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Background to the testing of a precast concrete hollowcore floor slab building

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ABSTRACT: Concern has been raised about the expected performance of some of New Zealand's precast concrete buildings during a severe earthquake. The intent of this paper is to give some insight into seismic performance issues. A principal concern is the affect that beam elongation has on seat width requirements for precast floor slabs. An experimental programme undertaken at the University of Canterbury is described.

1 INTRODUCTION

Precast concrete buildings that use prestressed hollowcore floor units have been the dominant form of construction used in New Zealand over the last two decades. Failures of several hollowcore flooring systems were observed after the Northridge earthquake (17 January 1994) have raised serious concern regarding the seismic performance and integrity of New Zealand's precast concrete multi-storey moment resisting frame buildings. Several buildings in Northridge collapsed as a result of the hollowcore flooring units loosing their seating from the supporting beams (Norton et al 1994), see Figure 1. Once the beam support was lost, the units collapsed onto the floor below causing a concertina effect with other floors. When the floor units lost their support they failed in one of three manners. The first being collapsing as a complete unit where the floor unit and topping came down in one piece (see Figure 1(a)). Another was when the support from the beam was lost, the hollowcore floor unit delaminated from the topping concrete and the unit dropped (Figure 1(b)). The third failure mechanism was when the webs of a hollowcore unit split once the support was lost (Figure 1(b)). This meant that part of the hollowcore unit and all the topping was left suspended by the beam while the remainder of the unit collapsed onto the floor below.

Following these observed failures in the 1994 Northridge earthquake a major research initiative has been undertaken at the University of Canterbury, to determine whether New Zealand designed and built structures have similar problems, and if so, to what extent these problem exist in a New Zealand context and what can be done about mitigation.

In order to test the performance of a precast concrete building constructed to the New Zealand Concrete Standard (Standards New Zealand 1995), a full size super-assemblage of a building was constructed in the University of Canterbury structures laboratory. By constructing a super-assemblage, it is possible to recreate the boundary conditions as they would exist in a real structure. Previous studies carried out at the University of Canterbury only focused on the individual components of a building. This project focuses on investigating the interaction of column-beam-slab performance of the large super-assembly.

(a) Complete collapse of a floor slab

(b) Partial collapse of a floor slab.

Figure 1. Figures observed after the Northridge Earthquake (17th January 1994)

The principal aim of this project is to focus on the floor-frame interaction and the effect that beam elongation has on the required seating lengths for hollowcore floor units. The strength enhancement to the perimeter beam negative moment capacity due to beam elongation will also be examined. The experimental evidence based on determining seismic capacities will be integrated into a computational analysis of seismic demands of a selection of low, medium and high rise frames. By investigating the balance between seismic capacities versus the demands associated with variable hazard exposure it will be possible to make recommendations on seat width requirements. Moreover, insight will be given into the seismic vulnerability of the existing building stock with precast concrete floor systems.

2 SPECIMEN DETAILS

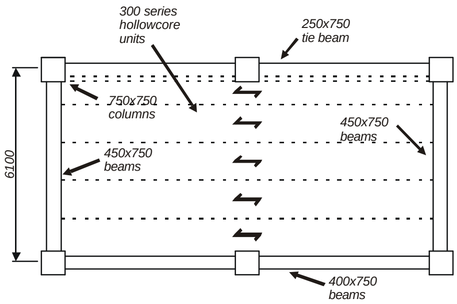
This test specimen represents a lower storey in a typical precast concrete building. The flooring system consists of 300mm deep hollowcore units with a 75mm cast insitu topping spanning 12m. The hollowcore unit itself spans past the central column, as this is a common detail used in New Zealand, and is seated on the two end beams with a nominal seat width of 50mm. Their actual length is 20mm on the east beam and 40mm on the west beam. These provided seats are considered to be representative of the range of seat width adopted in the field over the past two decades. Figure 2 shows the super-assemblage dimensions.

A complex test rig was developed for this large scale experiment. The test rig was required to apply realistic loads to the structure so that the specimen deforms in the correct manner. Special care was taken to ensure that any beam elongation that develops during the course of the experiment is neither promoted nor restrained by the lateral loading apparatus.

