



## Seismic performance of r/c perimeter frames with slabs containing prestressed units

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**ABSTRACT:** Three structural sub-assemblies were constructed and tested to investigate the influence of precast prestressed components in the floors on the seismic performance of ductile perimeter reinforced concrete frames. The sub-assemblies represented a bent of a frame with four beams, two of which were internal spans and two cantilever spans. One of these sub-assemblies had a floor slab, which contained precast prestressed components, consisting of two spans added to one side of the frame. In this situation the prestressed components restrain elongation in plastic hinge zones in the perimeter beams, which increased their flexural strength. The addition of the slab was found to increase the lateral strength of the frame by approximately 150 percent. The stiffness of the slab allowed bending moments to be resisted by the cantilever spans. If allowance is made for this effect, the average flexural strength increase of each plastic hinge, due to the addition of the floor, was 78 percent. This has major implications for capacity design of perimeter frames.

A computer model of a plastic hinge has been developed, which enables the influence of axial compression on the hinge elongation to be modelled. This is an initial step in investigating the influence of elongation in frames and their interaction with floor slabs containing prestressed components.

### 1 INTRODUCTION

Reinforced concrete members subjected to inelastic cyclic loading sustain three types of deformation, namely, flexure, shear and elongation. The flexure and shear components of deformation have been extensively studied. However, elongation is often not observed, or measured, as it only induces structural actions in statically indeterminate structures. Tests show that plastic hinges typically elongate by 2.5 to 3.5% of the beam depth when design ductility levels of approximately 6 are sustained. The resulting elongation has been shown to have important implications for the seismic performance of structures (Fenwick & Megget 1995, Fenwick et al 1999, Restrepo 1993). However, the significance of elongation in plastic hinge zones in the beams of reinforced concrete perimeter frames and its associated interaction with prestressed floor slabs, has been overlooked. A project at the University of Auckland has been investigating this phenomenon and as part of the project, sub-assemblies have been built and tested. These tests have been reported in the literature (Lau 2001, Lau et al 2002). This paper describes some of the results of these tests and the initial stages of the development of an analytical method of modelling both plastic hinge rotation and elongation. It is planned to use this model to investigate the interaction of frames and floor slabs in different structural arrangements.

#### 1.1 Description of test units

The primary objective of this project was to investigate the influence of precast prestressed flooring components on the structural performance of perimeter frames. Three beam-column frame subassemblies were tested. The first and third of these modelled a level of a 2-bay frame with cantilever beam extensions on each end. The columns extended to mid height of the storeys above and

below the level. The second sub-assembly had the same frame as in the other two, but with the addition of a floor slab, incorporating precast-prestressed ribs, which spanned between transverse beams located at the central column and at the ends of the cantilever beams. An overhead view of the frame-slab unit is shown by Figure 1. The sub-assemblies were scaled to approximately 1/3 full scale.

To represent seismic actions lateral forces were applied at the top and bottom of columns. The testing arrangement was designed to minimise axial restraint to elongation by the supports and to keep the columns parallel to each other. This was achieved by using very small loading steps and at the start of each step adjusting displacements at the top and bottoms of the columns so that the elongation measured on the beam was added to the distance between the loading points on the columns. Additional details of the test arrangement and results can be found in reference (Lau 2001).



