Experimental method and results from seismic testing of a two-storey, two-by-one bay, reinforced concrete slotted beam superassembly

C.A. Muir & S. Pampanin
University of Canterbury, New Zealand

D.K. Bull
University of Canterbury, New Zealand
Holmes Consulting Group, New Zealand

SUMMARY:
Displacement incompatibility between reinforced concrete moment frames and precast flooring systems during earthquakes has been shown experimentally, and in numerous earthquakes, to be an area of concern. Plastic hinge formation necessitates beam damage and the resulting elongation of the beam reduces the seating length of the floor, exacerbates the floor damage and induces unanticipated force distributions in the system. In severe cases this can lead to collapse.

The slotted beam is a detail that protects the integrity of the floor diaphragm, respects the hierarchy of strength intended by the designer and sustains less damage. The detail provides the same ductility and moment resistance as traditional details, whilst exhibiting improved structural performance. This is achieved with only a subtle change in the detailing and no increase in build cost.

This paper presents the experimental set up to test a large scale reinforced concrete slotted beam superassembly. The testing is described and results presented.

Keywords: Reinforced Concrete, Slotted Beam, Seismic Design, Low Damage, Experiment

1. INTRODUCTION

This paper is intended to be read in conjunction with the companion paper on the background, design and construction of specimen SA1 presented at this conference by Muir et at., (2012).

The above paper presents the development of the slotted beam detail for reinforced concrete moment resisting frames. The conceptual and detailed design of the two storey, two-by-one bay, superassembly is described. The manufacture of the precast concrete components is presented, and the erection of the specimen detailed.

Following on from the aforementioned paper, this paper presents the experimental set up, testing and results from a large scale experiment on a reinforced concrete slotted beam superassembly.

2. EXPERIMENTAL METHOD

Specimen SA1 was founded on double pin universal joints to represent the points of contraflexure in the columns. These were fixed in translation to recreate the fixity of the ground floor columns. Issues with excessive movement were encountered with these universal joints. This was mitigated through shimming. However, to avoid this issue true spherical universal joints are recommended.

The applied lateral displacement protocol was biaxial to simulate a realistic and worst case earthquake loading scenario. The protocol was based on ACI 371.1-05 (ACI Committee 374, 2005). However, because this standard specifies uniaxial loading only a combined uniaxial and biaxial protocol was required to enable the loading to conform to the standard along both axis. The applied displacement protocol can be seen in Figure 2.1. Loading was defined in terms of beam drift, which is the relative rotation between the beam and column. This is more generic than column drift and allows specimens with different geometries to be compared. The biaxial component was ‘cloverleaf’ in shape, this more demanding than a traditional 45° excursion as it enables different combinations of moments to be generated in the orthogonal beams.

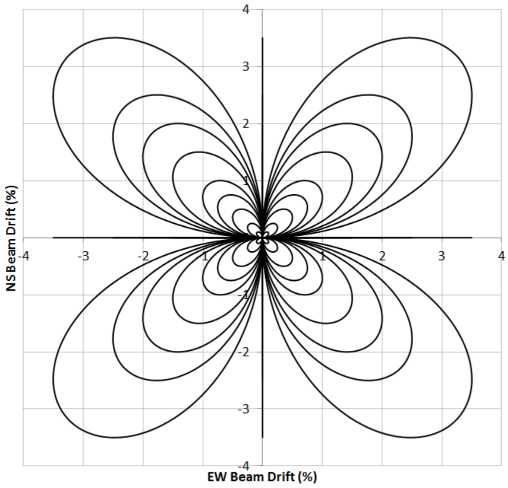


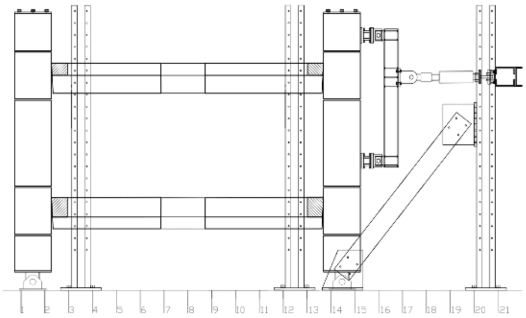
Figure 2.1. Combined uniaxial and biaxial displacement protocol.