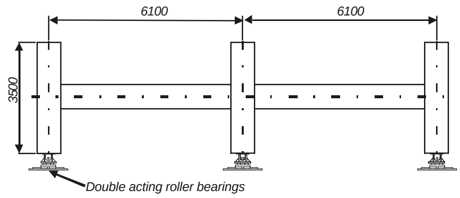
Figure 2(c) and 2(d) shows the loading frame set ups for both the longitudinal and transverse loading directions. The two main loading frames are the diagonal frames and they apply the shear forces to the columns. A set of secondary loading frames (that resemble an arrow shape) are provided to enforce displacement compatibility of the adjoining stories. The secondary frames ensure the drift angle on each column is the same.

3 BEAM ELONGATION

During a severe seismic attack, buildings that have been designed in accordance with modern codes behave by a preferred manner whereby a beam sidesway mechanism forms with plastic hinges at each end of the beams. Once plastic hinges form in a beam and the beam undergoes large inelastic rotations, the beam grows in length. This phenomenon has been demonstrated in various experimental studies undertaken by several groups of researchers (Douglas & Davidson 1992, Davidson & Fenwick 1993, Fenwick & Megget 1993, Fenwick & Fong 1979 and Restrepo et al 1993). The 1994 Northridge earthquake has also shown this elongation occurring.



(a) Plan



(b) Elevation

(c) Longitudinal loading set up

(d) Transverse loading set up

Figure 2. Super assemblage dimensions

The mechanics of beam elongation can be explained by referring to Figure 3 (Matthews et al 2001). This example describes the elongation for a typical plastic hinge zone where there is more top reinforcement in the beam than bottom reinforcement. This scenario is common as most beams have a symmetrical reinforcing cage layout. Extra top reinforcement comes from any activated slab reinforcement. For simplicity all the deformations are assumed to be rigid body rotations.

Stages of load reversal and the effect of cyclic loading are shown in Figure 3. Stages E^- and E^+ are for elastic negative and positive moments, respectively. Stages P1, P2 and P3 are inelastic negative, positive and negative amplitudes where the ductility factors exceed 1. Also shown in Figure 3 are bar stresses that lead to beam elongation. This process continues throughout the duration of the earthquake provided the earthquake imposed displacements are large enough to continue to yield the bars further. The amount by which the plastic hinge elongates depends on the number of inelastic cycles imposed on the beam.

Fenwick & Megget (1993) and Restrepo et al (1993) have derived mathematical expressions for the magnitude of expected beam elongation. The expression derived by Restrepo for elongation is a function of the amount of rotation the plastic hinge has undergone, the internal lever arm of the beam and the ratio between the column centrelines to the distance between plastic hinges. Typical magnitudes for the elongation have been observed to be 2-5% of the beam depth per plastic hinge.

The majority of research conducted on the beam elongation problem to date has not examined the presence of the floor slab system on the beam elongation. The presence of a floor slab is likely to restrain this elongation, but the extent is unclear.

