EXPERIMENTS ON THE SEISMIC PERFORMANCE OF HOLLOW-CORE FLOOR SYSTEMS IN PRECAST CONCRETE BUILDINGS

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Abstract:

This research follows on from recently completed work conducted by Matthews (2004) who investigated the seismic performance of precast concrete frame buildings with precast hollow-core floor slabs and demonstrated major shortcomings in standard construction practice that would lead to very poor seismic performance. Matthews (2004) outlined several areas of future research needs, several of which will be addressed in this research. The super-assemblage constructed by Matthews (2004) was severely damaged during his experiment; however, the precast frames were repaired and reused in this research. New end beams with wider (75mm) seat widths are cast and a simple (pinned) connection between the hollow-core units and the beam adopted. The redeveloped super-assemblage is tested in both the longitudinal and transverse directions up to interstorey drifts of ±4.0% and experimental and instrumental observations from the experiment are outlined. A forensic study is conducted to better understand the observed failure modes. A theoretical pushover curve for predicting the capacity of the super-assemblage is developed using the principles of yield line theory and progressive flange activation. Low cycle fatigue theory is used to show the final failure of such a precast frame is related to the fatigue life of the main beam longitudinal bars. Finally a fragility analysis is used to determine the seismic vulnerability of this class of precast concrete structure. The efficacy of the detailing improvements is clearly demonstrated. There is now at least a 90% confidence interval of satisfactory seismic performance. However, with the old details there is a virtual certainty of extensive irreparable damage and even the possibility of collapse.
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Section 1

Review and Experiment Design

1.1 Introduction

The use of precast hollow-core floor units in reinforced concrete moment resisting frames has been the conventional form of construction in New Zealand since the 1980s. However, following the 1994 Northridge (U.S.A.) earthquake serious concerns were raised as to the performance of precast buildings with hollow-core flooring systems (Norton et al 1994, Iverson and Hawkins 1994 and Holmes and Somers 1995). The major concern was with the connection of these precast hollow-core units to the lateral load resisting system. Should these hollow-core units loose their seating due to a connection failure with the surrounding perimeter beam, whole sections of the floor could collapse.

Recent earthquake engineering research undertaken at the University of Canterbury has aimed at determining whether New Zealand designed and built precast concrete structures, which incorporate precast concrete hollow-core floor slabs, also possess inadequate seating support details. A full scale precast concrete super-assemblage was constructed in the laboratory and tested in two stages. The first stage of testing investigated existing construction and demonstrated major shortcomings in standard construction practice that would lead to very poor seismic performance.

This thesis follows on from research work recently completed by Matthews (2004). His work investigated the seismic performance of precast concrete hollow-core floor slab buildings during a major earthquake. Further, Matthews’ (2004) study opened the door to many more questions that require immediate answering.
Matthews (2004) work also outlines several areas for future research including: improving the seating connection detail between the precast, prestressed hollow-core floor diaphragm and the perimeter moment resisting frame, isolating the first hollow-core unit, spanning parallel with the perimeter frame, from the frame for displacement incompatibility reasons and stopping the central column from displacing laterally out of the building due to insufficient lateral tie into the building.

This section discusses the previous research undertaken in this area in both the sub-assemblage and super-assemblage format. In addition, the test specimen used in both stages of testing is outlined along with the self-equilibrating loading frame system developed by Matthews (2004). In addition, Matthews’ (2004) findings are summarised. Finally, the remainder of this thesis is outlined by section and the significant contributing work is summarised.

### 1.2 Previous sub-assemblage research

Previous research undertaken in the area of hollow-core diaphragm performance has centred on the connection between the hollow-core floor unit and its supporting beam, Mejia-McMaster and Park (1994), Oliver (1998) and Herlihy and Park (2000). All of the research was undertaken to examine the seating requirements of a hollow-core unit during an earthquake when the supporting beam undergoes beam elongation. In all of these tests the hollow-core unit was seated on a supporting beam with various connection details and topping reinforcement. Under loading the hollow-core unit was pulled off the supporting beam by a cyclic horizontal load. Herlihy and Park (2000) describe a further set of tests that comprised a monotonic vertical push down of the hollow-core unit relative to the supporting beam to induce tension in the starter bars and to determine the amount of additional bending moment.
resistance added to the seating connection by the continuity steel. Test layouts for both the pull-off and rotation tests are shown in Figure 1-1.

Figure 1-1 Preceding 2-dimensional rotation and loss-of-support test set-ups

(Herlihy and Park 2000)
The failure of the hollow-core unit to supporting beam connection in the tests by Herlihy and Park (2000) was due purely to tension, as the hollow-core unit was pulled off its seat. Figure 1-2 shows the failure of the Herlihy and Park (2000) test when compared with the damage from an actual earthquake. In none of the previous sub-assemblage research has any program looked at the relative rotation between the hollow-core and supporting beam as the dominant source of the damage to the hollow-core units. Therefore, the failure mechanisms observed in these tests was not consistent and was unrealistic when compared with the failure mechanism observed in the Northridge earthquake.

![Image](image_url)

(b) Herlihy and Park (2000)

Figure 1-2 Failure of hollow-core in the 1994 Northridge earthquake (Norton et al, 1994) and Herlihy and Park (2000) tests.

### 1.3 Previous relevant super-assemblage work

Several researchers have attempted to look at the effect beam elongation plays in the performance of concrete frame buildings, some with and some without floor slabs present (Zerbe and Durrani 1989, 1990, Fenwick and Megget, 1993, Restrepo 1993, Fenwick et al 1995, 1996 and Lau 2002) with varying degrees of success. Only in the last few years have any researchers considered the effect of displacement...
incompatibility between the floor and frame systems. Lau (2002) discussed the vertical movement of the floor relative to the beams and identified that this relative movement could cause failure of the slab connecting the diaphragm to the perimeter beam. He identified the importance of this in cases where stiff precast units are situated close to the perimeter beam. In the PRESSS building tests (Priestley et al, 1999) the connection between the cast in place concrete topping of the hollow-core units and the frames showed only minor signs of damage which indicates a lack of severe displacement incompatibility. However, welded X plates connecting the lower level double-T’s to the perimeter frame showed significant inelastic action and permanent deformation, indicating a reasonable measure of displacement incompatibility.

1.4 Findings from Matthews (2004)

Overall, the performance of the precast, prestressed concrete floor system in the Matthews test was poor while the perimeter moment resisting frame behaved well. There were several questions answered in the testing regime which also allowed several more questions to be developed. Outlined in the following section are some of the critical questions raised from the Matthews testing that will be answered in this experimental program.

1.4.1 Details of the super-assembly

The building is a two-bay by one-bay corner section of a lower storey in a multi-storey precast concrete moment resisting frame building (see Figure 1-3). The new super-assemblage layout with modifications from the Matthews rig is shown in Figure 1-4. The floor units are pretensioned precast hollow-core and are orientated so that they run parallel with the long edge of the super-assembly, past the central
column. The columns are 750mm square at centrelines of 6.1m with an interstorey height of 3.5m. The perimeter beams are 750mm deep and 400mm wide on the longitudinal edge and 475mm at the ends where the hollow-core units are seated. The back tie beam is 400mm wide and 170mm deep. A full set of construction drawings can be found in Matthews (2004).

Figure 1-3 Origin of the super-assemblage (Matthews 2004)

Figure 1-4 Layout and dimensions of the super-assembly
The building was originally constructed as it would be on site in New Zealand, with most of the elements precast, leaving the top half of the beams, the mid-span beam lap splices poured insitu and the 75mm concrete topping on the floor all cast in place. The hollow-core floor units are cast off site using an extrusion machine on a long casting bed and are cut to the required length after the concrete has gone hard.

1.4.2 Load Frames

The key components Matthews (2004) used in the design of the loading frame for the experiment were to ensure that realistic forces were applied that did not promote or restrain beam elongation, that the columns remain essentially parallel as they would in a real building and that the correct deformed shape was allowed to occur. These requirements lead to the development of a self-equilibrating load frame where axial forces cancel within the system and a strong wall or floor is not required to load the super-assembly. The self-equilibrating primary load frame is shown in Figure 1-5.

In addition to the primary loading frame, a secondary load frame (SLF) is used to ensure that the columns remain parallel at all times while maintaining the intended displaced shape. The SLF also allows beam elongation to occur; the initial angle of 20 degrees reduces as the columns move further apart due to the elongation of the beams.

Figure 1-6 shows the SLF in place before and after beam elongation occurs. For longitudinal loading the back columns (north side) in the super-assembly require only one ram each to control the inclination with the centre back column fixed in position. The inclinations of the back columns are matched to that of the front to ensure that no torsion is induced in the transverse beams. A diagram of the load frames in place is shown in Figure 1-7.
(a) Loading points to ensure the system is self-equilibrating

(b) Self-equilibrating load frame in position

(c) Elevation of a set of scissor load frames

Figure 1-5 Details of the self-equilibrating loading frame (Matthews 2004)
After beam elongation occurs
Double acting rollers
Secondary loading frame
Before beam elongation occurs
6100mm

For simplicity, only the secondary frame components have been shown.

Figure 1-6 SLF before and after beam elongation occurs (Matthews 2004).

Actuator E
Actuator D
Actuator B
Actuator A

Plan

Secondary loading frame
Primary Loading frame

Side Elevation

Figure 1-7 Plan and elevations of the load frames for in-plane loading (Matthews 2004)

Front Elevation
1.4.3 Seating details

Initial damage to the seating connection of the hollow-core units to the perimeter frame in the Matthews test occurred at a much earlier stage than anticipated, with the failure mechanism being vastly different from that which was assumed by design. Figure 1-8 shows, by a schematic drawing, that the failure of the units was by the rotation of the hollow-core units relative to the supporting beam rather than the originally assumed sliding mechanism. This failure mechanism compares well with the failures observed in the 1994 Northridge earthquake, as seen by Figure 1-9. It was determined that it was this relative rotation that caused the fracturing of the ends of the brittle hollow-core units causing the seat support for all the units to be lost. Matthews (2004) recommended that to ensure adequate performance of the hollow-core unit it is required that any plasticity due to the relative rotation between the supporting beam and hollow-core unit does not occur within the hollow-core unit but rather at the connection between the hollow-core unit and its supporting beam.

![Assumed versus actual hollow-core to beam end connection mechanism](image)

Figure 1-8 Assumed versus actual hollow-core to beam end connection mechanism

(Matthews 2004)
1.4.4 Displacement Incompatibility

The first hollow-core unit (the one adjacent to the perimeter beam) showed further signs of damage due to the displacement incompatibility between the floor and perimeter beam. Figure 1-10 shows this displacement incompatibility. This was caused by the unit being forced to displace in a double curvature manner, due to the adequate ties along this south beam, when hollow-core units are not designed for such displacement profiles. This mode of displacement caused internal splitting of the webs of the hollow-core units, ultimately leading to the failure and collapse of the bottom half of the units.

Figure 1-10 Displacement Incompatibility between the frame and hollow-core floor units.
This displacement incompatibility caused large stresses in the floor resulting in a fracture of the reinforcing mesh in the cast insitu topping through the joint between the first and second (see Figure 1-4 for designation) hollow-core unit at a relatively low drift of 1.9%. Once this tear formed the central (intermediate) column was not restrained or tied back into the floor diaphragm and therefore, the column translated outwards posing a potential global instability issue for the column.

1.4.5 Analytical Results

For the southern frame it was found that the exterior plastic hinges in the beams (zone of plasticity adjacent to the corner or exterior columns) and interior plastic hinges in the beams (zone of plasticity adjacent to the central or interior column) formed different plastic mechanisms contributing to the frames lateral strength. That being a combination of overstrength moments and contributing continuity steel capacity. It was established that both NZS3101 (1995) and ACI318-02 (2002) very significantly miscalculate the contribution that the floor diaphragm makes to the frames lateral strength capacity.

A theory was developed by Matthews (2004), and verified against the results of other researchers, on beam elongation. The “rainflow counting” method predicts beam elongation using rigid body kinematics allowing far simpler equations to be developed than have been in the past. Knowing the total amount of beam elongation expected in the event of an earthquake is important in determining the required seat length when precast floor units are seated on the beams of the moment resisting frame. Based on this theory, larger seat widths are recommended to allow for irreversible beam growth and construction tolerances.

Matthews (2004) research work provided several answers to questions concerning the performance of hollow-core floors in precast concrete buildings in a
severe earthquake. These included estimating the failure mechanism for the floor units. However, Matthews neglected to actually measure the displacement incompatibility that is identified as one of the primary failure causing mechanisms. In addition to this, the torsion induced in the transverse beams is also a primary indicator of the performance of the ends of the hollow-core units as this influences the relative rotation, which in turn is what is pin-pointed as the actual failure mechanism of the hollow-core units. This torsion, accentuated by the eccentric loading of the hollow-core units, was not measured in Matthews (2004) experiment.

1.5 Current research

Following Matthews’ (2004) work, an industry based Technical Advisory Group on precast floors (TAG) was set-up to assimilate and discuss the results from the hollow-core research program. The TAG consisted of representatives from precast manufacturers, designers and researchers. From the results of the Matthews testing program the TAG recommended two new seating connection details that were expected to perform better, for inclusion in Amendment No. 3 to NZS 3101:1995 (2003). In order for these to be fully accepted by industry designers and builders, proof-of-concept experiments need to be conducted.

Further two-dimensional sub-assemblage testing was undertaken under the sponsorship of Precast NZ Inc. by Bull and Matthews (2003). This testing incorporated the relative rotation as the damage initiating factor, in keeping with Matthews (2004). The test set-up for these series of tests is shown in Figure 1-11. Four connection details were tested, shown in Figure 1-12. Firstly, a 300 series hollow-core unit seated on a mortar bed was tested as a control specimen to determine whether the failure mechanism was similar to that of the Matthews (2004) experiment.
Figure 1-11 Bull and Matthews (2003) test set-up.

(a) Control specimen (300 and 200 series tested) (b) Flexible seating detail

(c) Paperclip seating detail recommended by TAG.

Figure 1-12 Seating details tested by Bull and Matthews (2003).
A 200 series hollow-core unit was tested to determine the difference in performance between the 200 and 300 series units. Finally, two different connection details that were recommended by TAG were tested. The first being a flexible pinned type connection detail that consisted of replacing the plastic dam end plug with a low friction (PTFE or equivalent) bearing strip and 10mm of compressible material at the ends of the units. The second was a more rigid detail where two cores of a 300 series unit were filled with concrete and additional reinforcing in the form of a large paperclip.

The control specimen’s performance closely matched that of the full-scale three-dimensional Matthews (2004) test rig performance proving the validity of the test set-up. Both of the TAG recommended connection details performed well with the flexible connection slightly out-performing the paper-clip connection in the latter stages of testing.

Bull and Matthews (2003) noted some limitations in their research, including no net tension being applied to the units, no plastic hinge forming in the supporting beam, only an earthquake in the longitudinal direction is modelled, the displacement incompatibility issue is not considered and second order effects are not monitored, being just a few of them. All of these limitations can be addressed by testing in a three-dimensional super-assembly.

1.6 Summary of previous research findings

Based on the previous research described herein, the following conclusions may be drawn:

1. Previous researchers have not fully understood the mechanism that causes damage to the ends of hollow-core units. It is the relative rotation of the hollow-
core with respect to the supporting beam that causes a snapping action to the end of the hollow-core units.

2. The importance of testing structures in their three-dimensional format versus simple two-dimensional sub-assemblages is shown by comparing the previous work undertaken.

3. The importance of displacement incompatibility in the performance of hollow-core floors in precast prestressed moment resisting frames was shown. This displacement incompatibility was caused by the units being forced to displace in a double curvature manner due to being effectively connected to the edge of the perimeter beam, when hollow-core units are not designed for such displacement profiles.

1.7 Organisation of this thesis

Following this introductory section, there are two other major sections.

Section 2 discusses the repair undertaken on Matthews’ (2004) full-scale super-assembly as well as the new connection details used in this stage of testing. Secondly, the loading protocol used and instrumentation are discussed. Finally, the experimental and instrumental observations are outlined and conclusions are drawn.

Section 3 investigates the theoretical pushover curve developed for predicting the capacity of the super-assemblage. Theory previously developed by Matthews (2004) is used to predict the beam elongation of the super-assembly in all three loading phases (longitudinal, transverse and longitudinal re-loading). The contrasts between Matthews (2004) and this experiment, with respect to the seating details, lateral connection of the hollow-core to the perimeter frame and the overall system failure mechanism are discussed. A fragility analysis is conducted to compare the overall performance of Matthews (2004) and this experiment.
Effort has been made to keep strictly new material in the main body of the thesis. Considerable supplementary material is also given in the various appendices as follows:

- Appendix A incorporates repair photos from the reconstruction of the test specimen along with additional design calculations for the repair.
- Appendix B includes material properties for the test specimen measured prior to testing along with a full test account and photographic log.
- Appendix C has additional test data and graphs in addition to the calculations completed for the various theories used throughout the thesis.

1.8 What then is particularly new and significant in this thesis?

The major experiment described herein provides in itself valuable data and insights into the behaviour of precast hollow-core floor units in reinforced concrete moment resisting frames. The new connection details that are tested incorporate recommendations included in the 2003 draft amendment to the New Zealand concrete design standard (NZS3101, 2003). Previously these details were untested in a three-dimensional, full scale structure.

A theoretical pushover curve is developed that accurately predicted the superassemblages base shear capacity based on activated slab reinforcing and yield line theory and simplified design recommendations have been made for use of this theory in design.

Quantification of the displacement incompatibility between hollow-core floor and reinforced concrete frame systems is made. Instrumentation is placed to measure
this relative displacement and a greater understanding of displacement incompatibility is accomplished.

This research program also investigates whether it is possible to repair and restore, to good condition, a New Zealand designed and constructed precast concrete moment resisting frame that has survived a design basis earthquake (or worse). The results show that such a repair is possible, with very few repercussions that alter the new structural performance from the original performance.

Based on the test results, significant recommendations are made for improved detailing for all new construction of buildings of this type in New Zealand.

1.9 References

ACI 318-02, 2002, Building Code Requirements for Structural Concrete and Commentary, American Concrete Institute, Farmington Hills, USA.


Section 2

Experimental Investigation

2.1 Introduction

Code requirements for the seismic design of buildings nowadays require that life safety be assured and that collapse in a large earthquake does not occur. But, as a result of structural collapse and severe damage observed in the 1994 Northridge (U.S.A.) earthquake, the poor performance of steel and other buildings has been brought into question. Of these, precast buildings and buildings with precast hollow-core flooring systems did not perform particularly well. Due to the similarity of concrete construction in New Zealand and California it has become unclear as to the vulnerability of precast construction in New Zealand. Multi-phase research programmes were initiated at the Universities of Canterbury and Auckland to determine whether New Zealand designed and built precast structures have problems similar to those observed in the Northridge earthquake.

