively. This would ensure that no loading based torsion was induced into the transverse beams.

![Graph showing column inclination change with interstorey drift](image)

**Figure D-11** A plot showing how the columns inclination changed with interstorey drift

All the cracks that had formed in the floor slab were limited to the first hollow-core unit. The only exceptions to these were the cracks that formed at both ends of the units due to the rotation of the beam relative to the floor slab such as the discontinuity cracks. No new torsion cracked formed in the end beams, the existing cracks had continued to open. The 3-4mm continuity crack that had formed in the end of all the west hollow-core units during the previous load cycle did not fully close despite the reversal in inclination. This looked to be a visual sign of beam elongation.

More of the cover concrete of the east beams spalled due to the movement of the hollow-core unit relative to the supporting beam. The crack at the fractured end of the East hollow-core units was 2-3mm wide.

The residual drift at the completion of the load cycle was −0.41%. This recovery equates quite closely to the yield drift of the system (0.5% was the yield drift, refer to Appendix F for determination of the yield drift).

Upon the completion of the −1.0% cycle the delamination of the floor slab was checked and it was found that the zones of delamination were confined to regions
around all the columns (Figure D-10(b)). Figure D-3(c) shows the floor slab crack patterns at the end of the ±1.0% load cycle (the blue lines refer to cracks that formed during a positive inclination cycle while the red cracks refer to a negative). Figure D-12(a) shows the crack patterns on the underside of the floor slab at the end of the ±1.0% load cycle.

At zero drift the front frame was examined to determine whether all the cracks in the plastic hinge zones had closed. It was found that the two end plastic hinge zones cracks had not closed while the plastic hinges on each side of the central column had closed. This shows that the hollow-core unit passing the central column does provide some clamping force that is large enough to close the plastic hinge zone cracks at these relatively low levels of drift.

The next load cycle was to 2.5% drift. At 1.93% drift the reinforcing mesh within the topping started to fracture. The mesh fractured across the crack that had formed between the first and second hollow-core units (Figure D-13), the crack width measured 2mm. At 1.98% drift a continuous sound of fracturing mesh could be heard. Upon inspection it was found that a 9m long crack in the floor slab had opened and measured 3-4mm in width. It was assumed that most of the mesh had fractured over the entire length of the crack (due to the width of crack and the brittleness of the mesh). When the mesh fractured, a drop off in flexural strength of the longitudinal frame was observed. As loading was continued up to a drift of 2.5% more strands of the mesh could be heard to fracture.

At 2.5% drift, two additional strands of mesh fractured after the specimen had been held at that drift for approximately 1.5 hours. At 2.5% drift, the width of crack between the first and second hollow-core units had increased to 20mm in the centre and tapered to zero at the ends of the crack near the supporting beams. The vertical
(a) Damage to the hollow-core unit at the end of ±1.0%

(b) Damage to the hollow-core unit at the end of Phase I

(c) Hollow-core unit damage after the +2.0% and −2.5% displacement cycle during Phase II

(d) Finalized hollow-core damage at the completion of the test

Figure D-12 Damage to the underside of the hollow-core unit during the experimental programme.
Figure D-13. Longitudinal tear that formed within the floor diaphragm

offset between the two units across the diaphragm tear varied between 6-10mm. Upon inspection of the central columns rotary pots it was found that the column had displaced 25mm in the transverse direction (this movement is orthogonal to the direction of loading and away from the specimen). The drift of the central column in a transverse direction was at 0.5% (with the top of the column tilting in). As the tear in the floor slab was 20mm wide it was no longer possible to read the Demec points using the Demec gauge, a vernier was used in these regions.

The discontinuity crack at the east end now measured 12mm. Minor spalling of the beam cover concrete was observed on both sides of the central column. The regions of delamination on the floor slab had increased. The entire seating on the west beam had been completely lost. The seating had been lost in one of two ways: (a) the edge of the hollow-core unit had been broken off or (b) the cover concrete of the beam had spalled.

At 2.5% drift, the bottom of the hollow-core units were tapped to determine zones of web splitting. Zones where the webs were split made a distinctive hollow sound when tapped with a hammer. It was found that there was web splitting in the first hollow-core unit at the west end and it stretched approximately 3m into the floor slab. There was also a small region around the central column where the webs had split. Portions of the hollow-core unit had now dropped by at least 5mm.

Due to the extensive damage to the hollow-core units and the failure mechanisms observed to be occurring it was decided that additional catch frames
should be placed under the floor for safety concerns as the frames and angles presently being used may not be sufficient to hold the floor if it were to fail.

Upon unloading the super-assemblage had a residual drift of 1.8% at zero load. Several of the potentiometers used to measure the pull off of the hollow-core units from the supporting beam needed to be reattached as the cover concrete had spalled off the beam face affecting the potentiometers target. Figure D-14 shows some of the damage at the completion of this half load cycle.

Since the hollow-core units were performing in a manner different to that initially expected it was decided that some additional manual measurements should be taken. The vertical movement of the hollow-core units relative to their seat height was measured manually at various stages during the remainder of the experimental programme.

The next load cycle was to −2.0% drift. It was at this stage that it was clear that the seat on all the units at both ends of the hollow-core units were damaged. All the ends of the units had at least a 2-3mm wide residual crack when the sub-assemblage was at zero drift. This crack width increased as the inclination was increased. The first hollow-core unit had dropped by at least 5mm and all the others slightly less.

At −1.0% drift (on the way to −2.0%) the first signs of web splitting at the east end of the hollow-core units was noticed. This occurred in the first and second units and measured 500mm into the slab. At several intervals on the way up to −2.0% drift sections of the reinforcing mesh could be heard fracturing. The condition of the seating connections continued to degrade.
(a) Spalled beam cover on the hollow-core seat.

(b) The entire seat of the hollow-core unit has been lost.

(c) This photo clearly shows the first hollow-core unit dropping relative to the second unit.

Figure D-14. Photos at zero drift after the completion of the 2.5% drift half cycle
Due to the 25mm outward movement of the central column and the softening of the beam slab assembly the central column inclination in the transverse direction had increased to 1.3%.

Figure D-15 shows the diaphragm damage at the end of phase I. At the conclusion of the phase I loading, the first unit had dropped by some 10-12mm. All the others units had dropped by approximately 5mm. The damage seen in the hollow-core units seating detail is irreparable and had been for sometime. Figure D-10(c) shows the extent of delamination at the end of phase I. Figure D-3(d) shows the floor slab crack patterns at the end of the +2.5% and -2.0% load cycle (the blue lines refer to cracks that formed during a positive inclination cycle while the red cracks refer to a negative). Figure D-12(b) shows the crack patterns on the underside of the floor slab at the end of the phase I.

D.1.2 Phase II-transverse loading

Before the transverse loading could begin the loading and secondary frames needed to be connected to the side frames. All the columns were then straightened to ensure that all the columns started vertical.

During the elastic cycles not many new cracks formed, the torsion cracks from the previous load direction opened slightly.

The 0.5% drift cycle started to see a few small cracks forming in the first hollow-core unit around the central column. In both the southeast and southwest corners the topping started to lift. During this load cycle the specimen performed in a manner different to initially predicted. This was due to the influence that the phase I loading cycles had on the system. When a positive inclination (the top of the column displaces in the northern direction while the base of the column displaces in the southern direction, a negative inclination is the opposite) is applied to the super-
(a) Close up showing the width of tear within the floor diaphragm

(b) Extent of cracking within the floor

(c) The offset between the two hollow-core units

Figure D-15. Photos of the diaphragm at the end of Phase I
assemblage a crack was expected to open between the perimeter beam and the bottom of the first hollow-core unit. This was not the case since the soffit in the first hollow-core unit had been extensively damaged and had been cracked along its entire length creating a zone of weakness. This is where all the rotation occurred around. Figure D-16 shows the difference between the predicted and actual rotation.

![Tensile Yielding](image)

(a) Expected negative moment behaviour (by design)

(b) Observed behaviour

![Positive Moment Behaviour](image)

(c) Expected positive moment behaviour (by design)

(d) Observed behaviour

Figure D-16. Expected and actual rotation of the first unit and perimeter during the transverse loading.

The next load cycle was to -0.5% drift. Again the damage from the previous loading direction played a major role in the performance of this load cycle. Initially it was assumed that a discontinuity crack would form at the interface between the first hollow-core unit and the perimeter beam. This did not occur as the longitudinal crack that formed between the first and second unit caused the entire first hollow-core unit to lift. Figure D-16 shows the difference between the predicted and observed results.
The 1.0% drift load cycle saw small pieces of concrete falling out of the crack in the first hollow-core unit soffit. The topping around the central column started to lift. Until this point, all the delamination that had occurred was not visible but now a vertical lifting of the topping can be seen around the central column and the southwest corner. The northwest end of the fifth hollow-core unit is now starting to drop. It had dropped approximately 2-3mm.

The east beams seat became even more extensively damaged during the −1.0% drift cycle. The cracks within the two transverse beams are now forming as flexure cracks rather than being dominated by the torsion cracks that formed during the initial longitudinal loading. More reinforcing mesh started to fracture as the first hollow-core unit was lifted, leading to the extension of the crack between the first and second hollow-core units. At this stage it was clear that the west end of the first hollow-core unit is being held up (and had been for sometime) by a relatively small piece of concrete that has not failed, seen in Figure D-17. This portion of concrete could not be relied upon to hold up the unit, as it is not guaranteed to form every time. The unit has dropped by an additional 8-10mm at 2.0% drift. Just before the target drift of 2.0% was reached a portion of the soffit of the first hollow-core unit fell out. This enabled a camera to be placed in the core to verify the web splitting. The exact length of web splitting was difficult to measure but was at least 4m from the west beam. The width of web crack varied between 15-25mm. This explained why the front edge of the first unit had dropped by that amount relative to the section still attached to the beam. Figure D-18 shows the internal hollow-core damage looking in both the east and west directions.
During the $-2.5\%$ loading cycle the first hollow-core unit is clearly seen to be lifted. This lead to more reinforcing mesh fracturing and at $-2.5\%$ drift the difference in height between the two units varies between 27-33mm along the length of the unit (Figure D-19). At the west end of the floor slab the crack between the first and second units changed direction and started to propagate into the second unit, whereas at the east end the crack started to propagate into the first hollow-core. This is shown in Figure D-20.

Throughout the transverse loading the regions of delamination increased significantly. Most of the delamination was confined to within 500mm to 1000mm of the end of the perimeter beam. This coincides with the termination of the starter bars. shows the crack patterns on the underside of the floor slab at the end of the $+2.0$ and $-2.5\%$ load cycle.