**Figure 1: Frame with floor slab test unit.**

## 1.2 Findings from tests

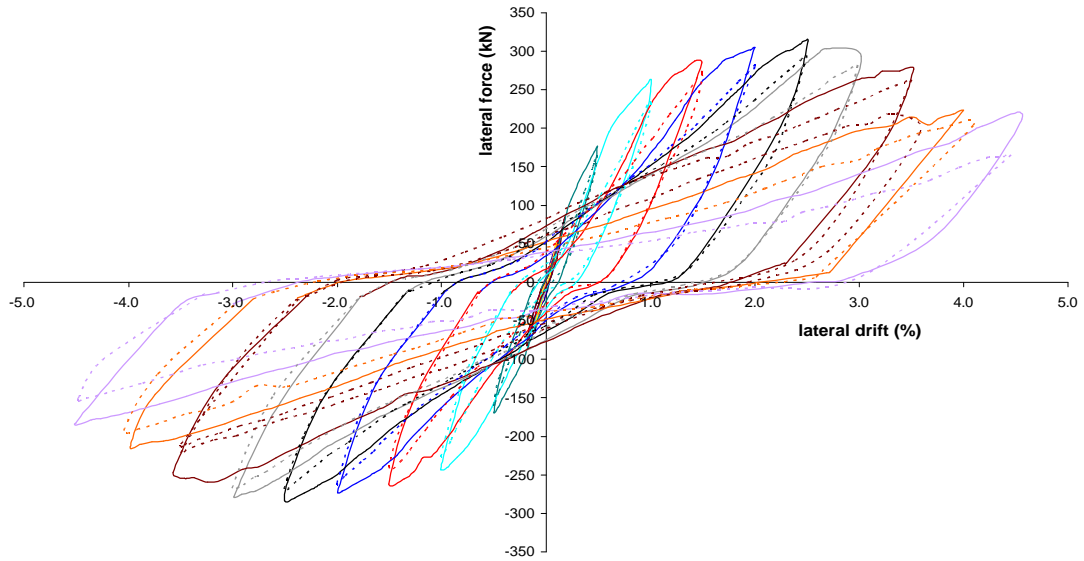
There was a substantial increase in both the strength and stiffness of the frame when a floor slab was added. Fig. 2 shows the storey shear versus inter-storey drift relationship (drift measured at the central column) for the units with and without floor slab. In the unit without the floor slab, only the beams in the internal bays were effective in contributing to the lateral strength. However, in the unit containing the floor slab it was found that yield strains were developed in the longitudinal reinforcement of the cantilever beams adjacent to the column faces in the load cycles with 1% and greater values of drift.

The addition of the slab had a major influence on the storey shear that was sustained, particularly in the cycles with maximum drifts between 1% to 3%. In this range the peak lateral forces varied from 2.3 to 2.7 times the corresponding values for the first unit and 2.4 to 3.2 times the corresponding values of the third unit. In the frames without the floor slabs only the 4 plastic hinges in the internal beams could develop. With the addition of the floor slab, as noted above, six plastic hinges formed. Allowing for this difference in the number of plastic hinges the corresponding average strength enhancement per plastic hinge, due to the addition of the floor slab, was 78 percent.

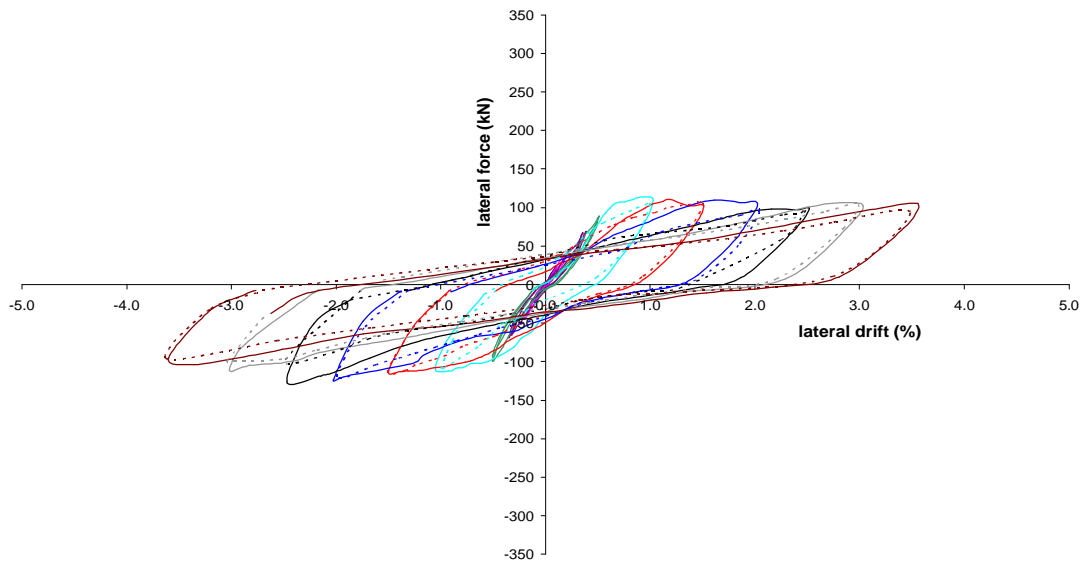
The strength enhancement can be attributed to two main features.