The research described herein follows Matthews (2004). His previously damaged full-scale super-assemblage specimen has been repaired and reconstructed with a new precast floor and retested. As shown in Figure 2-1, the super-assembly test specimen represents a two bay by one bay lower storey portion of a typical precast concrete building built in New Zealand from the 1980s. The overall dimensions of the specimen are approximately 12m by 6m with 750mm square columns and 750mm deep beams. The flooring system consists of 300mm deep precast prestressed hollow-core units with a 75mm cast insitu topping, spanning 12m, past the central column.
Based on findings from previous research (Matthews, 2004) several key issues were raised that were expected to improve the performance of the hollow-core seat and overall floor performance. These included: seating the hollow-core units on a low-friction bearing strip with a compressible backing replacing the plastic bungs in the end of the unit. This would allow the units to slide rather than snap off at the end under relative rotation between the supporting beam and floor. It would also reduce the compression forces applied to the bottom of the unit under a negative moment at the beam to floor connection. In addition to that, the recommended seat length was increased and it was further recommended that ductile reinforcing mesh replace the standard cold-drawn wire reinforcing mesh to increase the performance of the floor diaphragm.

This chapter firstly discusses the repair undertaken on the full-scale super-assemblage as well as the new connection details used in this second stage of testing. Secondly, the loading protocol used and instrumentation are discussed. The experimental and instrumental observations are outlined and finally conclusions are drawn from these observations.
2.2 Building repair details

At the end of the Stage 1 tests conducted by Matthews (2004) the super-assemblage’s floor failed and completely collapsed. However, though the frame was extensively cracked, it was relatively undamaged compared with the complete collapse of the flooring system. It was decided that instead of building a whole new specimen the existing frame could be repaired and re-used along with a new floor system. The remaining floor sections were removed, the transverse (east and west) beams were propped and the concrete removed. The damaged plastic hinge zones in the main longitudinal (South) beam were removed. Figure 2-2 shows a photograph of the repair in progress. A more complete series of repair photos are given in Appendix A. As the existing reinforcing bars were being re-used, the area where instrumentation stubs had been welded in place to the bars and where yielding of the bars occurred were heat treated to return the bars near to the original properties. This was done by heating the steel to a temperature around the critical transition point (~750-850°C) and allowing it to cool (Oberg et al., 2000). This process was used to restore ductility and reduce internal stresses caused by the prior plastic straining of the bars.

2.3 Support of Hollow-core Units

2.3.1 End seat details

The damage at the seats of the hollow core units in the Matthews (2004) experiment was associated with the rotation of the floor relative to the supporting beams. This led to a brittle snapping action and subsequent failure rather than the units sliding on the seat as originally intended by designers. From this it can be deduced that the damage is more a function of inter-storey drift rather than the
ductility of the beams next to the hollow-core units as was once the generally held opinion of designers.

Normally, hollow-core units are designed to act as a simply supported one-way floor system. In the Matthews (2004) experiment the first hollow-core unit adjacent to the perimeter beam (refer Figure 2-1 for layout) was sufficiently tied to the frame by starter bars and a concrete key that it was forced to deform in a quasi-two way manner with a displaced shape of the perimeter beam. The webs of the unit became overstressed and split as the hollow-core unit was forced into double curvature.

Hollow-core seats may be defined as rigid, semi-rigid, or simple (pinned) connections. Rigid connections include some form of reinforced concrete filled cells, semi-rigid connections are of the type tested similar to those of Matthews (2004) where the units are seated on mortar beds and are held with starter bars. They mostly inhibit rotations until failure is initiated. Simple connections attempt to transfer no end
moments. This investigation has used a simple end connection where the seating between the hollow-core unit and the supporting beam consists of replacing the plastic dam (plug) with a compressible material across the end of the unit. Units should be seated on low friction bearing strip. This detail is shown in Figure 2-3 along with its expected improved performance mechanism.

![Diagram of seating detail](image)

(a) Assumed to slide by design  
(b) Observed behaviour  
(c) Recommended detail  
(d) Expected performance

Figure 2-3 Matthews (2004) seating detail with actual performance and new seating detail and expected improved performance.

The reason for using the compressible material is to reduce the compression forces applied at the bottom or top of the unit under negative or positive moments respectively. If this large compression force forms in the bottom of the unit a shallow compression strut transfers this force through the unit at a relatively flat angle to the topping concrete. This compression strut has perpendicular principal tensile stresses that are not restrained by the hollow-core due to the lack of stirrups, as it would be in
an ordinary reinforced concrete beam. The prestressing strand in the hollow-core unit has not developed enough stress through bond in the short distance away from the supports to provide any supplementary shear resistance. This lack of shear capacity may cause splitting of the webs at small relative rotations. The required thickness of the compressible material is at least 3\% of the depth of the hollow-core unit, as described in NZS3101 (2003). Matthews (2004) recommended a thickness, \( t \), of:

\[
t = D_{HCT} \theta_{\text{max}}
\]  

(2-1)

in which \( D_{HCT} = \) the depth of the hollow-core unit plus topping (mm) and \( \theta_{\text{max}} = \) max expected rotation for a maximum considered earthquake, in this case taken as the interstorey drift of the structure. This is because the relative rotation between the beam and hollow-core unit is not quantifiable due to torsion affects and therefore the more conservative interstorey drift is used.

The low friction bearing strip allows the soffit of the floor unit to slide on its seat, relieving much of the moment that might arise from friction. This minimises the propensity for the build up of large diagonal tensile stresses in the hollow-core web.

Based on Matthews (2004) theory for the prediction of beam elongation, a formula for the required seat length, \( U_r \), to ensure the integrity of the hollow-core seat was recommended.

\[
U_r = \frac{\omega}{2} n e_{\text{c}} \left( \left| \theta_r \right| + \left| \theta_y \right| \right) \frac{L}{L_b} + 15 \text{mm} + \text{cover}
\]  

(2-2)

where \( \omega = \) a magnification factor or factor of safety (a value of 1.5 was suggested), \( n = \) number of hinges within the span of the floor section under consideration,
\( e_{cr} \) = average beam depth between the beam centreline and centroid of the compression force, \( \theta_p^+ \) = maximum positive rotation imposed on the beam hinges, \( \theta_p^- \) = maximum negative plastic rotation imposed on the beam hinges and \( \theta_y \) = yield rotation of the structure. The 15\text{mm} addition to the seat length is to allow for construction tolerances. The additional ‘cover’ is defined here as the required seat length after the beams have undergone plastic deformation. In this case the cover is equal to the width of bearing strip and the edge distance to the front of the beam from the bearing strip (15mm). Had (2-2) been strictly followed (with \( \omega = 1.5 \)) then a much larger seat length width would be required (120mm) than that used (75mm). However, due to the reluctance of designers to specify large seat widths in addition to the fact that the seat length for this experiment was adopted prior to this beam elongation work by Matthews (2004) and by accepting a smaller value of say \( \omega = 1.0 \) as a better test of the theory, a seat length of 75mm, as specified in the amendment to the New Zealand concrete standard (NZS3101, 2003) was adopted. The units were seated on a low friction bearing strip placed 15mm back from the face of the beam. Figure 2-4 shows the hollow-core unit in place with the backing board and compressible material.

![Figure 2-4 Hollow-core units with compressible backing board and low friction bearing strip in place.](image)
2.3.2 Lateral connection of the first hollow-core unit to the frame

To ensure one-way hollow-core action, as recommended from Matthews (2004) findings, it is necessary to separate the gravity load action of the floor from the lateral load resisting action of the adjacent perimeter frame. Therefore in this research, the connection between the first hollow-core unit and the adjacent perimeter beam consisted of moving the first unit away from the perimeter beam. The unit is replaced with a 750mm timber infill with 75mm insitu concrete topping, as shown in Figure 2-5. The thin infill slab allows a more flexible interface between the frame and first hollow-core unit and accommodates the expected displacement incompatibility between the floor and frame, as shown in Figure 2-6. Some cracking is to be expected in the thin slab due to the displacement incompatibility.

![Figure 2-5 First hollow-core unit to perimeter frame connection (Matthews 2004).](image1)

![Figure 2-6 Displacement incompatibility between the frame and floor system](image2)
2.3.3 Diaphragm reinforcement

Another finding in Matthews (2004) work is that it is essential that brittle reinforcement is not provided when ductile performance of the slab is required. Therefore, ductile reinforcing mesh was used in the topping concrete to improve the deformation capability of the topping.

In Matthews (2004) experiment, the cold drawn wire mesh in the topping concrete ‘unzipped’ through lack of inherent ductility while providing restraint to the central column (which attempts to move laterally, perpendicular to the southern perimeter frame). As a result the central column translated outward from the building. To remedy this NZS3101 (1995) stipulates “the columns of the frames shall be tied at each level of the floor system.” Also required is “the tie reinforcement shall be effectively anchored perpendicular to the frame and capable of resisting in tension the larger of 5% of the maximum total axial compression load on the column or 20% of the column shear force.” Due to these requirements, which seem to be largely ignored in the design practice, the central column of the perimeter frame requires to be tied back to the floor diaphragm.

By assuming an axial load of $0.2f'_cA_g$ in the column, it was found that two-YD20 ($f_y=500\text{MPa}$) drag bars were required. These were anchored over three hollow-core units to enable sufficient force transfer via an assumed 45 degree truss in the floor plate, with the final result being shown in Figure 2-7(a). To place the drag bars, 400mm deep holes were drilled into the existing concrete column, shown in Figure 2-7(b). The holes were drilled at an angle of 51 degrees to the horizontal in order to miss any reinforcing in the joint. A proprietary epoxy based post-installed reinforcing anchor system was used (Hilti HIT-RE500).
2.4 Material properties

The deformed reinforcing steel used had measured yield strengths of 306MPa for the beam bars, 430MPa and 480MPa for the HD24 and HD28 column bars respectively. Ready-mixed concrete had a specified strength of 30MPa with 19mm aggregate and a slump of 100mm. At the time of testing the concrete had the following measured strengths: beams 42MPa; Columns 48MPa and floor topping 38MPa. The stress-strain plots for the reinforcing bars and the compressive strength test results for the concrete are given in Appendix C.
2.5 Testing set-up

The load frames were attached to the building in two different set-ups, this depended on whether testing was undertaken in the longitudinal or transverse directions. The load frame set-up for the longitudinal loading is shown in Figure 2-8(a). The load frames were attached to the front of the super-assembly (South side) with one actuator in each pair being displacement controlled and the other force controlled. The back actuators (North side) were displacement controlled in order to keep the inclination of the back columns equal to that of the front so as not to induce torsion into the transverse beams.

Since buildings are rarely hit by an earthquake in one principal direction alone the load frames were developed to be able to attach to the transverse building frames (East and West), as shown in Figure 2-8(b). In order to load the central column in this direction another load frame was developed; see Figure 2-8(c). The column was loaded such that the applied shear forces top and bottom were equal and the inclination could be controlled to keep it the same as that of the other front columns to ensure no torsion was introduced into the longitudinal (South) beams.

2.5.1 Instrumentation

The instrumentation used in this experiment was extensive as many channels required recording at each load increment. An in-house data acquisition system was used to record data from some 200 different instruments while another computer actively controlled the application of actuator displacements and loads.

Load cells within the actuators as well as full bridge strain gauges within the secondary load frames were used to monitor load for both the controller and logger.
System is drift controlled (focusing on the central column)

(a) Hydraulic actuator locations for in-plane (Phase I and III) testing

(b) West end elevation showing actuator locations for out of plane (Phase II) testing

(c) Central column load frame and actuator locations for out of plane (Phase II) testing

Figure 2-8 Actuator locations for the three loading phases. Adapted from Matthews (2004)
side of the experiment. By strain gauging the secondary frames, this enabled the redistribution of the loads within the system to be monitored and meant that any degradation of a plastic hinge could be monitored. The universal joints at the base of each column were also stain gauged to act as a loadcell to monitor any change in axial load within the system.

Figure 2-9 shows the locations of the some 200 instruments measuring the various displacements and inclinations of the test specimen. These instruments included: rotary potentiometers, various sized linear potentiometers, inclinometers and displacement transducers. In addition to these, a grid of “Demec” points was set out over the floor diaphragm to measure changes in length and therefore strains in the diaphragm.

2.5.2 Loading protocol

A ductility based loading protocol has been used extensively at the University of Canterbury since the 1980s. However, Matthews (2004) developed a new loading sequence based on time history analyses because it was felt that when testing existing practise and construction details the loading should be representative of the type and intensity of loading that a real building would be expected to experience.

When testing a new building or new design, as is the case in this research, a conservative assessment of displacement capacity is considered to be more appropriate. Figure 2-10 shows details of the adopted loading protocol. In sum, the phases of experimentation consist of: (i) longitudinal loading – up to 2.0% drift; (ii) transverse loading – up to 3.0% drift; and (iii) longitudinal re-loading – from 2.0% to 5.0% drift.
(a) Location of transducers and rotary potentiometers to measure the inclination of the columns.

(b) Location of beam and beam-column joint pots.

(c) Enlargement of beam and beam-column joint section.

(d) Diaphragm instrumentation (underside)

(e) Pot gauges at ends of hollow-core units

(f) Pot gauges along timber infill (16 sets of).

Figure 2-9 Instrumentation used on the test specimen.
2.6 Experimental Observations

2.6.1 Phase I: Longitudinal Loading

Figure 2-10(a) presents the displacement (drift) path adopted for Phase I of the experiment. The experimental yield drift was determined to be 0.5% from linear approximation of the base shear-interstorey drift hysteresis loop. Figure 2-11 presents some key photographs at the end of this phase (2.0% drift). The drift notation for this phase of loading being that if the top of the columns in the southern frame move east and the bottom west, this is a positive drift. Figure 2-11(a) and (b) show that there was 10mm of hollow-core pull-off in the ±2.0% cycle with the low-friction bearing strip sliding out in some places instead of the unit sliding on the bearing strip. Some spalling occurred in the later cycles of this phase on the seat of the first unit due to the unit bearing on the unreinforced cover concrete. Diagonal cracks in the infill appeared at +1.0% and extended in the second ±1.0% cycle to reach the infill/hollow-core interface and running along the interface for almost the entire length of the floor, except around the central column where the drag bars appeared to tie the infill and floor together. By the end of Phase I, this interface crack grew to 2mm wide with a vertical displacement of 2mm, in the west end (Figure 2-11(c)). A crack in the south-
western corner of the first unit (refer Figure 2-1 for layout) developed at +2.0% and extended into the second core of the unit. The economic consequences to an owner of a building with damage like this may become an issue. However, the cracks are considered to be repairable with the only permanent damage being the residual interstorey drift of the building (approximately 0.8% drift).

(a) Corner crack in first hollow-core unit at +2.0%  
(b) Bearing strip sliding out at +2.0%  
(c) Crack at hollow-core/infill interface at ±2.0%

Figure 2-11 Damage to super-assemblage after Phase I loading.
2.6.2 Phase II: Transverse Loading

Under Phase II loading (refer to Figure 2-10(b)), key behaviour photographs are shown in Figure 2-12. The drift notation for this phase of loading is that if the top of the columns in the east and west frames move north and the bottom south, this is a positive drift. Little new cracking occurred in the early stages of Phase II because the transverse beams were already cracked from the longitudinal (Phase I) loading—these cracks simply opened more widely. During the ±1.0% cycle a crack (2mm at this stage, opening to 6mm at ±2.0%) opened up in the ends of both of the transverse beams about 1.0m from the column face as indicated in Figure 2-12(a). From the cracking profile it was evident that there was a positive moment plastic hinge that had formed at this location at both ends of the transverse beams. It had been expected that the positive hinge would have formed at the face of the column, rather than 1.0m away. Figure 2-12(b) shows a corner crack that formed at -1.0% in the north side corner of the fourth hollow-core unit at both ends. This crack progressed up from the bottom of the unit to run along the web as testing continued.

(a) Large crack forming in transverse beam, 1.0m from column face at +1.0%.
(b) Corner crack in north side of fourth hollow-core unit developing into a web-split at +1.0%.

Figure 2-12 Significant damage in Phase II, the transverse loading cycle
2.6.3 Phase III: Longitudinal Re-Loading

Figure 2-13 presents a photographic log of damage during Phase III of the experiment. Early in Phase III, beam spalling at the west end of the first hollow-core unit left the unit with some 75 percent of its length with at least 20mm of seat spalled off, as shown in Figure 2-13(a) and (b). At +2.25% drift the first crosswire of mesh fractured at the hollow-core/infill interface about 1.5m west of the central column. This first fracture was followed by nine other wires fracturing between 2.25% and 3.0% drift. Once the mesh had fractured it could be seen that the fracture was due to two mechanisms. Firstly, the tear was due to the floor diaphragm restraining the frame from elongating causing a transverse tension force as the beam tries to translate outwards instead; this produced a horizontal east-west dislocation between the infill and topping of the first hollow-core unit (15mm) once the mesh fractured (Figure 2-13(d)) as well as accentuating the transverse north-south displacement (i.e. crack width, 10mm) (Figure 2-13(c)). Secondly, the tear was due to the displacement incompatibility. This caused a maximum vertical offset of 10mm once the mesh had fractured (Figure 2-13(e)). The crack was 3m long at this stage. However, the central column was still adequately tied into the building. The transverse beams showed a significant amount of torsional twist due to degradation of the plastic hinge zones (PHZ)—this was principally due to the hollow-core load eccentricity.

A large section of the unreinforced seat of the fourth hollow-core unit at the west end began to drop away showing the necessity of reinforcing the seat to tie it to the beam. The load carrying capacity of this seat/cover concrete was lost at 3.0% drift (first cycle) and the concrete fell out during the second +4.0% cycle (Figure 2-13(f)). It was during these ±4.0% cycles that the PHZs showed some sign of distress with
(a) Underside of first hollow-core unit, west end, +3.0% drift.

(b) 45-50mm of seat exposed of first unit, west end, +3.0% drift

(c) Crack width of 10mm in places after mesh fractures (+3.0%)

(d) Displacement of 15mm after mesh fractures (+3.0%)

(e) Vertical offset of 10mm after mesh fractures (+3.0%)

(f) Section of unreinforced seat fallen out at +4.0%

(g) Fractured main bar at +3.56% on 2nd cycle to +4%

(h) Floor damage at end of test (5.0% drift).