After the completion of the $+2.0/-2.5\%$ load cycle the super-assemblage was bought back to zero drift so that a torsion test could be undertaken. The torsion test consisted of imposing a cycle of $+/0.5\%$ drift to the system with one frame being
(a) Section of the hollow-core unit that fell out allowing a camera inside

(b) Looking east. Large cracks can be seen in the web and soffit of the unit.

(c) Several cracks can be seen around the dam. Extensive damage to the seat connection as well as the unit

Figure D-18. Web splitting within the first hollow-core unit.
loaded in the positive direction while the other is loaded in the negative direction while the central column remained vertical. This loading cycle was undertaken so that the torsional stiffness of the frame could be assessed. A reading reflecting the state of an undamaged structure could not be obtained as the pre-existing damage in the super assembly greatly affected the buildings strength and performance. Figure D-12(c) shows the crack patterns on the underside of the floor slab at the end of 2.0% and -2.5% cycle.
Figure D-20. Changes in the crack direction at both ends of the first hollow-core unit

Just prior to the start of the torsion test the diaphragm delamination was mapped (Figure D-10(c)). The figure clearly shows that most of the starter bars around the perimeter of the floor had caused the topping to delaminate. One region near the south west corner did not delaminate, this is due to the affect the tear in the floor slab had on the system.

As the drift was increased up to 3.5% drift various noises were heard as the structure slowly deteriorated. These noises varied from the sound of small pieces of
hollow-core units falling out to more reinforcing mesh fracturing. All the hollow-core units were now slowly dropping down the beam face. At 3.0% drift a large section of the first hollow-core unit fell out (Figure D-21). This exposed some of the prestressing strands at the west end. It could be seen that only the one strand was holding the bottom half of the first hollow-core unit from collapsing (as seen in Figure D-17). Once this piece of concrete had fallen it was possible to look up all the cores of the first unit. Shining a torch down the core revealed that all the webs were split to at least the central column (6m from the end). The crack width was at least 25mm.

(a) West end damage

(b) Close up after a large section had fallen out. Note the curved strands no longer supporting the hollow-core unit.

Figure D-21. Damaged section of the first hollow-core unit at the west end.
On the load reversal this hollow-core unit started to drop significantly. At zero drift it had lowered by some 22-26mm, at −1.0% it was down 25-28mm. More mesh across the interface of the first and second hollow-core units fractured at −1.77% drift. The second and third hollow-core units were starting to suffer more damage. At a drift of −2.45% there was a loud thud. Upon investigation it was found that the webs in the northern most hollow-core had split (Figure D-22). The split was over two thirds of the length of the hollow-core unit and occurred around the central column. The webs split due to the horizontal shearing force applied to the hollow-core unit as it bore against the northern central column. The lifting of the first hollow-core unit had lead to the diaphragm crack propagating to the end of the sub-assemblage. Once at the end of the frame the crack then caused the beam cover concrete to be lifted. This is shown in Figure D-23. At −3.5% drift the centre of the first hollow-core unit had lifted 55-60mm relative to the second unit and the width of crack was 15-20mm.

The damage to the west end of the first hollow-core at the end of phase II is shown in Figure D-24.

**D.1.3 Phase III- Longitudinal loading**

The intention for this final loading direction was to complete the following displacement cycles of ±0.5%, ±2.5% and ±3.5% drift. The initial “yield” cycle was undertaken so that a comparison between the initial phase I longitudinal elastic stiffness and the phase III longitudinal stiffness could be made after the transverse loading has been completed. The ±2.5% cycle was carried out to compare the loss of strength the transverse loading had on the longitudinal strength. Refer to Chapter 5 for details on the strength comparison and initial stiffness for the various displacement phases.
(a) The east side of the back central column

(b) West side of the back central column

Figure D-22. The northern hollow-core units split webs.

Figure D-23. Lifting of the beam cover concrete due to the imposed displacement to the first hollow-core unit.
(a) Damage to the first hollow-core unit at the west end. Note the kinked strand holding the unit up.

(b) Photo showing the width of web crack in the hollow-core unit

Figure D-24. Damage at the completion of Phase II.

During the first half of the 0.5% cycle there was a 9mm difference in height between the levels of the first and second hollow-core unit at the east end. All the units had dropped further during this load cycle.
At -0.48% drift another piece of hollow-core fell out at the west end. This meant that it was possible to look up all the cores of the first hollow-core unit. At this end of the hollow-core unit it can be seen that all the strands in the first unit except one had been pulled out by 20mm. The concrete around the one strand that is holding this unit is starting to crack. This is shown in Figure D-25.

(a) Looking up at the west end of the first hollow-core unit

(b) Internal damage to the hollow-core unit

Figure D-25. Damage at -0.5% drift

During the next load cycle there was constantly small pieces of concrete falling out from the underside of the first hollow-core unit. A new longitudinal crack 500mm long formed in the bottom of the third hollow-core unit at the east end. It
looks to have formed due to the hollow-core unit bearing against the beam face since it has dropped relative to its initial seat. This bearing imposes a longitudinal compression strain into the unit. At 1.59% drift more beam cover concrete spalled on the west beam. The southwest corners hollow-core unit continues to break up and drop lower. There is evidence of web splitting at the east end for the first time; units 2 and 4 had split webs for the first 0.5-1m into the slab. The length of split webs in the hollow-core unit is now three quarters of the length of the super-assemblage. At 1.88% drift a region of the topping around the central column has lifted, due to the beam and the hollow-core units having different displacement patterns (Figure D-26). At 2.0% drift the small piece of concrete holding the first hollow-core unit (at the west end) started to be pulled off the beam. There are still continually pieces of concrete falling out of the hollow-core units.

At 2.5% drift the piece of concrete holding the first hollow-core unit failed. The web cracked allowing the bottom of the unit to fall onto the catch frame. The section of floor was then propped so that the catch frames could be removed. The prop was then removed to see whether the remaining uncracked webs had enough strength to hold the bottom of the hollow-core unit up. When the prop was removed the entire bottom of the first unit peeled away. Figure D-27 shows the set up when the unit is propped and then after the prop was removed. Note the similarity with the photos taken following the Northridge earthquake (Chapter 1).

As the remainder of the floor was still present is was decided to complete the load cycle to -2.5% drift. Unloading the specimen from 2.5% down to 0% drift went without incident. The hollow-core units closest to the south beam dropped significantly more than the hollow-core units closest to the north beam. At was at this stage that it was realised that the starter bars used in the tie beam were actually
holding the remainder of the floor up (in a real structure these starter bars would not be present). At \(-1.3\%\) drift it could be seen that the reinforced section of the topping slab on the east beam is pushing the floor down. This is due to the rotation being imposed by the edge beam (seen in Figure D-28(a)). Some of the cracks within the beam plastic hinge zones were now 5mm wide. At \(-1.5\%\) drift the hollow-core units supported by the east beam were checked for split webs. It was found that all the five units now had split webs for at least their first 500mm into the floor slab. The east beams hollow-core units had dropped an additional 15mm. Since
(a) The failed unit resting on the catch frame. Highlighted is the piece of concrete that was holding up the unit.

(b) The first hollow-core unit just before the prop (far end) was removed.

(c) After the prop was removed. The top of the unit remains in place, the soffit of the unit has fallen away.

(d) Looking East. Note the end of the hollow-core units are still attached to the beam.

(e) Looking West

Figure D-27. Failure of the first hollow-core unit.
(a) Note the floor has been pushed down by the combination of the end beam rotating and the reinforced topping.

(b) Close up of the second hollow-core unit at the East end (this unit has dropped from 90mm)

(c) The floor prior to the load test

Figure D-28 Photographs of the remaining floor prior to the load test
Figure D-29 The concrete mass being used to simulate the live load

The first hollow-core unit no longer had any reinforcement, since the bottom failed, a crack formed across its entire width near the central column. An additional crack formed in the topping at the termination of the starter bars at the west end. The west end hollow-core units did not drop any further during this load cycle. Once the target drift of –2.5% drift was reached if was found that at the east beam the hollow-core units had dropped by the following: 90mm at the unit 2 to 30mm at unit 5 (northern unit i.e. unit 5). This difference in the amount dropped clearly shows the influence the tie beam starters had on the system. As previously mentioned, in a real building these starter bars would not be present as there are usually no intermediate beams spanning in a direction parallel with that of the hollow-core units.

The floor was then load tested to see whether the floor could carry its design load. The key issue with the load test was to ensure the correct shear force was applied to the floor/beam connection. A concrete mass was used to load the floor (Figure D-29) and apply a load equivalent to a load of 1.75kPa (1.0kPa Reduced live load and 0.75kPa Superimposed dead load). When the load was applied, the second, third and fourth hollow-core units failed in one complete unit. This type of failure was
very similar to the failures observed in Northridge (Chapter 1). The photos, before and after load test, are shown in Figure D-28 and Figure D-30, respectively.

Upon inspection of the failed units it was found that what was thought to be split webs in the hollow-core unit near its seat was in fact the fractured end of the hollow-core unit. Refer to Figure D-12(d) for a summary of the damage to the hollow-core units at the completion of the test.

This signified the end of the testing programme.
(a) Failure of the floor after the load test. Units 2, 3 and 4 failed as one.

(b) Looking up at the floor

(c) The starters and the end of the hollow-core units remain attached to the East beam.

Figure D-30. Photos of the floor slab following the load test.
Appendix E
Experimental Results 1: Photo Log

E.1 Phase I

E.1.1 ±0.25 Drift photos

Central Column
- First sign of shear lag cracks

SE column
- Diaphragm crack at the end of the starter bars formed

SW column
- Plastic hinge cracks at the completion of the ±0.25% cycle.
Central Column
- Plastic hinge cracks at the completion of the ±0.25% cycle.

SE Column
- Plastic hinge cracks at the completion of the ±0.25% cycle.

SW Column
- Diaphragm cracks at the end of the cycle

E.1.2 ±0.5% Drift Photos

SW Column
- Damage to the hollowcore units connection
Central Column
- More shear lag cracks formed
- Longitudinal crack between the first and second hollowcore unit formed

NE Column
- Torsion cracks forming in the transverse beam

Central Column
- Longitudinal cracks formed in the soffit of the first hollowcore unit
SW column
- Plastic hinge cracks at the completion of the ±0.5% cycle.

Central Column
- Plastic hinge cracks at the completion of the ±0.5% cycle.

SE column
- Plastic hinge cracks at the completion of the ±0.5% cycle.