1. Firstly, the restraint provided by the pretensioned units to the elongation of the plastic hinge zones induces high shear forces at the interface between the slab and the beam. This effectively induces an eccentric axial compression force on the beam, which leads to a significant increase of the negative moment flexural strength of the beam.
2. Secondly, elongation of perimeter frame beams generates wide cracks in the topping concrete on each side of the central transverse beam, on which the pretensioned components are seated. Forces were transferred across these cracks by reinforcement in the in-situ concrete. In

addition the opening up of these cracks bends the slab in its own plane so that it acts as a deep beam, with bending moment and shear forces being sustained. These actions, together with the force transmitted across the cracks by the reinforcement, effectively apply an axial force to the beams in the region close to the central column in the frame. This action can substantially increase the flexural strength (Fenwick et al 1999).



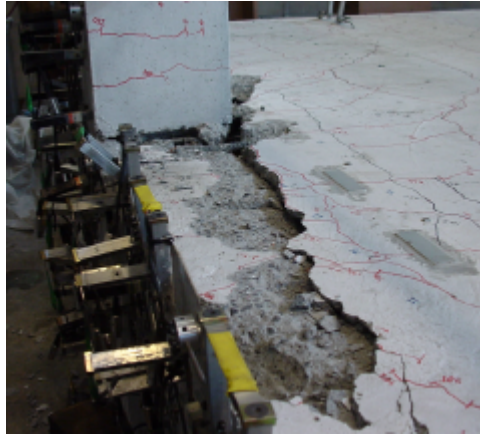
(a) Frame-floor slab unit



(b) Frame only unit

**Figure 2: Lateral load-displacement response of test units.**

In the frame-floor slab sub-assembly, wide cracks formed in the junction zone between the beam and the slab. These occurred over an appreciable distance along the length of the beam as can be seen from Fig. 3. It is believed that these cracks arose as a result of a combination of the in plane shear stresses, arising from the restraint to elongation, and the differential vertical movement between the beams and floor slab. As the frame is displaced laterally, the beams rotate about the ends (positive rotation one end and negative rotation on the other) causing the transverse beams at the ends of the cantilevers to rise and fall. However, the stiff prestressed ribs in the floor slab, try to remain straight, which results in relative vertical deflection between the slab and beam. The relative displacement across this crack caused the starter bars to kink, leading to failure of the beam slab connection.