Figure 2-13 Damage in Phase III testing
large sections of cover concrete completely spalling off. The first main longitudinal beam bar fractured at +3.56% in the second ±4.0% cycle (Figure 2-13(g)) in the South-West PHZ with the remaining beam bars in that PHZ fracturing in the following cycles. A final +5.0% cycle was performed and during this cycle further seat damage was observed along with fracturing of the main bars and topping mesh. A photograph of the infill section of the floor at the end of test is shown in Figure 2-13(h). It was at this stage it was considered that life safety would be a major concern—enough of the hollow-core seat had been damaged to question the stability of the floor diaphragm and nine main bars had fractured in total leading to concern about the stability of the frame elements.

2.7 **Hysteretic performance and system strengths**

Figure 2-14(a) shows the overall behaviour of the super-assemblage for Phase I loading. The hysteresis loops have minor necking arising from opening and closing of cracks in the PHZ. A maximum positive base shear of 1394kN was recorded, while a maximum negative value of -1323kN occurred—both in the first cycle to ±2.0%. It can be seen that in the second cycle of loading little loss in base shear capacity was noted.

The theoretical base shear strength of the super-assemblage, shown in Figure 2-14(a), was calculated from the mechanism capacity using individual hinge strengths from the hinges in both the front and back frames and summed together. The individual hinge capacities are shown and explained in section 3.3, with the calculations given in Appendix C.
Figure 2-14 Hysteresis loops for the three loading phases

(a) Hysteresis loop for Phase I loading

(b) Hysteresis loop for Phase II loading

(c) Hysteresis loop for Phase III loading, including Phase I for comparison.
Figure 2-14(b) shows the hysteresis loop for Phase II loading. Phase I loading appeared to have not adversely affected on the performance of the super-assemblage in Phase II. The hysteresis loops have essentially no necking, and little strength deterioration. The maximum positive base shear was 917kN which occurred in the first cycle to +2.0% drift. The maximum negative base shear was -967kN which occurred in the first cycle to -3.0% drift.

The difference in base shear capacities between the positive and negative base shears is due, in part, to the non-symmetrical reinforcing layout. On the northern side of the super-assemblage the floor is not tied to the tie beam with starters therefore these can not be activated in a positive drift (negative hinge moment) cycle. A crack line also forms at the beam/infill interface through the starter bars. These reasons also account for why the base shear in Phase II is considerably less than Phase I as well as the use of the nominal yield stresses for the positive moment hinges in Phase II due to these hinges forming in steel that was not pre-yielded and subsequently heat treated. The calculated value for the capacity is also plotted on Figure 2-14(b). Full theoretical pushover curves for the individual hinges are analysed in section 3.3, with the calculations presented in Appendix C.

Figure 2-14(c) shows the hysteresis loop for Phase III loading with the same calculated mechanism as Phase I plotted for comparison. The maximum positive base shear is 1322kN while the maximum negative base shear is -1420kN, both occurring during the first cycle to ±4.0% drift. The base shear value at the ±2.0% cycle after, compared with before, Phase II loading was slightly less with a drop of approximately 200kN that could be attributed to damage accrued in the frame in the transverse (Phase II) testing. This drop did not adversely affect the continuing behaviour of the
super-assemblage as the maximum negative base shear occurred in Phase III in the first -4.0% cycle. The fracturing of the main longitudinal bars had a large affect on the base shear capacity of the frame. It is possible, from Figure 2-14(c), to pin-point the fracture of each bar at those locations where there is a sudden drop of load.

Also plotted in Figure 2-14(c) is the theoretical base shear prediction for Phase III. The mechanism is the same as observed in Phase I. The locations where the observed strength exceeds the theoretical capacity is attributed to strain-hardening in the longitudinal reinforcing bars.

### 2.8 Discussion of the efficacy of the construction changes made in this experiment

#### 2.8.1 Low friction bearing strip

Figure 2-11(b) shows the bearing strip sliding out in Phase I by 10mm indicating that the strip slid out from the early stages of testing. It is clear that the low friction bearing strip did not perform in the way it was intended by design. The bearing strip has teeth on one side and is smooth on the other allowing the hollow-core unit to slide on the smooth surface. In this case there was insufficient bond/friction resistance between the toothed surface and the beam seat and in some places the bearing strip slid with the unit instead. Based on manufacturers test data (McDowels Concrete Accessories, 2000) values for the static coefficient of friction of the bearing strip used in the test were determined to be 0.502 and 0.612 for the smooth and toothed side, respectively. These values are quite close and it is therefore not surprising that the bearing strip may slide with the unit instead of the unit sliding on the bearing strip.
The teeth on the underside of the bearing strip are designed to compress and reduce the high points of the beam seat to allow full force transfer. If the compressed teeth do not even out the high points enough the hollow-core unit could bear on two or three points rather than being evenly distributed. This could account for why large sections of cover concrete spalled off.

### 2.8.2 Compressible backing board

The total average pull-off along the seating of the units up to ±4.0% was 15mm in the west end and 16mm in the east. From Figure 2-15 it is also evident that the compressible backing board did not actually compress beyond 0.5mm. The reason that the compressible material does not compress significantly is that prior to yield of the super-assemblage, the rotation of the beam and hollow-core cause only a small compression force as well as the occurrence of elastic beam elongation of the longitudinal beams. However, after yield of the super-assemblage plastic beam elongation of the longitudinal beam has occurred and the backing board will not compress and it is therefore not required. This is due to a net elongation of the perimeter beams as discussed in section 3.5. It is worth noting that a stiff bond-breaker of some sort is required to stop concrete from entering the cores of the hollow-core units.

Figure 2-16 shows the vertical displacements of the hollow-core units relative to the supporting beam during Phase I loading. It can be seen that as the west beam rotates clockwise due to torsion, the weight of the hollow-core unit is taken on the back of the bearing strip and the beam appears to drop relative to the floor—the displacement therefore increases. It can be seen that in the east end, where very little
(a) Southern unit (Unit 1) pull-off during Phase I

(b) West end average pull-off

Figure 2-15 Pull-off of hollow-core units during test.

Figure 2-16 Average vertical Floor Movement during Phase I
damage occurred to the seat or ends of the hollow-core unit, the unit demonstrated a minimal drop below the level of the beam.

2.8.3 Supporting beam

The need to reinforce the seat formed in the beam supporting the hollow-core is evident, as can be seen in Figure 2-13(a) and (f). The weight of the hollow-core unit bearing on the front surface of the concrete seat causes large forces to be transferred through the unreinforced concrete overloading the capacity of the concrete. Reinforcing the seat in the beam with an additional longitudinal bar and stirrup would prevent large sections of the unreinforced seat from spalling off.

A concentration of plasticity in the plastic hinge zones of the supporting beams leads to areas of high stresses and strains. This appeared to be a contributing factor to the failure of the floor in Matthews (2004) experiment. It was also a contributing factor in the corner cracking of the first and fourth hollow-core unit in this experiment. It is therefore recommended that hollow-core units not be seated in the potential plastic hinge zones of the supporting beams. Instead, infill sections should be placed over the highly deformed zones with extra reinforcing placed within the beam to force the hinge zone to occur beneath the infill section only and not under the brittle hollow-core units.

2.8.4 Displacement Incompatibility

Figure 2-17(a) shows the relative displacement incompatibility between the floor and beam at ±2.0% drift during Phase I. It can be seen in the graph that the maximum positive and negative values are not the same, this is because the ‘zero’ value at the central column was not zero, meaning that the floor initially dropped
away from the beam near the start of test. This occurred after the discontinuity crack formed at the end of the hollow-core units, between the supporting beam and the units, at ±0.25% drift.

From the relative displacements the actual displacement profiles can be determined. Figure 2-18 shows the displacements around the central column at +2.0% drift in Phase I with an assumed displacement profile. If the total displacement across the column is summed and divided by the width of the column then a rotation of 0.016 or 1.6% is determined. This is sufficiently close to 2.0% to conclude that if the beams deform in double curvature then the floor remains essentially flat, albeit sagging after the discontinuity crack forms. This concurs with the initially assumed displacement profile of Figure 2-6.

In Phase I the maximum positive incompatibility was 10.6mm (i.e. the floor dropping away from the beam) while the maximum negative (i.e. the beam dropping away from the floor) was 6.2mm. A majority of this deflection occurred at the hollow-core to infill interface with almost twice the amount occurring here compared with that at the infill to beam interface. This is because the starters stopped prior to the hollow-core to infill interface, making it more flexible.

2.8.5 Reinforcing Mesh Fracture

Figure 2-17 shows the difference in relative displacement between the floor and beam before and after the topping mesh fractured. As can be seen, prior to the mesh fracture the maximum relative displacement was 7.3mm but after the mesh fractured in the west end of the floor, between the +2.0% and +3.0% cycles, this displacement jumped to 15.9mm, almost a 9mm increase in relative displacement.
(a) Displacement incompatibility (floor with respect to beam) during Phase I loading

(b) Displacement incompatibility (floor with respect to beam) during Phase III loading

Figure 2-17 Displacement incompatibility of relative floor movement with respect to the beam (positive displacement means that the floor has dropped relative to the beam).

Figure 2-18 Central column displacement incompatibility
Figure 2-19 shows that by examining the cross-section through the floor slab it allows the first zone of weakness to be determined. This zone is determined to be the interface between the infill and first hollow-core unit because this interface is only reinforced with ductile reinforcing mesh. The analysis required to determine why the floor mesh fractured at 2.25% drift is complicated. What is known is that there is a tension force generated in the floor slab by a bowstring effect (further explained in Section 3.2). Along with this, the displacement incompatibility between the frame and floor system adds to the complexity of the problem. To fully model the parameters affecting the mesh fracture a finite element model needs to be developed that incorporates the interaction between the hollow-core units, topping slab and the beam. Further work on this topic is required.

Figure 2-19 Section through the floor and beam showing the first zone of weakness.

2.9 Concluding Remarks

Based on the experimental study presented herein, the following conclusions are drawn.

1. Improved one-way floor action was observed in the testing of the two-bay by one-bay super-assembly, such that drifts up to 5.0% could be sustained without
collapse of the flooring system. However, irreparable damage (that was not life threatening) occurred to the infill slab that tore apart. It is evident that mild steel reinforcing bars should be used in the concrete topping, not welded-wire mesh.

2. The low friction bearing strip allowed the hollow-core unit to slide on the supporting beam. However, it did not perform in an ideal manner. A bearing strip with bigger teeth, to grip the beam better, or bonding the underside of the bearing strip to the beam is required.

3. The compressible material reduced the compression force applied to the beam under negative moments and restricted concrete from entering the cores of the unit. However, the compressible backing board did not actually compress significantly and therefore only a bond breaker/barrier may be necessary to restrict concrete from entering the cores of the units. On the other hand, a compressible backing board may be required in some building geometries, especially buildings with shallow beams where the neutral axis is above the soffit of the hollow-core unit.

4. Providing that the seismic damage is not excessive, it is evident from the experimental results that it is possible to repair New Zealand designed and constructed precast reinforced concrete frames successfully.

2.10 References


McDowels Concrete Accessories, 2000, Data obtained from McDowels Concrete Accessories testing at University of Auckland, New Zealand, April 2000


Section 3

Forensic Analysis of Experimental Observations

3.1 Introduction

This research is the second stage of an ongoing research programme at the University of Canterbury aimed at determining the likely performance of New Zealand designed and built precast concrete moment resisting frame structures with prestressed flooring systems, particularly precast hollow-core floor systems, during an earthquake. Two seating connection details have been tested in the full-scale super-assembly consisting of a two-bay by one-bay precast concrete test specimen built to the current New Zealand Concrete Standard, NZS3101 (1995). Figure 3-1 shows the dimensions and layout of the super-assemblage.

Figure 3-1 Layout and dimensions of the super-assemblage.
This chapter presents a forensic analysis of the major experimental findings. First a theoretical pushover analysis is undertaken to assess the lateral force-deformation capacity of the super-assemblage. Secondly, the “rainflow” analysis methodology proposed by Matthews (2004) is used to predict the beam elongation of the super-assembly in all three phases of loading. The contrast between the observed beam elongation results from both Matthews (2004) and the present study are compared and discussed. Thirdly, a comparison is made between the investigation of Matthews (2004) and this present study with respect to the seating details, lateral connection of the hollow-core to the perimeter frame and the overall system failure mechanism. Finally, a fragility analysis is undertaken to compare the overall performance. General conclusions are drawn regarding the seismic vulnerability of hollow-core floor systems with the old (semi-rigid) details and the proposed (simple, pinned) connection details of the present study. Both of these fragility outcomes are compared with the fragility of the surrounding frames. It will be concluded, that although there is a decided improvement in performance of the simple connection examined herein (compared to Matthews (2004) floor connections), the outcome is not optimal. Therefore suggestions are made for improving performance through enhanced detailing on the critical parts of the structural system.

3.2 Bowstring Effect

As outlined in Section 2.8.5, a bowstring effect is present in the floor slab. This is due to the plastic hinges in the perimeter frame being restrained by the floor slab. The restraint causes the floor slab to go into tension while the beams go into compression. This tension force is transferred to the beam in the form of a diagonal compression strut. For equilibrium, a transverse tension force develops thereby
forming a truss system. This effect has been explained by Fenwick et al (1999), Paulay and Priestley (1992) and is shown by Bull (2003) in Figure 3-2(a-d). In this case, the floor slab acts as the string in the bow with a large compression strut forming in the floor and longitudinal beams, as shown in Figure 3-2(e). A simplified calculation, using the starter bars and hollow-core units seating friction as the components of the tension tie with the mesh in the topping forming the perpendicular strut, can be performed as follows.

\[ Te = \frac{wL^2}{8} \]  

(3-1)

Where \( e \) = midspan eccentricity of the tie force with respect to the centroid of the strut; \( T \) = combined tie force (taken here as \( A_s(1.25f_y)2e/s+\mu WL_e \) where \( A_s \) = starter bar area, \( (1.25f_y) \) = probable yield stress, \( s \) = starter bar spacing, \( m \) = coefficient of friction of the bearing strip, \( W \) = weight of hollow-core units); \( w \) = uniformly distributed load given by the reinforcing mesh at the onset of fracture (taken here as \( A_{wire}f_{wu}/1000/s \) where \( A_{wire} \) = wire area, \( f_{wu} \) = ultimate tensile strength of the mesh and \( s \) = mesh wire pitch in mm); and \( L \) = overall span of arch.

This calculation leads to an eccentricity of approximately 1.7m, or the centre of the bow tie being located near the northern edge of the first hollow-core unit. This simplified calculation is consistent with the observed crack patterns, especially the diagonal cracking observed at the end of the first unit.
3.3 Lateral Capacity of the System

3.3.1 General Strength

A width of activated slab at the exterior plastic hinges in the southern beams, which contributes to the capacities of the individual hinges, can be proposed (Matthews 2004). At the interior plastic hinges a new mechanism is proposed for accounting for the capacities of these hinges, that accommodates 3D displacement incompatibilities. In order to determine the width of slab that contributes to the overall resistance, the performance of the exterior hinges and the amount of progressive activation of the starter bars in the transverse beams needs to be better understood.
Ideally, if the transverse beam is torsionally stiff then the lag in activation of the starter bars between the end of the beam and the centre of the beam is small (Velez and French, 1989). However, if the beam is torsionally weak then the difference in activation is large. This is because as the column rotates it drags the end of the beam with it, if the beam is torsionally weak then it will undergo large amounts of torsion before a wide enough crack forms to cause the starter bars to activate in tension at the centre of the beam, Figure 3-3 shows this pictorially.

To determine the theoretical capacity curve two distinct points need to be determined: firstly, the width of activated flange at first yield of the beams and secondly the width of activated flange at full plasticity of the super-assembly. The first is determined by observing the experimental crack patterns at 0.5% drift (Figure 3-4). It can be seen from the cracks in Phase I (refer loading scheme described in Section 2.5.2) that at the onset of yield the width of flange that should be adopted is the width of the infill (0.75m) because that is the width of cracking and hence deformation at the point of yield. The latter point, the point of full plasticity and activation, it was concluded, from Matthews (2004) work, that 3.05m (or one-half of the bay width in this experiment) was activated at full plastification at a drift of 2.0%.
Figure 3-3 Activation of the starter bars along the transverse beam as the drift increases is affected by the torsional stiffness of the beam (Matthews 2004).

This theory only applies when a transverse beam with continuity steel runs perpendicular to the longitudinal beam. For the interior column, as there is no transverse beam present, this theory does not apply. Based on crack patterns and other measured deformations a new theory is proposed herein.

Figure 3-4 Crack in the topping after the ±0.5% cycles

3.3.2 Local Effects of 3D behaviour: Folded Plate Theory for the infill slab capacity

From observation of the crack patterns in the infill around the central column in Phase I of the experiment, a folded plate mechanism is evident. Figure 3-5(a) and (b) shows these crack patterns with the proposed folded plate mechanism superimposed. Under a positive drift, the beam on the western side of the central column is lifted, (refer Figure 3-5(c) and (d) for the observed displacement between beam and slab) which leads to a folded plate mechanism propagating along the infill.

Yield line theory can be used to calculate the moment contribution of this mechanism to the capacity of the central column hinges by taking into account the angle of crack and equating external and internal work done. The reverse is true under
(a) Folded plate mechanism evident due to the crack patterns in the infill region.

(b) Enlargement of folded plate mechanism, with yield lines labelled.

(c) Displacement incompatibility between frame and hollow-core units.

(d) Experimental displacement incompatibility of floor relative to beam

(e) Demecs crossings the beam/infill and infill/hollow-core interfaces

Figure 3-5 The effects of 3-D displacement incompatibility on global response

3-7
a negative drift. The calculations for this mechanism are given in Appendix C which show that an additional resistance of 30kNm is needed to form this mechanism.

By plotting the strains in the floor at ±2.0% drift, as shown in Figure 3-5(e), it can be seen that the top surface of the infill/hollow-core interface is in tension (north-south sense) when the yield line theory requires it to be in compression for a moment to be generated. This is also the case for the ±1.0% drift cycles. It is clear, from Figure 3-6(a), that this moment is originally generated on the lower tension quadrant of the moment interaction diagram. It is not long after yield of the super-assemblage that the dominate mechanism changes from a flexural yield line mechanism to a tensile membrane system due to the tension induced in the floor by a “bowstring” effect (as discussed in Section 2.8.5. The moment capacity along this interface is lost and the contribution to the overall base shear capacity is reduced as shown by the schematic load path shown in Figure 3-6(b).

As a consequence of the moment capacity vanishing along yield line H,I,J,K,L,M in Figure 3-5(b), the contribution of the folded plate mechanism to the overall moment capacity drops from 30kNm to 24kNm. Because of this small proportion of the total resistance (c.f. 24kNm with 30kNm total at the central column) the folded plate yield line mechanism contributes approximately 1% of the total resistance: therefore it may be ignored in an overstrength analysis.