West Beam
- Formation of the continuity crack between the end of the hollowcore unit and the beam
E.1.3 ±1.0% Drift Photos

Central Column
- Lots of shear lag cracks present.
- A few small cracks within the column can be seen.

East Beam
- Spalling of the transverse beams cover concrete due to the relative movement between the hollowcore and the beam.

SE column
- The inclination of the column can be seen when compared to the white board.
Central Column
- A lot more cracks formed within the topping around the central column
- The crack between the first and second hollowcore unit increases

SW Column
- The torsion cracks within the transverse beams met with the continuity cracks on top of the beam

Central Column
- Beam column joint cracks at the end of ±1.0% cycle
E.1.4 +2.5%, -2.0% Drift Photos

**SW Column**
- Major damage to the hollowcores seat.
- Crack width is approximately 12mm

**Central Column**
- First beam spalling seen
- Longitudinal crack in soffit of hollowcore unit shows a change in height on each side of the crack

**Bottom of first hollowcore unit**
- Difference in height each side of the soffit crack
Central Column
- Shear lag cracks on the bottom of the first hollowcore unit

SE Column
- Torsion cracks at the end of Phase I loading

Central Column
- Lifting and deterioration of the topping concrete around the central column
- Full delamination of the topping in this region
Floor slab
- Diaphragm tear that formed at 1.93% drift

Floor slab
- Difference in height on either side of the diaphragm tear

East Beam
- Hollowcore unit seat damage
East Beam
- Entire seat lost at the end of Phase 1

SW Column
- Plastic hinge cracks at the completion of the +2.5% and -2.0% cycle.

Central Column
- Plastic hinge cracks at the completion of the +2.5% and -2.0% cycle.
SE Column
- Plastic hinge cracks at the completion of the +2.5% and -2.0% cycle.

East Beam
- Diaphragm cracks at the end of phase I

West Beam
- Diaphragm cracks at the end of phase I
Floor Slab
- The diaphragm tear at the completion of the +2.5% and -2.0% cycle.

Central Column
- Plan showing the extent of diaphragm cracking at the end of Phase I

Central Column
- Damage to the underside of the first hollowcore unit at the completion of Phase I
E.2 Phase II

E.2.1 ±0.5% Drift Photos

NW Column
- Plastic hinge zone cracks at the completion ±0.5%

SW Column
- Plastic hinge zone cracks at the completion ±0.5%

SE Column
- Plastic hinge zone cracks at the completion ±0.5%
NE Column
- Plastic hinge zone cracks at the completion ±0.5%

Bottom of first hollowcore unit
- Soffit crack worsening

East Beam
- General set up of the loading frames
E.2.2 ±1.0% Drift Photos

SE Column
- Plastic hinge zone cracks at the completion ±1.0%

NE Column
- End of the Hollowcore unit has fractured

NW Column
- Plastic hinge zone cracks at the completion ±1.0%
NE Column
- Further cracks form within the floor slab around the corner column
- Cracks form at the termination of the starter bars

E.2.3 +2.0%, -2.5% Drift Photos

NE Column
- Plastic hinge zone cracks at the completion ±1.0%

Floor Slab
- Close up showing the difference in height between the first hollowcore slab and the remainder of the floor
West Beam
- The first section of hollowcore unit falls out of the first unit

Hollowcore Unit Damage
- Looking at the west beam support
- Numerous cracks within the unit are seen

Hollowcore unit damage
- Soffit cracks in the first hollowcore unit
- Spit webs are also be seen
First Hollowcore Unit
- The diaphragm tear shifted from between the first and second hollowcore unit into the first unit near the east beam.

SW Column
- First hollowcore unit has dropped 20mm

NW Column
- Plastic hinge cracks are now a mixture of torsion cracks from Phase I and new flexure cracks from Phase II.
NE Column
- Plastic hinge cracks are now a mixture of torsion cracks from Phase I and new flexure cracks from Phase II.

NE Column
- The NE column at 2.5% drift

SW Column
- The diaphragm tear propagated into the second hollowcore unit near the west beam.
- Note: Lack of cracks in the second unit, most of the damage is localised to the first hollowcore unit.
SE Column
- The diaphragm tear propagated into the first hollowcore unit near the east beam.
- Note: Lack of cracks in the second unit, most of the damage is localised to the first hollowcore unit

E.2.4 ±3.5% Drift Photos

NW Column
- Large flexure cracks form near column

NE Column
- Large flexure cracks form near column
Hollowcore Unit Damage
- The 25mm web cracks relates to the 25mm drop of the unit

SE Column
- Deterioration of the East end of the hollowcore unit

Hollowcore Unit Damage
- Dropping of the first unit
SE Column
- The diaphragm tear propagated to the end of the super assembly and started to peel off the beam cover concrete

NW Column
- Plastic hinge zone damage at the end of Phase II

Hollowcore Unit Damage
- West end of the first unit
NE Column
- Plastic hinge zone damage at the end of Phase II

North Beam
- The fifth hollowcore units web split around the central column
- Crack propagated for approximately 4m

SE Column
- Lifting of the first unit relative to the floor slab
Hollowcore Unit Damage
- A section of the west end of
  the first hollowcore unit fell
  out.
- The sagging prestressing
  strands are not anchored into
  the beam

Hollowcore Unit Damage
- Large section of hollowcore
  unit has failed

Floor Slab
- Lifting of the first
  hollowcore unit relative to
  the floor slab
E.2.5 End of phase II

Hollowcore Unit Damage
- West end of hollowcore unit is held up by kinked strand at top of the picture

Floor Slab
- Final difference in height between the first unit and the floor slab at the end of Phase II

E.3 Phase III

E.3.1 ±0.5% Drift Photos

Hollowcore Unit damage
- Strand pull out at the west end of the unit
E.3.2 +2.5% Drift Photos

**Hollowcore Unit Damage**
- West end after failure
- Hollowcore unit is resting on catch frame
- Section at the top of the photo that was holding up the hollowcore unit has now failed

**Hollowcore Unit Damage**
- Prestressing strands are sliding down beam face
- Not supporting the unit due to no anchorage

**Hollowcore Unit Damage**
- Fractured section of hollowcore unit that had been holding up the west end
Hollowcore Unit Damage
- East beam damage
- End of prestressing strands can be seen

Central Column
- Lifting of the topping around the central column

Hollowcore Unit Damage
- Damage to the end of the unit at the NW column
Hollowcore Unit Damage
- Propped hollowcore unit after the west end failed
- Extensive damage at east end as well

Hollowcore Unit Damage
- Failed first unit when the prop supporting the unit was removed

Hollowcore Unit Damage
- The ends of the unit is still attached to the beam
- The end of the unit fractured rather than sliding (design assumption)
Hollowcore Unit Damage
- Failed bottom section of hollowcore

E.3.3 -2.5% Drift Photos

Floor slab
- As the east beam rotates the reinforced topping slab push the floor slab down

Hollowcore Unit Damage-East beam
- The second unit had dropped by 90mm prior to the load test been carried out
Seat Connection
- Completed damaged floor slab connection just prior to the load test

Hollowcore Unit Damage
- Once the bottom of the unit failed a large crack opened near the central column as the floor slab did not restrain any beam elongation

Floor Slab
- The floor slab just prior to the load test being undertaken
**Load Test**
- Applying a load to represent the design live load on the floor

**Load Test**
- Failure of the floor during the load test

**Floor slab failure**
- Complete floor slab failure when the floor was loaded tested
East Beam
- Failed floor when the starters and topping still attached to the beam

Floor Failure
- Failed floor with the frame still standing

Hollowcore Unit Damage
- End of the hollowcore unit still attached to the east beam
Hollowcore Unit Damage
- Hollowcore ends and topping still attached to the east beam

West Beam
- Large crack forming at the termination of the starter bars
West Beam
- Failed floor with the starter bars and topping still attached to the beam

E.4 End of test

End of Test
- Super assembly stripped down to be a basic frame

East Beam
- End of the hollowcore unit still attached to the beam
Failed Floor
- Plan of the floor failure

West Beam
- Distorted beam due to the torsion induced by the floor slab and the inclination of the columns

West Beam
- By using a straight edge it is possible to see the final distorted shape of the beam.
Hollowcore Unit Damage
- Failed unit sitting on the laboratory floor

Hollowcore Unit Damage
- Kinked prestressing strands from when they were supporting the unit once its seat was lost

E.4.1 Plastic hinges

NW Column
- Final plastic hinge zone damage
SW Column
- Final plastic hinge zone damage

Central Column
- Final plastic hinge zone damage
SE Column
- Final plastic hinge zone damage

SE Column
- Final plastic hinge zone damage

NE Column
- Final plastic hinge zone damage
**SW Column**
Final plastic hinge zone damage

**SW Column**
- Final plastic hinge zone damage

**Central Column**
- Final plastic hinge zone damage
Central Column
  - Final plastic hinge zone damage

SE Column
  - Final plastic hinge zone damage

SE Column
  - Final plastic hinge zone damage
NE Column
  - Final plastic hinge zone damage
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Appendix F

Hysteretic response

F.1 Determination of the yield displacement and effective stiffness

To determine the yield displacement of the super-assemblage the system was broken up into its individual components.

**Beam Contribution**

Using a moment-curvature programme the beams yield moment \( M_{yb} = 522 \text{ kNm} \) and yield curvature \( \phi_{yb} = 4.6 \times 10^{-6}/\text{mm} \) were determined. Knowing the yield moment and curvature it was possible to determine the effective stiffness \( (EI_{eff}) \) for the beams.

\[
\phi_{yb} = \frac{M_{yb}}{EI_{eff}} \Rightarrow EI_{eff} = \frac{M_{yb}}{\phi_{yb}} \quad (F-1)
\]

or in terms of the effective stiffness over the gross stiffness \( (EI_g) \)

\[
\frac{EI_{eff}}{EI_g} = \frac{M_{yb}}{\phi_y EI_g} = \frac{522 \times 10^6}{4.6 \times 10^{-6} \times 30000 \times 400 \times \frac{750^3}{12}} = 0.27 \quad (F-2)
\]

in which \( E \) = Modulus of elasticity of the concrete (At the time of the test, the compressive strengths \( f'_c \) of the topping slab and beam concrete was 45 MPa whereas the actual hollow-core units concrete compressive strength was unknown.

