INVESTIGATING THE LOADPATHS OF FLOOR DIAPHRAGM FORCES DURING SEVERE DAMAGING EARTHQUAKES

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INTRODUCTION

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 is described including the testing to be completed at the University of Canterbury.

BACKGROUND

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 observed after the Northridge earthquake (17 January 1994) have raised some concern regarding the performance of New Zealand's precast concrete multi-storey moment resisting frame buildings. This is because New Zealand construction methods are similar to that used in the United States and several of their buildings did not perform adequately during the Northridge earthquake.

Several buildings in Northridge collapsed as a result of the hollowcore flooring units loosing their seating from the supporting beams [1]. Once the beam support was lost, the units collapsed onto the floor below causing a concertina effect with all 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 come down in one piece (see Figure 1). Another was when the support from the beam was lost, the hollowcore floor unit delaminated from the topping concrete and the unit drops. The third failure mechanism was when the webs of a hollowcore unit split once the support was lost. 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.

After observing the failures in Northridge a study has been undertaken at the University of Canterbury, to determine whether New Zealand designed and built structures have a similar problem, and if so, to what extent the problem exists and what can be done about it.

Figure 1. Flattened car under a hollowcore floor unit that has lost its support. [1]

PROJECT DESCRIPTION

In order to test the performance of a precast concrete building constructed to the New Zealand Concrete Standard [2], a full size subassemblage of a building has been constructed in the laboratory. By constructing a subassemblage, 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 have focused on the individual components of a building. This project focuses on investigating the interaction of column-beam-slab performance of a large subassembly.

PROJECT AIMS

The principle aim of this project is to focus on the effect that beam elongation has on the required seating lengths for hollowcore floor units. The strength enhancement to the negative perimeter beam moments 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

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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.

BEAM ELONGATION

During a severe seismic attack, buildings that behave by the preferred beam sidesway mechanism form plastic hinges at each end of the beams, as shown in Figure 2. Once plastic hinges form in a beam and the beam undergoes large inelastic rotations, the beam significantly grows in length. This phenomenon is known as “beam elongation”. The recent 1994 Northridge earthquake has shown this elongation occurring. Experimental studies undertaken by several groups of researchers [3, 4, 5, 6 and 7] have observed elongation occurring.

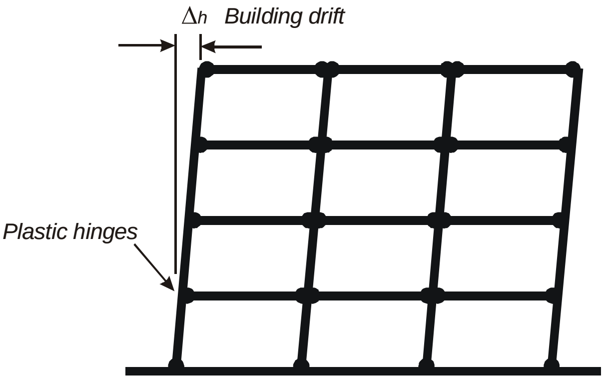


Figure 2. Desirable weak beam strong column mechanism

The mechanics of beam elongation can be explained by referring to figure 3. 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 any deformations are assumed to be rigid body rotations.

Stage 1: (figure 3(a))
The top beam reinforcement reaches yield in tension. At this point the bottom beam reinforcement is well below yield in compression as the compression load is resisted by both compression in the steel and in the concrete.

Stage 2: (figure 3(b))
The loading direction reverses and the bottom steel reaches yield in tension while the compression force is resisted by the top reinforcement in compression and bearing of the concrete.

Stage 3: (figure 3(c))
Up until this stage, whenever the load has reversed the reinforcing steel in tension has regained its elastic recovery and the crack at the beam column interface has closed. During the next cycle the top steel yields in tension and undergoes plastic deformation. The bearing of the concrete and the bottom reinforcement in compression resists the compressive force.

