Effect of precast-prestressed flooring systems on the seismic performance of reinforced concrete frames

B.H.H. Peng, R.P. Dhakal & R.C. Fenwick
University of Canterbury, Christchurch, New Zealand

ABSTRACT: A one story, two bays, approximately half scaled, perimeter moment frame containing precast-prestressed floor units was built and tested at the University of Canterbury to investigate the effect of precast-prestressed floor units on the seismic performance of reinforced concrete moment resisting frame. This paper gives an overview of the experimental set up and summarizes the results obtained from the test. The results show that elongation in the beam plastic hinges is partially restrained by the prestressed floor, which increases the strength of the beams much more than that being specified in the codes around the world.

1 INTRODUCTION

A collaborate research effort between USA, NZ, Japan and China in the early eighties was initated to investigate the seismic performance of interior/exterior beam-column joints with cast-in-situ floor slabs in one-way/two-way reinforced concrete frames (Jirsa 1991). It was found that the presence of floor slabs significantly increases the negative flexural strength of the beam due to elongation of the plastic hinges and its interaction with the floor (Cheung et al. 1991; French and Moehle 1991). This phenomenon is commonly referred to as flange effect. Equations accounting for the floor slab contribution to the overall strength of the beams were developed based on the experimental results available at the time and these equations were adopted by the American Standard, ACI 318, (American Concrete Institute 1995), and New Zealand Standard, NZS3101, (Standards New Zealand 1995).

More recently, experimental projects were carried out in New Zealand (Lau et al. 2007; Lindsay 2004; MacPherson 2005; Matthews 2004) to investigate the seismic performance of reinforced concrete frames containing precast-prestressed floor units. It was found that the presence of prestressed floor units increased the strength of the beams to a greater extent than those had been observed in studies where cast-in-situ slabs were used. This is because the prestressed floor units, unlike the cast-in-situ slabs, provide additional tensile strength along the floor thereby confining the major cracks to develop in the weak sections at the supports. In some cases, the prestressed floors also provided additional restraining force to the beams which resulted in different level of strength enhancement from different structural arrangements used in the tests.

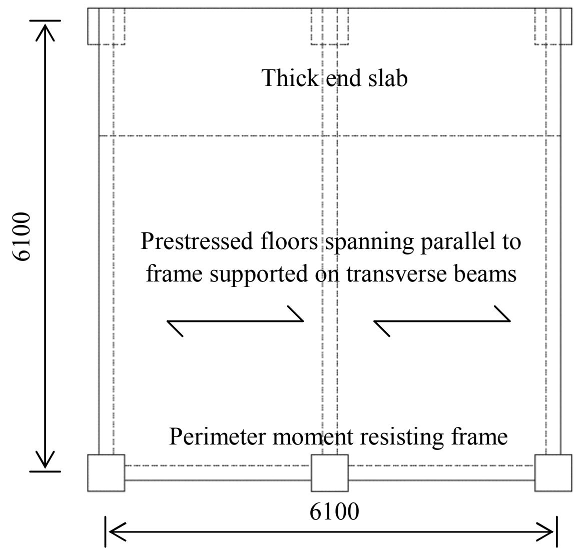
While these tests gave some insight into the mechanisms associated with the interaction between the perimeter beams and the slabs, they cannot by themselves be used to develop satisfactory design methods due to the wide range of structural arrangements and precast floor units used in practice. In this study, a 3D sub-assembly test of a one story reinforced concrete frame with prestressed floor slab was carried out to further investigate the mechanisms associated with the interaction of prestressed concrete slab with moment resisting frames. This paper gives an overview of the experimental observations and highlights some of the key results obtained from the test.

2 EXPERIMENT

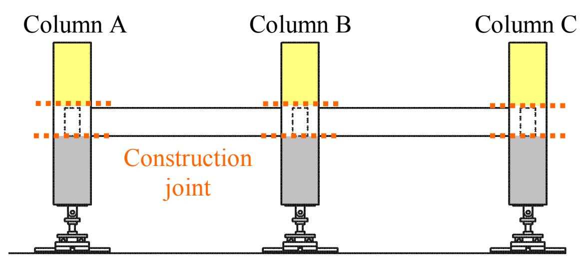
2.1 *Sub-assemblage construction*

The sub-assembly is an approximately half scale model of a portion of a mid-height multi-story RC building. As illustrated in Figure 1 and Figure 3, the sub-assembly consists of a two bay moment resisting frame in the longitudinal direction and three gravity beams in the transverse direction.