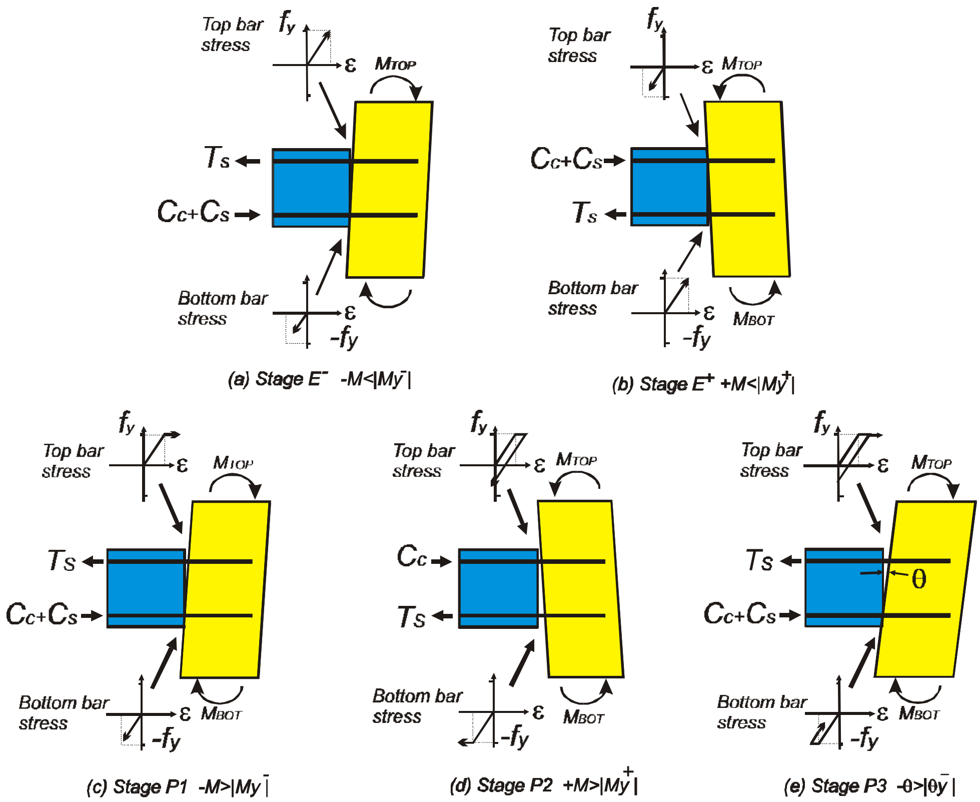


Figure 3. Mechanics behind beam elongation (Matthews et al 2001)

4 THE ROLE BEAM ELONGATION PLAYS IN THE SYSTEM

4.1 Hollowcore seating length:

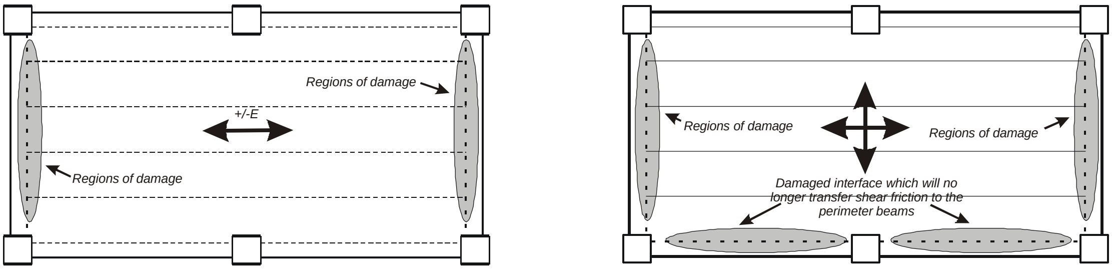
As beam elongation occurs the available seat width for the hollowcore units is reduced. If this length is insufficient to handle the amount of elongation demand then the hollowcore units become unseated. The reliance of bond with the cast in place topping slab to restrain collapse is questionable. Concern has been expressed as to whether this bond is sufficient (Herlihy et al 1995; Mejia-McMaster and Park 1994; and Oliver 1998). Certain failures observed in the 1994 Northridge earthquake showed that bond is insufficient in providing restraint as shown in Figure 1(b). Concern has also been raised regarding whether the hollowcore unit itself will remain intact during an earthquake.

4.2 Negative Moment enhancement:

As beam elongation starts to occur some of the reinforcement within the floor slab becomes activated. This acts as additional beam tensile reinforcement and increases the negative moment capacity of the beam. If this enhancement is significant, there is a chance that the building will not perform in the expected mechanism of a strong column-weak beam as the beams have become stronger than the columns. Researchers (such as Cheung et al 1991) have partially investigated this enhancement for monolithic slab construction, but the effect the hollowcore units have on the system has not been studied. If enough columns on a particular floor are damaged then there is a possibility that a soft storey failure could result.