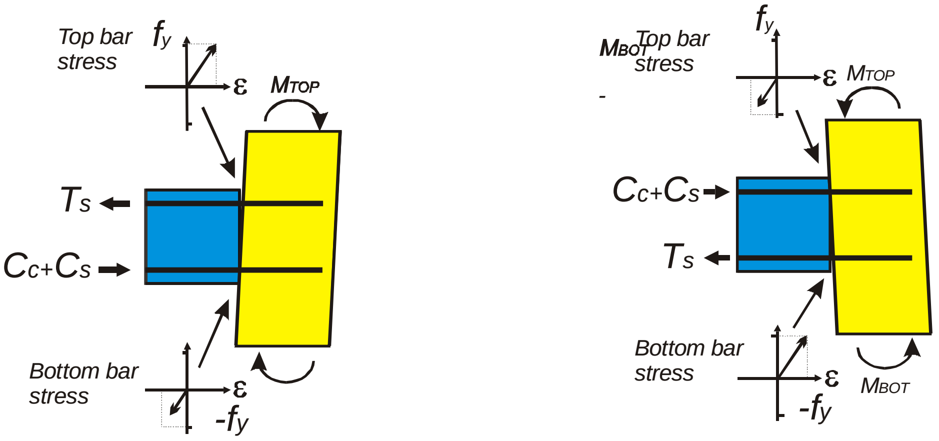
Stage 4: (figure 3(d))
Upon the load reversal the bottom reinforcing yields in compression before the top reinforcing is able to yield back in compression as there is significantly more top reinforcement. This is the onset of beam elongation because the top crack has not closed fully and the bottom reinforcement has been pulled out of the beam. The compressive force now is resisted by the compression reinforcement alone.

Stage 5: (figure 3(e))
As the load reverses the bottom crack closes, as there is sufficient force to yield these bars back in compression due to the large area of top reinforcement. Now the top beam reinforcement undergoes further plastic deformation and the crack width increases.

Stage 6:
The load reverses and now the residual crack at the top of the beam has increased due to further yielding in the last cycle (stage 5). Again, the bottom reinforcement yields in tension before the top reinforcement yields back in compression. 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.

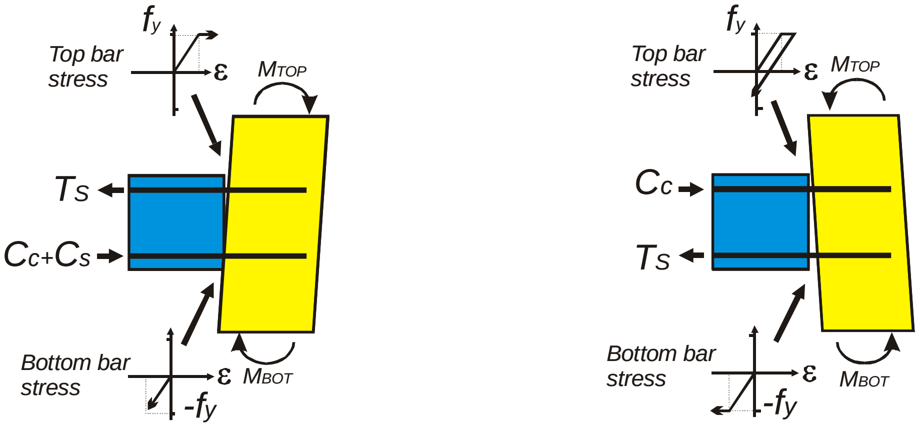
A mathematical expression for the magnitude of expected beam elongation has been derived by Fenwick and Megget [5] and Restrepo [7]. The expression for elongation is a function of the amount of rotation the plastic hinge has undergone, the internal lever arm of the beam and the ratio between the column 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.