It is important that designers make a post-earthquake assessment of the ability of the infill portion of the slab to continue to carry load. One simple approach is to use the lower bound Hillerborg strip method. Consequently assuming cantilever action only as illustrated in Figure 3-6(c) then the residual (post-failure) capacity may be
Initially have moment capacity

Probable load path

Moment capacity lost when tensile membrane action takes over

Potential tensile fracture line

Assessed from $w_u = \frac{2M'}{L^2}$ where $L$ = the strip length to the fracture line on the hollow-core to infill interface and $M'$ = negative moment capacity from the starter bars from the main beam. Based on this experiment, back calculating the capacity of the infill slab gives $w_u = 20$ kPa which is sufficient for a wide range of design load combinations.

### 3.3.3 Theoretical Pushover curves

The capacity mechanism used to determine the theoretical push-over curve capacity of Phase I and Phase III were the same. The mechanism for Phase II is
similar, however, in Phase II, relocated hinges in the positive hinge direction increased the beam moments in all cases. Appendix C gives a full explanation of the details and shows the complete calculations, a summary is given here. For Phases I and III, in the exterior hinges, at the initial yield point three starter bars are activated in the tension flange for a negative moment and in the positive moment it is assumed that the compression block is roughly the depth of the flange section. For the interior hinges, the folded plate mechanism outlined in section 3.3.2 adds to the capacity of the hinges around the central column. However, the small moment contribution this makes can be effectively ignored in practise.

At full plastification, in the exterior hinges, half of the bay length of starters in the transverse beam contributes to the front frames capacity with the other half contributing to the back tie beam. For the interior hinges, the folded plate mechanism is taken over by a tension membrane mechanism as the floor is put into more tension and the infill section is unable to sustain the moment couple along the hollow-core/infill interface.

In Phase II, relocated, uni-directional, hinges increased the beam moments in all cases. The reason that the total positive and negative base shear is not equal is due to the non-symmetrical reinforcement layout of the specimen.

**Longitudinal Behaviour: Phases I and III**

Figure 3-7(a) and (c) show the overall base shear hysteresis loops for Phase I and Phase III respectively with the calculations for the theoretical pushover curves given in Appendix C. The figures show satisfactory agreement between the
(a) Experimental Base shear capacity and theoretical pushover curve for Phase I

(b) Experimental Base shear capacity and theoretical pushover curve for Phase II

(c) Experimental Base shear capacity and theoretical pushover curve for Phase III

Figure 3-7 Hysteresis loops for the three phases of loading with the theoretical push-over curve.
experimental and theoretical push-over curves. The difference at the higher drift could be due to increased stresses due to strain hardening in the longitudinal beam bars as they approach ultimate stress. This would increase the capacity by approximately 1.25, to account for strain hardening—based on standard design assumptions. Using this approximation the theoretical capacity would match the experimental results better in the later stages of loading.

**Transverse Behaviour: Phase II**

Figure 3-7(b) shows that the experimental base shear hysteresis loops compare well with the theoretical push-over curve. The theoretical curve was plotted using the same theory assumed in Phases I and III. Good agreement is observed. However, once again the later cycles would be better predicted with the inclusion of strain hardening in the determination of flexural capacity.

### 3.4 Individual hinge strengths

The overall system base shear strengths can be broken down into the individual hinge capacities. This enables each hinge to be looked at and therefore the actual versus theoretical progressive slab reinforcement activation for each hinge to be examined.

#### 3.4.1 Phase I

The individual hinge moment-rotation graphs for Phase I are plotted in Figure 3-8(a) with the calculations for all three phases given in Appendix C. Figure 3-8(a) shows that the experimental capacity for the exterior column hinges agree well with the theoretical push-over curve. At the central column the infill section is already well distorted due to three-dimensional effects and significant strain
(a) Phase I individual hinge strengths.

(b) Phase II individual hinge strengths

Figure 3-8 Individual hinge strengths, Experimental and theoretical push-over

3-13
hardening can be expected in both the infill and longitudinal bars. Therefore, it is not surprising to see an appreciable degree of apparent flexural overstrength in the vicinity of the central column.

3.4.2 Phase II

The individual hinge moment-rotation graphs for Phase II are plotted in Figure 3-8 with full calculations given in Appendix C. Figure 3-8(b) show the theoretical pushover capacity curves for each of the columns in the east and west frame. Full starter activation of the starter bars, contributing to the negative moment in the southern hinges only, along the length of the super-assembly was assumed to occur at 2.0% drift and a shorter distance of activated starters was used at yield.

The small discrepancy in the positive moment of the northern hinges can be attributed mostly to additional actions in the back roller support arrangement. Additional moment could be gained in the positive drift direction because no movement was allowed in the back column in the North-South direction and bearing on the restraining plates could have occurred. Any interaction the column has with the pin mechanism at its base could alter the moment capacity of that column.

3.5 Beam Elongation

A new theory developed for predicting elongation in structural concrete members was presented by Matthews et al (2004). They used a type of rainflow counting method to explain the increase in permanent elongation from plastic flexure. The theory is based on simple rigid body kinematics. Basically, major beam elongation will only take place as the plastic deformation exceeds a previous peak. If
additional cycles to the same or lesser plastic rotation occur elongation should not increase.

From Matthews et al (2004) the following equations were used to determine the beam elongation with a frame/bent. Firstly, the amount that a plastic hinge elongates for a given rotation is expressed by

$$
\delta_i = \theta e_{cr}
$$

(3-2)

where $\delta_i$ = elongation of the $i^{th}$ hinge; $\theta$ = rotation the plastic hinge undergoes; and $e_{cr}$ = eccentricity between the centre of gravity of the concrete (c.g.c) of the beam and the centroid of the compression force (instantaneous centre of rotation, I.C.R). This elongation is shown in Figure 3-9(a) in terms of $\theta$ and $e_{cr}$.

The maximum total elongation of a frame/bent ($\delta_{max}^{el}$) can be determined in terms of interstorey drift by:

$$
\delta_{max}^{el} = \delta_e^{el} + \delta_p^{el} = \left[ |\theta_p^+| + |\theta_p^-| \right] \left| \sum_{i=1}^n e_{cri} \right|
$$

(3-3)

in which $\delta_p^{el}$ = non-recoverable plastic component of beam elongation, $\delta_e^{el}$ = unloading recoverable component of beam elongation, $\theta_p^+$ = maximum positive plastic rotation imposed on the frame/bent, $\theta_p^-$ = maximum negative plastic rotation imposed on the frame/bent, $\theta_y$ = yield rotation of the super-assemblage which is assumed to be the same for each direction of loading.

The values of $\theta_p^+$ and $\theta_p^-$ are determined from the loading history imposed on the structure. Whereas as shown by Figure 3-10, $e_{cri}$ is structure dependant. From
Figure 3-9 it can be seen that the values for $e_{cr}$ change depending on the presence and activation of slab reinforcing as well as the presence of the prestressing strands in the hollow-core units reducing the internal lever arm.

(a) Exterior plastic hinge lever arms for reinforced concrete beam systems

(b) Interior plastic hinge lever arms. Note: Prestress reduces $e_{cr}$

Figure 3-9 Internal lever arms for an interior and exterior joint (Adapted from Matthews 2004)

Figure 3-10 Detailed schematic of beam elongation showing the effect of both the elastic and plastic components (Adapted from Matthews, 2004).
3.5.1 Phase I and III

Figure 3-11 presents the observed and theoretically predicted beam elongation for the total frame/bent resulting from Phase I and Phase III loading respectively. The values of $e_{cr}$ were determined to be 0.425D and 0.475D for a negative and positive exterior hinge respectively and 0.225D and 0.275D for a negative and positive interior hinge respectively (in which $D$ is the overall member depth) from compatibility, equilibrium and a moment-curvature analysis (Figure 3-9). It was assumed that the hollow-core units had a sufficient effect on the performance of the interior hinges to warrant an inclusion of their effects. It can be seen that the observed and predicted results agree well, with the predicted values generally falling between the first and second cycle of the experimental results. In Phase III the beam elongation due to the earlier Phase I and II cycles is added to the experimental elongation and the total average observed elongation at the -4.0% drift amplitude was 77mm. Using (3-3) the predicted maximum elongation in the -4.0% cycle is $\delta_{max}^{el} = 79$mm. The difference between the two is small and not significant.

3.5.2 Phase II

Figure 3-12 shows the total frame beam elongation resulting from phase II loading. By averaging the elongation from the east and west beams the total frame elongation can be determined. Due to the fact that the beams were entirely reconstructed and no pre-existing cracks from Matthews (2004) experiment were present, the elongation formula predicts the elongation very well. The overall predicted beam elongation after the end of phase II was $\delta_{max}^{el} = 30$mm with the experimental data also being 30mm. This shows the good match in experimental versus theoretical data.
Figure 3-11 Total beam elongation graph for Phases I and III
Figure 3-12 Total beam elongation graph for Phase II
3.6 Low cycle fatigue failure of longitudinal reinforcing

Reinforcing bars in plastic hinges can fail due to low cycle fatigue (Mander et al., 1994). Using the low cycle fatigue theory proposed by Dutta and Mander (2001) it can be shown that the plastic rotation fatigue life capacity can be determined from

\[ \theta_p = 0.16 \left( \frac{D}{D'} \right) \left( \frac{L_p}{D} \right) \left( 1 - \frac{1}{2} \frac{L_p}{L} \right)^{0.5} \left( 2N_f \right)^{0.5} \]  

where \( \theta_p \) = plastic drift; \( L \) = lever arm of the cantilever column; \( D \) = overall member depth; \( D' \) = distance between the outer layers of steel and \( 2N_f \) = number of reversals to the appearance of first fatigue crack where \( N_f \) is the effective number of constant amplitude cycles for a variable amplitude displacement history and \( L_p \) = equivalent plastic hinge length given by

\[ L_p = 0.08L + 4400\varepsilon_yd_b \]  

where \( \varepsilon_y \) = yield strain; and \( d_b \) = diameter of the longitudinal reinforcement.

The equivalent number of cycles can be found using the well-known Miner’s hypothesis:

\[ N_f = \sum \left( \frac{\theta}{\theta_{ref}} \right)^2 \]  

where \( \theta \) = drift for the \( i \)th cycle of loading and \( \theta_{ref} \) = reference drift for an equivalent constant amplitude loading history.

Applying (3-6) on the loading history (Figure 3-13) for a reference \( \theta_{ref} = 4\% \) drift gives an applied cyclic demand of \( N_f = 3.3 \) cycles. Applying this cyclic demand to
leads to a plastic rotation drift capacity of $\theta_p = 0.032$ radians. Then adding the elastic drift of $\theta_e = 0.01$ radians, this gives a required capacity of $\theta_u = \theta_p + \theta_e = 0.042$ radians. This compares well with the maximum observed drift that the longitudinal beam bars were able to withstand before fracture, $\theta_{\text{max}} = 0.04$ radians.

Fatigue is not a particularly exact science and solutions within $\pm 20\%$ are considered very satisfactory. It should be noted that low cycle fatigue is a function of steel fatigue-life and system geometry alone. The failure of the bars is therefore the defining failure point of the structure because it is not a parameter that can be altered without changing the mechanical properties of the reinforcing steel or altering the drift magnitudes felt by the structure.

![Figure 3-13 Loading cycles imposed in the two stages of testing](image)

(a) Matthews (2004) loading

(b) Loading protocol used in this experiment

Figure 3-13: Loading cycles imposed in the two stages of testing
3.7 Comparison between Matthews (2004) and this experiment

The first major difference between the two stages of testing was the loading cycles imposed; the two loading protocols are shown in Figure 3-13 with the reasons for choosing a different loading regime outlined in Section 2.5.2. The significant detailing changes made between the two stages of testing are outlined in Figure 3-14. A detailed study of the failure mechanisms for Matthews study is shown in Matthews (2004). The changes to the super-assembly were brought about by the poor performance of these details during Matthews (2004) experiment. Modifications were made to improve the performance of the hollow-core floor system in both the lateral and end seating details as well as more adequately tying the central column into the floor system.

3.7.1 Seating connection

In Matthews (2004) experiment the initial crack, at the underside of the hollow-core unit at the junction between the unit and the beam seat, was visible at 0.35% drift. This crack propagated at a relatively flat angle to the topping concrete due to the principal tensile force, at an early stage. This mechanism was activated because the unit was seated on a mortar bed and was unable to slide. In this test, this mechanism did not appear because the unit was able to slide on the low friction bearing strip. The low friction bearing strip allowed the deformation, and relative rotation, to occur at the end of the hollow-core unit rather than being forced into the body of the hollow-core unit in front of the beam face.

During Matthews (2004) experiment the units began to drop down the beam at an early stage, with 10-12mm drop at the end of Phase I loading, whereas during this
Figure 3-14: Differences between the two stages of testing.
experiment the units did not drop significantly down the beam at any stage during the test.

### 3.7.2 Lateral connection of first hollow-core unit to the perimeter frame.

A hollow-core floor unit is designed to act as a one-way floor slab, by tying the unit along its longitudinal edge to the perimeter frame the unit is forced to act in a quasi two-way manner, leading to displacements and forces induced in the diaphragm that the units are not designed for. Since a hollow-core unit has no form of redundancy, due to the lack of transverse reinforcement and load sharing mechanisms, the unit then splits along the web and the bottom half of the unit is not tied to the structure. The seating crack has already developed, as explained in section 0, and the bottom half of the unit drops away.

In Matthews (2004) experiment, web splitting of the first hollow-core unit was noticed late in Phase I of the testing and failure of the unit occurred early in Phase III. In this stage of testing, no significant web splitting occurred.

The mesh fracture initiation point in both Matthews (2004) and this investigation occurred at approximately 1-1.5m west of the central column. Photos of the two fracture planes can be seen in Figure 3-15. By examining the cross-section through the floor slab it allows the first zone of weakness to be determined and identified, as was determined in Matthews (2004) experiment as shown in Figure 3-16. In this experiment, this zone is determined to be the interface between the infill and first hollow-core unit whereas in Matthews (2004) it was determined to be the interface between the first and second hollow-core unit.
(a) Mesh fracture in Matthews (2004) (entire floor)  (b) Mesh fracture in this experiment (west end)

Figure 3-15 Mesh failure plane in the two stages of the experiment.

(a) Zone of weakness in Matthews (2004) experiment

(b) Zone of weakness in this experiment

Figure 3-16 First zone of weakness in the floor diaphragm from the southern perimeter beam in the two testing stages

3-25
It is worth noting that, in this test, although the mesh fractured at an interstorey drift of 0.35% above that when it fractured in the Matthews test (1.9% versus 2.25%), at that time the ductile mesh had undergone more than six times the plastic rotation than the mesh in Matthews (2004) experiment. This means that although the mesh fractured by a similar mechanism in this test its performance was definitely better than the mesh used in Matthews (2004) experiment.

### 3.8 Fragility Analysis

Hysteresis loops are beneficial in determining the overall capacities of test super-assemblies but their usefulness is limited when looking at the performance of individual elements. As can be seen by the graphs in Figure 3-17, the overall comparison of the two super-assemblies would be that similar overall base shears were observed, Matthews (2004) super-assembly was slightly stiffer than this experiment and the present super-assembly was loaded to higher drifts and therefore underwent more plastic deformation.

![Hysteresis loops for comparison of performance.](image)

(a) Matthews (2004) hysteresis loop for Phase I and Phase III

(b) Hysteresis loop for this experiment for Phase I and Phase III

Figure 3-17. Hysteresis loops for comparison of performance.
What is not apparent is that the overall system hysteresis loops are dominated by the performance of the perimeter concrete frame, and in Matthews (2004) experiment, the overall performance of the super-assembly was vastly inferior due to premature failure of the hollow-core flooring system. In Matthews (2004), there was little evidence in the hysteresis loops to indicate the poor performance of the hollow-core floor system. Therefore, assessment of the frame performance alone is not satisfactory in determining the damage state and consideration of the performance of all of the elements in the system needs to be undertaken.

An investigation has been undertaken by Matthews (2004) that determined the expected interstorey drift demand on the class of structure tested in this programme. The findings were, in terms of the expected (median) drift,

\[ \tilde{D}_D = 2.0(F_vS_{1})_D \]  
(3-7a)

or

\[ \tilde{D}_D = 2.0(PGA)_D \]  
(3-7b)

in which \(\tilde{D}_D\) = the median (50\(^{th}\) percentile) drift demand as a percentage of the storey height, \((F_vS_{1})_D\) = one second spectral acceleration for tall structures (above four stories) and \((PGA)_D\) = peak ground acceleration for low rise structures (up to four stories). From (3-7a) it follows;

\[ (F_vS_{1})_C = 0.5\tilde{D}_C \]  
(3-8a)

or

\[ (PGA)_C = 0.5\tilde{D}_C \]  
(3-8b)

where \(\tilde{D}_C\) = expected drift capacity of the structure.
Analysis by Matthews (2004) showed that the distribution of drift outcomes is lognormal with a coefficient of variation of $\beta_D = 0.52$. When combining distributions, to give an overall composite distribution, Kennedy et al (1980) showed that by using the central limit theorem the coefficient of variation for a lognormal distribution can be found from:

$$\beta_{C/D} = \sqrt{\beta_C^2 + \beta_D^2 + \beta_U^2}$$

(3-9)

where $\beta_C = \text{coefficient of variation of the capacity, taken herein as } \beta_C = 0.2$ (Dutta, 1999); and $\beta_U = \text{dispersion parameter to account for modelling uncertainty, taken here as } \beta_U = 0.2$.

Applying (3-9) gives $\beta_{C/D} = 0.60$. By using a lognormal cumulative distribution that can be described by a lognormal variate $\xi_\beta$ (where the median = 1 and the lognormal coefficient of variation, $\beta_{C/D} = 0.60$), the distribution of ground motion demands needed to produce a given state of damage can be found by:

$$F_\xi S_1 = 0.5\tilde{D}_\xi (DS)\xi_\beta$$

(3-10)

or

$$PGA = 0.5\tilde{D}_\xi (DS)\xi_\beta$$

(3-11)

where $\tilde{D}_\xi (DS) = \text{the expected value (in this case, the experimentally observed drift) for a given damage state (DS).}$

The state of damage after an earthquake may be assigned by inspection engineers using a colour-coded format, as shown in Table 3-1. Based on the experimental observations made in this research, expected values of the response...
parameters for different post-earthquake states of damage in terms of coloured tagging is presented in Table 3-2.