The displacement pattern that the building undergoes during an earthquake will affect the amount by which the negative beam strength is enhanced. If a building is to displace solely in a direction parallel to the span of the hollowcore planks, then this is where the maximum enhancement is expected (see Figure 4(a)). This displacement pattern is highly unlikely to occur in an earthquake as a building will usually move in an irregular biaxial motion. However, the strength enhancement could be used as an upper bound limit state. The reason this gives the maximum enhancement is that the interface between the topping and the perimeter beam parallel to the planks is undamaged thus allowing enhancement from diaphragm action to be transferred to the beam by shear friction within the concrete and by dowel action via kinked starter bars. If the building has undergone displacement in both orthogonal directions then this interface, between the topping and the perimeter beam, would be damaged and the shear friction component of the diaphragm action can no longer be transferred to the beam (Figure 4(b)). Therefore the maximum permissible enhancement transferred is that due to kinking of the starter bars.

The width of reinforcing within the floor diaphragm being activated is being studied so that an effective flange width can be used to calculate this strength enhancement.



(a) After initial longitudinal loading

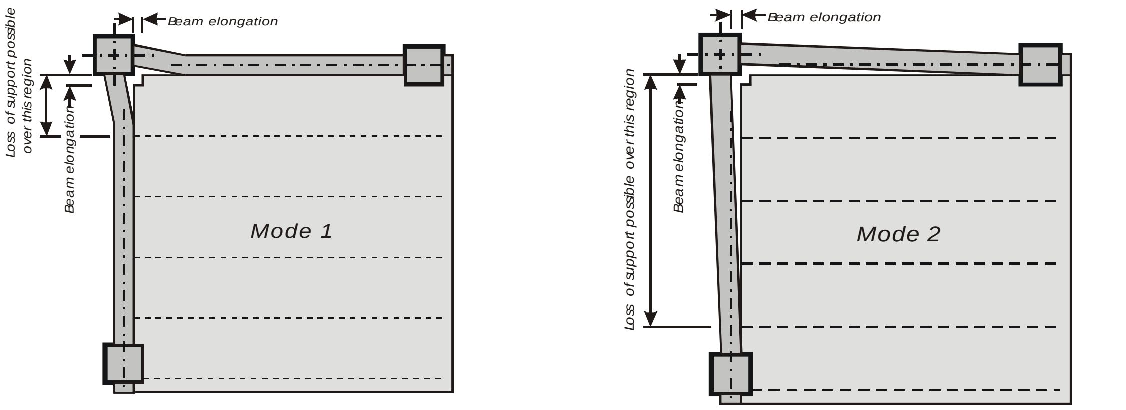
(b) After longitudinal and transverse loading

Figure 4. Regions of damage affecting the potential for beam strength enhancement

4.3 How does beam elongation affect the displacement of the orthogonal perimeter beam?

As the beams start to elongate the orthogonal beam must start to rotate out-of-plane to account for this beam growth. The way in which this beam displaces will affect the number of hollowcore to loss their seating.

Two possible mechanisms are expected to occur. The first, and most likely, is where the beam rotates out about the plastic hinge zone next to the corner column. If this occurs then the number of units that loose their seating will be low and will only be a problem in the corners of buildings. This is referred to as a “Mode 1” mechanism as shown in Figure 5(a). The second mechanism is where the entire beam rotates as shown in Figure 5(b). The mechanism could lead to more units being pulled off their support. This is referred to as a “Mode 2” mechanism.



(a) Beam plastic hinge rotates to allow for beam elongation

(b) Entire beam rotates to allow for beam elongation

Figure 5. Particular deformation modes to deal with beam elongation.

4.4 **Strut and tie solutions for floor diaphragm forces:**

Traditionally during a strut and tie analysis for a floor diaphragm of a monolithic frame construction, the corner columns have been used as nodes to allow the compression struts within the diaphragm to be transferred to the perimeter frame (Park et al 1997). This may not be possible for precast concrete frames because the area around these columns is likely to be extensively damaged. There is a possibility that a large crack occurs along the interface between the floor slab and the column (as shown in Figure 5) not allowing the compression force to be transferred to the perimeter beam.