The sub-assemblage was built in four different stages. First, three bottom columns with reinforcement sticking out, full depth longitudinal beam including beam-column joints with ducts allowing the protruding column reinforcing bars to pass through and three half height transverse beams were precast. These precast members were then erected and the beam-column joints were grouted. Next, the top of the columns and the lap splices between the transverse beams and the columns were cast-in-place. Finally, the prestressed ribs were placed between the transverse beams and the floor topping and the rest of the transverse beams were poured.



(a) Plan view of the sub-assembly.



(b) Elevation of the sub-assembly showing the precast members

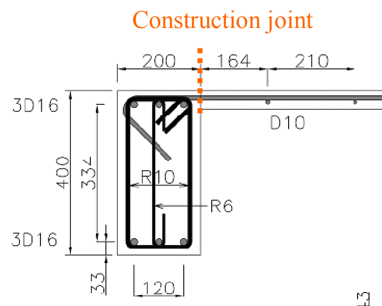
Figure 1. Plan and elevation of the sub-assembly.

Figure 2 shows the cross-sections of the key members in the sub-assembly. The stirrup sets in all members were spaced at 90mm centers. The columns were designed to be stronger than the beams to ensure plastic hinges will form in the beams. The flooring system consists of 100mm deep prestressed Stahlton ribs with 45mm thick cast-in-situ topping. The ribs are supported on low friction McDowel bearing strips and 40mm wide seating on the transverse beams. The reinforcement in the topping consists of Grade 300 deformed 10mm bars at 210mm centers in both directions. The floor is connected to a 175mm thick solid end slab to act like a rigid body simulating the rest of the structure.

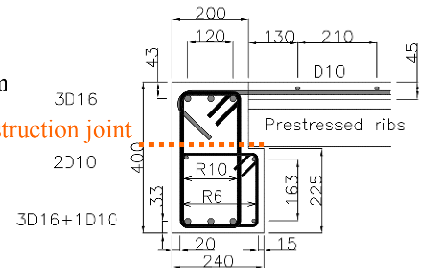
The averaged compressive strength of the concrete in the three casts was 31.2, 42.4 and 33MPa respectively. The averaged yield stress of the reinforcing bars used in the test is summarized in Table 1.

Table 1. Average yield stress of reinforcing bars.

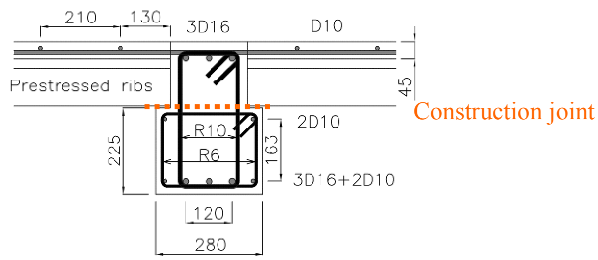
	D10	D16	D20	R6	R10
Yield stress (MPa)	370	365	317	445	391



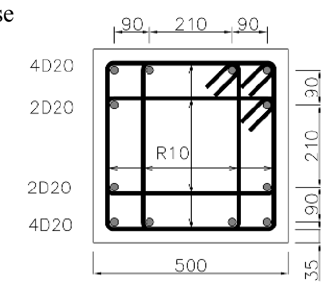
(a) Main beam



(b) Exterior transverse beam



(c) Central transverse beam



(d) Column

Figure 2. Key cross sections of the sub-assembly

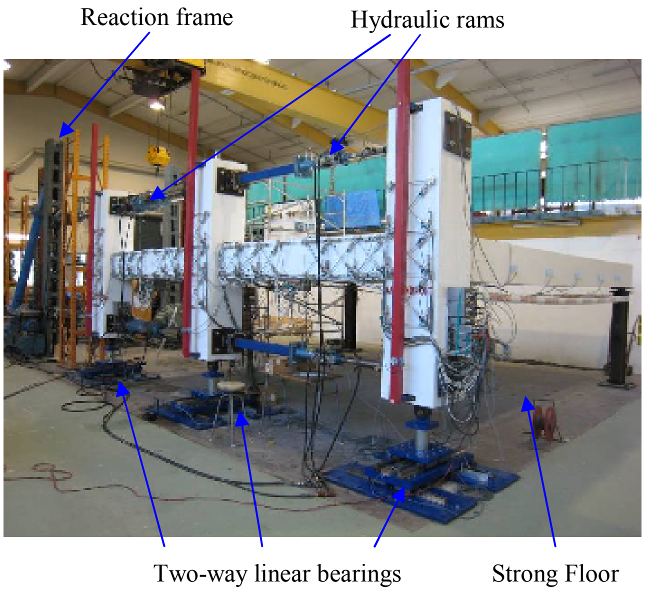
2.2 *Test set-up*

The experiment was set up as shown in Figure 3 to ensure that:

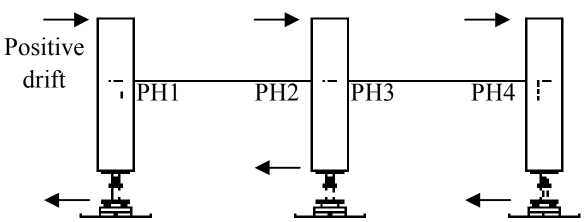
1/ Elongation in the plastic hinges is not restrained by the loading system;
2/ The columns remain parallel to each other throughout the test;
3/ Equal and opposite shear forces are applied to each individual column during the test.

To fulfill the first requirement, the columns were supported on two way linear bearings allowing movement in two directions. The exterior transverse beams were supported on steel columns with one way linear bearing for floor movement parallel to frame and the interior transverse beams were supported on ball bearings allowing movement in the horizontal plane.

The loading was displacement controlled; the displacements were applied at the top and bottom of each column using six hydraulic rams with an inter-story height of 1.9m. The loading sequence was carefully designed to ensure that the rest of the requirements were met.



(a) Photo of the test arrangement



(b) Labeling and sign convention used for the sub-assembly

Figure 3. Schematic diagram showing positive drift and label

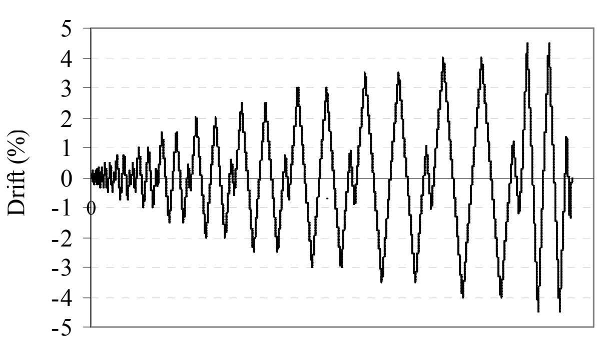
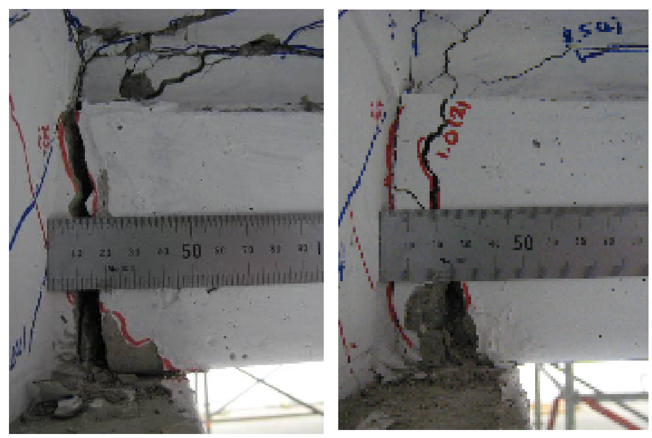


Figure 4. Applied displacement history.

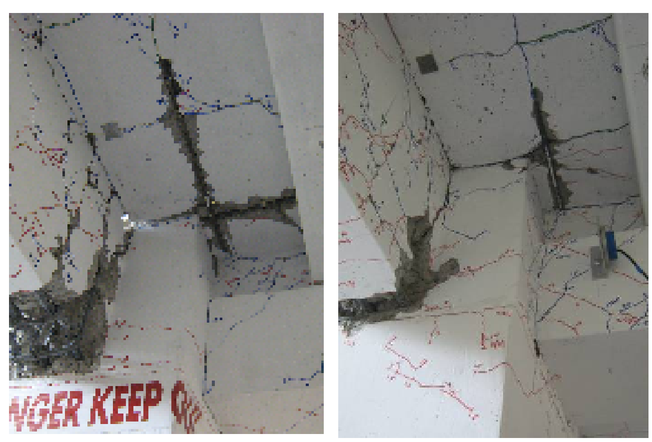
The loading history applied in the experiment is shown in Figure 4. Initially, two elastic cycles at 0.25%, 0.35% and 0.5% drift were applied to check the loading and the logging systems. Following these elastic cycles, two large cycles and a small cycle, equal to 30% of the large cycle amplitude, were applied. The peak displacement was increased gradually in increments of 0.5% drift until the maximum shear force in the frame has dropped by more than 30%.