The hollow-core manufacturers brochure stated that the minimum compressive strength was 42 MPa. Therefore, for the purposes of the calculations the hollow-core units concrete compressive strength was assumed to be 45 MPa); and \( I_g \) = gross beam moment of inertia (based on the beams rectangular dimensions).
Now determine the beam deflection in terms of a column deflection using moment area.

\[
\delta_b = \frac{M_y L_b^2}{12EI_{\text{eff}}} = \frac{522 \times 10^6 \times 5350^2}{12 \times 30000 \times 0.26 \times 400 \times 750^3/12} = 10.9\text{mm} \tag{F-3}
\]

in which \(\delta_b\) = displacement of the beam at its point of inflection; \(L_b\) = the distance between the beam plastic hinges (5350mm).

To convert a beam displacement to a column displacement the following is used

\[
\delta_c^b = \delta_b \frac{L_c}{L_b/2} = 10.9 \times \frac{3500}{6100/2} = 12.5\text{mm} \tag{F-4}
\]

in which \(\delta_c^b\) = the column displacement due to the beam deflecting; and \(L_c\) = The height of the column between inflection points.

**Beam Column joint contribution**

To determine the contribution that the beam column joint makes to the lateral displacement of the super-assemble an expression must be written for the horizontal beam column joint shear \((V_{jh})\).

Using Figure F-1 and solving for equilibrium

\[
2R \frac{L_b}{2} = H_{col}H \Rightarrow H_{col} = \frac{R L}{H} = \frac{2M_b}{H} \frac{L}{L_b} \tag{F-5}
\]

in which \(R\) = shear force within the beam; \(H_{col}\) = column shear force; \(M_b\) = beam moment at the column face; \(L_b\) = distance between plastic hinges; \(L\) = the centreline distance between beam inflection points; and \(H\) = column height.

Therefore, \(V_{jh}\) (Figure F-1) can be written as
\[ V_{jh} = T + C - H_{col} = 2 \left( \frac{M_b}{d - d'} \right) - 2 \frac{M_b}{H} \frac{L}{L_b} \]  
(F-6)

\[ \text{Applied loading to a cruciform section} \]

\[ \text{Applied loads to the column} \]

\[ \text{Column SFD} \]

(a) A diagram showing the determination of \( V_{jh} \)

(b) The relationship between the horizontal beam-column joint shear (\( V_{jh} \)), joint stiffness (\( K_j \)) and joint distortion (\( \gamma_j \))

Figure F-1 Determination of the beam column joint (\( \gamma_j \)) distortion

in which \( (d-d') = \) beams internal leverarm. Equation (F-6) can be simplified to

\[ V_{jh} = \frac{2M_b}{(d - d')} \left[ 1 - \frac{d - d'}{H} \frac{L}{L_b} \right] \]  
(F-7)

Also an expression for cracked stiffness (\( K_{scr} \)) of the joint is required (Kim and Mander, 1999) can be written as a ratio of the uncracked stiffness (\( K_{sun} \)).

\[ \frac{K_{scr}}{K_{sun}} = \frac{\rho_v n E_c A_j}{(1 + 7.33 \rho_v n)0.4 E_c A_j} \]  
(F-8)

F-3
in which $\rho_v = \text{beam column joint steel ratio}; n = E_s/E_c; A_f = \text{cross sectional area of the joint}; E_s = \text{Modulus of elasticity of the reinforcement}; \text{and } E_c = \text{Modulus of elasticity of the concrete.}

Equation (F-8)) was simplified by Kim and Mander (1999) to

$$K_{scr} = \frac{0.129E_cA_f}{1 + 0.02/\rho_v} \quad (F-9)$$

Knowing both $V_{jh} = \text{the horizontal joint shear force and } K_j (= K_{scr})$ it is now possible to determine the joint distortion ($\gamma_j$) (Figure F-1).

$$\gamma_j = \frac{V_{jh}}{K_j} = \frac{2M_b}{(d-d')\left(1 - \frac{d-d'}{H\frac{L}{L_b}}\right)} \quad (F-10)$$

in which $B = \text{breadth of the column}; \text{and } H = \text{width of the column}$

$$\gamma_j = \frac{V_{jh}}{K_j} = \frac{2A_s f_y (d-d')\left(1 - \frac{d-d'}{H\frac{L}{L_b}}\right)}{0.052E_cBH} \quad (F-11)$$

in which $A_s = \text{area of column reinforcement}; \text{and } f_y = \text{yield stress of the column reinforcement.}$

By rearranging Equation (F-11) and substituting in actual values, $\gamma_j$ is determined.

$$\gamma_j = 38.5 \frac{A_s f_y}{BH E_c} \left(1 - \frac{d-d'}{H\frac{L}{L_b}}\right) \quad (F-12)$$

$$\gamma_j = 38.5 \frac{2714}{750^2} \frac{300}{31500} \left(1 - \frac{564}{3500} \frac{6100}{5350}\right) = 0.0014 = 0.14\% \quad (F-13)$$

When $\gamma_j$ is converted to an interstorey drift, the column displacement ($\delta_c'$) due to the beam column joint distortion was 4.9mm.
Column contribution

From statics and knowing the beam yield moment it is possible to determine the column moment. More important in determining the yield displacement is knowing the $EI_{eff}$ for the column. Priestley et al (1996) have shown that it is acceptable to use the yield secant stiffness based on the column yield moment rather than using the exact secant stiffness at the applied moment, as the variance is minimal. This can be seen in Figure F-2.

![Figure F-2 Comparison between the yield effective stiffness and the exact secant stiffness at a point less than yield (Priestley et al, 1996)](image)

To calculate the $EI_{eff}$ and hence the displacement, the columns yield moment ($M_{yc}=790$ kNm) and curvature ($\phi_{yc} = 4.44\times10^{-6}$/mm) were determined using moment-curvature.

To determine $EI_{eff}$ the following was used

$$\frac{EI_{eff}}{EI_g} = \frac{M_{yc}}{\phi_{yc}EI_g} = \frac{790\times10^6}{4.44\times10^{-6}\times30000\times750\times\frac{750^3}{12}} = 0.22$$  \hspace{1cm} (F-14)

To determine the deflection of the column due to the beam yielding the actual column moment was required. From equilibrium the column moment ($M_c$) was determined as 234 kNm.

F-5
Therefore the column deflection is

\[
\delta_c = \frac{2M_c h_c^2}{3EI_{cf}} = \frac{2 \times 234 \times 10^6 \times 3500^2}{3 \times 30000 \times \frac{750^3}{12} \times 750 \times 0.22} = 1.6\text{mm}
\]  \hspace{1cm} (F-15)

Therefore the total displacement due to the three components is summed to give the yield displacement (and hence yield drift).

\[
\delta_t = \delta_{cb} + \delta_{cb}' + \delta_c = 10.9 + 4.9 + 1.6 = 17.4\text{mm}
\]  \hspace{1cm} (F-16)

If this lateral displacement is converted into an interstorey drift the yield drift \((\theta_d)\) is 0.49%.

**F.2 Torsion test**

Part way during the second phase of loading a torsion test to \(\pm 0.5\%\) was undertaken to examine the torsional stiffness of the super-assemblage. The torsion test was undertaken between the \(-2.5\%\) cycle and the \(+3.5\%\) cycle as shown in Figure F-3(a). A torsion test is one in which the East and West frames are displaced in opposite directions (i.e. the East frame was displaced in a positive direction while the West frame displaced in a negative direction and vice-versa as shown in Figure F-3(b)).

Figure F-3(c) shows the hysteretic response for both the East and West bays. When comparing the stiffness of the two bays it is evident that the longitudinal tear that had formed within the floor slab during Phase I affected the torsional stiffness. The East bay had a stiffness of 10MN/m whereas the West bay had a stiffness of 6.2MN/m. This difference can be explained by examining the loading directions for each bay. When the East bay was displaced in a positive direction (the top of the column displaces in a northern direction while the bottom displaces in a southern direction) the starter bars in the northern beam were activated as additional tension.
(a) Phase II displacement history

(b) A schematic showing the torsion test

(c) Hysteresis loop for the torsion test

Figure F-3 Details of the torsion test
reinforcement. Whereas when the West bay was displaced by a negative direction (the
top of the column displaces in a southern direction while the bottom displaces in a
northern direction) that causes the first hollow-core unit to be lifted rather than
activating the starter bars in the southern beam. It was this lifting of the hollow-core
unit that caused the stiffness to be reduced for a negative inclination.

Due to the damage caused by the diaphragm tear very little information was
obtained from the torsion test.

F.3 General super-assemblage performance

Longitudinal bar slip

Phase I

During the experimental programme, the reinforcing bars passing through the central
beam column joint were monitored for longitudinal bar slip. As can be seen in Figure
F-4(a), minimal bar slip occurred. The top bar moved a maximum of 0.8mm while the
bottom bar moved a maximum of 0.1mm. The difference between the two bars is due
to the formation of air bubbles on the underside of the top reinforcement when the
beams were cast. These air bubbles reduce the bond of the top reinforcement to the
concrete making it more prone to sliding through the joint earlier than the bottom
reinforcement.

The amount of bond slip was low as the ratio of the beam depth to column
depth ($d_b/h_c$) is a lot less than the maximum allowable by New Zealand Concrete
through the beam column joint. This maximum bar size is limited by the bond
requirements of the bar. For a ductile frame Cl 7.5.2 of NZS3101:1995 governs.
Figure F-4 Bar slip of the longitudinal reinforcement through the central beam column joint

\[ \frac{d_b}{h_c} = 3.3\alpha f' \sqrt{\frac{f'c}{\alpha_0 f_y}} \]  \hspace{1cm} (F-17)

in which \( d_b \) = longitudinal bar diameter; \( h_c \) = the depth of column; \( \alpha_f = 1.0 \) for one way frames; \( f'c \) = the specified concrete strength (MPa); \( \alpha_0 = 1.25 \) when the plastic hinge forms on the column face; and \( f_y \) = the nominal yield strength of the longitudinal reinforcement.

From the Equation (F-17) the maximum allowable bar size to pass through the joint is

\[ d_b = 3.3 \times 1.0 \times \frac{\sqrt{45}}{1.25 \times 300} \times 750 = 44mm \] \hspace{1cm} (F-18)
The actual longitudinal bar size used was 24mm, 55% of the allowable maximum bar size. This difference between the maximum allowable bar to be used and the actual bar used explains why there was minimal bar slip as the stresses within the beam column joint would have been low.

Phase III

The degree of bar slip through the central beam column joint for Phase III was similar to that observed during Phase I. This can be seen in Figure F-4(b).

F.3.1 Beam Dilation

Dilation is the increase in the depth of the beam across a vertical section due to the beam forming a plastic hinge.