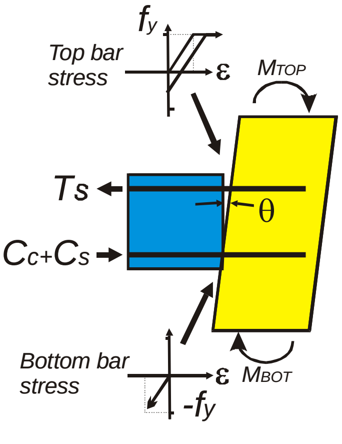
(a) Stage 1 $-M<|M_y^-|$

(b) Stage 2 $+M<|M_y^+|$



(c) Stage 3 $-M>|M_y^-|$

(d) Stage 4 $+M>|M_y^+|$



(e) Stage 5 $-\theta>|\theta_y^-|$

Figure 3. Mechanics of beam elongation

THE ROLE BEAM ELONGATION PLAYS IN THE SYSTEM

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. Serious concern as too whether this bond is sufficient as shown by several researchers [8,9 and 10]. Certain failures observed in the 1994 Northridge earthquake showed that bond is insufficient in providing restraint as shown in Figure 4.

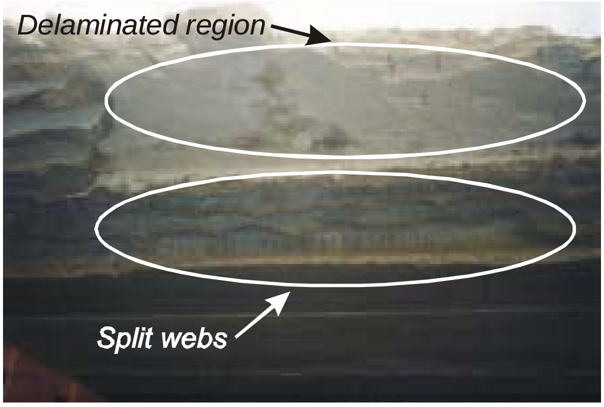


Figure 4. Delaminated section of hollowcore topping

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 (see figure 2) as the beams have become stronger than the columns. Researchers such as Cheung [11] 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. This type of mechanism in a multi-storey building is undesirable due to excessive plastic rotation demands on the columns as shown in figure 5.

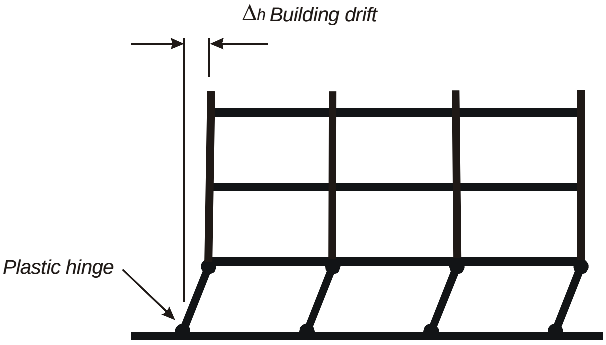


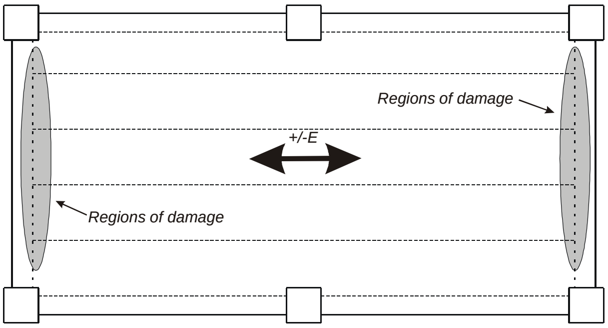
Figure 5. Undesirable soft storey mechanism

The displacement pattern that the building undergoes during an earthquake will affect the amount but 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 6(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 (see figure 6(b)). Therefore the maximum permissible enhancement transferred is that due to kinking of the starter bars.

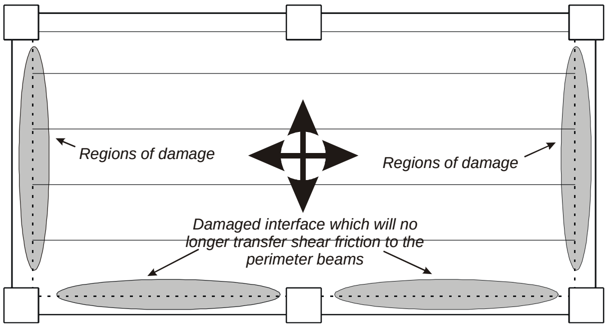
The width of reinforcing within the floor diaphragm being activated is being studied so that an effective flange width can be used to calculate this strength enhancement.