Extensive instrumentation consisting of load cells, linear potentiometers, inclinometers, rotary potentiometers, sonic displacement transducers and DEMEC gauges was used to measure the lateral and axial forces in the columns, deformation of the beams, columns, beam-column joints, and the floor.

(a) Damage in the exterior and interior plastic hinges.



(b) Damage in the prestressed floor connection next to the exterior and interior plastic hinges.



(c) Damage under-neath the floor next to the exterior and interior plastic hinges.

Figure 5. Damage to the sub-assembly at the end of test.

3 EXPERIMETNAL RESULTS

3.1 *General Observations*

Photographs of the key damage sustained till the end of the test are shown in Figure 5 and Figure 6. Flexural cracking first appeared in the longitudinal beams and the floor slabs at 0.25% drift. Minor cracking in the columns and prestressed floor connections appeared at 0.35% drift. At 0.5% drift, diagonal cracks appeared in the longitudinal beams and the slabs. First sign of yielding occurred at 0.75% drift. At 1.0% drift, torsional cracks developed in the transverse beams. The differential movement between slabs and beams became apparent at 1.5% drift. Spalling occurred in the exterior plastic hinges at 2.0% drift. At 2.5% drift, spalling occurred underneath the floor starter bars. The exterior plastic hinges lost concrete cover exposing buckling of bottom reinforcement at 3.0% drift and the bottom bars in the plastic hinge next to Column A fractured at 3.5% drift. All the bottom reinforcement in the exterior plastic hinges fractured under low cycle fatigue at 4.5% drift and the test was terminated at this stage.

It can be seen from Figure 5 that the region around the exterior plastic hinges sustained more damage than the region around the interior plastic hinges. Severe concrete spalling, reinforcement buckling and bar fracture were observed in the exterior plastic hinges where as the interior plastic hinges only sustained minor spalling and minor bars buckling. This difference may be attributed partially to the participation of the floor slabs acting as deep beam as shown in Figure 8, which provides additional strength and stiffness to the interior plastic hinges. The difference in the anchorage conditions between the interior and exterior beam-column joints would also affect the deterioration of the plastic hinges.

It can be seen from Figure 7 that many cracks formed in the floor parallel, perpendicular and diagonal to the frame. It should be noted that the cracks parallel and perpendicular to frame formed mostly along the line of the topping reinforcement where the reinforcing bars act as crack initiator. The cracks perpendicular to the frame developed due to elongation of the plastic hinges. The cracks were the greatest at the prestressed floor supports. It is interesting to note that all the diagonal cracks were inclined towards the interior column. This is because elongation of the plastic hinges tried to push the floor apart. The cracks parallel to frame were wider near the interior transverse beam and finer towards the exterior transverse beams. The cracks parallel and diagonal to frame indicate that the floor was acting like two deep beams trying to restrain the growth in the plastic hinges as illustrated in Figure 8. These actions would significantly increase the flexural strength of the beams as observed later in Table 2.

Figure 6. Torsional cracks in the transverse beam at the end of test.

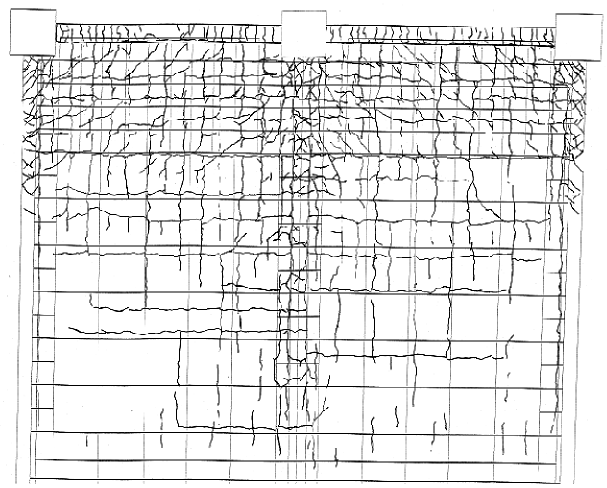


Figure 7. Crack patterns in the floor at the end of the test.