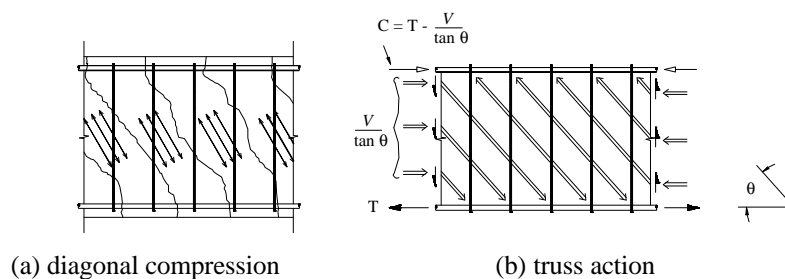


**Figure 3: Formation of wide cracks at interface of beam and floor slab.**

## 2 DEVELOPMENT OF ELONGATING HINGE MODEL

### 2.1 Mechanisms of elongation

With the formation of plastic hinges, such as may be expected in a major earthquake, elongation increases due to the extensive yielding of the longitudinal flexural tension reinforcement. In the initial inelastic displacement a small compression strain is sustained by the compression reinforcement. With the reversal of loading direction, the reinforcement in the compression zone, which had been yielding in tension in the previous loading half-cycle, does not fully yield back in compression. There are two reasons for this. Firstly aggregate particles become displaced in the cracks and these prevent the closure of the cracks, which is known as contact stress effect. Secondly, with the formation of intersecting diagonal cracks, shear is resisted by a truss-like action, as illustrated in Figure 4.

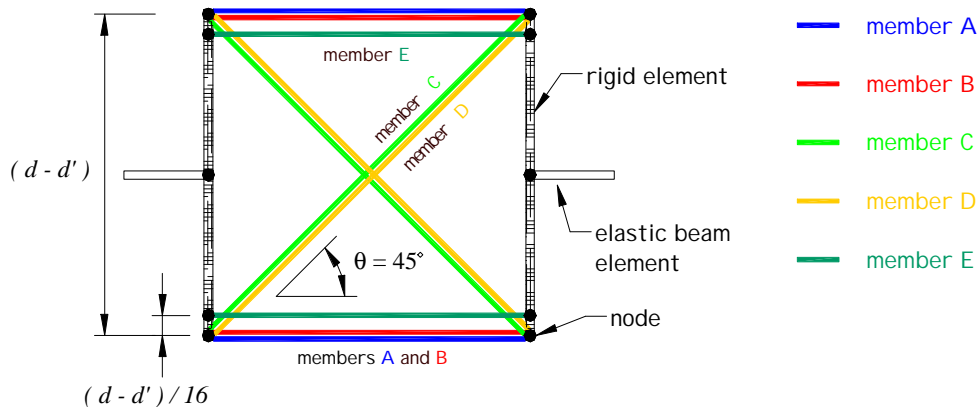


**Figure 4: Mechanism of shear resistance.**

With this mechanism the concrete sustains diagonal compression forces and the stirrups resist tension forces. From the corresponding equilibrium requirements shown in Fig. 4 (b), it can be seen that the flexural tension force,  $T$ , is always larger than the flexural compression force,  $C$ , at the same section. As a consequence inelastic rotations in the plastic hinge zone occur more by the yielding of the reinforcement in tension than by yielding in compression, and this leads to elongation.

### 2.2 Description of hinge model

The main intention is to develop an inelastic hinge model that includes the influence of axial load on both strength and elongation, so that it may be used to study the interaction between floor slabs and frames. The hinge model was developed using a non-linear building analysis program, SAP 2000 Non-linear v8. It is essentially a simple empirical plastic hinge model, which reproduces some of the features described in the previous section. It is suited to study the effects of elongation on overall structural behaviour.



**Figure 5: Model of elongating hinge.**

The model of the hinge is shown in Fig. 5. All the members within the hinge, except the rigid vertical end elements, are axial (truss) members only. Therefore their member stiffness is defined by:

$$\frac{EA}{L} \quad (1)$$

where  $E$  is the modulus of elasticity;  $A$  the area of member; and  $L$  its length.

The following is a brief description and parameters of each member in the plastic hinge model:

*Member A* – longitudinal steel reinforcement, where  $E$  and  $A$  have the appropriate elastic modulus and area for the reinforcement in the beam. A bilinear relationship is used and it is given a tension yield strength,  $A_s f_y$ , such that the product of  $A_s f_y (d - d')$  is equal to the nominal flexural strength of the rectangular beam (with no allowance for reinforcement in the slab). The strength of the member in compression is discussed in the next section. The strain hardening stiffness is matched to the properties of the reinforcement.