Phase I

All the plastic hinge with the South frame showed very little dilation except for the left hand side of the central column which grew by 21mm as shown in Figure F-5(a). This growth was large because the hollow-core unit spanned past the central column and restricted some of the beam elongation (Chapter 7). This restraint caused the beam to dilate more than the other plastic hinge zones where the beam elongation was not restrained. This plastic hinge was also the hinge that showed the greatest amount of damage during the first phase of loading.

Phase II

During Phase II very little beam dilation was seen. The West beam grew by approximately 4mm while the East beam grew by 2mm. This growth can be seen in Figure F-5(b).
(a) South beam dilation during Phase I

(b) Transverse beams dilation during Phase II

(c) South beam dilation during Phase III

Figure F-5 Perimeter frame beam dilation
Phase III

Figure F-5(c) shows no major dilation occurred. The plastic hinge that formed on the left hand side of the central column actually recovered some of the dilation gained during Phase I. The reason for this recovery was that the bottom section of the first hollow-core unit failed during Phase III allowing the plastic hinge to grow unrestrained and hence relieve some of the restraint forces with the plastic hinge.

F.3.2 Hollow-core pull off

Refer to Figure F-6 for the layout of the potentiometers used to measure the hollow-core pull off.

![Diagram showing the location of potentiometers measuring hollow-core pull off](image)

Figure F-6 Plan showing the location of the potentiometers measuring hollow-core pull off

Phase I

Figure F-7 shows the amount that the hollow-core units moved relative to the perimeter frame. Since the hollow-core units did not slide, but fractured (Chapter 4), the measurement recorded at the East and West beams was the hollow-core units crack width at the end of the hollow-core unit rather than the amount the units slid. A maximum crack width of 12mm was located at the SE corner on the East beam during the-2.0% load cycle. It should be noted that five (out of six) potentiometers positioned
Figure F-7 Hollow-core unit pull off during Phase I
on the two transverse beams never went negative (in displacement) during Phase I. This meant that for both a positive and negative inclination the bottom of the hollow-core unit was being pulled of its seat.

The two South beams (Beam B and C) potentiometers measured the width of crack that formed in the soffit of the hollow-core unit next to the perimeter frame (Figure F-7(b) and (c)). Most of the movement occurred in sixth and seventh potentiometers located either side of the central column (Figure F-6).

**Phase II**

Phase II saw an increase in the crack widths for the East and West beams (Figure F-8(a) and (d)). This increase was due to the two transverse beams rotating due to the eccentric load being applied to the beam from the hollow-core units as shown in Figure F-9. This rotation increased as the plastic hinge zones torsional stiffness reduced during the Phase II loading.

Within the southern beams (Figure F-8(b) and (c) only potentiometer number four recorded some significant growth (potentiometer six and seven was removed as it was positioned within a zone of localised damage). The damage that had occurred during Phase I had caused a crack to form in the bottom of the soffit of the hollow-core unit rather than having a crack forming between the hollow-core unit and the perimeter beam (refer to Figure 4-14, Chapter 4). This explains the low readings for most of the potentiometers as the crack occurred outside the instrumented region.

**Phase III**

No readings were taken during Phase III as the potentiometers were removed to ensure the instrumentation would not be damaged if the floor failed.
Figure F-8 Hollow-core unit pull off during Phase II
F.4 Initial cracking of the precast units under positive moments.

The failure mechanism that caused the initial cracks in the end of the hollow-core unit to form is shown in Figure F-10. The point at which the unit cracked can be analytically predicted. Firstly the section properties of the topping and hollow-core unit are required as a composite section. These properties are summarised in Table F-1.

Table F-1 Section properties for the hollow-core unit and topping section about the x-axis (refer to Figure F-11 for a diagram showing the composite section)

<table>
<thead>
<tr>
<th>Part</th>
<th>b (mm)</th>
<th>d (mm)</th>
<th>A (mm$^3$)</th>
<th>y (mm)</th>
<th>Ay</th>
<th>y-$\bar{y}$</th>
<th>A(y-$\bar{y}$)$^2$</th>
<th>$I_{local}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Topping</td>
<td>1200</td>
<td>75</td>
<td>90000</td>
<td>337.5</td>
<td>30.4×10$^6$</td>
<td>118.5</td>
<td>1.26×10$^9$</td>
<td>42.2×10$^6$</td>
</tr>
<tr>
<td>H/C unit</td>
<td>1200</td>
<td>300</td>
<td>160600</td>
<td>153</td>
<td>24.6×10$^6$</td>
<td>66</td>
<td>0.7×10$^9$</td>
<td>2.04×10$^9$</td>
</tr>
<tr>
<td>Σ</td>
<td></td>
<td></td>
<td>250600</td>
<td>55×10$^6$</td>
<td></td>
<td></td>
<td>1.96×10$^9$</td>
<td>2.08×10$^9$</td>
</tr>
</tbody>
</table>

\[
\bar{y} = \frac{\sum A_y y}{\sum A} = \frac{55 \times 10^6}{250600} = 219\text{mm} \tag{F-19}
\]

in which $\bar{y}$ = centroid of the transformed section.
\[ I_{xx} = A(y - \overline{y})^2 + I_{local} = 1.96 \times 10^9 + 2.08 \times 10^9 = 4.05 \times 10^9 \text{ mm}^4 \]  \hspace{1cm} (F-20)

in which \( I_{xx} \) = second moment of inertia about the \( x \)-axis of the transformed section; \( A \) = cross sectional area of the transformed section; \( y \) = distance to the centroid of the localised section; and \( I_{local} \) = localised moment of inertia.

Now determine the cracking moment \( (M_{cr}) \) of the hollow-core unit. Since the crack formed within 50mm of the edge of the hollow-core unit the presence of prestressing stands in the bottom of the unit is ignored as the strand needs approximately 500mm to develop its full capacity.

\[
\frac{M_{cr}}{I_{xx}} = \frac{f_{cr}}{\overline{y}} \Rightarrow M_{cr} = f_{cr} \frac{I_{xx}}{\overline{y}} = f_{cr} S_x
\]  \hspace{1cm} (F-21)

in which \( f_{cr} \) = tensile strength of the concrete (determined from NZS3101:1995); and \( S_x \) = section modulus of the composite section.

\[
f_{cr} = 0.5\sqrt{f'_c} = 0.5\sqrt{65} = 3.4 \text{ MPa} \text{ and } S_x = \frac{4.05 \times 10^9}{219} = 18.5 \times 10^6 \]  \hspace{1cm} (F-22)

in which \( f'_c \) = concrete compressive strength of the hollow-core unit (=45MPa, as explained earlier).

By substituting in the values for \( f_{cr} \) and \( S_x \), Equation (F-21) becomes

\[ M_{cr} = 3.4 \times 18.5 \times 10^6 = 62.1 \times 10^6 \text{ Nmm or } 62.1 \text{kNm} \]  \hspace{1cm} (F-23)

Now determine the rotation \( (\theta_{cr}) \) required to generate this cracking moment by assuming the hollow-core unit and supporting beam can be approximated using moment area.

\[
\delta_{cr} = \frac{F_{cr} L^3}{3EI} = \frac{M_{cr} L^2}{3EI} \Rightarrow \theta_{cr} = \frac{M_{cr} L}{3EI}
\]  \hspace{1cm} (F-24)
\[ \theta_{cr} = \frac{62.1 \times 10^6 \times 5900}{3 \times 30000 \times 4050 \times 10^6} = 0.0010 \] (F-25)

in which \( \delta_{cr} \) = tip deflection of the cantilever required to cause the hollow-core unit to fracture; \( F_{cr} \) = shear force required to be applied to the tip of the cantilever to cause the end of the hollow-core unit to fracture; \( L \) = length from the support of the hollow-core unit to where the shear force is applied, \( E \) = modulus of elasticity of the concrete; and \( I \) = second moment of area of the topping slab and hollow-core unit.

At an interstorey drift of 0.10\%, the end of the hollow-core unit cracks. Once the concrete cracked, the induced strain in the concrete either side of the crack is relieved. Now determine the width of crack \( w \) that would be present at 0.32\% interstorey drift (the drift at which the crack was noticed)

\[ w = \Delta \theta \times jD = (0.0032 - 0.0010) \times 335 = 0.74 \text{mm} \] (F-26)

in which \( \Delta \theta \) = change in rotation between the rotation at first crack and when the crack was first observed; \( jD \) = the internal leverarm between the centre of rotation and the extreme tension fibre as shown in Figure F-10.

Assumed to slide

Actual behaviour

Figure F-10 Assumed versus actual hollow-core to beam performance.

F-18
F.5 Bowstring effect

When a reinforced concrete frame elongates due to plastic hinges forming within the beams, the floor slab provides restraint to the beam growth, leading to an increase in the capacity of the frame. This restraint causes the floor slab to act in tension while the beam goes into compression. This phenomena is referred to as the "bowstring effect" and has been explained by Fenwick et al (1999). It is called the "bowstring effect" because the beam in compression acts like a bow and the floor slab in tension acts like the string within a bow.

During the current testing programme a slightly different event occurred. When the floor diaphragm tore the width of floor slab that was in tension reduced to the width of one hollow-core unit (the unit that was still attached to the perimeter beam). The tear then enabled the central column to freely translate in a direction transversely to the direction of loading (out of the building), as the column was not tied into the remainder of the floor slab. The amount that the column translated can be calculated based on a force couple and a calculated moment of inertia.

Firstly, the section properties for the combined beam and floor slab section about the y-axis need to be determined as seen in Figure F-11 and calculated in Table F-2.

\[ \bar{y} = \frac{\sum A y}{\sum A} = \frac{301.6 \times 10^6}{541160} = 557 \text{mm} \]  \hspace{1cm} (F-27)

in which \( \bar{y} = \) centroid of the transformed section.

\[ I_{yy} = A(y - \bar{y})^2 + I_{Local} = 96.5 \times 10^9 + 24.5 \times 10^9 = 120.9 \times 10^9 \text{mm}^4 \]  \hspace{1cm} (F-28)
in which $I_{yy}$ = second moment of inertia about the y-axis of the transformed section; $A$ = cross sectional area of the transformed section; $y$ = distance to the centroid of the localised section; and $I_{local}$ = localised moment of inertia.

To determine the amount that the column translates due to a horizontally applied moment, caused by the offset between the centroid of the compression and tension forces, moment area was used. Using symmetry (about the central column of the super-assemblage) an expression for the translation ($\delta$) can be written as

![Diagram](image-url)

Figure F-11 Section properties for the composite hollow-core and topping sections
\[ \delta = \frac{ML^2}{3EI} \]  

in which \( M \) = horizontal bending moment induced by the tension field within the floor slab; \( L \) = length of the beam; \( E \) = modulus of elasticity of the concrete; and \( I \) = second moment of inertia of the member.