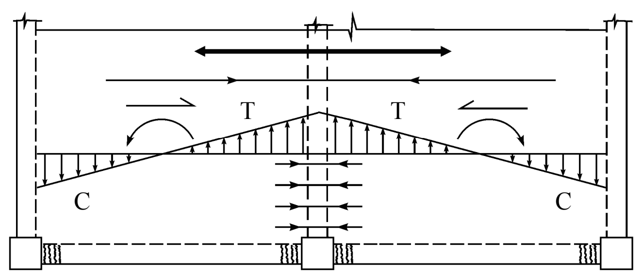


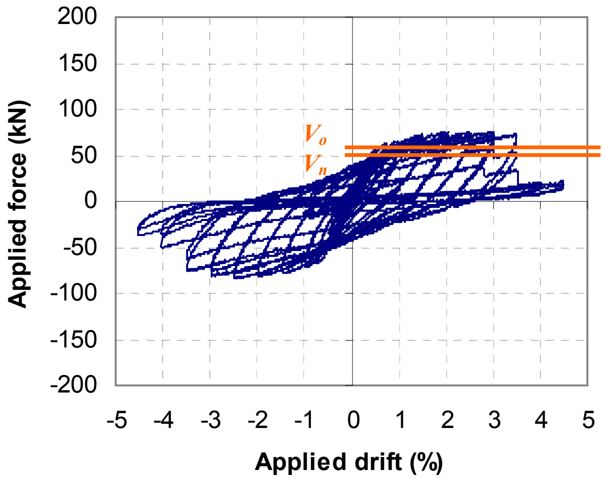
Figure 8. Deep beam actions in floor slabs

3.2 *Preliminary Results*

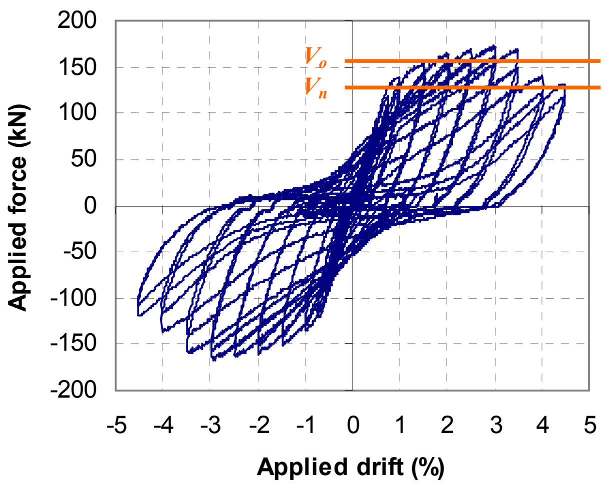
3.2.1 *Strength increase from prestressed floor*

The force-displacement responses for all three columns are shown in Figure 9. Also plotted on the diagrams are the shear forces corresponding to the theoretical flexural strength, V_n, and over-strength, V_o of the beams. The values were calculated using the measured material properties and considering the flange effect according to the current New Zealand code (NZS3101:2006). The values are summarized in Table 2. In this case, the shear forces are calculated for a positive drift as shown in Figure 3.

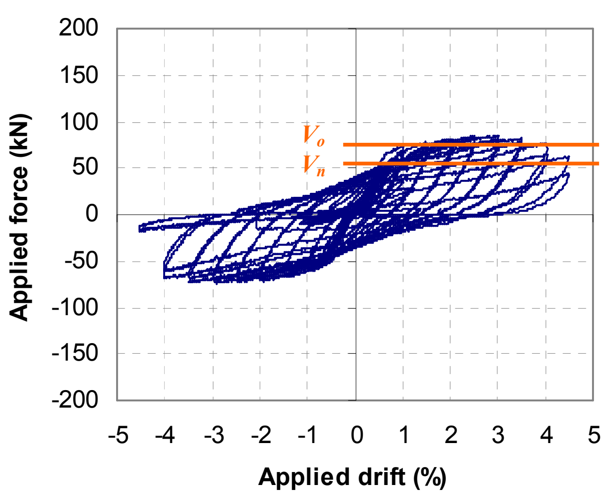
Hence, the shear forces in Column A and Column C correspond, respectively, to the positive and negative moment capacity of the beam hinge adjacent to the corresponding column. Similarly, the shear force in Column B originates from the combination of positive and negative moment capacities of the hinges in the two sides of the column.



(a) Column A



(b) Column B



(c) Column C

Figure 9. Force-displacement relationship of each column

Table 2. Summary of the theoretical and measured strength.