Figure 3-18 shows the fragility curves for the floor and precast frame performance when classified under the colour-coded scheme. For comparative purposes, similar curves for the floor performance in Matthews (2004) experiment are also given. On each of the graphs the 10% in 50 years, Design Basis Earthquake (DBE), $F_{S1} = 0.40g$ for Wellington, New Zealand is shown, as well as the 2% in 50 years, Maximum Considered Earthquake (MCE), $F_{S1} = 0.72g$, for Wellington, New Zealand. If the structure investigated here is classified in terms of the critical element (floor or frame) then it can be seen that under a MCE only 23% would be expected to sustain damage such that the building could not be entered (Figure 3-18(a)). This performance is dictated by the performance of the floor. Under a DBE, 65% would sustain no damage allowing immediate occupancy and 29% would sustain moderate damage to the floors while only 5% of floors would be red tagged. No buildings would collapse from inferior floor or frame performance (Figure 3-18(a) and (c)). If the floor performance is compared to that of Matthews (2004), it can be seen that for Matthews (2004) experiment, under the same DBE, that essentially all buildings would sustain some form of damage to the floors whether it is moderate (5%), heavy (60%), near collapse (27%) or total collapse (8%) (Figure 3-18(b)). The superior performance of the floors in this experiment over the performance in Matthews (2004) is clearly seen by comparing Figure 3-18(a) and (b).
Table 3-1. Definition of colour coding used to classify building damage following an earthquake.

<table>
<thead>
<tr>
<th>Tag Colour</th>
<th>Description of damage level</th>
</tr>
</thead>
<tbody>
<tr>
<td>Green</td>
<td>No Damage, building occupiable</td>
</tr>
<tr>
<td>Yellow</td>
<td>Moderate levels of damage. Building can be entered to remove belongings.</td>
</tr>
<tr>
<td>Orange</td>
<td>Heavy damage. Building can be entered for brief periods to remove essential items only</td>
</tr>
<tr>
<td>Red</td>
<td>Near collapse. Building can not be entered</td>
</tr>
</tbody>
</table>

Table 3-2 Colour coding classification for the super-assemblage

<table>
<thead>
<tr>
<th>Tag Colour</th>
<th>Floor (Interstorey Drift)</th>
<th>$F_V S_1$ or PGA for Floor $(g)$</th>
<th>Frame (Interstorey Drift)</th>
<th>$F_V S_1$ or PGA for Frame $(g)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Green</td>
<td>1.0%</td>
<td>0.5</td>
<td>1.0%</td>
<td>0.5</td>
</tr>
<tr>
<td>Yellow</td>
<td>2.0%</td>
<td>1.0</td>
<td>2.0%</td>
<td>1.0</td>
</tr>
<tr>
<td>Orange</td>
<td>2.25%</td>
<td>1.13</td>
<td>3.0%</td>
<td>1.5</td>
</tr>
<tr>
<td>Red</td>
<td>4.0%</td>
<td>2.0</td>
<td>4.0%</td>
<td>2.0</td>
</tr>
</tbody>
</table>
Figure 3-18 Fragility curves for the different classes of floor details compared with the expectations of a capacity designed frame, under the colour coded damage index format.
The damage is also described in terms of numerical damage states in Table 3-3 and building drift classifications for the damage states given in Table 3-4. If the damage is classified in terms of different damage states, then in the current experiment under a MCE, 71% of the buildings would sustain repairable damage to the frame or floor, of the remaining 29% irreparable damage, 23% of floors and 5% of frames would sustain major damage (Figure 3-19(a) and (c)). These figures also show that under a DBE, 94% of buildings would sustain repairable damage to the floor and frame with 5% of the remaining 6% of floors sustaining heavy irreparable damage while none of the remaining 6% of frames sustains heavy irreparable damage. However, when the performance of the two flooring systems are compared it can be seen that in Matthews (2004) only 2% of structures would be expected to sustain slight or repairable damage under a MCE and under a DBE 92% of structures of this type would sustain irreparable damage, requiring demolition with 8% leading to possible loss of life. This is almost a complete reversal of the damage states identified in this experiment and therefore the analysis shows the improved performance due to the enhanced details.

As can be seen, in this experiment, the performance of both the frame and floor are very similar whereas in Matthews (2004) experiment the performance of the floor is vastly inferior to the performance of the frame and therefore, in Matthews (2004) the overall performance is dictated by the poor performance of the floor. The findings from this experiment adhere to the expectations of ductile structures designed and detailed in accordance with the principles of capacity design as well as meeting the target objective that the confidence interval at the onset of irreparable damage under a DBE exceeds 90%.
Table 3-3. Definition of damage states used to classify building damage following an earthquake (Mander, 2003).

<table>
<thead>
<tr>
<th>Damage State</th>
<th>Description of Damage</th>
<th>Post-earthquake utility of structure</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>None (pre-yield)</td>
<td>Normal</td>
</tr>
<tr>
<td>2</td>
<td>Minor/Slight</td>
<td>Slight Damage</td>
</tr>
<tr>
<td>3</td>
<td>Moderate</td>
<td>Repairable Damage</td>
</tr>
<tr>
<td>4</td>
<td>Major/Extensive</td>
<td>Irreparable Damage</td>
</tr>
<tr>
<td>5</td>
<td>Complete Collapse</td>
<td>Irreparable Damage</td>
</tr>
</tbody>
</table>

Table 3-4 Damage state classification for the super-assemblage.

<table>
<thead>
<tr>
<th>Damage State</th>
<th>Floor (Interstorey Drift)</th>
<th>$F_{V,S_1}$ or PGA for Floor (g)</th>
<th>Frame (Interstorey Drift)</th>
<th>$F_{V,S_1}$ or PGA for Frame (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>1.0%</td>
<td>0.5</td>
<td>1.0%</td>
<td>0.5</td>
</tr>
<tr>
<td>3</td>
<td>2.0%</td>
<td>1.0</td>
<td>2.0%</td>
<td>1.0</td>
</tr>
<tr>
<td>4</td>
<td>2.25%</td>
<td>1.13</td>
<td>4.0%</td>
<td>2.0</td>
</tr>
<tr>
<td>5</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
Figure 3-19 Fragility curves for the different classes of floor details compared with the expectations of a capacity designed frame, under the damage state rating format.
3.9 Conclusions

Based on the results presented herein, the following conclusions may be drawn;

1. Yield line theory can be used to predict the behaviour of infill slabs based on a folded plate mechanism. For the yield line adjacent to the first hollow-core unit the flexural resistance diminishes quickly as tensile membrane evolves due to the “bowstring” effect. The contribution of the slab was calculated to be less than one percent of the overall system strength. A simple strip method is suggested for assessing the residual post-failure capacity of the infill.

2. Beam elongation of the longitudinal beams affects the seat requirements of precast elements, especially elements able to slide on bearing strips. The theory proposed by Matthews (2004) accurately predicts the overall beam elongation of systems with different seat arrangements and can be used by designers to determine the required seat length and expected overall building growth in an earthquake.

3. The failure of the longitudinal reinforcing bars can be predicted by low cycle fatigue theory. This failure mode is a function of steel fatigue-life and system geometry. Its effect can not be easily mitigated, and therefore becomes the defining failure point for the super-assemblage.

4. By using fragility curves to assess individual elements of a system it is possible to determine the implications of the drift damage on New Zealand constructed buildings of this type. The target objective that the confidence interval at the onset of irreparable damage under a Design Basis Earthquake exceeds 90% was easily met by all elements in this experiment. Fragility analysis allows comparisons to be
made between separate elements and drift limits to be placed on different performance levels.

3.10 References


Section 4

Summary, Conclusions and Recommendations

4.1 Summary

This research has followed on from recently completed experimental and analytical work conducted by Matthews (2004) who investigated the seismic performance of precast concrete frame buildings with precast hollow-core floor slabs. Matthews (2004) outlined several areas of future research needs, several of which have been addressed in this research, specifically the use of hollow-core flooring systems with improved details to better cope with displacement incompatibilities induced by earthquake shaking.

The super-assemblage constructed by Matthews (2004) was severely damaged during his experiment; however the precast frames were repaired and reused in this research. New end beams with wider (75mm) seat widths were cast and a simple (pinned) connection between the hollow-core units and the beam adopted. The redeveloped super-assemblage was tested in both the longitudinal and transverse directions up to interstorey drifts of ±4.0%. Experimental and instrumental observations from the experiment were outlined.

A forensic study was conducted to illuminate understanding of the observed failure modes. A theoretical pushover curve for predicting the capacity of the super-assemblage was developed using the principles of folded plates, yield line theory and progressive flange activation to accommodate 3D performance effects. Low cycle fatigue theory was used to show the final failure of such a precast frame is related to
the fatigue life of the main beam longitudinal bars. The Matthews et al (2004) theory for predicting beam elongation of the super-assembly was successfully applied.

Finally a fragility analysis was used to infer the general seismic vulnerability of this class of precast concrete structure. A comparison was made of the results based on new seating details. The efficacy of the detailing improvements is clearly demonstrated whereby there is now at least a 90% confidence of satisfactory seismic performance (damage limited to repairable levels), whereas with the old details there is a virtual certainty of extensive irreparable damage and even the possibility of collapse.

4.2 Conclusions

Based on the research presented in this thesis, the following conclusions may be drawn:

1. Previous researchers did not fully understand the importance of the relative rotation between the hollow-core unit and the supporting beam. It is this relative rotation that causes a snapping action at the ends of the hollow-core units. The failure mechanism observed the previous tests that only looked at loss of support due to unit pull-off was very different to that observed in the 1994 Northridge (U.S.A) earthquake. However, Mathews (2004) addressed the issue of this relative rotation by testing in a full-scale three-dimensional super-assemblage. Under cyclic loading the brittle failure mechanism observed (hollow-core units falling from supporting beams) was very similar to that seen in the Northridge earthquake.
2. Displacement incompatibility is important in the performance of hollow-core units in precast concrete frames. Displacement incompatibility is caused by the floor system being forced to displace in double curvature via connection to adjacent perimeter frames. However, hollow-core units are not designed for such displacement profiles. Matthews (2004) and Lau (2002) have both identified this as a failure causing mechanism, but neither researcher actually measured the amount of incompatibility to accurately quantify the effect. In this research, the amount of displacement incompatibility between the frame and floor systems was measured and quantified.

3. Matthews (2004) identified the rotation between the hollow-core and supporting beam as one of the most significant factors in the failure of the floor unit and yet the torsion of the supporting beams was not measured. This torsion, due to the eccentric loading from the hollow-core unit influences the amount of relative rotation between the hollow-core unit and beam. By measuring it, the effect of the rotation between the beam and hollow-core can be better quantified. In addition, Matthews (2004) looked back at past construction techniques to determine if there was an issue. This research has looked to the future, as well as the present, and determined a way forward.

4. This experiment showed that provided the damage is not too extensive, then precast concrete frames designed and constructed in New Zealand since the early 1980s can be repaired and restored to full use. Improved one-way slab performance was observed in the experiment; this indicated the success of the detailing changes. The low friction bearing strip and compressible material both
did not perform in the exact way they were intended by design. However, simple solutions have been proposed to improve performance.

5. A beam elongation theory developed by Matthews (2004) was used to predict the overall perimeter frame elongation, with good agreement. Yield line theory was used to predict the behaviour of the infill slab section. This proposed mechanism diminishes quickly as tensile membrane action evolves due to the “bowstring” effect. A simplified design approach utilising the lower bound Hillerborg strip method was found to be a useful way for defining the residual post-earthquake slab-to-hollow-core fracture load carrying capacity of the infill section.

6. The hysteresis loops were found to be dominated by the performance of the structural frame, and the poor performance of the floor in the Matthews (2004) test would not have been indicated by looking at hysteretic performance of the super-assembly. This was also largely true for the present investigation. It is important that the individual details be closely examined in addition to global performance. For precast structures, integrity of the precast parts with the whole should be systematically scrutinised.

7. Fragility analysis was used to better quantify the expected damage outcomes in large earthquakes to a class of precast concrete structures. The fragility analysis showed that when looking at the results from the Matthews (2004) experiment, classified using different damage states, under a Design Basis Earthquake (DBE), 92% of structures of this type would sustain irreparable damage or collapse, requiring demolition with some 8% of the structures leading to possible loss of life. However, in this experiment, under a DBE 94% of buildings would sustain only slight to repairable damage to the floor and frame with no collapse likely.
Also, in the current experiment under a Maximum Considered Earthquake (MCE), 71% of the buildings would sustain repairable damage to the frame or floor, while in Matthews (2004) only 2% of structures would be expected to sustain slight or repairable damage under a MCE. This is a vast improvement in performance.

### 4.3 Recommendations for Professional Practice

The experimental program has given many insights into the performance of hollow-core units in precast concrete buildings. Although many detailing improvements were made in this stage of testing, following Matthews (2004) recommendations, some of the details require further enhancement for superior performance of the hollow-core floor units. The following recommendations are made for improving design standards and professional practice.

1. Reinforcing mesh, ductile or cold-drawn, should not be used within concrete topping slabs over precast hollow-core units. Conventional reinforcing, in the form of HD10’s at 300 centres each way, should be used instead.

2. When an infill slab is present adjacent to the perimeter beam, starter bars should be extended to cross the interface between the infill slab and terminate midway in the adjacent precast floor unit as shown in Figure 4-1. This will increase the ductility of the interface and lower the risk of fracture of the reinforcement at the hollow-core-to-infill interface.

3. Hollow-core units should not be seated in potential plastic hinge zones of their supporting beams. Instead, infill sections should be placed over the highly deformed zones with extra reinforcing placed within the beam to force the hinge
zone to occur beneath the infill section only and not under the brittle hollow-core units. This detail is schematically presented in Figure 4-1.

**Figure 4-1 Recommended detail for the improved performance of the hollow-core units.**

4. The seat of the hollow-core unit in the supporting beam should be reinforced. This requires an extra longitudinal bar and stirrups to tie the seat back into the beam and prevent large sections of concrete peeling off, as shown in Figure 4-2.

5. A second generation low friction bearing strip is required. Increasing the coefficient of friction on the underside of the bearing strip or bonding the underside of the bearing strip to the beam should stop the bearing strip from sliding out from under the unit during cyclic loading. Bonding the bearing strip to the beam or increasing the teeth height should improve the performance as well by further levelling the seat and lowering the risk of concrete spalling, as shown by Figure 4-2.

6. The compressible material placed across the end of the unit is only required when the neutral axis of the beam is above the soffit of the hollow-core. In all other
cases, only a stiff bond breaker is required to restrict concrete from entering the cores of the unit. See Figure 4-2.

![Figure 4-2 Recommended seating support detail.](image)

7. It is important that designers make a post-earthquake assessment of the ability of infill portions of slabs to continue to carry load if partial failure occurs. One simple approach is to use the lower bound Hillerborg strip method.

### 4.4 Recommendations for Future Research

Although much progress has been made since the commencement of Matthews (2004) research, it is evident from this experimental programme at the University of Canterbury that the work that has been undertaken on the topic of precast floor systems in reinforced concrete moment resisting frames is only still in its infancy. Several areas require further work.

**Hollow-core seating details**

Only two different seating details have been investigated in the full-scale, three-dimensional test rig due to the time it takes to set-up and test on such a large scale. Two-dimensional testing similar to that undertaken by Bull and Matthews (2003) can be continued as proof-of-concept tests as these can be done in a relatively
short space of time. The failure mechanism observed in these tests was similar to Matthews (2004) but the limitations of these tests also have to be noted by that researcher. It is possible to alter the set-up of the Bull and Matthews (2003) tests to incorporate the effect of beam elongation of the perimeter beams on the seating of the hollow-core units; this would make the testing more realistic.

Performing several of these component tests in the two-dimensional format will allow a better understanding of the performance of many different seating connection details. This would allow a database of tests to be used to build an analytical model such that the performance of other connection details can be predicted. Ideally, all of the details used for seating precast elements on supporting beams in New Zealand should be tested, at least in the two-dimensional format and an allowable drift level placed on the connection due to its performance. However, there are some details that require urgent assessment, these include:

1. Two details were recommended by the Technical Advisory Group on Precast Floors to be included in the 2003 amendment to the New Zealand Concrete Structures Standard (NZS 3101, 2003). The second of these details needs to be investigated more fully as presently it is untested. Ideally, because of its inclusions in the standard this detail also needs to be examined in the three-dimensional test rig to incorporate all realistic second order effects.

2. In determining a “way forward” for new precast concrete buildings incorporating hollow-core floors something needs to be done about the existing buildings with the proven inferior connection details. A retrofit solution needs to be developed such that these buildings at least maintain life-safety in a severe earthquake.
Matthews (2004) proposed several retrofit solutions that, as yet, are essentially untested.

3. Hollow-core floors are certainly not the only common precast floor type used in New Zealand. All of the major precast floor types used in New Zealand should be tested to determine their vulnerability to catastrophic damage during a major earthquake. It is believed that hollow-core may not be the only precast floor type to exhibit poor performance.

4. Owners of buildings may have an intrinsic belief that their buildings are ‘earthquake proof’. Designers are moving towards trying to specify details that ensure damage avoidance or at least damage control. Future building construction should attempt to ensure structures are undamaged following a major earthquake. On the list of potential new details is rocking or articulated connections. New connection details are required because all of the displacement is concentrated at the rocking joints rather than dispersed over a plastic hinge length. How do precast floors perform when seated on beams with rocking connections?

The list could go on, but some additional important questions requiring resolution are:

- What is the affect of negative seating (precast units being cut short and placed with a bridging connection) on the hollow-core performance?
- What is the affect of having different supporting beam geometries? (this may change the compressible backing board material requirements)
What is the affect of placing additional shear reinforcement in the cores of the units?

**Lateral support of the first hollow-core unit to the perimeter frame**

The detail investigated in this research worked well but has its own limitations. A possible requirement for a flat bottomed soffit for architectural reasons means that the infill slab portion would not be desirable. Instead, a detail that allows vertical movement and yet transfers diaphragm forces to the perimeter frame is required. Shown below in Figure 4-3 are two possible solutions that require experimental investigation.

![Diagram of possible architectural finish solutions for first hollow-core unit connection to perimeter frame.](image)

(a) Debonding of first 300 series hollow-core unit.

(b) Replacing first 300 series unit with a 200 series unit and compressible, sacrificial filler.

Figure 4-3 Possible architectural finish solutions for first hollow-core unit connection to perimeter frame.
**Analytical Research**

The experimental data collected from this study and Matthews’ (2004) study would provide an excellent basis for an extensive analytical study to develop and analytical model that could accurately predict the complex three-dimensional behaviour and interaction between the hollow-core and concrete moment resisting frame. In order to accurately model the behaviour and interaction several influencing parameters need to be fully understood and modelled individually. In addition to the hollow-core connection details outlined above, these include:

1. The point of full activation of the starter bars in the transverse beams. This is obviously a function of the torsional stiffness of the beams and from Matthews (2004) test data was assumed to be 3.05m (in this case, half the bay length) at 2.0% drift. The torsional stiffness of the beams needs to be modelled to determine the point at which full activation occurs.