Force was applied to the specimen through the columns at a constant force ratio of 2:1 between the top and bottom floors respectively. Because the specimen represents the lower two stories of a seven storey structure the inertial forces would be small in comparison to the column shear from the floors above. A perspective of the final experimental set up can be seen in Figure 2.2 and the elevations and plans in Figures 2.3 (a) – (c). The loading was quasi-static in nature. The laboratory is not capable of pseudo-dynamic or high speed testing on this scale at present. Even with a reduced loading rate the forces required in this test were at the maximum of what is achievable given the current laboratory facilities. A full analysis of the structural extension laboratory strong floor was undertaken to assess capacity and safe working limits.

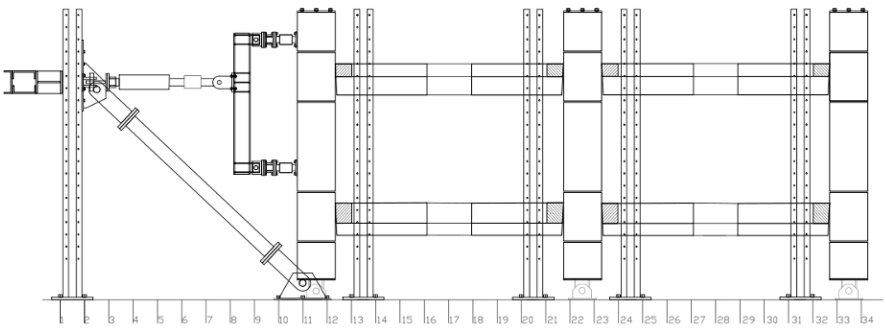
Many issues were encountered controlling the five hydraulic rams displacing the specimen. A program to control the rams through the precise displacement protocol was developed. However, it was found that the methods that had been applied to smaller and less complex specimens did not work effectively in this more complex, stronger and stiffer specimen. As a consequence that rams would interact with each other, both along the same axis and against the orthogonal. This caused the rams forces to increase over numerous displacement increments until the maximum allowable force limits were reached.


Figure 2.2. Experimental setup perspective.

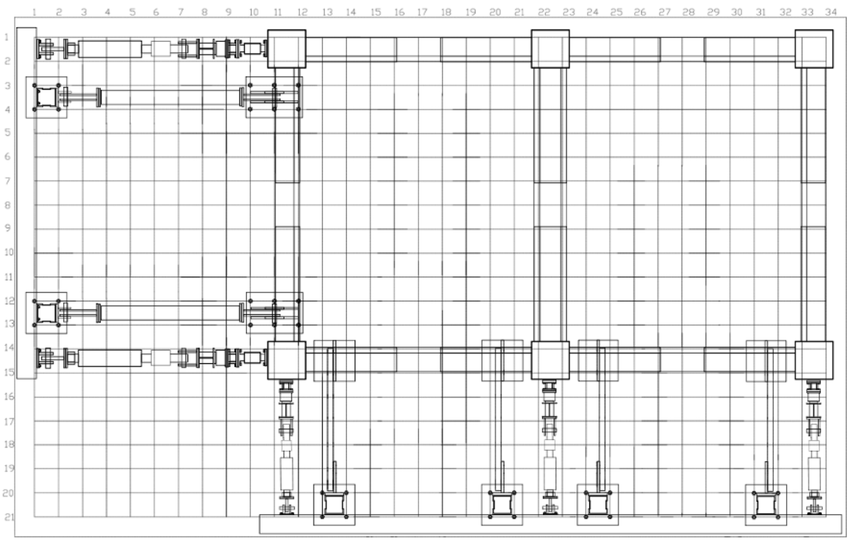
The issue was addressed through several methods. Firstly, a hybrid of displacement and force control was used. Pure displacement control caused severe ram interaction, however pure force control does not allow precise application of the intended displacement protocol. A compromise had to be sought between the two. The order which the rams were fired in was adjusted, this helped to reduce specimen torsion influencing the orthogonal rams, and specimen bending influencing rams long the same axis. Finally, the ram controller microchips were forced to shut down after reaching the required displacement or force target. In the past these have remained live to maintain the required targets; however this meant that a situation could develop where rams were pushing each other off their targets to maintain their own, which causes forces to increase without bound. However, the continued development of the controller software, in conjunction with selective weakening, allowed this issue to be largely mitigated and was a significant outcome of this project.