Table F-2 Section properties for the combined beam and floor slab section about the y-y axis  
(refer to Figure F-11 for the cross section)

<table>
<thead>
<tr>
<th>Part</th>
<th>b (mm)</th>
<th>d (mm)</th>
<th>A (mm²)</th>
<th>y (mm)</th>
<th>Ay</th>
<th>(y-\bar{y})</th>
<th>A(y-\bar{y})²</th>
<th>I_{Local} (mm⁴)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>108</td>
<td>1200</td>
<td>129600</td>
<td>1000</td>
<td>129.6×10⁶</td>
<td>443</td>
<td>25.4×10⁹</td>
<td>15.6×10⁹</td>
</tr>
<tr>
<td>2</td>
<td>40</td>
<td>1200</td>
<td>48000</td>
<td>1000</td>
<td>48×10⁶</td>
<td>443</td>
<td>9.5×10⁹</td>
<td>5.8×10⁹</td>
</tr>
<tr>
<td>3</td>
<td>227</td>
<td>60</td>
<td>13620</td>
<td>1570</td>
<td>21.4×10⁶</td>
<td>1013</td>
<td>14×10⁹</td>
<td>4.1×10⁶</td>
</tr>
<tr>
<td>4</td>
<td>227</td>
<td>55</td>
<td>12485</td>
<td>1290</td>
<td>16.1×10⁶</td>
<td>733</td>
<td>6.7×10⁹</td>
<td>3.1×10⁹</td>
</tr>
<tr>
<td>5</td>
<td>227</td>
<td>50</td>
<td>11350</td>
<td>1000</td>
<td>11.4×10⁶</td>
<td>443</td>
<td>2.2×10⁹</td>
<td>2.4×10⁹</td>
</tr>
<tr>
<td>6</td>
<td>227</td>
<td>55</td>
<td>12485</td>
<td>710</td>
<td>8.9×10⁶</td>
<td>153</td>
<td>0.3×10⁹</td>
<td>3.1×10⁹</td>
</tr>
<tr>
<td>7</td>
<td>227</td>
<td>60</td>
<td>13620</td>
<td>430</td>
<td>5.9×10⁶</td>
<td>127</td>
<td>0.2×10⁹</td>
<td>4.1×10⁹</td>
</tr>
<tr>
<td>8</td>
<td>750</td>
<td>400</td>
<td>300000</td>
<td>200</td>
<td>60×10⁶</td>
<td>357</td>
<td>38.2×10⁹</td>
<td>4×10⁹</td>
</tr>
<tr>
<td>Σ</td>
<td>541160</td>
<td></td>
<td>301.3×10⁶</td>
<td></td>
<td></td>
<td></td>
<td>96.5×10⁹</td>
<td>25.4×10⁹</td>
</tr>
</tbody>
</table>

Knowing the compression strengths (=45MPa, as explained earlier) it was possible to determine the modulus of elasticity \( E \) for the composite section.

\[ E = 3320 \sqrt{f'_c} + 6900 = 3320\sqrt{45} + 6900 \approx 30000MPa \]  

(F-30)

To determine the applied moment due to the centroid of the compression and tension forces being offset (Figure F-12), the magnitude of the tension force \( T \) and the eccentricity \( e \) from the centroid of the compression force is required.

The tension force \( T \) was determined by assuming all the slab reinforcement within the first hollow-core unit was at yield. This assumption agrees with the
effective slab width used to determine the contribution to the super-assemblages lateral strength in Chapter 5.

\[ T = \text{mesh} + \text{prestress} = A_{\text{smesh}} f_{\text{ymesh}} + A_{\text{shec}} f_{\text{yhc}} \quad \text{(F-31)} \]

\[ T = (174 \times 585) + (1000 \times 1030) = 1132 kN \quad \text{(F-32)} \]

![Diagram showing offset compression and tension forces](image)

Figure F-12 Plan and elevation showing the offset compression and tension forces that cause the central column to displace laterally.

in which \( A_{\text{smesh}} = \) area of the reinforcing mesh within one hollow-core unit (=174mm\(^2\)); \( f_{\text{ymesh}} = \) yield stress of the reinforcing mesh (=520MPa, obtained from the steel test results in Appendix B); \( A_{\text{shec}} = \) area of the prestressing strands in the hollow-core unit within one hollow-core unit (=10 strands of 12.5mm diameter-7-wire strand); and \( f_{\text{yhc}} = \) yield stress of the prestressing strands (taken as 1030MPa)
The eccentricity (e) is assumed to be the distance between the centroid of the section in tension and the centroid of the section in compression (perimeter beam).

\[ e = 1000 - 200 = 800 \text{mm} \]  
\[ (F-33) \]

Therefore, the applied moment is determined by

\[ M = Te = 1132 \times 0.8 = 906 \text{ kNm} \]  
\[ (F-34) \]

In order to determine the deformation of the floor slab in the horizontal plane, the stiffness of the beam (EI) and first hollow-core unit needs to be determined. A constant value for EI was assumed based on the gross section properties (denoted as the average EI. for the explanation refer to Figure F-13). Now equate the deflection caused by this applied moment in Equation (F-29).

\[ \delta = \frac{906 \times 10^6 \times 6100^2}{3 \times 30000 \times 121 \times 10^9} = 3.1 \text{mm} \]  
\[ (F-35) \]

This deflection of 3.1 mm compares well to the observed displacement of approximately 2 mm at the initiation of the tear (1.93%).

![Diagram](image-url)

**Figure F-13 Determination of EI in the transformed section**
F.6 Determining the applied base shear

Using the applied base shear hysteresis loops it is possible to plot different capacity lines on the graph that represent different approximations used to predict the lateral strength of the super-assemblage. Four different mechanisms have been investigated: (i) a proposed theory for predicting the components that contribute to the lateral strength; (ii) the NZS3101:1995 recommendations; (iii) the ACI 318-02 recommendations; and (iv) the progressive activation of the floor slab based on the experimental observations.

To calculate the moment capacities at the various plastic hinge locations, the reinforcement cross sectional areas, yield stress and leverarms for the super-assemblage were required. These values are summarised in Table F-3 and F-4 for the longitudinal (Phase I and III) and transverse (Phase II) loading, respectively. The values in Table F-3 and F-4 for the starter bars, reinforcing mesh and prestressing strands are stated as individual bars/ wires or strands. The yield stresses are measured values, obtained from steel tests on the actual reinforcement used (Appendix B) rather than specified fifth percentile values.

\[ A_s = \text{cross sectional area of reinforcement, } f_y = \text{measured yield stress of the reinforcement and } j_d = \text{internal leverarm between the centre of compression and the centre of the particular reinforcement in tension.} \]
Table F-3 Reinforcement details for the longitudinal loading direction

<table>
<thead>
<tr>
<th></th>
<th>Perimeter Beam</th>
<th>Starter Bar</th>
<th>Reinforcing Mesh</th>
<th>P/S Strands (negative moment)</th>
<th>P/S Strands (positive moment)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_s$ (mm$^2$)</td>
<td>2714</td>
<td>113</td>
<td>22</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>$f_y$ (MPa)</td>
<td>299</td>
<td>508</td>
<td>520</td>
<td>1030</td>
<td>1030</td>
</tr>
<tr>
<td>jd (mm)</td>
<td>560</td>
<td>610</td>
<td>610</td>
<td>360</td>
<td>250</td>
</tr>
<tr>
<td>Bending moment (kNm)</td>
<td>455</td>
<td>35</td>
<td>7</td>
<td>37</td>
<td>26</td>
</tr>
</tbody>
</table>

Table F-4 Reinforcement details for the transverse loading direction

<table>
<thead>
<tr>
<th></th>
<th>Perimeter Beam</th>
<th>Starter Bar</th>
<th>Cold-drawn Wire reinforcing mesh</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_s$ (mm$^2$)</td>
<td>2714</td>
<td>113</td>
<td>22</td>
</tr>
<tr>
<td>$f_y$ (MPa)</td>
<td>299</td>
<td>508</td>
<td>520</td>
</tr>
<tr>
<td>jd (mm)</td>
<td>512</td>
<td>560</td>
<td>560</td>
</tr>
<tr>
<td>Bending moment (kNm)</td>
<td>416</td>
<td>32</td>
<td>6</td>
</tr>
</tbody>
</table>

Proposed Theory:

This theory uses rigid body kinematics to determine the moment capacity for the perimeter beam and slab (Figure F-14).

For an exterior plastic hinge, a tripartite curve was proposed to estimate the lateral strength (Chapter 5). A tripartite curve was required because the width of floor slab activated increased with interstorey drift. The amount of activation is dependent on the torsional stiffness of the transverse beam. To define this curve two data points are required: (i) the moment capacity at first yield (the activated slab width =1.2m)
and (ii) at full plasticity (the activated slab width = 3.05m) as shown in Figure F-15. It should be noted that for the proposed theory the prestressing strands within the hollow-core unit contributes to the lateral strength for both a positive and negative moment on either side of the central column.

For an interior plastic hinge, a bi-linear approximation was proposed to estimate the lateral strength. To define this curve only the capacity at first yield is required (the activated slab width =1.2m) as shown in Figure F-15.
(a) A sketch showing how rigid body kinematics determine the displaced shape and in turn are used to generate the moment capacities for the various plastic hinges.

(b) Section A-A at all stages of plastification

(c) Section B-B at the onset of plastification (yield)

(d) Section B-B at full plastification

Figure F-14 The use of rigid body kinematics to explain how the additional tension reinforcement contributes to the lateral strength of the super-assembly at different stages of progressive plastification
Figure F-15 Proposed floor slab activation with increasing interstorey drift.