Another option has been to place a series of 'drag' bars in the floor slab just off the perimeter beams to allow the diaphragm forces to be directed to a relatively undamaged zone in the centre of the beams. This solution may be inappropriate as any additional reinforcing steel placed in the floor slab may unduly enhance the perimeter beam's negative moment capacity causing the beams to become excessively strong and potentially lead to column hinging.

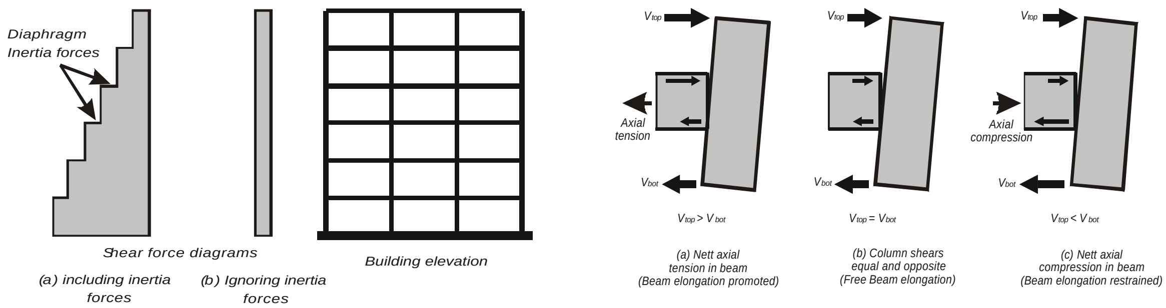
A complete rethink on the strut and tie analysis of floor diaphragms is considered necessary.

5 EXPERIMENTAL APPLICATION OF SEISMIC LATERAL LOADS.

The earthquake simulated loads are applied to the structure as a series of column shear forces to the top and bottom of the columns.

The fundamental component ensuring that beam elongation is not promoted nor restrained is the applied column shear forces. The column shear forces induced in a building during an earthquake represent a series of arrows up the building height. A typical shear force diagram is shown in Figure 6(a). These steps in the shear force diagram are due to the floor inertia forces from each floor level. If inertia forces are ignored, as is the case in this testing programme since the floor diaphragm itself is not loaded, then the shear force up the height of the building is constant (Figure 6(a)). Since this testing programme is a pseudo-static test, rather than a real time test, then the assumption of zero floor inertia forces is true. The key issue to allow beam elongation to form naturally is to keep the external applied loads from the column shear forces equal and opposite. This seems to be an area that other researchers have overlooked. If there is an out of balance force between the top and bottom applied shear forces then this elongation is either restrained or promoted. This principle is shown in Figure 6(b).

Since the external applied column shear forces are equal and opposite does not mean that there are no compression or tension fields formed within the beams. As testing proceeds there will be compression fields formed within the beams and these will be equalised by tension fields within the floor diaphragm.



(a) Shear forces induced from an earthquake (b) Types of beam elongation

Figure 6. Shear forces induced from earthquake versus potential s shear forces induced by experiment

5.1 **Time History studies to determine experimental loading protocol.**

As recommended by Park (1989) the traditional the loading history used to test various concrete elements at the University of Canterbury has required the specimen to be subjected to *two completely reversed loading cycles* at ductility amplitudes of 0.75, 2, 4, 6 and 8.For the present experimental structure whose yield drift is assessed to be 0.5%, this translates into two cycles at ±0.4%, ±1.0%, ±2.0%, ±3.0% and ±4.0%. It is considered unrealistic to impose these drifts on the super-assemblage as such demands are unlikely to be experienced during a real earthquake. One reason the Park method is considered inappropriate for this experimental programme was because the test being undertaken is one in which existing structural performance is being examined. Therefore the structure should be subjected to a realistic displacement history. When verifying new construction methods, a more conservative experimental protocol is considered acceptable.