(a) North-South Elevation.



(b) East-West Elevation.



(c) Plan.

Figure 2.3. Experimental setup drawings.

The specimen was extensively instrumented to capture all aspects of connection behaviour, the influence on other structural elements and the resulting deformation in the floor diaphragm. In total 587 strain, displacement and rotation channels were logged. A large number of logging boxes were required, both due to the number of channels and the physical distance between instruments. A consequence of this was issues with voltage supply, and subsequently operation of the microchips within the logging boxes. In addition, measurement of the floor strain was undertaken using demountable mechanical strain gauges. This involved approximately 40,000 manual measurements.

3. TEST SUMMARY

Extensive cracking had occurred in the specimen prior to testing. This was a result of shrinkage strain and the on-going aftershocks in the Christchurch region. The first cracking in the frame, during testing, was observed at 0.2% beam drift. The cracking was hairline and concentrated at the beam ends and column precast connections.

First cracking in both the dapped double-tee and hollow-core flooring was observed at 0.5% beam drift. This was minor and did not comprise gravity support. Debris started to be ejected from the slotted region at this displacement. This was expected and consisted of a mixture of residual slot formwork, reinforcement scale and cement paste that had adhered to the unbounded reinforcement. During testing to this displacement, excessive movement was observed in the universal joints the specimen is founded upon. The specimen had to be jacked up and the universal joints shimmed to mitigate this issue.

Specimen yield occurred at approximately 0.7% beam drift during the 0.75% cycle. Cracking within the seating of the precast floor continued. The first signs of warping damage within the floor appeared around columns. Activation of the timber infill and double-tee flange of the bottom and top floors respectively occurred to accommodate the displacement incompatibility between the frame and floor systems. During this displacement cycle, the ram force limit in one of the EW rams was reached. This was caused by interaction between the rams in the East-West and North-South directions and a higher than anticipated system lateral resistance. This issue was mitigated through loading program development and selective weakening. Selective weakening was undertaken through removal of a number of longitudinal bars. This reduced the lateral capacity of the system. The removed bars were selected so as to not influence the overall specimen response and results of the test.

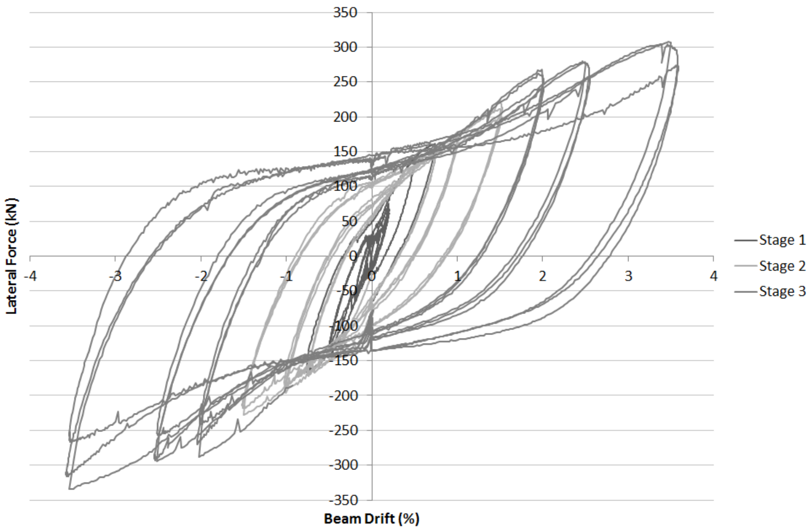
During the 1.0% cycle displacement incompatibility issues were encountered with the North-South rams. This was addressed thorough the fabrication of new ram connectors. Specimen damage was mainly restricted to widening of existing cracks. However, the first sign of concrete cone pull out of the lower longitudinal reinforcement in the column was observed. This is caused by strain penetration of the bars into the beam-column joint. During the 1.5% cycle, the maximum ram force limit was again reached. The rams had to be driven manually to bring the specimen back to a neutral position. The issue was mitigated in the same manner as previously. However, the latest version of the controller software worked significantly better than the previous and the issue alleviated.