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.



(a) After initial longitudinal lateral loading



(b) After longitudinal and transverse lateral loading

Figure 6. Regions of damage affecting the potential for beam strength enhancement due to diaphragm interaction

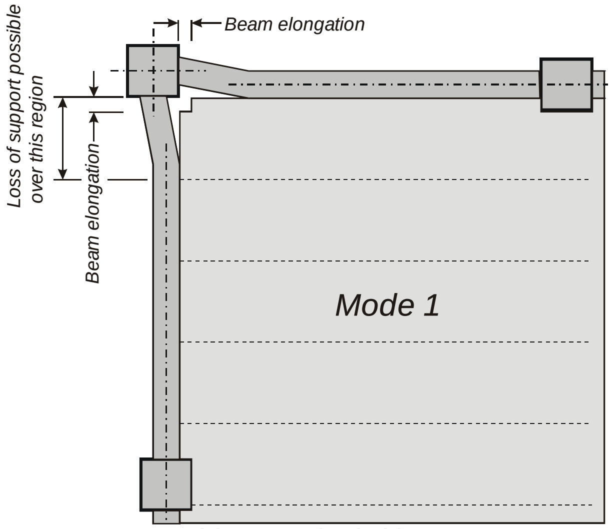
Two possible mechanisms are expected to occur. The first, and most likely, is where the beam rotates out about the plastic hinge zone next to the corner column. If this occurs then the number of units that loose their seating will be low and will only be a problem in the corners of buildings. This is referred to as a “Mode 1” mechanism as shown in Figure 7(a). The second mechanism is where the entire beam rotates as shown in Figure 7(b). The mechanism could lead to more units being pulled off their support. This is referred to as a “Mode 2” mechanism.

Strut and tie solutions for floor diaphragm forces:

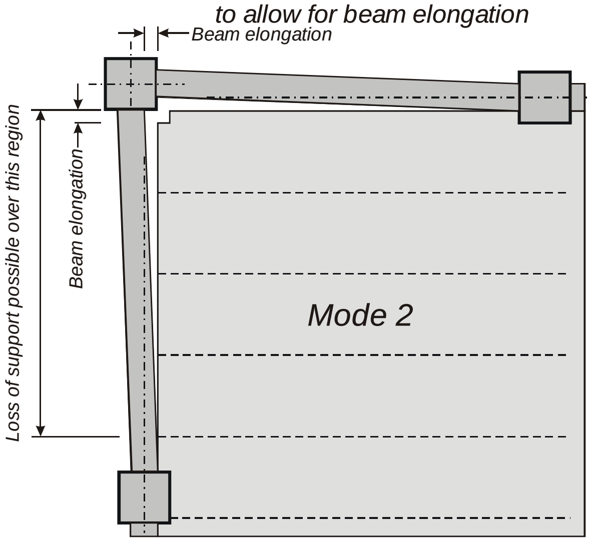
Traditionally during a strut and tie analysis for a floor diaphragm of a monolithic frame construction, the corner columns have been used as nodes to allow the compression struts within the diaphragm to be transferred to the perimeter frame [12]. This may not be possible for precast concrete frames because 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 7) 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.



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



(b) Entire beam rotates to allow for beam elongation

Figure 7. Particular deformation modes to deal with beam elongation.

ANALYSIS

The analysis of a building has historically been carried using the member’s centrelines as line elements. End blocks are added to simulate the member widths. This method is sufficient when looking at the entire building and ignoring beam elongation. However, when studying a subassembly and when beam elongation is to be accounted for, the one dimensional structural elements are insufficient in fully describing behaviour. By studying the figures explaining beam elongation (Figure 3) it can be seen that the point of rotation within the beam changes depending whether a positive or negative moment is applied to the beam. This is important when determining the principles behind beam elongation as well as determining exactly how the system as a whole deforms. By assuming rotation occurs about the centre of the beam column joint, the centreline dimension does not change during load cycles. Using the actual displacement pattern that the beam undergoes reveals that the centreline length between the centre of the beam column joints actually increases during the load cycle. The amount by which the centreline dimension increases is equal to twice the width of the crack on the beam centreline (see Figure 8). The greater the inclination of the columns the larger this distance becomes. Beam elongation will also increase this distance further.