*Member B* – longitudinal concrete element representing the concrete in the compression zone, where  $E$  is based on the elastic modulus for concrete and the area,  $A$ , is taken as  $3A_s f_y / f_c'$ . This is an elastic compression member (forms gap under tension) and is sufficiently stiff (hence the multiplier 3) to avoid crushing under compression, particular when subjected to high axial loads.

*Member C* – diagonal concrete element, which represents the resistance provided by the concrete in the web. This is an elastic compression member, and it forms a gap when subjected to tension. Its stiffness is defined by the elastic modulus of the concrete and its area,  $A$ , is such that the vertical component of  $A_c f_c'$  is 80 percent of the shear resistance provided by the stirrup reinforcement in the potential plastic hinge zone. This shear resistance is calculated by the standard code equation (SNZ 1995) (ie.  $A_v f_v d / s$ ).

*Member D* – is a diagonal element, which resists tension and compression. The two “D” members together with the two “C” members resist the shear in the plastic hinge zone. The “D” members are important for shear resistance particularly at low load levels during unloading and reloading cycles, as they provide some shear resistance when both the diagonal compression members contain gaps. They effectively allow some shear to be resisted and they model the shear that would be carried by the contact stress effect by the diagonal compression members in the web. The stiffness of each of these members is 20% of the stiffness of member “C”. The vertical component of the sum of the strengths of both members is equal to 20% strength of the transverse reinforcement. Thus if the strength of a member is  $A f_y$  then  $A f_y \cos 45^\circ = \frac{1}{2} * 0.20 A_v f_v d / s$ .

*Member E* – represents the axial flexural compression member in a beam. This member, together with the adjusted yield strength in compression of member A, is used to calibrate the elongation characteristics of the hinge, particularly to account for axial load effects. The stiffness and yield strength vary with the reinforcing steel to concrete ratios in beams. Therefore a trial and error approach is required to calibrate this member and the yield strength in compression of member A.

### Calibration of Members A and E

To calibrate the hinge model so that it can simulate the effects of axial compression on elongation, the results of tests on cantilever beams (Fenwick & Davidson 1993) were used. The test beams were 500mm deep and 200mm wide with five 20mm longitudinal reinforcement in both the top and bottom of the beams. A “trial and error” method was used where the compression strength of member “A”, the stiffness of member “E” and both the tension and compressive strength of member “E” were adjusted so that the numerical results best fitted those recorded from the experiments. The results from this exercise were:

- (i) the compression strength of member “A” is  $0.6 A_s f_y$
- (ii) the stiffness of member “E” is  $0.15 E_s A_s$
- (iii) the tensile strength of member “E” is 0.0
- (iv) the compressive strength of member “E” is  $0.7 A_s f_y$

A model of the test specimen was formed with the stiffness and strength values described above and the “experiments” were simulated. In each test, the cantilever beam was subjected to the set of cyclic displacements and axial loads used in the tests. The axial forces were; zero,  $0.039 A_g f_c'$ ,  $0.068 A_g f_c'$  and  $0.145 A_g f_c'$ . The load-displacement response of the cantilever beams for both the experimental and numerical tests are shown in Fig. 6. It can be seen that the numerical results follow the experimental results quite closely. Fig. 7 shows the “force – deflection” curve for numerical test with no axial compression. It can be seen that the model fails to accurately follow the actual response that would be expected of a reinforced concrete member during the reloading phase. This is mainly due to the fact that the model does not predict shear deformation. However, the strength and displacements levels achieved are in satisfactory agreement with the experimental values.