The first signs of torsional cracking in the East-West beams were observed during the 2.0% cycle. Partial spalling of the hollow-core seating occurred up to the ledge reinforcement. This did not affect gravity support. During the 2.5% cycle strain penetration in to the column worsened, as did warping damage. Full activation of the timber infill and double-tee flange had now occurred. Other existing cracks lengthened.

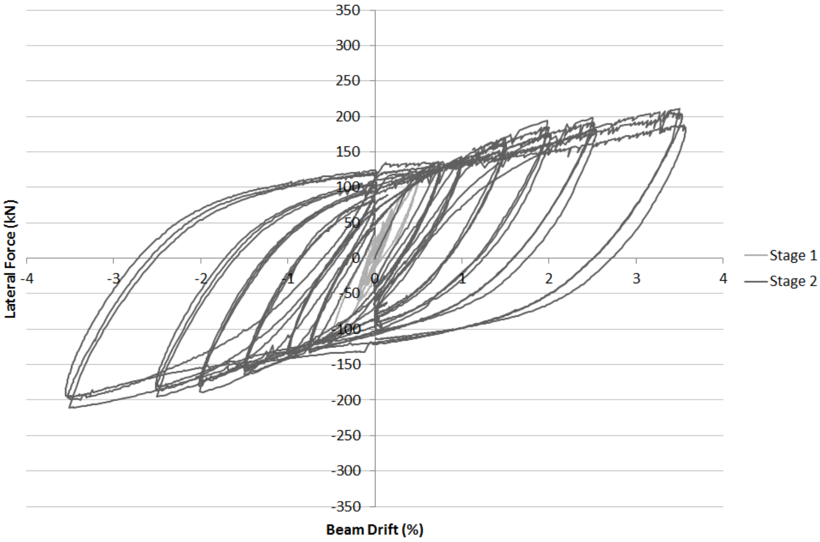
During the 2nd and 3rd cycles at 3.5% beam drift the lateral resistance softened slightly. Extensive compression spalling at the top of the concrete hinge was now occurring. The main tensile crack that develops through the top hinge during closing rotations was now almost the entire depth of the top hinge. One lower longitudinal bar in the North-South direction fractured.

4. EXPERIMENTAL RESULTS

In general, the overall performance of SA1 was extremely satisfactory. A stable response was observed to 3.5% beam drift with high levels of hysteretic energy dissipation observed. The overstrength in both directions was larger than anticipated based on previous experimental tests (Au, 2010). Larger overstrength was observed in recent subassembly tests (Byrne, 2012), which suggests that the floor alone, acting as a flange to the beam, is not responsible for the overstrength. It can be seen in Figures 4.1 (a) and (b) that while the shapes of the global hysteretic responses are similar; the East-West direction has a more pronounced post-yield strength gain. This discrepancy between the two directions is likely due to the one-way flooring system, which runs parallel to the East-West frames. Stages 1, 2 and 3 in the plots refer to the aforementioned selective weakening. The plots have been normalised to account for this. There was little difference in observed response between uniaxial and biaxial loading. This means that the two directions can effectively be decoupled for lateral design purposes. However, biaxial demands still need to be considered on an elemental level, such as joint and column design.



(a) East-West direction.