Therefore, an analytical study has been undertaken on four different building heights (Figure 7) using numerous earthquake records to determine the expected demand on the sample precast concrete buildings. The earthquake records used in the analytical study have included both near and far field effects. Some records were scaled so that they represented the amount of energy expected from a New Zealand earthquake. From these results it is possible to determine a more realistic loading history that better matches the expected cyclic capacity with the demand.

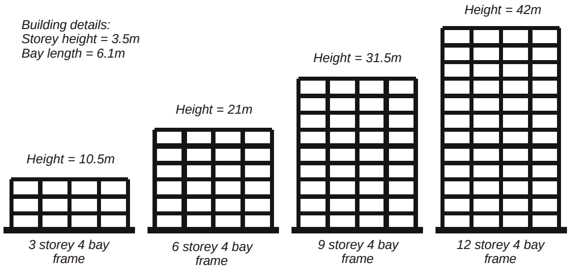
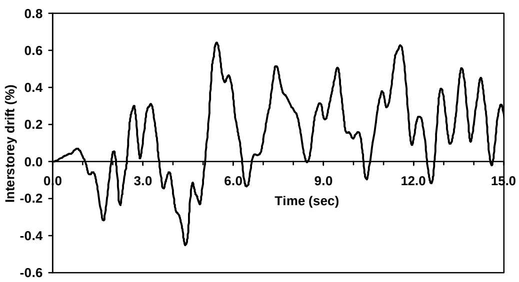


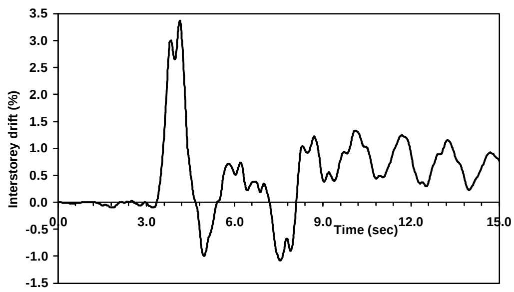
Figure 7. Buildings analysed

The results show that the number of cycles that a structure is likely to experience is significantly less than proposed by Park (1989). Therefore a new loading history was devised based on the time history results.

When examining the results from the time history studies there were two main trends seen. The first was when a far-field type event occurred (1.5xEl Centro 1940). These results showed several cycles of modest amplitudes. An example of the results from this type of earthquake is shown in Figure 8(a). The second trend was seen in a near-field earthquake (Northridge 1994, Syff943) where there was one large pulse and several smaller cycles (see Figure 8(b)). None of the results showed two reversing cycles of increasing magnitude.



(a) 1.5xEl Centro earthquake, 1940, PGA=0.52G , 12 storey structure



(b) Northridge earthquake (Syff943), 1994, PGA=0.83G, 9 storey structure

Figure 8. Typical time history results.

The finalised loading history to be used to load the super-assemblage consisted of one completely reversing load cycle at the following interstorey drift levels: 0.5%, 1.0% and 2.5% (if a maximum credible event is to be imposed then a additional cycle of 3.5% is added). Note that this proposed cyclic loading protocol is in stark contrast with the above mentioned Park method.

A companion written by the same authors (Matthews et al 2003) is to be presented within the same session and focuses on the experimental observations and results from the testing programme.

6 SUMMARY LARGE SCALE TESTING RECOMMENDATIONS

Based on the foregoing analysis and discussion the following recommendations are made concerning large scale testing.

1. Inelastic performance of concrete frame buildings lead to elongation of beams. An experimental method and loading apparatus must be so devised to ensure the beams are permitted to grow, as they would naturally want to in a real structure. Therefore beam elongation should be neither promoted nor restrained by the loading frames during testing. The arrangement of diagonal (scissor) frames and actuators proposed herein permits this objective to be achieved.
2. The traditional ductility based test protocol is suitable for defining dependable inelastic capacity for code development purposes because it is conservative. However, such a loading protocol is unduly restrictive in defining the expected performance of existing structures. Analytical studies of real moment resisting frame concrete buildings show that there are only a few large amplitude cycles of an earthquake motion that cause damage and the loading protocol should be adjusted accordingly.

7 ACKNOWLEDGEMENTS

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