2. As stated in section 2.8.5, determining the mechanism that caused the fracture of the mesh is complicated because so many factors played a part. Ideally a finite element model needs to be developed that incorporates the interaction between the hollow-core units, topping slab and the beam including the displacement incompatibility and force transfer between them.

3. Modelling of the “bowstring” effect is required. It is known that the affect exists but as yet no quantification of it has been done. Once again, this is a complicated model as there is so many influencing parameters. In addition to this the beam moment enhancement due to the compression field generated in the beam from the bowstring effect needs to be modelled.
4.5 References


Appendix A

Building Reconstruction Photographs and Calculations

A.1 Building repair photos

East Transverse Beam.

- Removal of concrete and stirrups.

Pictured are helpers from C.Lund and Son Ltd.

East Transverse Beam.


East Transverse Beam.

- Removal of concrete complete, ready for reconstruction.
East Transverse Beam.

- Beam boxed and ready for concrete

East Transverse Beam.

- Lap bars in central section. Designed such that bars not curtailed in potential plastic hinge zone (PHZ).

East Transverse Beam.

- Beam ready for concrete

East Transverse Beam.

- Instrumentation rods embedded in concrete beam.
East Transverse Beam.

- Slump test of concrete for east beam (slump = 110mm)

East Transverse Beam.

- After pour. Seat levelled and finished.

West Transverse Beam.

- Chipping out concrete with concrete breakers.
West Transverse Beam.
  Pictured are helpers from C.Lund and Son Ltd.

West Transverse Beam.
- Beam ready to be boxed. Note the lap bars and new stirrups in place.

West Transverse Beam.
- Ready for pour

West Transverse Beam.
- After concrete pour, seat levelled and finished.
South Longitudinal Beam.

- Plastic hinge removed.
  Chipped out back to uncracked concrete

South Longitudinal Beam.

- Exposed column face

Building ready for hollow-core units to be placed with beams and hinges boxed.
Hollow-core units

- Stacked and awaiting transport to site.

Hollow-core units

- First unit being craned in and placed.

Hollow-core units

- In place, note backing board attached to end of units

Hollow-core units

- Entire floor in place.
Floor

- Mesh and starters in place ready for concrete topping to be poured.

Floor pour

- First section of floor poured and screeded, awaiting second truck load.

Floor pour

- Entire floor topping poured

Floor pour

- Concrete being finished with a Kelly float.
Building after concrete has cured and beams and columns have been painted (floor yet to be painted.).

Note:
Blue = existing (Matthews) concrete.
White = new concrete

Figure A-1 Building repair photos.

A.2 Column drag bar design.

The design of the drag bars were based on a moderate to high axial load ratio of $0.2f'_c A_g$. This is because the super-assembly is self-equilibrating and therefore no axial loads should be induced into the system. However, in the later stages of testing when the plastic hinges begin to degrade and forces redistribute, the forces applied to the columns are not equal and opposite and axial load is induced into the column. The magnitude of these axial forces are nowhere near the magnitude of axial compression and tension force experienced in a real building. It was for this reason that a conservatively high axial load of $0.2f'_c A_g$ was chosen for the design of the drag bars. In a real building the magnitude of axial load would be known so this assumption would not be required in design.

The New Zealand Concrete Structures Standard (NZS3101:1995) requires that “the columns of the frames shall be tied at each level of the floor system. The tie reinforcement shall be effectively anchored perpendicular to the frame and capable of resisting in tension the larger of 5% of the maximum total axial compression load on the
column or 20% of the column shear force.” In addition to this the draft joint Australian and New Zealand Structural Design Actions Standard (AS/NZS 1170.4:2003) states that “all parts of the structure shall be interconnected. Connections shall be capable of transmitting 5% of the value of \((G+\Psi_cQ)\) for the connection under consideration.”

Design based on 5% of \(0.2f'_cA_g\) axial compression load or 20% of the column shear force (based on Matthews’ experimental results):

\[
N^* = 5% \times 0.2 \times 30MPa \times 750mm \times 750mm = 169kN \tag{A-1a}
\]

or

\[
N^* = 20% \times 320kN = 64kN \tag{A-1b}
\]

Therefore, use 169kN force.

If use 2YD20 bars:

\[
N^* \leq \phi N_t
\]

\[
169000N \leq 0.8 \times 2 \times 314mm^2 \times 500MPa
\]

\[
169kN \leq 251kN
\]

Therefore, 2YD20 bars acceptable by design (note: 2YD16 bars would not be acceptable.)

**A.3 References**


Appendix B

Experimental Results 1: Testing Record and Photographic Log

B.1 Material Testing

B.1.1 Steel Testing

Below in Table B-1 are the tensile test results for the reinforcing steel used in the experiment. Figure B-1 shows graphs of these results.

Table B-1 Experimental test results of properties of reinforcing steel used in the test (some values obtained from Matthews (2004)).

<table>
<thead>
<tr>
<th>Type</th>
<th>Location</th>
<th>$f_y$ (MPa)</th>
<th>$f_{sh}$ (MPa)</th>
<th>$f_u$ (MPa)</th>
<th>$E_s$ (GPa)</th>
<th>$\varepsilon_y$</th>
<th>$\varepsilon_{sh}$</th>
<th>$\varepsilon_{su}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>D24</td>
<td>Beam Bars – new</td>
<td>309</td>
<td>307</td>
<td>450</td>
<td>215</td>
<td>0.0019</td>
<td>0.019</td>
<td>0.21</td>
</tr>
<tr>
<td>D24</td>
<td>Beam Bars – old</td>
<td>302</td>
<td>301</td>
<td>451</td>
<td>238</td>
<td>0.0017</td>
<td>0.020</td>
<td>0.19</td>
</tr>
<tr>
<td>HD24</td>
<td>Column Bars – old</td>
<td>430</td>
<td>432</td>
<td>582</td>
<td>231</td>
<td>0.0028</td>
<td>0.015</td>
<td>0.12</td>
</tr>
<tr>
<td>HD28</td>
<td>Column Bars – old</td>
<td>479</td>
<td>477</td>
<td>650</td>
<td>240</td>
<td>0.0021</td>
<td>0.014</td>
<td>0.12</td>
</tr>
<tr>
<td>HD12</td>
<td>Starter Bars – old</td>
<td>504</td>
<td>512</td>
<td>674</td>
<td>186</td>
<td>0.0028</td>
<td>0.012</td>
<td>0.14</td>
</tr>
<tr>
<td>YD12</td>
<td>Starter Bars – new</td>
<td>565</td>
<td>568</td>
<td>677</td>
<td>218</td>
<td>0.0027</td>
<td>0.020</td>
<td>0.12</td>
</tr>
<tr>
<td>R12</td>
<td>Stirrup – old</td>
<td>318</td>
<td>325</td>
<td>438</td>
<td>160</td>
<td>0.001</td>
<td>0.008</td>
<td>0.17</td>
</tr>
<tr>
<td>Mesh</td>
<td>Diaphragm</td>
<td>-</td>
<td>-</td>
<td>546</td>
<td>213</td>
<td>-</td>
<td>-</td>
<td>0.034</td>
</tr>
</tbody>
</table>
Figure B-1 Individual steel test results.
B.1.2 Concrete compressive strength tests

Table B-2 shows the concrete compressive strengths and Figure B-2 shows photos of the concrete cylinders being tested.

<table>
<thead>
<tr>
<th>Pour Location</th>
<th>7-day strength (MPa)</th>
<th>28-day strength (MPa)</th>
<th>Strength @ start of test (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>East ½ beam</td>
<td>25.6</td>
<td>30.7</td>
<td>39.8</td>
</tr>
<tr>
<td>West ½ beam</td>
<td>21.8</td>
<td>29.6</td>
<td>40.3</td>
</tr>
<tr>
<td>Topping (avg)</td>
<td>23.0</td>
<td>30.2</td>
<td>37.6</td>
</tr>
<tr>
<td>Matthews’ Columns</td>
<td>-</td>
<td>-</td>
<td>48.2 (average)</td>
</tr>
<tr>
<td>Matthews’ Beams</td>
<td>-</td>
<td>-</td>
<td>44.5 (average)</td>
</tr>
</tbody>
</table>

(a) Concrete cylinder testing  
(b) Concrete cone failure

Figure B-2. Concrete cylinder testing and failure of a concrete cylinder
B.1.3 Hollow-core properties

The hollow-core floor units are cast off site using an extrusion machine on a long casting bed and are cut to the required length after the concrete has gone hard. Four seven strand hollow-core units were supplied to site. Figure B-3 shows a cross-section of a typical seven strand hollow-core unit and Table B-3 shows the typical properties of the 300mm deep (designated “300 series”) hollow-core unit that was supplied to site.

![Cross section of hollow-core unit](image)

**Figure B-3 Cross section of hollow-core unit used in testing program**

**Table B-3 Section properties of a typical 300 series hollow-core unit.**

<table>
<thead>
<tr>
<th>Strand type</th>
<th>Area (m²)</th>
<th>Yb (mm)</th>
<th>I (m⁴)</th>
<th>Self Weight (kPa)</th>
<th>Strand @ 28 days</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dycore/partek</td>
<td>0.1606</td>
<td>153</td>
<td>2.04x10⁻³</td>
<td>3.20</td>
<td>7 wire stress relieved</td>
</tr>
</tbody>
</table>
B.2 Instrumentation photographs

The following photos show the locations and set-up for some of the instrumentation used throughout the test specimen.

![Image of Loadcell attached to actuator in place for test.](image)

Figure B-4 Loadcell attached to actuator in place for test.

(a) Rotary pot mounted on instrument frame and attached to top of NE Column

(b) Displacement transducers mounted under universal bearings in the longitudinal and transverse directions.

Figure B-5 Instrumentation used to measure calculate the interstorey drift of the columns
(a) Inclinometers attached to NW column in both in plane and out of plane directions.

(b) East beam inclinometers to monitor the torsion in the beam

Figure B-6 Reiker inclinometers used to measure inclination of the beams and columns

B.3 Full Test Account

B.3.1 Phase I: Longitudinal loading

The loading began with a cycle to 0.1% drift to check that the programme was working properly and had been set up correctly. No damage, other than existing shrinkage cracks, was noted at this early stage. At the 0.25% cycle hairline cracks were visible at the old concrete/new concrete interface at the beam-beam/column boundary and discontinuity cracks formed at both ends of the super-assemblage (Figure B-7). The 0.5%
cycles showed a little cracking in the plastic hinge zones (PHZ) and some existing cracks from the Matthews testing in the columns and beam/column joints opening.

The super-assemblage appeared to yield later than in the Matthews test, but this could be due to the previous heat treatment of the bars altering the apparent yield stress slightly. At +1.0% drift there was a small amount of topping delamination around the North East column and in the -1.0% cycle around the North West column as well. A delamination map for the ±1.0% cycle can be seen in Figure B-8. In the +1.0% drift cycle the displacement incompatibility between the perimeter frame and first hollow-core unit became obvious with diagonal cracks showing in the infill section (Figure B-9). The diagonal cracks in the infill extended in the second ±1.0% cycle reaching the infill - hollow-core interface and running along the join for almost the entire floor length, except around the central column where the drag bars appeared to tie the infill and hollow-core unit together.

Cracking in the PHZ extended further into the beam than in the Matthews test although most of the cracks never opened past hairline width with most of the deformation occurring at the beam – column interface (i.e. the joint between old and new
At this stage the super-assemblage was performing well, any damage like this in a real building would lead to very little economic loss to the owner.

The +2.0% cycle produced extensions of existing cracks with the infill – hollow-core crack opening up on the west end to 2mm wide with a vertical displacement of 2mm (Figure B-10(a)). The first hollow-core unit developed a large corner crack that extended into the web of the second core and this unit showed signs of seat damage that was...
visible due to the 10mm pull-off of the first unit. Cover concrete in the area also spalled due to the hollow-core unit bearing on the front edge of the beam. The low-friction bearing strip was visible from the underside of the unit showing that in some places it was being pulled out as the units were sliding. All of this can be seen in Figure B-11. A hairline crack formed in the underside of the first hollow-core unit propagating out from the central column. This crack never grew any longer or any wider during test.

(a) Infill crack at +2.0% drift (2mm wide and 2mm vertical displacement)

(b) Infill crack at -2.0% drift, 1.0mm wide

Figure B-10 Infill crack at ±2.0% drift
In the -2.0% cycle a soffit crack formed in the first and third hollow-core unit extending for 1.5m from the transverse beams into the floor and a corner crack formed in the fist unit at the east end although this did not extend to any cracking in the second core. The soffit crack in the first unit formed directly above a large crack developing in the transverse beams. The east end of the infill showed much less damage than the west end with a 1.0mm wide crack with no vertical or horizontal displacement (Figure B-10(b)). An overall photo of the infill is shown in (Figure B-12).
Once again the PHZs were faring well; most of the damage was occurring at the interface with a 10mm opening visible at both ends of the south beam (Figure B-13). Some spalling was noticeable in the compression zones of the PHZ. Delamination maps for the ±2.0% cycle are shown in Figure B-8. Economic loss to an owner of a building with damage like this would be moderate. The cracks and damage are considered to be easily repairable with the only sign of concern at this stage being the residual interstorey drift of the building (about 0.8% drift).

![Figure B-13 South East PHZ beam – column interface, 10mm opening.](image)

**B.3.2 Phase II: Transverse loading**

Very little new cracking occurred in the early stages of the transverse loading. This was because the transverse beams were pre-cracked from the longitudinal loading. In the ±1.0% cycle a crack (2mm at this stage) opened up in the ends of both of the transverse beams about 1.0m from the column face (Figure B-14 (a)). The north side corner of the fourth hollow-core unit at both ends formed a corner crack at -1.0% drift that progressed up from the bottom of the unit to run along the web (Figure B-14 (b)).

In the ±2.0% drift cycles the large crack in the transverse beams opened up to 6mm wide in places and cracks were forming around the columns in the transverse beam.
that seemed to isolate the floor from the columns (Figure B-15). Further cracks developed in the corners of the first unit. Additional cracking of the soffit occurred and extended almost the entire length of the floor although the crack was no more than hairline in width.

Following the second cycle of ±2.0% drift a torsion test to ±0.5% was completed. A third cycle of ±2.0% drift showed no loss in load carrying capacity following the torsion test. The ±3.0% drift cycle generated much more cracking in the infill section. Sections of the interface crack were 8mm in width, especially near the ends, while the central column was tied to the floor by the drag bars. Damaged concrete in the east end...
was removed to find the mesh in tact (Figure B-16). In the +3.0% cycle the floor had dropped by 20mm with respect to the beam at the west end (Figure B-17) while in the -3.0% cycle it rose by 20mm. This is expected when the transverse beams deform in double curvature; in the positive drift direction the south end of the transverse beam drops, taking the floor with it while in the negative drift direction it rises pushing the floor up with it relative to the longitudinal beams. Delamination maps for the transverse loading are shown in Figure B-8.

![Figure B-16 Infill crack in ±3.0% cycle](image1)

(a) Infill crack, east end (8mm wide)  (b) Mesh crossing infill crack

![Figure B-17 Diaphragm dropping away from beam at west end in +3.0% cycle](image2)

Figure B-17 Diaphragm dropping away from beam at west end in +3.0% cycle
B.3.3 Phase III: Longitudinal re-loading

Early in the phase III testing beam spalling at the west end of the first hollow-core unit left the unit with almost ¾ of its length with at least 20mm of seat spalled off (Figure B-18). At +2.25% drift, on the way to +3.0% drift, the first crosswire of mesh fractured at the infill/hollow-core interface about 1.5m west of the central column. The first fracture was followed by nine others on the way to +3.0%.

(a) Underside first hollow-core unit, west end. Note beam spall and bearing strip sliding

(b) 45-50mm of seat exposed of first hollow-core unit, west end

Figure B-18. Seating performance of first hollow-core unit, west end at +3.0% drift

Once the mesh had fractured it could be seen that the fracture was due to two mechanisms. Firstly the tear was due to the floor diaphragm restraining the frame from elongating causing a transverse tension force as the beam tries to translate outwards instead, this produced a horizontal east-west displacement (15mm) once it fractured (Figure B-19(a)) as well as accentuating the transverse north-south displacement (i.e. crack width, 10mm) (Figure B-19(b)). Secondly, the tear was due to the displacement incompatibility. This caused a vertical offset of 10mm once the mesh had fractured (Figure B-19(c)). A close-up of the mesh fractured is shown in Figure B-19 (d). Even
though at this stage the mesh fractured over about 3m in the west end of the floor, the central column remained tied into the building.

![Images of displacements](image1.jpg)  ![Images of displacements](image2.jpg)

(a) Horizontal displacement of 15mm after mesh fractures  (b) Crack width of 10mm in places after mesh fractures

![Images of displacements](image3.jpg)  ![Images of displacements](image4.jpg)

(c) Vertical offset of 10mm in west end after mesh fractures  (d) Mesh fracture in infill section

Figure B-19 Displacements in west end of infill after mesh fractures.

At ±3.0% more spalling in the PHZs was apparent due to the cyclic compression/tension loading. The transverse beams showed more torsion due to degradation of the PHZs accentuating the load eccentricity and causing more torsion of the beams. The web split in the north edge of the fourth hollow-core unit grew to about 2m from the support with the broken section displacing by 5mm both vertically and horizontally (Figure B-20). At this stage the units had pulled off by 15mm with the low friction bearing strip sliding by 15mm in some places. The building itself was in relatively good repair (even with a residual drift of 1.7%), there was some areas of serious damage but life safety was not an issue, people would be able to safely get out of the building.
In the first ±4.0% cycle eight further mesh strands fractured with most of the cracking at this time being small extensions of existing cracks. A large section of the unreinforced seat of the fourth hollow-core unit at the west end began to drop away showing the necessity of reinforcing the seat to tie it to the beam, the concrete fell out during the second +4.0% cycle (Figure B-21). A marked amount of spalling and end fracturing occurred under the first hollow-core unit at the west end leaving the unit with very little support and the prestressing strand visible through a hole (Figure B-22).
The PHZs began to show signs of distress, large enough sections of concrete had fallen and the beam had separated from the column by cracking that it began to slide up and down the column as the column rotated. This is evident in the kinking of the main bars by as much as 10mm (Figure B-23 (a)). At 3.56% and 3.78% in the second +4% cycle two bars in the bottom layer of the south beam, west end PHZ fractured (Figure B-23(b)), with two more in the bottom layer of the south beam east end PHZ and one in the top layer of the west PHZ necking and fracturing (Figure B-24) in the -4% cycle.
A cycle to +5.0% was undertaken as it was felt that drifts of this magnitude could be expected in high seismic areas of the likes of Wellington, New Zealand, especially in near fault fling effects and with the draft New Zealand and Australian Loadings Standard, (AS/NZS 1170.4:2003) requiring that structures are capable of surviving an earthquake with a return period of 2500 years without collapse and with a margin of safety. This means that interstorey drifts in the order of 4.5% drift should be sustained without collapse. In this cycle further main bar fractures were observed eventually leaving the south beam west end PHZ with no main bars in the bottom layer at +4.32% drift. This failure signified end of test after completion of the +5.0% cycle because all of the moment capacity had been lost in that beam. The seat of the units was significantly damaged with the ends of the units visible in large section along the beam (Figure B-25(a)). The crack at the infill/hollow-core interface had become significantly worse in the west end with at 60mm vertical displacement in the middle of the crack (Figure B-25(b)).
Delamination maps for the three phases are shown in Figure B-8 and a full photographic log of the observed photographic results follows.