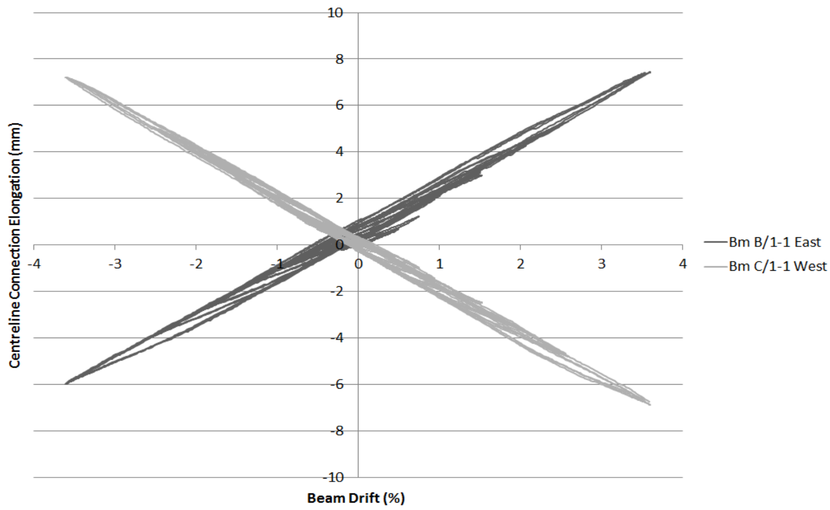


(b) North-South direction.

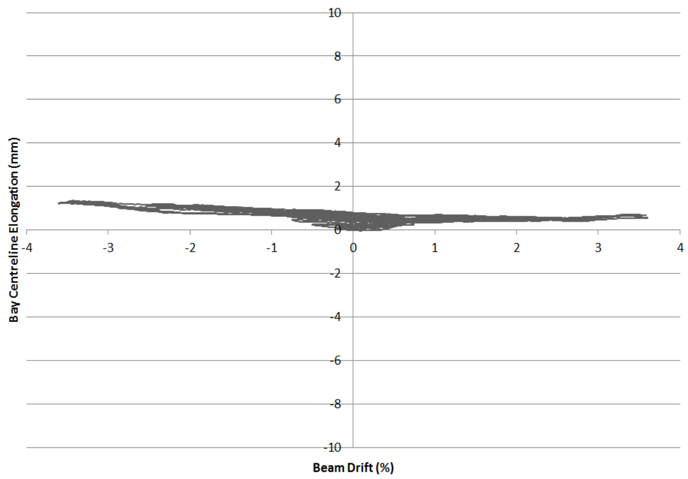
Figure 4.1. Global hysteretic response.

The beam elongation recorded in specimen SA1 was significantly lower than would be expected in a structure with traditional monolithic detailing. Figure 4.2 (a) shows the beam elongation recorded at the level 1 interior and exterior connections. Elongation is recorded at the centreline of the beams, so opening and closing rotations are recorded as effective elongation. However, the important feature is residual elongation as it returns to the origin. It can be seen in Figure 4.2 (b) that the effective elongation at each end of the beams, which is caused by slot opening and closing, are opposite and cancel each other out. Hence, the bay elongation, which is what causes structural issues, is vastly reduced compared to traditional monolithic details.

Interior connections tend to exhibit less beam elongation than external due to restraint by the floor. In traditional monolithic reinforced concrete structures it is typical to record approximately 1.5% and 4% of beam depth for interior and exterior connections respectively (Peng, 2009). Whereas the slotted system peaked at 0.3% and 0.5%, or approximately eight times lower. This reduces damage to the floor, reduces beam flexural overstrength and preserves the hierarchy of strength that the designer intended.



(a) Centreline connection elongation.



(b) Total bay elongation.

Figure 4.2. Beam elongation.

The damage sustained by specimen SA1 at the ultimate limit state was far less than would be expected in a monolithic structure. The damage sustained at serviceability limit state would not require any repair. Recommendations by Priestley et. al. (2007) are used to define limit states. Figures 4.3 (a) and (b) compare the damage to the beam end for specimen SA1 and a monolithic reinforced concrete detail at the ultimate limit state. It is clear that the slotted beam has sustained significantly less damage. With the exception of the primary crack through the top concrete hinge, the cracks are typically less than 0.15mm. Strain penetration and cone-type pull out mechanism was observed in the lower longitudinal reinforcement. Full development of this damage coincided with the observed softening of response during the 2nd and 3rd cycles at 3.5% beam drift. Supplementary reinforcement welded to the lower longitudinal reinforcement through the interior connection has been effective in reducing the amount of strain penetration developed and increasing bond.