Phase 1: at the onset of plastification

Table F-5 Nominal moment capacities for the South Frame

<table>
<thead>
<tr>
<th></th>
<th>SW</th>
<th>CC-LHS</th>
<th>CC-RHS</th>
<th>SE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td>455</td>
<td>455</td>
<td>455</td>
<td>455</td>
</tr>
<tr>
<td>Starter bars</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>4×35</td>
</tr>
<tr>
<td>Reinforcing mesh</td>
<td>-</td>
<td>47</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>P/S strands</td>
<td>-</td>
<td>11×37</td>
<td>11×26</td>
<td>-</td>
</tr>
<tr>
<td>Total</td>
<td>455</td>
<td>909</td>
<td>741</td>
<td>595</td>
</tr>
</tbody>
</table>

Sum of nominal moments = 455+909+741+595

(Column face) = 2700kNm

(Convert to column centreline) = 2700 × 1.14 = 3078kNm

\[
\sum V_{col} = \frac{\sum M_n}{h_z} = \frac{3078}{3.5} = 757\text{kN}
\]

in which \( V_{col} \) = lateral force applied to the column; \( M_n \) = nominal beam moment; and \( h_z \) = interstorey height of the columns. Refer to Figure F-16 shows the location of \( V_{col} \), \( M_n \) and \( h_z \).
Figure F-16 Notation used to determine the sum of $V_{col}$ (Applied base shear)

**Phase II:** at the onset of plastification

<table>
<thead>
<tr>
<th>Table F-6 Nominal moment capacities for the East and West Frames</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
<tr>
<td>Beam</td>
</tr>
<tr>
<td>Starter bars</td>
</tr>
<tr>
<td>Reinforcing mesh</td>
</tr>
<tr>
<td>P/S strands</td>
</tr>
<tr>
<td>Total</td>
</tr>
</tbody>
</table>

Sum of nominal moments = $2(416+544)$

(Column face) = 1912kNm

(Convert to column centreline) = 1920 × 1.14 = 2189kNm

$$
\sum V_{col} = \frac{\sum M_s}{h_s} = \frac{2189}{3.5} = 625kN
$$
Phase I: at full plasticity

Table F-7 Nominal moment capacities for the South Frame

<table>
<thead>
<tr>
<th></th>
<th>SW</th>
<th>CC-LHS</th>
<th>CC-RHS</th>
<th>SE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td>455</td>
<td>455</td>
<td>455</td>
<td>455</td>
</tr>
<tr>
<td>Starter bars</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>10x35</td>
</tr>
<tr>
<td>Reinforcing mesh</td>
<td>-</td>
<td>47</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>P/S strands</td>
<td>-</td>
<td>11x37</td>
<td>11x26</td>
<td>-</td>
</tr>
<tr>
<td>Total</td>
<td>455</td>
<td>909</td>
<td>741</td>
<td>805</td>
</tr>
</tbody>
</table>

Sum of nominal moments = 455+909+741+805

(Column face) = 2910kNm

(Convert to column centreline) = 2910 x 1.14 = 3317kNm

\[ \sum V_{col} = \frac{\sum M_n}{h_z} = \frac{3317}{3.5} = 948kN \]

Phase II: at full plasticity

Table F-8 Nominal moment capacities for the East and West Frames

<table>
<thead>
<tr>
<th></th>
<th>SW=SE</th>
<th>NW=NE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td>416</td>
<td>416</td>
</tr>
<tr>
<td>Starter bars</td>
<td>-</td>
<td>10x32</td>
</tr>
<tr>
<td>Reinforcing mesh</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>P/S strands</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Total</td>
<td>416</td>
<td>736</td>
</tr>
</tbody>
</table>

Sum of nominal moments = 2(416+736)

(Column face) = 2304kNm

(Convert to column centreline) = 2304 x 1.14 = 2627kNm

\[ \sum V_{col} = \frac{\sum M_n}{h_z} = \frac{2627}{3.5} = 750kN \]

F-30
NZS3101 prediction (Phase I and II):

The New Zealand Concrete Structures Standard (NZS 3101:1995) gives guidance on determining the amount of strength enhancement from the floor diaphragm. This prediction is based on the work carried out by Cheung et al (1991). This mechanism assumes the beam longitudinal reinforcement plus a contribution from the floor slab are used to calculate the nominal beam moment. Determining the width of slab activated according to NZS3101 can be difficult to follow. Confusion leads as to whether the prestressing strands within the hollow-core units should be included for determining the Phase I mechanism (the direction in which the prestressing strands run parallel to the direction of loading). This mechanism assumes that the prestressing stands do not contribute to the strength enhancement. The width of floor slab assumed to contribute (according to NZS3101:1995) was taken as 1.53m \((L_b/4)\) for both the longitudinal and transverse loading directions.

Phase I:

For any slab reinforcement to be considered to contribute to the negative moment capacity of the beam it must be effectively anchored reinforcement within the zone of possible activation.

For an exterior hinge, the number of activated bars equals one HD12 starter bar plus 10 cross wires of cold-drawn wire reinforcing mesh. This is illustrated in Figure F-17.
Figure F-17 Determining the number of activated bars in an exterior joint

(reinforcing mesh is not shown for clarity)

For an interior joint, only the reinforcing mesh is assumed to contribute as there are no starter bars present at that location.

| Table F-9 Nominal Moment capacities for the South Frame |
|----------------|---------|---------|---------|---------|
|               | SW      | CC-LHS  | CC-RHS  | SE      |
| Beam          | 455     | 455     | 455     | 455     |
| Starter bars  | -       | -       | -       | 1×35    |
| Reinforcing mesh | -     | 10×7    | -       | 10×7    |
| P/S strands   | -       | -       | -       | -       |
| Total         | 455     | 525     | 455     | 560     |

Sum of nominal moments = 455+525+455+560 = 1995kNm

(Column face) = 1995 × 1.14 = 2274kNm

\[ \sum V_{col} \sum \frac{M_n}{h_x} = \frac{2274}{3.5} = 650kN \]

Phase II:

For any slab reinforcement to be considered to contribute to the negative moment capacity of the beam it must be effectively anchored reinforcement within the zone of possible activation.
For an exterior hinge, the number of activated bars equals one HD12 starter bar plus 10 cross wires of cold-drawn wire reinforcing mesh. This is illustrated in Figure F-18.

![Diagram](image)

**Figure F-18** Determining the number of activated bars in the floor slab (reinforcing mesh is not shown for clarity)

<table>
<thead>
<tr>
<th>Table F-10 Nominal moment capacities for the East and West Frames</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
<tr>
<td>Beam</td>
</tr>
<tr>
<td>Starter bars</td>
</tr>
<tr>
<td>Reinforcing mesh</td>
</tr>
<tr>
<td>P/S strands</td>
</tr>
<tr>
<td><strong>Total</strong></td>
</tr>
</tbody>
</table>

Sum of nominal moments = 2(416+518)

(Column face) = 1868kNm

(Convert to column centreline) = 1868 × 1.14 = 2130kNm

\[
\sum V_{col} = \frac{\sum M_n}{h_x} = \frac{2130}{3.5} = 608\text{kN}
\]

**ACI 318-02 prediction (Phase I and II):**

This mechanism assumes the beam longitudinal reinforcement plus a contribution from the floor slab as assumed by ACI 318-02 is used to calculate the nominal beam moment. According to ACI 318-02, the effective flange width was taken as the lesser
of one-twelfth the span of the beam, six times the slab thickness, and one-half the clear distance to the next web. One-twelfth the span of the beam governs in this case and equates to 510mm.

For Phase I, the diaphragm contribution is made up of the starter bars, reinforcing mesh and the prestressing strands within the hollow-core units while for Phase II only the starter bars contribute.

Phase I:
For an exterior hinge, the number of activated starter bars equals two HD12 starter bars. For an interior joint, the amount of activation is three wires of cold-drawn wire reinforcing mesh and four prestressing strands.

<table>
<thead>
<tr>
<th>Table F-11 Nominal moment capacities for the South Frame</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
<tr>
<td>------------------</td>
</tr>
<tr>
<td>Beam</td>
</tr>
<tr>
<td>Starter bars</td>
</tr>
<tr>
<td>Reinforcing mesh</td>
</tr>
<tr>
<td>P/S strands</td>
</tr>
<tr>
<td>Total</td>
</tr>
</tbody>
</table>

Sum of nominal moments = 455+624+455+525

(Column face) = 2059 kNm

(Convert to column centreline) = 2059 × 1.14=2347 kNm

\[
\sum V_{col} = \sum \frac{M_n}{h_z} = \frac{2347}{3.5} = 671 \text{kN}
\]

Phase II:
For an exterior hinge, the number of activated starter bars equals two HD12 starter bars.
Table F-12 Nominal moment capacities for the East and West Frames

<table>
<thead>
<tr>
<th></th>
<th>SW=SE</th>
<th>NW=NE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td>416</td>
<td>416</td>
</tr>
<tr>
<td>Starter bars</td>
<td>-</td>
<td>2x32</td>
</tr>
<tr>
<td>Reinforcing mesh</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>P/S strands</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>416</td>
<td>480</td>
</tr>
</tbody>
</table>

Sum of nominal moments $= 2(416+480)$

(Column face) $= 1792 \text{kNm}$

(Convert to column centreline) $= 1792 \times 1.14 = 2043 \text{kNm}$

$$\sum V_{col} = \sum \frac{M_n}{h_s} = \frac{2043}{3.5} = 584 \text{kN}$$

F.7 Phase III moment diagrams

Phase III:

For both the SW and SE columns (Figure F-19 (a) and (b)), the positive moment capacity agrees well with the observed moments. The SE observed moment was slightly larger than the prediction because the plastic hinge did not form exactly on the column face. Also, the reinforcement within the plastic hinge had started to strain harden. For a negative moment, the predicted capacity overestimated the capacity. The overestimation was due to the degrading condition of the hollow-core unit and its connection detail to the supporting beam. It was this deterioration that caused the lateral strength to reduce.

For the central column (Figure F-19(c)), the expected capacity was less than that observed as the expected capacity consisted of the negative beam moment plus the contributing slab consisting of the reinforcing mesh and prestressing strands. The
deterioration of the hollow-core unit contributed to the overestimated prediction of the lateral strength.

(a) SW columns moment versus rotation  (b) SE columns moment versus rotation

(c) Central Columns moment versus rotation plot for Phase III

(d) LHS central column beam moments  (e) RHS central column beam moment

Figure F-19 Outer column moment versus rotation plots for Phase III

As explained in Chapter 5, the axial load recorded by the instrumented universal joints at the base of the columns was not consistent. This meant that it was not possible to accurately determine the centre column beam moments. The calculation of the beam moments was generally good at low levels of drift, this was because the loading frames were self-equilibrating which meant that all the loads were equalised.
within the frame and the column axial load did not change. Once moment redistribution occurred, the column axial loads changed, this caused the incorrect moment to be calculated. Figure F-19(d) and (e) shows the incorrectly calculated beam moment capacities for the interior plastic hinge on either side of the central column. The actual bending moment plots should look similar (in shape) to that of the interior plastic hinge moment diagrams in Phase I (refer to Chapter 5).