(a) Damaged seat of west end hollow-core units  
(b) Infill damage at west end

Figure B-25 Significant damage at +5.0% drift

B.4 Testing photographic log

B.4.1 Phase I

(a) West side of south central column  
(b) North-west column discontinuity crack forming

Figure B-26 Damage at ±0.25% drift
(a) East end discontinuity crack

(b) Central column floor cracks – beam, column and infill cracks

(c) South central column flexure cracks

(d) East beam torsion cracks

Figure B-27 Damage at +0.5% drift
(a) Infill crack  
(b) Small beam spall under joint between 1st and 2nd hollow-core unit at east end  
(c) South-East PHZ flexural cracks  
(d) South-centre column joint cracks

Figure B-28 Damage at -0.5% drift
(a) North-East discontinuity crack, floor dropped by about 1mm near NE column.

(b) Infill, west end. Diagonal cracks forming in infill and hollow-core/infill crack extends length of floor.

(c) Hollow-core seat crack, 1st unit west end

(d) South-West beam to column joint, 1mm crack.

Figure B-29 Damage at +1.0% drift
(a) Infill, east end. Diagonal cracks forming in infill and hollow-core/infill crack extends length of floor.

(b) West end discontinuity crack, 1mm wide at ends

(c) South-west corner. Beam flexure, infill and discontinuity cracks

(d) Torsion cracks in east beam

Figure B-30 Damage at -1.0% drift
(a) East end discontinuity cracks

(b) North-east corner. 35-40mm deep crack, 7mm vertical displacement

(c) Infill crack. Note extends over 1st hollow-core unit at west end. 20mm deep, 2mm vertical displacement

(d) South centre column, on floor. 50mm deep crack in this and other PHZs.

(e) Hollow-core corner crack, 1st unit east end

(f) Hollow-core seat and corner crack, 1st unit west end

Figure B-24 Damage at ±2.0% drift

B-24
(g) Hollow-core hairline crack in middle of 1st unit

(h) Spalling cover concrete, second cycle

(i) Crack in second core of first hollow-core unit, west end

Figure B-31 cont. Damage at +2.0% drift
(a) First hollow-core unit soffit crack

(b) First hollow-core unit corner and seat crack

(c) Infill crack, middle east end. 1.5-2mm wide. 10mm deep in places

(d) South centre beam to column connection. Ruler could be lost in joint, 6-7mm wide around centre column, 10-11mm at SE and SW hinges.

Figure B-32 Damage at -2.0% drift
B.4.2 Phase II

(a) Cracks in East beam, North end
(b) West beam flexural cracks

Figure B-33 Damage at +0.5% drift

(a) Flexure cracks in East beam, South end
(b) Corner crack in North-West end, 4th hollow-core unit.
(c) East beam flexure cracks
(d) Corner crack in North-East end, 4th hollow-core unit.

Figure B-34 Damage at -0.5% drift
(a) Cracks in East beam, south end. Two cracks approx 1-1.5mm wide

(b) Flexure cracks in West beam

Figure B-35 Damage at +1.0% drift

(a) Cracks in West beam, north end. Two cracks approx 1mm wide

(b) Corner crack in 4th hollow-core unit, west end

(c) Soffit crack in hollow-core unit.

(d) Corner crack in 4th hollow-core unit, East end

Figure B-36 Damage at -1.0% drift
(a) Beam flexure cracks in North-West corner.

North East corner. 3-4mm wide floor crack, 7-8mm crack at column, 10-11 mm between hollow-core and timber

(c) East beam, south end. Crack 4-5mm wide

(d) West beam, south end. Crack 4-5mm wide

(e) Bearing strip sliding out (9mm)

(f) Corner crack, 1st hollow-core unit, west end

Figure B-37 Damage at +2.0% drift
(a) Infill cracking, east end. 4-5mm wide at ends.

(b) East beam, south end PHZ

(c) Corner crack 4\textsuperscript{th} hollow-core unit west end. Crack 1.5-2m long both ends in hollow-core web.

(d) Soffit crack

Figure B-38 Damage at -2.0\% drift

(a) North-East column to floor. 12mm displacement at edge of column. 5mm floor crack, 4mm vertical

(b) Vertical displacement of 19mm floor to beam at infill crack

Figure B-39 Damage at +3.0\% drift

B-30
(a) Floor, South-East corner. Infill crack 8-15mm wide this end

(b) Exposed mesh – kinked but not visibly fractured

(c) Crack in second core of first hollow-core unit, west end at end of out-of-plane testing

(d) Floor lifting relative to longitudinal beams

(e) Cracking in West PHZ, south end

(f) Cracking in West PHZ, north end

Figure B-40 Damage at -3.0% drift
B.4.3 Phase III

(a) Infill crack, note: topping cracks extended into end of 1st hollow-core unit (delamination).

(b) Fractured mesh

(c) North-west corner. 10mm vertical hollow-core crack, cover concrete lost load carrying capacity

(d) First unit west end seat and corner damage

(e) 15mm seat and bearing strip sliding

(f) Damage to seat of first hollow-core unit, west end.

Figure B-41 Damage at +3.0% drift
(a) Infill cracks, east end

(b) Transverse beam, west end

(c) South-East column floor cracks (15mm wide)

(d) Seat of first hollow-core unit, east end. Note: damaged ends under core locations

(e) Infill cracks underside east end.

Figure B-42 Damage at -3.0% drift
Figure B-43 Damage at +4.0% drift, first cycle

(a) Central column drag bar cover concrete spalled

(b) Infill crack, west side central column (10mm)

(c) Vertical floor drop of 25mm

(d) Cover concrete spalling, 2nd unit west end
(a) South-East PHZ deformation (~20mm)  
(b) Kinked main bar

**FigureB-44 Damage at -4.0% drift, first cycle**

(a) Seat concrete lost 4\textsuperscript{th} hollow-core uni  
(b) Entire seat damaged and cracked, no load carrying capacity.

(c) Hollow-core seat, west end. Spalled concrete exposes bearing strip and end of units.  
(d) Fractured main bars in South-west PHZ

**FigureB-45 Damage at +4.0% drift, second cycle**

B-35
(a) Necking main bar in south-west PHZ

(b) Fractured main bar in south-west PHZ

(c) Fractured main bars in South-East PHZ

(d) Corner crack 4th hollow-core unit, west end.
Strand visible through 10mm crack and 15mm vertical drop of unit

Figure B-46 Damage at -4.0% drift, second cycle
Figure B-47 Damage at +5.0% drift

(a) South-West hinge damage at end of test
(b) All main bars fractured in bottom of SW hinge
(c) Seat damage west end
(d) Fractured end hollow-core units west end
(e) Infill crack at end of test (60mm vertical)
(f) Fractured mesh in fill crack.
(g) Floor at end of test
B.5 References


Appendix C

Experimental Results 2: Forensic Analysis

C.1 Yield drift of the super-assemblage

Based on the experimental hysteresis loop for Phase I the yield drift was assumed to be 0.5% drift. This is although the specimen appeared to yield later due to the pre-cracked beams and columns increasing the apparent yield drift. This experimental yield drift can be compared to the theoretical yield drift calculated in a manner similar to Matthews (2004) and found to be 0.48% drift. This yield drift is made up of contributions from the beam, beam column joint and column elastic deflections;

C.1.1 Beam Contribution

The yield moment was determined from simple mechanics and the curvature from straightforward relationships proposed by Priestley and Kowalsky (2000);

\[ \phi_{yb} = 1.70 \frac{\varepsilon_y}{h_b} = 3.4 \times 10^{-6} \text{mm} \text{ and } M_{yb} = 460 \text{kNm} \]  

\[ (C-1) \]

where \( \varepsilon_y \) = yield strain of the longitudinal reinforcing bars and \( h_b \) = beam height. Now, knowing the beam nominal moment and the yield curvature the effective stiffness can be determined, as a function of the gross stiffness;

\[ \phi_{yb} = \frac{M_{yb}}{E_c I_{eff}} \]  

\[ (C-2) \]
where $E_c$ = Modulus of elasticity of the concrete, $I_g$ = gross beam moment of inertia and $I_{eff}$ = effective beam moment of inertia.

Now the beam deflection in terms of the column deflection ($\delta^b$) can be determined using moment area.

$$\delta_b = \frac{M_{sb}l_b^2}{12EI_{eff}} = 8.1mm$$  \hspace{1cm} (C-4)

in which $\delta_b$ = displacement of the beam at its point of inflection and $L_b = the$ distance between the beam hinges. Finally, the beam displacement can be converted to an overall system column displacement using:

$$\delta^b_c = \frac{\delta_b L_c}{L_b/2} = 10.6mm$$ \hspace{1cm} (C-5)

in which $\delta^b_c$ = column displacement due to the beam deflecting and $L_c = column$ height between inflection points.

**C.1.2 Beam Column Joint Contribution**

From Matthews, an expression for the beam column joint contribution to the overall yield drift was derived;

$$\gamma_j = 38.5 \frac{A_f}{BH} E_c \left( 1 - \frac{(d - d') L_c}{H L_b} \right) = 0.0014or0.14\%$$ \hspace{1cm} (C-6)
in which $\gamma_j = \text{joint distortion}$, $B = \text{breadth of the column}$, $(d-d') = \text{beam internal leverarm}$, $H = \text{column height}$, $L = \text{centreline distance between beam inflection points}$. 0.14% drift equals 4.9mm column displacement due to the beam column joint distortion ($\delta_j$).

### C.1.3 Column Contribution

From calculating the column yield moment and determining the yield curvature from simple relationships (Priestley and Kowalsky, 2000) the effective stiffness can be determined.

$$\phi_{yc} = 2.12 \frac{E}{h_c} = 6.77 \times 10^{-6} / \text{mm} \quad \text{and} \quad M_{yc} = 750 \text{kNm} \quad \text{(C-7)}$$

where $h_c = \text{depth of the column}$. The effective stiffness can be calculated from;

$$\frac{EI_{eff}}{EI_g} = \frac{M_{yc}}{\phi_{yc} EI_g} = \frac{750 \times 10^6}{6.77 \times 10^{-6} \times 28000 \times \frac{750^2}{12}} = 0.15 \quad \text{(C-8)}$$

Now the column moment at the yield of the beam can be determined using equilibrium of the moments at the joint. The column moment at yield of the beam is determined to be ($M_c$) = 222kNm. From moment area theorem the deflection of the column is;

$$\delta_c = \frac{M_c l_c^2}{3 EI_{eff}} = 1.2 \text{mm} \quad \text{(C-9)}$$

where $l_c = \text{height of the column above the joint to the point of inflection}$.

Therefore the total displacement of the top of the column due to the three components (beam, column and beam-column joint) and therefore the yield drift of the super-assembly can be found.
\[ \delta_e = \delta_e^b + \delta_e^j + \delta_e = 10.6 + 4.9 + 1.2 = 16.7 \text{mm} \]  
(C-10)

If this displacement is turned into an interstorey drift it equals 0.48%.

The reasons why the experimental yield point appeared to be higher than the calculated theoretical one is because almost 40% of the calculated yield drift is made up of elastic deformations of the columns and beam-column joints and with the sections pre-cracked the effective stiffness of the section is reduced significantly therefore increasing the super-assemblages yield displacement. The cracking in the connection between the beam and beam-column joint also started early in the testing indicating that the bond between the old and new concrete was lost at an early stage therefore the super-assemblage appeared to yield at a larger drift due to the larger system displacements incurred at the yield stresses.

C.2 Stiffness of the super-assemblage

Figure C-1 shows the initial stages of loading used to calculate the initial stiffness of the super-assemblage for the three phases of loading. In Phase I the stiffness was calculated to be 34.7MN/m (Figure C-1(a)). The initial stiffness for Phase II was calculated to be 28.3MN/m (Figure C-1(b)); about 82% of the Phase I initial stiffness. This loss in stiffness in Phase II is due to the cracks in the transverse beams from Phase I loading reducing the beam stiffness. The initial stiffness of Phase III was 19.4MN/m which was 56% of the Phase I initial stiffness. This reduction in stiffness was due to the damage experienced from the earlier cycles of loading and the existing cracks, such as the discontinuity cracks isolating the floor from the frame, therefore reducing the impact of the floors stiffening effect.
C.3 General Super-assembly Performance

C.3.1 Central Column Bar Slip through Joint

The slip of the bars through the central column joint was monitored by two potentiometers attached to the reinforcing steel. In the first phase of loading less than 1.0mm of slip was experienced. In phase III about 1.2mm slip occurred in the later stages of loading. Most of the slip occurred in the top bars, as expected, due to less bond strength in the top reinforcing (Figure C-2). The New Zealand Concrete Code, (NZS3101, 1995) specifies a maximum allowable bar size to pass through a beam column joint. This maximum bar size is limited on the basis of bond requirements;

\[
\frac{d_b}{h_c} = 3.3\alpha_a \sqrt{\frac{f_c}{\alpha_a f_y}} \tag{C-11}
\]
where \( d_b \) = longitudinal beam bar diameter, \( h_c \) = depth of the column, \( \alpha_f = 1.0 \) for one way frames, \( f'_{c} \) = specified concrete compressive strength, \( \alpha_o = 1.25 \) when the plastic hinge forms at the column face and \( f_y = \) yield strength of the reinforcement.

The equation shown in (C-11) gives a maximum allowable bar size to pass through the beam column joint of 44mm. Therefore, the slip experienced is minimal as the beam bar diameter passing through the joint is only 55% of the maximum allowed by NZS3101 (1995). Approximately the same amount of slip was experienced in Matthews (2004) experiment showing that no significant degradation of the joint occurred because of the Matthews (2004) testing.

**C.3.2 East Beam Torsion**

Figure C-3 shows the torsion induced into the east transverse beam as a function of the scan number (or time). As can be seen, as the interstorey drift increases the beam
(a) Torsion in east transverse beam as a function of time (scan number) for Phase I

(b) Torsion in east transverse beam as a function of time (scan number) for Phase III

Figure C-3 Torsion in the East transverse beam.
undergoes more and more negative (anti-clockwise) rotation. The inclinometer in the joint region is fairly consistent with the overall building interstorey drift showing that the joint, in general, follows the inclination of the columns, as expected. However, the centre of the beam, around the intersection between the 1\textsuperscript{st} and 2\textsuperscript{nd} and the 2\textsuperscript{nd} and 3\textsuperscript{rd} hollow-core units the torsion in the beam is significantly noticeable. In the initial stages, pre-yield, the beam follows the joint. After yield, the beam undergoes significant torsion due to the eccentric loading of the hollow-core units added to the fact that the beams are already cracked, and the beam rotates in an anti-clockwise direction. The torsion grows significantly worse as time goes on, with the east beam remaining in negative drift throughout Phase III loading.

This torsion is one of the contributing factors for why the compressible material will not compress. If the beam rotates in an anti-clockwise direction then the base of the hollow-core unit is pulled further and further off the seat.

**C.3.3 Displacement incompatibility**

Figure C-4(a) and (b) show the displacement incompatibility along the length of the super-assembly between the floor and beam in Phase I and Phase III loading respectively. Each peak loading cycle is showed as a different coloured line. The displacement incompatibility does not increase significantly between loading cycles to the same drift amplitude.

A small project was undertaken on the computer program Ruaumoko (Carr, 2002) to try and predict the displacement incompatibility between the frame and floor systems. The analysis involved utilising a plasticity concentrated approach to model the interaction between the slab and perimeter frame using springs. The properties of the springs
Figure C-4 Displacement incompatibility between floor and beam in the longitudinal loading cycles
determined the level of interaction between the slab and frame. Initial analysis involved assuming several properties and the level of interaction and although the model produced results in the ball park of the actual experimental results (Figure C-5(a) and (b)) further work needs to be completed on the model to incorporate all of the influencing parameters. For example, these include, the initial offset of the hollow-core units due to the pre-tensioning, calibration of the end seat support connection springs, modelling the effects of beam elongation on the performance of the diaphragm, including strength degradation of the hinges in the model and expanding on the simplifications made during modelling. However, as stated above the model produced a similar deformed shape with similar displacement incompatibilities.

**C.3.4 Pull-off of Hollow-core Units**

Figure C-6 shows the location of the potentiometers measuring the hollow-core pull-off and vertical movement. Figure C-7 to Figure C-9 show the hollow-core unit pull-off in the three stages of testing for the east and west beams. As expected, very little pull-off occurred in Phase II (Figure C-8(a) and (b)), with only 4mm occurring in the worst case. This increase in elongation was due to the hollow-core units applying an eccentric load to the beams by being supported near the edge of the beams rather than close to the axis of symmetry of the beam. This caused a rotation of the beams in torsion under this load which extended the potentiometer situated under the hollow-core units.

In all Phases, more elongation occurred in the first unit than any of the others with the least amount of elongation occurring in the fourth unit. This is because the southern frame undergoes a lot more elongation due to the elongation of the beam hinges, which results in more pull-off of the first unit than any others. In total, a maximum elongation of
(a) Displacement incompatibility in the positive drift cycles during Phase I testing

(b) Displacement incompatibility prediction using Ruaumoko.

Figure C-5 Displacement incompatibility, actual versus predicted for Phase I
approximately 19mm was experienced in the first unit at the west end before damage to the seat resulted in the loss of the target (Figure C-9(b)). All of the potentiometers situated under the ends of the hollow-core units were removed after the ±3.0% cycles in Phase III to ensure that the instrumentation would not be damaged by spalling concrete.