Figures 4.3 (c) and (d) compare the damage to the hollow-core floor diaphragm for specimen SA1 and a monolithic reinforced concrete system at the ultimate limit state. Again, it is clear that there is significantly less damage in the slotted beam system. The cracks in the slotted system are fewer and smaller, peaking at approximately 0.45mm. Whereas the traditional system has developed an 8mm tear between the infill and the floor. The absence of diagonal cracks in the slotted system is indirect evidence of reduced beam elongation. Beam elongation is restrained by the floors, inducing compressive force in the beams and tension stresses in the floors. The diagonal cracks form as the force struts between the two. This is termed the “bowstring effect” (Matthews, 2004).

(a) Slotted beam end damage.

(b) Monolithic reinforced concrete plastic hinge damage (MacPherson, 2005).

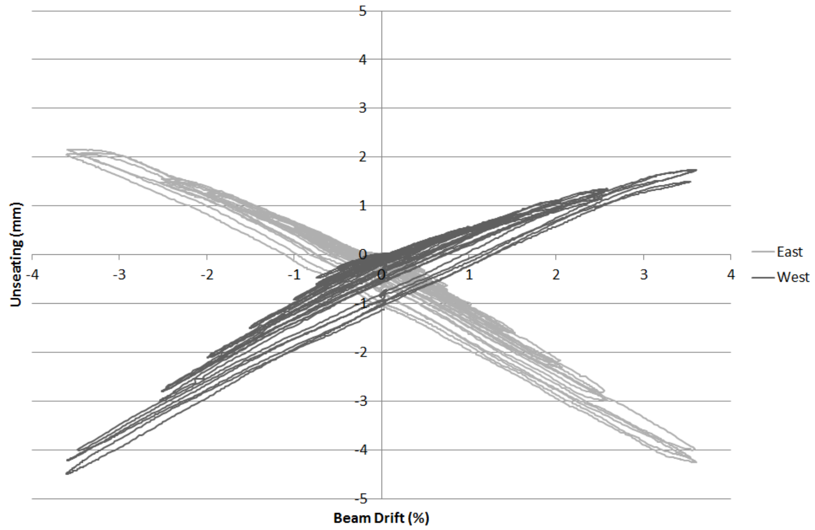
(c) Timber infill cracks, less than 0.45mm.

(d) Timber infill cracks, approximately 8mm (Lindsay, 2004).

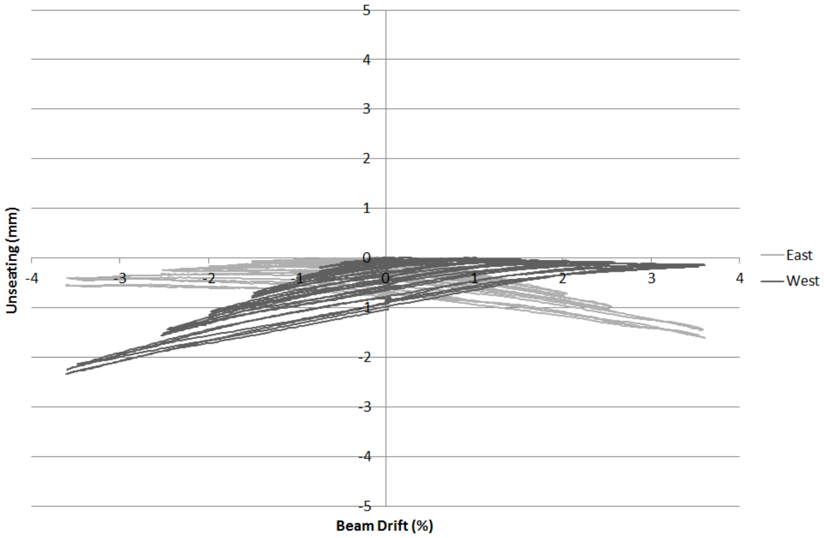
Figure 4.3. Comparison of ultimate limit state damage in slotted specimen SA1 (L) and a conventional monolithic reinforced concrete structure (R).

Due to the width of the columns large warping deformations are induced in the floor. These deformations were able to be accommodated by the infill and double-tee flange on the first and second stories respectively. Minor delamination of the topping and the precast flooring was observed in both stories around the warped sections.