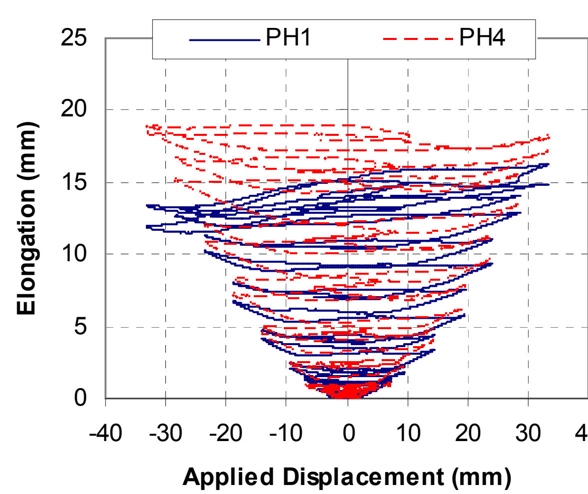
	Column average shear force (kN)		
	Column A	Column B	Column C
Theoretical strength	51.0	109.2	58.2
Measured yield strength (0.75% drift)	62.4	119.1	59.7
Over-strength	58.0	145.9	82.4
Measured maximum strength (3.0% drift)	76.4	171.6	85.6

From these comparisons, it can be seen that both the code specified theoretical and over-strength values are lower than the experimental measured values. The differences in the theoretical and over-strength are 10% and 17% respectively. This reduction in strength predicted by the code is of a great concern as it can potentially result in undesirable strong-beam/weak column mechanism in the event of a large earthquake.

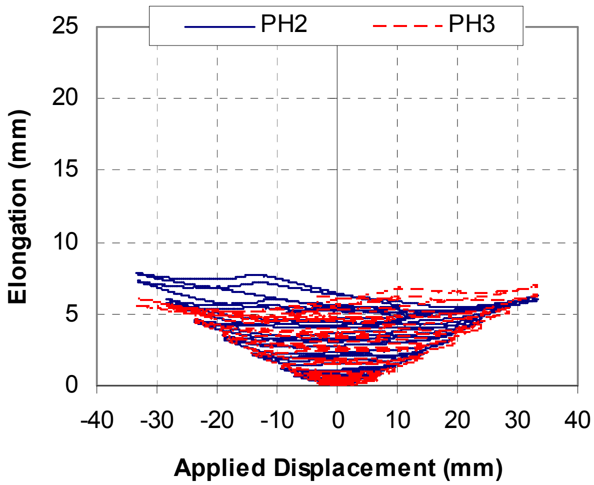
It can also be seen that the majority of strength enhancement arises from the positive flexural strength in Column A and the combined flexural strength in Column B. It is not clear at this stage why there was appreciable increase in the positive flexural strength of the exterior column and further study is underway.

3.2.2 *Elongation in the plastic hinges*

Elongations in the four plastic hinges are plotted in Figure 10. As can be seen in the figures, elongation within the two exterior plastic hinges is similar. This is also the case for the two interior plastic hinges. Elongation in PH1 is smaller than in PH4 at larger cycles because the longitudinal bars in PH1 buckled and fractured at an earlier drift. It can be observed that elongations in the exterior plastic hinges, PH1 and PH4, are much larger than those in the interior plastic hinges, PH2 and PH3. This is because the floor continuity provides greater restraint to the growth around the interior column. This result matches well with the physical observations made earlier in this paper.



(a) Elongation in the exterior plastic hinges



(b) Elongation in the interior plastic hinges

Figure 10. Elongation history in the beams

4 DISCUSSIONS AND CONCLUSIONS

The experimental results have shown that floor slab containing prestressed floor units can have a major influence on the seismic performance of RC moment resisting frames. The presence of prestressed flooring increases the strength of beams much more than that have been accounted for in the current design codes.

The basis of capacity design depends on the strength of the primary plastic hinges to be accurately assessed and the other structural members to be proportioned so that ductile beam-sway mechanism will form in an earthquake. The test results have highlighted the importance of the interaction between the floor slabs and the beams in assessing the strength of the plastic hinges.

To determine the level of strength enhancement, an analytical model that has the capability of analyzing elongation in the plastic hinges as well as the interaction between the floors and the beams is required. An attempt is currently being made to develop such a model. Once it has been verified with the experiment, it can be used to assess the seismic performance of RC buildings containing precast-prestressed floor units.

5 ACKNOWLEDGEMENTS

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