For a further check of the performance of the hinge model, two tests performed by (Liddell 2000) were simulated. The cantilever beams used in his experiment were 600mm deep, 270mm wide, with four 16mm diameter bars in top and bottom of the beam. They were not subjected to axial compression, but were subjected to differing cyclic displacements. The first was subjected to the New Zealand test regime (Park 1989), the second a procedure often used at the University of California at Berkeley. The first requires two displacement cycles at each ductility for ductilities 2, 4, 6 and 8, the second, three displacement cycles at each ductility from ductility 2 through to ductility 8 at single ductility steps. The model of the hinge for these tests used the parameters according to the recommendations described in the section above. A summary of the results is given in Table 1. The model gives reasonable elongation readings at ductility 6 for both test procedures. It tends to overestimate the elongation at ductility 8 by quite a large margin for the New Zealand loading history. This may be due to the inability of the hinge to model to represent the contact stress effect due to aggregate particles becoming jammed in the diagonal cracks, and the buckling of reinforcement (causing elongation to decrease). It should be noted that there is considerable variability in recorded elongation values in different tests. For example, an elongation of 3.35% of beam depth was measured for the 500mm deep beam at ductility 6 (Fenwick and Davidson 1993), while with the same test procedure (Liddell 2000) recorded an elongation of 1.8% of beam depth for a 600mm deep beam. Typically, an elongation of 2.0 to 3.0% is expected for beams reaching a deformation of ductility 6, and as such these values can be used as a benchmark for comparison with the hinge model. From these initial tests, it is considered that the proposed plastic hinge model is working reasonably well.

**Table 1. Summary of elongation measured in beam tests and analytical model.**

	Elongation ( % of beam depth)			
	physical experiment		Analytical model	
	ductility 6	Ductility 8	ductility 6	ductility 8
New Zealand load history	1.8	2.7	2.5	4.4
UC Berkeley load history	3.9	4.7	3.2	5.4

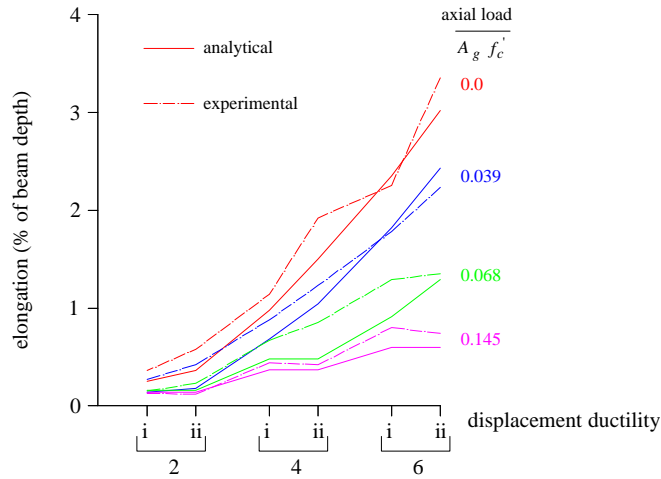


Figure 6: Elongation of beam with varying axial load levels.

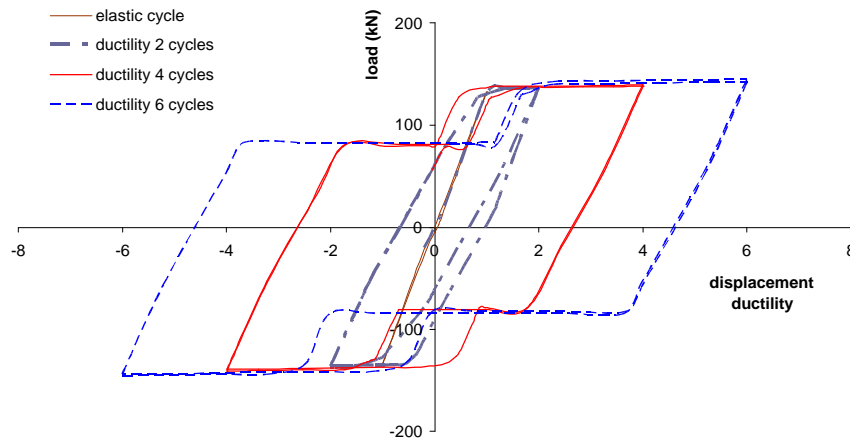


Figure 7: Load-displacement of cantilevered beam model.