It is imperative the gravity support of the floor diaphragm is not lost during an earthquake. In a system such as SA1 with one-way precast flooring the key indicator is floor unseating. Figures 4.4 (a) and (b) show the floor unseating for the the northern precast units on the first and second stories respecively. It can be seen that although the behaviour of the two types of floor differs due to the depth of the floor, the resisdual unseating is similar and peaks at approximately 1.2mm. This equates with the recorded beam elongation in the slotted beams. Due to the pragmatic approch taken in designing the supporting ledge width this seating loss is easily accommodated and will not affect current or future performance.



(a) North Unit Level 1.

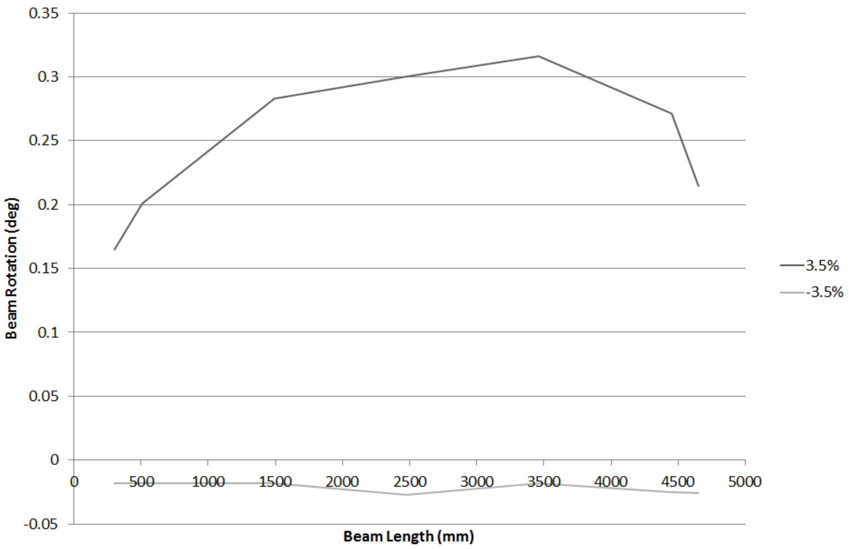


(b) North Unit Level 2.

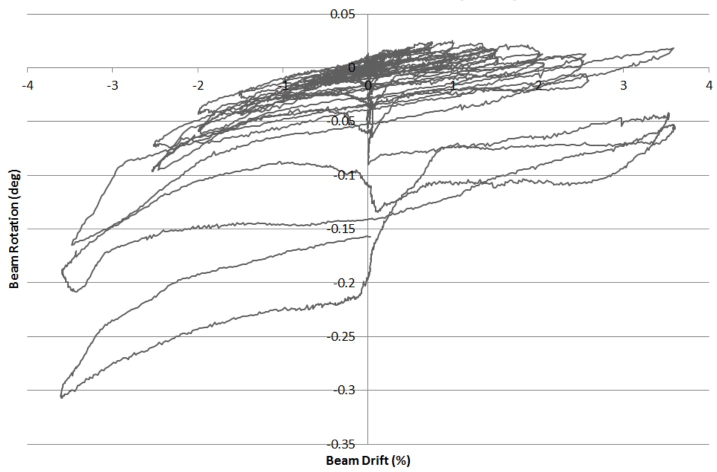
Figure 4.4. Floor unseating.

Torsional demand on the exterior beams is greater in one direction than the other due to the eccentric loading from the precast floors. This can be seen in Figure 4.5 (a) with greater torsional deformation along the beam length for this direction. In the slotted beam, the confined concrete core that is relied upon to transfer torsion in a traditional reinforced is significantly reduced; instead a force couple between the diagonal shear hangers is relied upon. For this reason a three-hanger detail is used to increase capacity in this direction (Muir, et al., 2012).