Within the structural analysis software available there is a lack of ability to be able to model the plastic hinge performance in terms of beam elongation. This beam elongation element modelling is important when designing structures as the buildings performance could change quite significantly when beam elongation is considered.

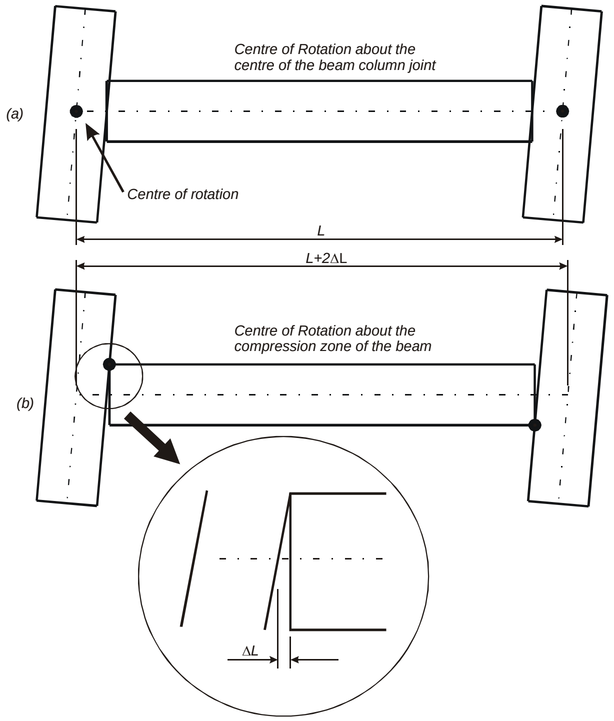


Figure 8. Real behaviour versus structural analysis based on (a) conventional one-dimensional beam-column elements. Note the displacement incompatibility at the beam-column junction and (b) two-dimensional solid elements that allow for gap movement.

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 floor units are seated on a nominal ledge 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 9 shows the sub assemblage dimensions.

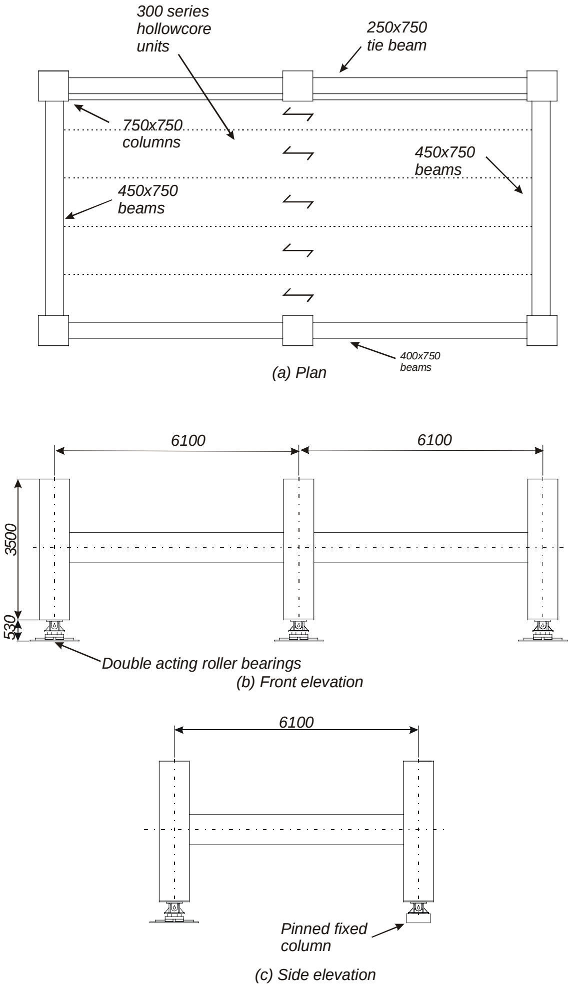


Figure 9. Plan and elevations of the test specimen

Loading

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 test rig developed for this large scale experiment is a complex set up as it is required to apply realistic loads to the structure so that the specimen deforms in the correct manner. Care has to be 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.