### 3 FRAME MODEL WITH ELONGATING HINGE

A model of the two-bay bent test unit is shown in Fig. 8. This incorporates plastic hinge models on both sides of all the columns, a floor slab including the prestressed rib units and support beams. Non-linear elements were placed along the end supports of the floor spans to connect the floor units to the transverse support beams. The properties of these elements were defined to match the actual reinforcement (starter bars and mesh) used in the test unit. Along the interface of the floor slab and the beams of the frame, non-linear elements were placed and defined such that the dowel or kinking action in the reinforcement along this interface are modelled (at  $0.8 A_{st} f_y$ ). Further details are described in (Lau 2003).

The elongation measured for the experimental units and the model are shown in Fig. 9. The elongation is slightly underestimated by the computer model, where at 3% drift, the average value is 0.88% of beam depth compared to the measured value of 1.3% for the frame-slab unit.

The peak lateral strengths for different values of drift are compared in Figure 10. To enable a comparison to be made between the frames with the floor slab (six plastic hinges acting) and without the floor slab (four hinges acting), the lateral strengths of the frames without the slabs have been multiplied by 1.5. From this figure we see that over the drift range of 1.0 to 3.0%, the frame with the floor slab had an average increase in flexural strength per plastic hinge of 78% compared to the frame without the slab. The average increase in strength predicted from the analysis over this range was 52%. This underestimate of the strength increase by the model may be due to a number of factors, such as an underestimation of elongation, the stiffness of the prestressed ribs, and/or the shear force resisted by the starter bars along the edge of the frame beams and the floor slab. However, there is

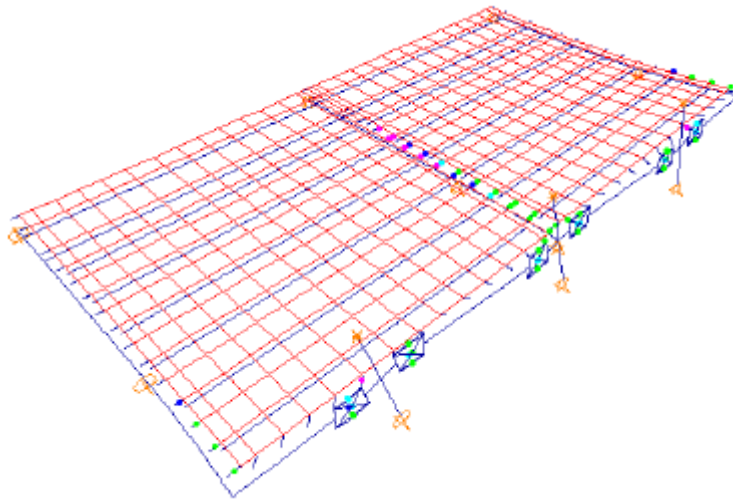


Figure 8: Graphical output of frame-slab unit incorporating elongating hinge.

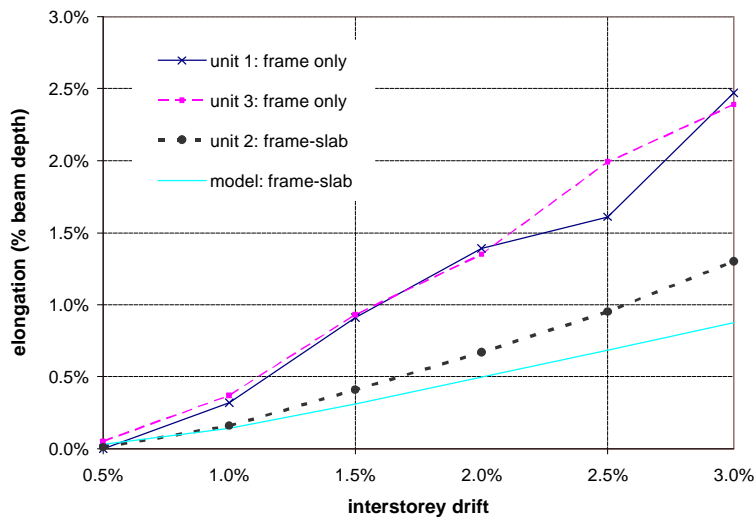


Figure 9: Elongation measured in experimental units and analytical model.