**F.8 References**

ACI 318-02, *Building Code Requirements for Structural Concrete and Commentary*, American Concrete Institute, Farmington Hills, USA


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Appendix G

Diaphragm Performance

G.1 Technical Advisory Group (TAG) on precast floors testing

The sub-assemblage used to test the connection details recommended by the TAG is shown in Figure G-1. The shear force pattern experienced during the super-assemblage testing (12m span) is shown in Figure G-2. The 2-D set up used to test the TAG recommended details was based on the expected deformations and shear forces found from the super-assemblage testing programme. Clamping the supporting beam to the structural floor and articulating the hollowcore relative to the beam achieved this. The background to the development of this set up is also shown in Figure G-2.

Figure G-1 Sub-assemblage experimental set up (Bull and Matthews, 2003)
Figure G-2 Schematic showing the determination of the experimental set up

(Bull and Matthews, 2003)

G.2 Diaphragm Performance

Included below are all the various graphs showing the diaphragm growth during the experimental programme. The growth within the floor is obtained from summing the demec points across the floor, refer to Figure G-3 for the location of the demec points.
Figure G-3 Demec point location

In the figures below is the longitudinal growth for all the positive drift cycles.
Row 2 for Phase I

Row 3 for Phase I

Row 4 for Phase I
Row 5 for Phase I

Row 6 for Phase I

Row 7 for Phase I
In the figures below is the longitudinal growth for all the negative drift cycles.
Row 2 for Phase I

Row 3 for Phase I

Row 4 for Phase I
Row 5 for Phase I

Row 6 for Phase I

Row 7 for Phase I
Overall longitudinal diaphragm growth for Phase I

Positive drift growth for Phase I

Negative drift growth for Phase II
Transverse diaphragm growth for Phase I

Positive drift transverse growth for Phase I

Negative drift transverse for Phase I
Longitudinal growth for Phase II

![Graph showing longitudinal growth](image)

Note: No significant change in the length of the diaphragm occurred during Phase II

Transverse growth for Phase II

![Graph showing transverse growth](image)

Note: The length of the transverse diaphragm only changed during the 3.5% drift load cycle. All the pervious growth occurred during Phase I.

### G.3 References

Appendix H

Beam Elongation

H.1 Observed beam elongation

All the plots herein show beam elongation starting from “zero elongation”. “Zero elongation” is defined as the initial length of the beam at the start of that particular phase of loading. The total beam elongation for each beam is given at the end of this appendix.

H.1.1 Phase I

During loading in Phase I (in the longitudinal direction), beam elongation was seen to occur in all the four beams instrumented. The major elongation occurred in the beams parallel to the direction of loading (Beams B and C), a smaller amount of elongation was also seen in the two transverse beams (Beams A and D). The elongation in all the beams is shown in Figure H-1.

The elongation that formed in beams B and C was similar as the two beams had symmetric reinforcement. A more detailed breakdown of the elongation within beams B and C is shown in Figure H-2. The response from the two plastic hinges within each beam is quite different. The reason for this difference is due to the support connections at each end of the beam and the reinforcement ratios. The exterior plastic hinges in both beams represents a simple support, for the connection of the hollow-core unit to beam, while the central connection acts as a continuous support because the hollow-core unit spans past the central column restraining some of the elongation that tries to form whereas the simple support does not restrain the growth, it is simply...
pulled off its support. The reinforcement ratio affects $e_{cr}$, the force eccentricity of the beam depth that is the distance between the beam centreline and the instantaneous centre of rotation (centroid of the compression force), which in turn affects the amount of elongation.

(a) Beam elongation in beams B and C (longitudinal beams)

(b) Beam elongation in beams A and D (transverse beams)

Figure H-1 Observed Phase I beam elongation

Figure H-2 shows that the exterior joints (simple support) beam elongation was larger than the amount of elongation at the interior joint (continuous support). When a plastic hinge forms next to the central column and the beam tries to grow, the
floor slab goes into tension and restrains the growth. For a simple support, the floor unit is not sufficiently anchored to the supporting beam to restrain the beam elongation. In the case of this experimental programme the end of the hollow-core unit had fractured before the beam started to gain any substantial beam elongation so there was no appreciable restraint.

(a) The plastic hinge contribution to beam B's elongation

(b) The plastic hinge contribution to beam C's elongation

Figure H-2 The individual components of beam B and C contributing to the total beam elongation

Beams A and D grew in length because of the torsion cracks that formed within these transverse beams (Figure H-1).
H.1.2 Phase II

Phase II loading saw the transverse beams grow in length while the longitudinal beams remained essentially unchanged (Figure H-3). The torsion-induced beam elongation in the beams perpendicular to the direction of loading is seen in Figure H-3(b). Beams B and C grew in length during the displacement cycles but recovered the growth when unloaded. Beam C had a net gain in elongation of 4.7mm at the completion of Phase II while beam B grew under 1mm.

(a) Observed beam elongation in beam A and D

(b) Observed beam elongation in beam B and C

Figure H-3 Observed Phase II elongation plot.
The transverse beams grew approximately 20mm per plastic hinge, as shown in Figure H-4. The reason why the plastic hinges grew the same amount was because all the plastic hinges had similar reinforcement ratios and support details (a simple support).

(a) Phase II beam A’s elongation plot.

(b) Phase II beam D’s elongation plot

Figure H-4. Individual components of beams A and D contributing to the total beam elongation during Phase II

Figure H-5 shows the rainflow plot for the predicted beam elongation for the West beam versus the observed elongation. Again there was good agreement between the theory and the observations.
H.1.3 Phase III

During Phase III, the total growth within the longitudinal beams (beam B and C) was smaller than during Phase I. This is because no new significant plastic rotation was imposed when compared to Phase I. As stated in Chapter 5, a large increase in beam elongation occurs due to an increase in new rotation from a previous maximum. The amount of beam elongation in both the longitudinal beams is shown in Figure H-6.

![Beam B Elongation](image)

**Figure H-6 Observed Phase III elongation plot**

Figure H-7 shows the individual hinge elongation within each beam for the third phase of loading. The first section of floor failed at 2.5% drift and from that point onwards the interior plastic hinges grew more than the outer hinge. This is due to the change in $e_{cr}$, from approximately 0.5 to 0.85 since the central column connection no longer acts as a continuous support. Figure H-7 showed the interior hinge elongated the same amount as the exterior hinge once the floor failed, this is not the case in Figure H-2 where the interior hinge elongation was approximately 40% of the exterior hinge.
H.2 Derivation of the elastic beam elongation

The elastic component of beam elongation can be derived from theory by examining on the strain within the beam member at its centre of gravity for the concrete section (c.g.c) (Figure H-8(a)). The elastic elongation component \( \delta_{\varepsilon} \) can be found by integrating the neutral axis (c.g.c) strain along the length of the beam as follows:

\[
\delta_{\varepsilon} = \frac{1}{2} \varepsilon_0^+ \xi L_b + \frac{1}{2} \varepsilon_0^- L_b (1 - \xi)
\]  

(H-1)
in which \( \varepsilon_0^+ \) – the strain at the onset of first yield at the member end at the c.g.c for a positive bending moment \( (M^+) \); \( \varepsilon_0^- \) = the strain at the onset of first yield at the other member end at the c.g.c for a negative bending moment \( (M^-) \); and \( \zeta \) = portion of the total beam length \( (L_b) \) to where the beam bending moment is zero (Figure H-8)

\[
\delta^d_\varepsilon = \frac{1}{2} \varepsilon_0^+ L_b - \frac{1}{2} \varepsilon_0^- L_b \zeta + \frac{1}{2} \varepsilon_0^- L_b \zeta
\]

\[
\delta^d_\varepsilon = \frac{L_b}{2} \left[ (\varepsilon_0^+ - \varepsilon_0^-) \zeta + \varepsilon_0^- \right]
\]

\[
\zeta = \frac{M^+}{M^+ + M^-} \implies \delta^d_\varepsilon = \frac{L_b}{2} \left[ \varepsilon_0^- + \frac{\varepsilon_0^+ - \varepsilon_0^-}{1 + \frac{M^-}{M^+}} \right]
\]

\[
\delta^d_\varepsilon = \frac{L_b}{2} \left[ \frac{\varepsilon_0^- + \varepsilon_0^+ \frac{M^-}{M^+} + \varepsilon_0^- - \varepsilon_0^-}{1 + \frac{M^-}{M^+}} \right]
\]

\[
\delta^d_\varepsilon = \frac{L_b}{2} \left[ \frac{\varepsilon_0^+ + \varepsilon_0^- \frac{M^-}{M^+}}{1 + \frac{M^-}{M^+}} \right]
\]

\[
\delta^d_\varepsilon = \frac{L_b}{2} \left[ \frac{\varepsilon_0^+ M^+ + \varepsilon_0^- M^-}{M^+ + M^-} \right]
\]

From a strain diagram (Figure H-8(b)) it is possible to write expressions for the strain at the c.g.c \( (\varepsilon_c) \) and curvature \( (\phi) \).
(a) Beam bending moment and strain diagrams used to determine the elastic component of beam elongation.

(b) Strain diagram used to determine the yield curvature for a beam

Figure H-8 Figures used to determine the elastic beam elongation

\[ \varepsilon_o = \varepsilon_y - \frac{d}{2} \phi \]  

\[ \phi = \frac{\varepsilon_y}{d - kd} = \frac{\varepsilon_y}{d(1 - k)} \]

in which \( \varepsilon_y \) = yield strain of the beam reinforcement; and \( d \) = effective depth of the beam.

\[ \varepsilon_o = \varepsilon_y - \frac{d}{d(1 - k)} \varepsilon_y = \varepsilon_y - \frac{\varepsilon_y}{2(1 - k)} \]  

in which \( k \) = fraction of the beam depth from the extreme compression fibre to the neutral axis depth.

H-10
\[ \varepsilon_o = \varepsilon_y \left( 1 - \frac{1}{2(1-k)} \right) \]  

\[ \varepsilon_o = \varepsilon_y \left( \frac{2 - 2k - 1}{2 - 2k} \right) = \frac{1 - 2k}{2 - 2k} \]  

Substituting Equation (H-12) into Equation (H-7) becomes

\[ \delta_e^{st} = \frac{L_b}{2} \varepsilon_y \left[ \frac{1 - 2k^+}{2 - 2k^+} M^+ + \frac{1 - 2k^-}{2 - 2k^-} M^- \right] \]  

To simply Equation (H-13) assume that the positive yield moment equals the negative yield moment \((M^+ = M^-)\) and hence the fraction of member depth \((k)\) at yield will also be equal \((k = k^+ = k^-)\).

\[ \delta_e^{st} = \frac{L_b}{4} \varepsilon_y \left[ 1 - 2k \right] \approx \varepsilon_y \frac{L_b}{4} [1 - k] \]