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 10(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 (see Figure 10(b)). 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 over looked. 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 11.

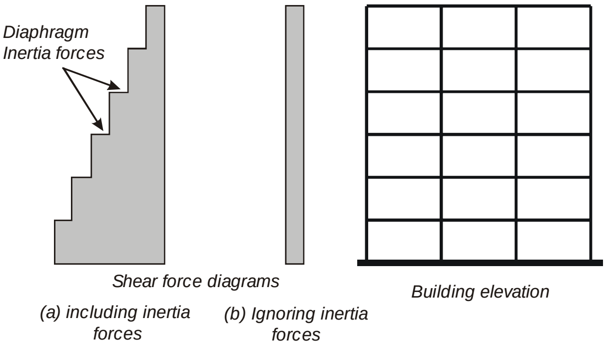


Figure 10. Shear forces induced from an earthquake

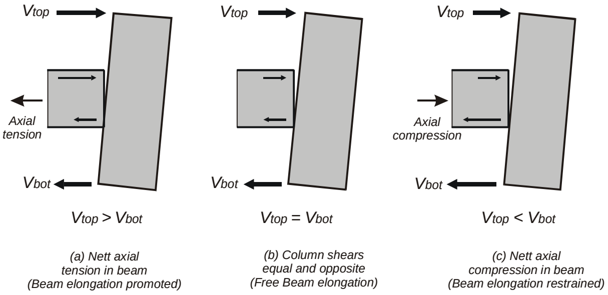


Figure 11. Types of beam elongation.

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.

Two main loading frames are the diagonal frames on the front of the specimen as shown in Figure 12. These 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. Basically the secondary frames ensure the drift angle on each column is the same.

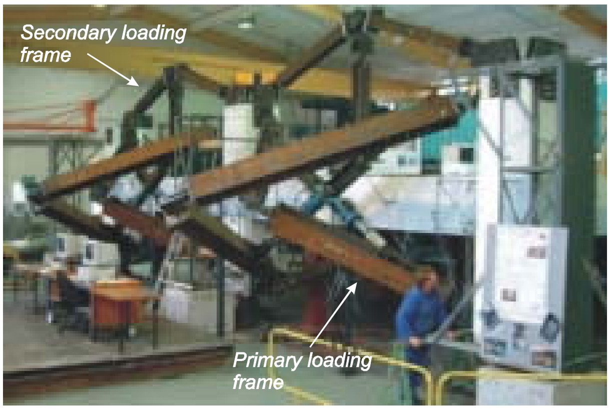


Figure 12. Photograph of the test set up just prior to the commencement of testing.

Time History studies

Traditionally the loading histories used to test various concrete elements at the University of Canterbury have required the element to be subjected to two loading cycles at various levels of displacement ductility. The imposed ductility levels are usually 1,2,4,6 and 8. Figure 14(a) shows a typical loading history. By imposing these ductilities to the element requires it to be subjected to unrealistic demands that the element is unlikely to experience during a real earthquake. An analytical study has been undertaken on four different building heights (see Figure 13) using numerous earthquake records to determine the expected demand on the various buildings. From these results it is possible to determine a more realistic loading history that better matches the expected cyclic capacity with the demand.

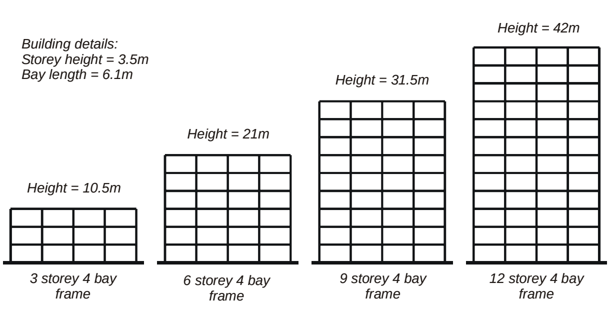


Figure 13. Buildings analysed

Several earthquake histories from around the world have been studied to determine the loading history. These records 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. Comparing the results shown in Figure 14 shows the difference in expected demands. The time history results are focussing on the interstorey drift of a building rather than on their displacement ductilities demands. It can be clearly seen that the number of large loading cycles in the time history response is significantly less than in the traditional method.