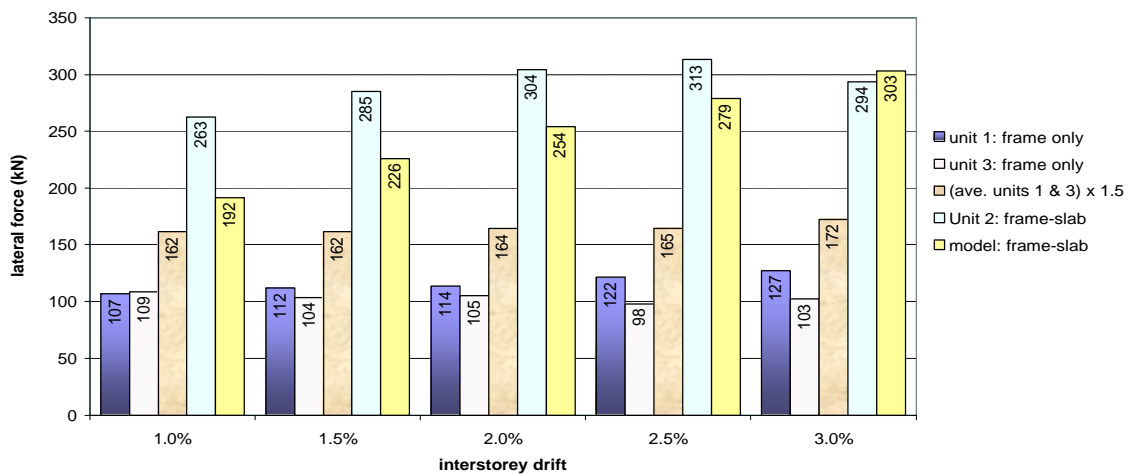


Figure 10: Lateral strength of experimental units and analytical model.

good agreement in the increase of lateral strength at the maximum drift level of 3%, where an increase of 81% was recorded in the frame-slab experimental unit and the predicted value from the analysis was 76%.

## 4 CONCLUSIONS

Experimental work shows that floor slabs, which contain prestressed components, can greatly increase the strength and stiffness of perimeter moment resisting frames. The extent of the increase in strength of the potential plastic hinge zones in the beams is such that in a major earthquake a non-ductile column failure mechanism may develop instead of the intended ductile beam sway failure mode.

A non-linear model of a plastic hinge zone is described in the paper. It is shown that the proposed model can predict the correct order of both elongation and strength enhancement of plastic hinge zones when axial loads, or up to a level of  $0.15A_g f'_c$ , are applied. An analysis of the composite frame–slab test unit indicated that it predicted the correct order of strength enhancement. Some further refinement of the proposed model is desirable to enable it to incorporate the effects of shear deformation and more realistically represent behaviour at low load levels. However, it does represent a significant step forward in that it enables interaction effects, which arise between different structural elements due to elongation associated with the formation of plastic hinges, to be assessed.

The response of structures in major earthquakes is more complex than is usually assumed in both design and research. The effects of elongation are seldom included in analyses. The analytical and experimental work described in this paper indicates just how significant elongation can be on behaviour, with strength and stiffness values for a frame and floor slab, which was examined both experimentally and analytically, more than doubling due to this effect.

The results of this study provide a warning against allowing nonlinear analysis as a means of obtaining a design form. Whereas existing nonlinear analysis computer programs may, with some effort, be used to model actual structural response, we presently have insufficient results from testing full scale (indeterminant) structures to include into the computer models second order effects (such as elongation) that can markedly influence structural behaviour.

## 5 ACKNOWLEDGEMENTS

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