**C.4 Torsion Test**

At the end of the second cycle to ±2.0% in Phase II, the transverse loading cycle, a torsion test to ±0.5% drift was undertaken to determine the torsional stiffness of the super-assemblage. After the torsion test, a further cycle to ±2.0% was done to determine if the torsion test had any influence on the base shear capacity of the super-assemblage. The torsion test entailed displacing the east and west frames in opposite directions to ±0.5% drift (i.e. the East frame was displaced in a positive drift direction while the west frame was displaced in a negative drift direction while the central column was kept vertical and vice-versa, as seen by Figure C-10(a)). This introduced torsion into the longitudinal beams. However, no significant cracking was noticed other than a few small cracks in the centre of the southern beams.
Figure C-7 Hollow-core pull-off during Phase I

Figure C-8 Hollow-core unit Pull-off during Phase II

Figure C-9 Hollow-core Pull-off during Phase III
Figure C-10(b) shows the hysteresis loop for the east and west bays during the torsion test. The two bays vary in stiffness; when the east bay undergoes a positive drift it has a larger stiffness (10.0MN/m) than the corresponding west bay undergoing a negative drift (6.4MN/m). However, similar stiffness’s occur for both bays in the positive or negative drift directions. This can be explained by the presence of the infill slab. When each bay undergoes a negative drift, the southern hinge has a negative moment and the cracked and damaged infill slab in tension reduces the stiffness in that hinge and thus the bay. This is the same for both bays under a negative drift.

(a) Schematic of the torsion test (two cycles in each direction)

(b) Hysteresis loop for the torsion test

Figure C-10 Details of the torsion test
The overall hysteresis loops in section 3.3.2 show that the torsion test had very little influence on the base shear capacity of the frame. No significant drop in base shear capacity was observed after the torsion test and this is because very little damage was observed during the torsion test.

**C.5 Capacity Mechanism Calculations**

**C.5.1 Phase I and Phase III**

The capacity mechanism used to determine the theoretical push-over curve capacity of Phase I and Phase III were the same. The following calculations show how the values were determined.

**Initial Yield Point**

**Exterior Hinges:**

Figure C-11 shows the lever arms and activated flange at both the initial yield point and at full plastification for the exterior hinges. At the initial yield point three starter bars are activated in the tension flange for a negative moment and in the positive moment it is assumed that the compression block is roughly the depth of the flange section. The moments are transferred to column centreline moments in keeping with all other calculations. A similar mechanism occurs at the back tie beam, the nominal moment capacity of the beam is added with the activated starters. Therefore, the moments are calculated as follows:
Positive Hinge:

Beam Moment \( 525 \text{ kNm} \)

Convert to column centreline \( 697 \text{ kNm} \)

Negative Hinge:

Beam Moment \( 472 \text{ kNm} \)

Starter Bar Contribution \( 119 \text{ kNm} \)

\( \Sigma = 591 \text{ kNm} \)

Convert to column centreline \( 760 \text{ kNm} \)

Interior Hinges:

It is in the interior hinges that the folded plate mechanism comes into play, added to the beam moment capacity. As explained in section 3.3.2, the folded plate mechanism
is present in the early stages of testing but is overtaken by tensile membrane action around yield of the super-assembly.

Positive Hinge:

Beam Moment 525 kNm

Convert to column centreline 697 kNm

Negative Hinge:

Beam Moment 508 kNm

Yield Line Mechanism 30 kNm

\[ \Sigma = 538 \text{ kNm} \]

Convert to column centreline 713 kNm

where the calculations for the yield line mechanism are shown below:

From Figure C-12 and Table C-1 the following calculations for the internal work done can be made.

![Infill section with proposed crack pattern overlaid](image)

![Layout of yield line mechanism](image)

Figure C-12 Crack Pattern of the infill section and proposed yield line mechanism.
Table C-1 Calculation of the yield line mechanism.

<table>
<thead>
<tr>
<th>Yield Line</th>
<th>$\theta_x$</th>
<th>$\theta_y$</th>
<th>$M_1\theta_{xy}$</th>
<th>$M_2\theta_{yx}$</th>
<th>Total</th>
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<tr>
<td>A</td>
<td>1.29</td>
<td>-</td>
<td>$M_1 \times 1.29 \times 0.75$</td>
<td>-</td>
<td>0.97 $M_1$</td>
</tr>
<tr>
<td>B</td>
<td>1.14</td>
<td>-</td>
<td>$M_1 \times 1.14 \times 0.75$</td>
<td>-</td>
<td>0.86 $M_1$</td>
</tr>
<tr>
<td>C</td>
<td>0.94</td>
<td>-</td>
<td>$M_1 \times 0.94 \times 0.75$</td>
<td>-</td>
<td>0.71 $M_1$</td>
</tr>
<tr>
<td>D</td>
<td>0.70</td>
<td>-</td>
<td>$M_1 \times 0.70 \times 0.75$</td>
<td>-</td>
<td>0.53 $M_1$</td>
</tr>
<tr>
<td>E</td>
<td>0.70</td>
<td>-</td>
<td>$M_1 \times 0.70 \times 0.75$</td>
<td>-</td>
<td>0.53 $M_1$</td>
</tr>
<tr>
<td>F</td>
<td>0.23</td>
<td>-</td>
<td>$M_1 \times 0.23 \times 0.75$</td>
<td>-</td>
<td>0.17 $M_1$</td>
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<tr>
<td>G</td>
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<td>-</td>
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<td>-</td>
<td>3.75 $M_1$</td>
</tr>
<tr>
<td>H</td>
<td>-</td>
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<td>-</td>
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<td>1.74 $M_2$</td>
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<tr>
<td>I</td>
<td>-</td>
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<td>-</td>
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<tr>
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<td>-</td>
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<td>3.37 $M_2$</td>
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<td>-</td>
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<td>-</td>
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<td>3.05 $M_2$</td>
</tr>
<tr>
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<td>-</td>
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<tr>
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<td>-</td>
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<td>5.00 $M_2$</td>
</tr>
<tr>
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<td>-</td>
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<td>-</td>
<td>$M_3 \times 1.28 \times 1.25$</td>
<td>1.60 $M_3$</td>
</tr>
<tr>
<td>O</td>
<td>-</td>
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<td>-</td>
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<td>2.42 $M_3$</td>
</tr>
<tr>
<td>P</td>
<td>-</td>
<td>3.36</td>
<td>-</td>
<td>$M_3 \times 3.36 \times 0.75$</td>
<td>2.52 $M_3$</td>
</tr>
<tr>
<td>Q</td>
<td>-</td>
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<td>-</td>
<td>$M_3 \times 4.06 \times 0.75$</td>
<td>3.05 $M_3$</td>
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<tr>
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<td>-</td>
<td>$M_3 \times 4.77 \times 0.25$</td>
<td>1.19 $M_3$</td>
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<tr>
<td>S</td>
<td>1.29</td>
<td>1.29</td>
<td>$M_4 \times 1.29 \times 0.75$</td>
<td>$M_4 \times 1.29 \times 1.35$</td>
<td>2.71 $M_4$</td>
</tr>
<tr>
<td>T</td>
<td>1.14</td>
<td>1.14</td>
<td>$M_4 \times 1.14 \times 0.75$</td>
<td>$M_4 \times 1.14 \times 1.25$</td>
<td>2.28 $M_4$</td>
</tr>
<tr>
<td>U</td>
<td>0.94</td>
<td>0.94</td>
<td>$M_4 \times 0.94 \times 0.75$</td>
<td>$M_4 \times 0.94 \times 1.00$</td>
<td>1.65 $M_4$</td>
</tr>
<tr>
<td>V</td>
<td>0.70</td>
<td>0.70</td>
<td>$M_4 \times 0.70 \times 0.75$</td>
<td>$M_4 \times 0.70 \times 0.75$</td>
<td>1.05 $M_4$</td>
</tr>
<tr>
<td>W</td>
<td>0.70</td>
<td>0.70</td>
<td>$M_4 \times 0.70 \times 0.75$</td>
<td>$M_4 \times 0.70 \times 0.75$</td>
<td>1.05 $M_4$</td>
</tr>
<tr>
<td>X</td>
<td>0.23</td>
<td>0.23</td>
<td>$M_4 \times 0.23 \times 0.75$</td>
<td>$M_4 \times 0.23 \times 0.25$</td>
<td>0.23 $M_4$</td>
</tr>
<tr>
<td>Y</td>
<td>5.00</td>
<td>5.00</td>
<td>$M_4 \times 5.00 \times 0.75$</td>
<td>$M_4 \times 5.00 \times 0.75$</td>
<td>7.50 $M_4$</td>
</tr>
</tbody>
</table>

Total = $7.52 M_1 + 19.78 M_2 + 10.78 M_3 + 16.47 M_4$
Yield Line 1 – mesh in topping = 3.9kNm/m
Yield Line 2 – mesh in topping = 3.9kNm/m
Yield Line 3 – starters in topping = 8.0kNm/m
Yield Line 4 – Starters and mesh in topping = 12.4kNm/m

Therefore, the sum of the internal work done = 39.6 \delta (assuming \delta = 10)

External work done = M\theta but \delta = \theta \times 0.75.

Therefore, external work done = \frac{M\delta}{0.75} \quad \text{(C-12)}

Finally, by equating external and internal work;

IWD = EWD

\[ 39.6 \delta = \frac{M\delta}{0.75} \quad \text{(C-13)} \]

M = 30 kNm

Finally, the total base shear at first yield can be calculated:

\[ \sum V_{cat} = \sum \frac{M_k}{h_c} = \pm 890kN \]

**Full Plasticisation**

**Exterior Hinges:**

At full plastification, 3.05m of the starters in the transverse beam contribute to the front frames capacity (this equals half the bay length in this case) with the other half contributing to the back tie beam. For the front frame, the capacities are calculated as:

Positive Hinge:

Beam Moment 525 kNm
Negative Hinge:

Beam Moment 472 kNm
Starter Bar Contribution 353 kNm
\[ \Sigma = 825 \text{ kNm} \]

Convert to column centreline 1084 kNm

Interior Hinges:

At full plastification (2.0% drift), the yield line mechanism has been taken over by a tension membrane mechanism as the floor is put more and more into tension and the infill is unable to sustain the moment couple along the hollow-core/infill interface. This leads to a slight drop in moment capacity:

Positive Hinge:

Beam Moment 525 kNm

Convert to column centreline 697 kNm

Negative Hinge:

Beam Moment 508 kNm
Tension membrane contribution 24 kNm
\[ \Sigma = 532 \text{ kNm} \]

Convert to column centreline 705 kNm

Finally, the total base shear at full plastification can be calculated:

\[ \sum V_{cal} = \sum \frac{M_h}{h_c} = \pm 1062 \text{ kN} \]

C-20
C.5.2 Phase II

The mechanism used to determine the individual hinge theoretical moment capacities is similar to that used in Phase I and III with beam moments and yielding starters. However, in Phase II, relocated hinges in the positive hinge direction increased the beam moments in all cases. The centre of the negative hinges were $0.5D$ from the column face as well, where $D$ = the overall depth of the member. The following calculations show how the values used in the theoretical push-over curve were determined for Phase II.

**Initial Yield Point**

**Northern Hinges:**

The positive moment hinge is made up of a relocated uni-directional hinge whereas the negative moment hinge forms at $0.5D$ from the column face.

Positive Hinge:

- Beam Moment – relocated hinge 460 kNm
- Convert to column centreline 738 kNm

Negative Hinge:

- Beam Moment 460 kNm
- Convert to column centreline 634 kNm

**Southern Hinges:**

In the southern end of the transverse beams a smaller beam cage was used in order to allow the longitudinal beam bars to cross in the South-West and South-East beam column joints. Therefore, the nominal beam moment is smaller. The positive moment hinge in the southern hinge consists of a relocated, one-way hinge forming
0.75m from the column face with the compression block assumed to be roughly the depth of the flange section. In the negative moment, the capacity consists of the beam moment forming \(0.5D\) from the column face with activated starter bars contributing to the strength of the hinge.

Positive Hinge:

Beam Moment – relocated hinge \(420\) kNm

Convert to column centreline \(774\) kNm

Negative Hinge:

Beam Moment \(420\) kNm

Starter Bar contribution \(105\) kNm

Convert to column centreline \(684\) kNm

Finally, the total base shear at first yield can be calculated:

\[
\sum V_{col}^+ = \sum \frac{M_b}{h_c} = 805kN \quad \text{and} \quad \sum V_{col}^- = \sum \frac{M_b}{h_c} = -820kN
\]

The reason that the positive and negative base shear are not equal is due to the non-symmetrical reinforcement layout of the specimen.

**Full Plastification**

The capacity of the northern hinges does not change at full plastification (2.0% drift) because there is no additional reinforcing to contribute to the base shear capacity. However, the negative moment hinge capacity in the southern hinges is increased due to more starter bar activation in the flange at the beam/infill interface.

**Southern Hinges:**

Positive Hinge:
Beam Moment – relocated hinge  
420 kNm

Convert to column centreline  
774 kNm

Negative Hinge:

Beam Moment  
420 kNm

Starter Bar contribution  
313 kNm

Convert to column centreline  
950 kNm

Finally, the total base shear at full plastification can be calculated:

\[ \sum V_{col}^+ = \sum \frac{M_b}{h_e} = 805 kN \quad \text{and} \quad \sum V_{col}^- = \sum \frac{M_b}{h_e} = -977 kN \]

The individual hinge capacities for all three loading phases are shown in Figure C-13.

### C.6 Beam Elongation

Beam elongation occurred in both the longitudinal and transverse beams in all phases of testing. However, in Phase I loading, very little beam elongation was seen in the transverse beams even though the beams exhibited cracking. In Phase II, the longitudinal beam hinges appeared to undergo negative elongation or recovery. This could have been due to the significant large cracks at the column face closing further under the transverse loading, a relaxation recovery or an error in the instrumentation. The recovery was only 2mm in total across the four hinges.

The determination of the value of \( e_{cr} \) is very important in determining the beam elongation for a hinge. It is strongly affected by the type of support, i.e. an end support hinge or an interior hinge, and by the reinforcement ratio. The value of \( e_{cr} \) is the distance between the centre of the compression force and the beam centreline in both directions of loading. This means that the amount of reinforcing present in the beam and topping influences the force eccentricity. In this stage of testing the same values of \( e_{cr} \) were used...
Figure C-13 Individual hinge strengths – experimental and theoretical push-over
as in Matthews et al (2004) because the same frame was used and the change in topping reinforcement did not alter the force eccentricity by a significant amount. It was also felt that the presence of the hollow-core units, even situated away from the beam, would still influence the elongation of the interior hinges, by restraining the growth, enough to warrant the use of the reduced $e_{cr}$ values for the central column hinges.

### C.6.1 Phases I and III

Figure C-14 shows the elongation for the individual southern hinges, plotted on the column lines, for Phases I and III. In Phase I (Figure C-14(a)) the interior hinges did not elongate as much as expected in the -2.0% cycles, in fact the two hinges appeared to undergo negative elongation in this cycle. This could be due to an instrumentation fault, damage in the infill affecting the hinge performance or the self-centring type connections clamping back around the column due to the prestressing in the hollow-core units. However, the interior hinges were better predicted in Phase III.

### C.6.2 Phase II

Figure C-15 shows the individual hinge performance of the four hinges in the Phase II loading. Figure C-15(a) shows the transverse east beam hinges while Figure C-15(b) shows the transverse west beam hinges. The southern hinges performed better than the northern hinges in the early stages of testing in Phase II. This is because the elongation was suppressed by the presence of the torsion cracks, from Phase I loading, that were more prevalent in the northern hinges. This meant that initially the northern hinges did not develop as expected. Once sufficient flexural cracking developed in the hinge (in the ±2.0% cycle) the beam elongation matched the data better.
Figure C-14 Predicted versus experimental beam elongation for the longitudinal loading phases.
Figure C-15 Predicted versus experimental beam elongation for the transverse loading phase.
C.7 Low Cycle Fatigue of Main Longitudinal Reinforcing Bars

As explained in section 3.6 the failure of the main longitudinal beam bars can be predicted by low cycle fatigue theory (Mander et al, 1994 and Dutta and Mander, 2001). The following calculations show the results:

Plastic rotation fatigue-life capacity:

\[
\theta_p = 0.16 \left( \frac{D}{D_L} \right) \left( \frac{L_p}{D} \right) \left( 1 - \frac{L_p}{2L} \right) \left( 2N_f \right)^{0.5}
\]  \hspace{1cm} (C-14)

where \( \theta_p \) = plastic drift; \( L \) = lever arm of the cantilever column; \( D \) = overall member depth; \( D' \) = distance between the outer layers of steel and \( 2N_f \) = number of reversals to the appearance of first fatigue crack where \( N_f \) is the effective number of constant amplitude cycles for a variable amplitude displacement history and \( L_p \) = equivalent plastic hinge length given by

\[
L_p = 0.08L + 4400\varepsilon_y d_b
\]  \hspace{1cm} (C-15)

where \( \varepsilon_y \) = yield strain and \( d_b \) = diameter of the longitudinal reinforcement and is the generalized form of the equation proposed by Paulay and Priestley (1992).

Miner’s well-known cycle counting method can be used to determine the equivalent number of cycles \( (N_f) \):

\[
N_f = \sum_i \left( \frac{\theta}{\theta_{ref}} \right)^2
\]  \hspace{1cm} (C-16)

where \( \theta \) = drift for the \( i^{th} \) cycle of loading and \( \theta_{ref} \) = reference drift for an equivalent constant amplitude loading history. Table C-2 shows the calculations for the equivalent number of constant amplitude cycles.

C-28
Table C-2. Determination of the number of effective cycles

<table>
<thead>
<tr>
<th>Drift amplitude (θ)</th>
<th>Number of cycles (n)</th>
<th>$n \left( \frac{\theta}{\theta_{\text{max}}} \right)^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>0.125</td>
</tr>
<tr>
<td>2</td>
<td>3</td>
<td>0.75</td>
</tr>
<tr>
<td>3</td>
<td>2</td>
<td>1.125</td>
</tr>
<tr>
<td>4</td>
<td>1.3</td>
<td>1.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\Sigma = 3.3 \text{ cycles}$</td>
</tr>
</tbody>
</table>

The value for the plastic rotation capacity of the longitudinal main bars is therefore:

$$\theta_p = 0.16 \left( \frac{0.75}{0.632} \right) \left( \frac{0.348}{0.75} \right) \left( 1 - \frac{1}{2 \times 2.675} \right) \left( 2 \times 3.3 \right)^{-0.5} = 0.032 \text{ rad} \quad (C-17)$$

When the value of the elastic recovery component is added to the plastic component, the total rotation capacity can be determined:

$$\theta_r = \theta_e + \theta_p \quad (C-18)$$

where $\theta_e$ is determined to be the elastic recovery component after several cycles of loading and is equal to 0.008 from experimental observations. This leads to a total rotation capacity of 0.04 radians.

This compares well with the observed capacity of 0.04 radians being the maximum drift capacity of the longitudinal beam bars.
C.8 References