The selected time history record to be applied to the test sub assemblage will be generated from various portions from the individual time history response predictions. Figure 14(b) shows one response for the first floor of a nine storey building during one of the Northridge earthquake records while Figure 14(c) shows the response for the second storey in a twelve storey building in a modified El Centro earthquake. Various cycles from different time histories will be combined to make one composite test sequence that incorporates three discrete phases. The three phases are the foreshock, main shock and after shock. This loading history approach is based on the method explained by Dutta *et al* [13].

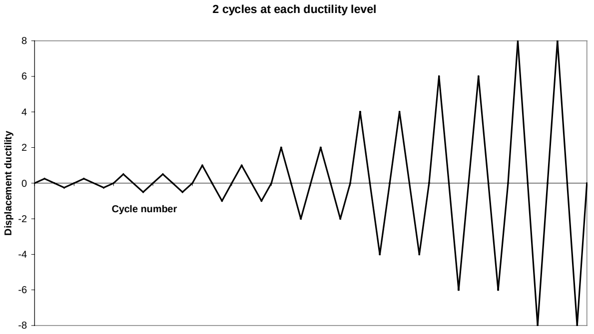
It is of interest to note that if a conventional ductility based test protocol were to be adopted then this would result in histories of the following drift amplitudes; +/-0.5%, +/-1%, +/-2% and +/-3%.

This is in contrast with a more realistic event that is representative of a near-field main shock. Thus using Figure 14(b) as a basis, the loading protocol would resemble one-half cycle at +3% followed by two cycles at +/-1% drift. Figure 14(c) represents a far-field main shock event. This loading protocol would resemble one cycle at +/-0.6% followed by four half cycles of +0.5%. The demands from these two records are significantly less than that for the ductility protocol.

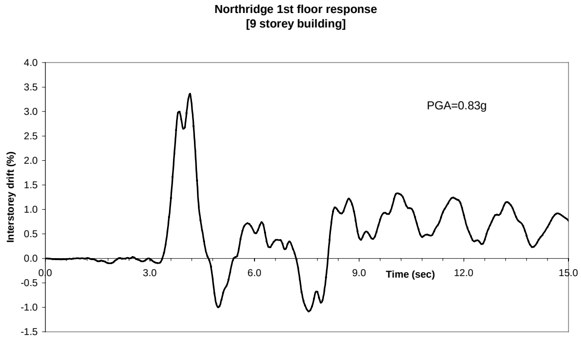
SUMMARY

The global performance of a precast concrete building during a severe earthquake is dependent on the effects that beam elongation has on the system. Once a proper understanding of this beam elongation and its related issues have been gained then it is possible to determine the buildings performance. This is of particular importance as there is two decades worth of building stock constructed in this nature in New Zealand.

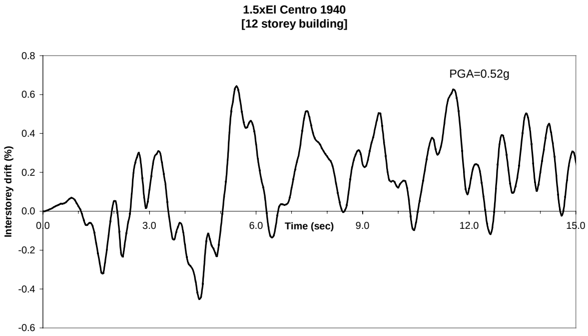
Currently there is a shortage of appropriate structural analysis software that is capable of modelling beam elongation. When accounted for the buildings performance could be significantly different.



(a) Traditional displacement ductility based loading sequence



(b) Time history based loading sequence: Near-field (Note: One or two large cycles then a number of smaller loading cycles)



(c) Time history based loading sequence: Far-field

Figure 14. Time history results versus traditional loading history.

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