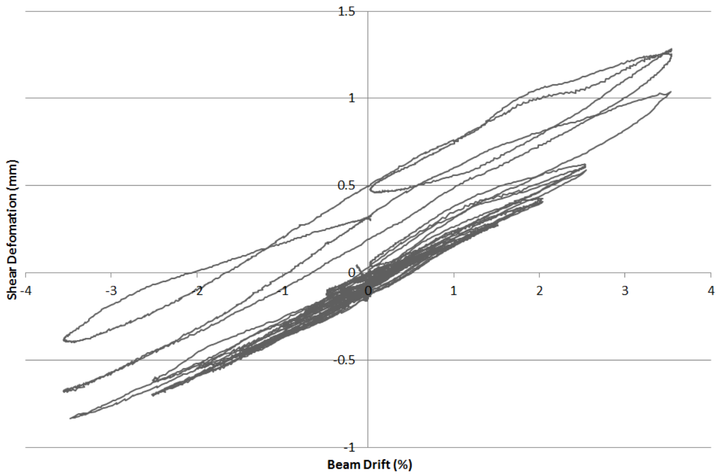
The diagonal hangers are heavily loaded in combined shear, torsion and flexure. Due to the interaction between these modes, ideally the demand should be kept within each modes elastic capacity. Figures 4.5 (b) and (c) show the shear and torsional deformation of Beam C/1-1. It is clear from these plots that the response is elastic up to 2.5% beam drift, during the 3.5% drift cycle there is limited plasticity in the hangers. This displacement represents a state beyond ultimate limit state and may be justifiable depending on the designer's priorities in the structural design. Certainly, plasticity would be undesirable during design level loading.



(a) Peak beam torsion along length.



(b) Beam C/1-1 torsional deformation.



(c) Beam C/1-1 end shear deformation.

Figure 4.5. Beam deformation on level 1 grid C.

The next phase of this research will involve the extraction of two subassemblies from superassembly SA1. These are exterior beam-column connections with precast prestressed floors. These will be tested to destruction to determine the residual capacity, which is a critical consideration when assessing a structure for future use. The subassemblies will then be retrofitted using external replaceable dissipaters to assess the viability for use in both retrofit, and new build applications.

5. CONCLUSIONS

The experimental method for testing a large scale reinforced concrete slotted beam superassembly has been described. The experimental testing has been summarised and the selected results presented and discussed. The experimental program has demonstrated that the reinforced concrete slotted beam is a viable substitution for the monolithic detail. Extremely promising structural performance was observed. High energy dissipation and stable response were observed to 3.5% beam drift. The system displayed reduced levels of damage to the frame and floor diaphragm when compared to traditional monolithic details. Beam elongation for the slotted beam was shown to be significantly less than traditional reinforced concrete beams. Floor unseating is very low and does not affect structural performance during an earthquake, or subsequently. The diagonal hangers at the ends of the beams are very heavily loaded and a conservative design must be undertaken to ensure the integrity of the structure.

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REFERENCES

ACI Committee 374. (2005). Acceptance Criteria for Moment Frames Based on Structural Testing and Commentary. Farmington Hills, Michigan.

Au, E. (2010). The mechanics and design of a non-tearing floor connection using slotted reinforced concrete beams. University of Canterbury, Christchurch, New Zealand.

Byrne, J. D. R. a. Bull., D. K. (2012). Design and testing of reinforced concrete frames incorporating the slotted beam detail. *New Zealand Society for Earthquake Engineering*.

Lindsay, R. (2004). Experiments on the seismic performance of hollow-core floor system in precast concrete buildings. University of Canterbury, Christchurch, New Zealand.

MacPherson, C. (2005). Seismic performance and forensic analysis of a precast concrete hollow-core floor super-assemblage. University of Canterbury, Christchurch, New Zealand.

Matthews, J. G. (2004). Hollow-core floor slab performance following a severe earthquake. University of Canterbury, Christchurch, New Zealand.

Muir, C. A., Bull, D. K., & Pampanin, S. (2012). Background, design and construction of a two-strorey, two-by-one bay, reinforced concrete slotted beam superassembly. *15th World Conference on Earthquake Engineering.*

Peng, B. H. H. (2009). Seismic performance assessment of reinforced concrete buildings with precast concrete floor systems. University of Canterbury, Christchurch, New Zealand.

Priestley, M. J. N., Calvi, G. M., & Kowalsky, M. J. (2007). Displacement-based seismic design of structures. Pavia, Italy: IUSS Press.