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

NZSEE 2002 Conference

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ABSTRACT: Three tests were conducted to assess the influence that floors constructed with precast prestressed components have on the seismic performance of ductile perimeter reinforced concrete frames. All units represented a bent of a frame with two internal bays and two cantilever spans. One unit had a near identical frame but a floor slab containing precast concrete members was added to one side of the frame.

The addition of the precast floor was found to increase the lateral strength by a factor of about 2.7 for inter-storey drifts of between 1 and 3 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 zone due to the addition of the floor was 78 percent. Damage occurred at the interface between the beams and slab, due to both the shear transfer and relative vertical movement between the two.

1 BACKGROUND

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 significant structural actions in statically indeterminate structures. Elongation in a plastic hinge zone occurs as the tensile strains in the yielding reinforcement are greater than the corresponding compression strains. Tests show that plastic hinges typically elongate by 2 to 3% 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 1993, Fenwick et al 1999, Mejia-McMaster & Park 1994, Restrepo 1993).

With the formation of plastic hinges, such as may be expected in a major earthquake, elongation increases due to the extensive yielding of the reinforcement. This is illustrated in Fig. 1, which shows measurements

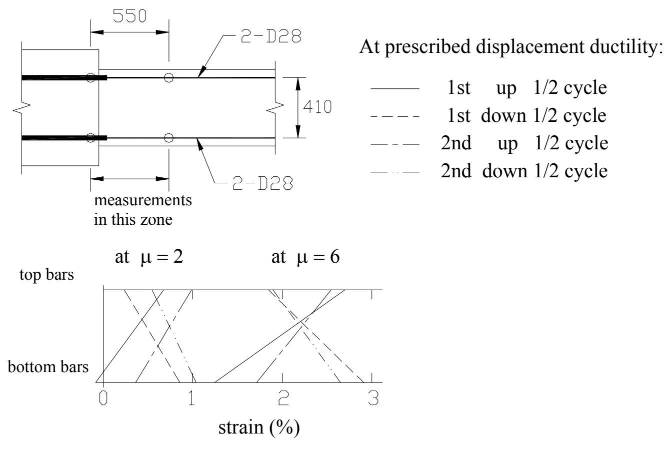


Fig. 1: Elongation of reinforcement in plastic hinge zone (Fenwick & Megget 1993).

made on the plastic hinge zone in a test beam. On 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 wedge the crack open. Secondly diagonal compression forces associated with shear reduce the magnitude of the flexural compression and as a consequence the tension reinforcement yields in preference to the compression reinforcement. With continued inelastic loading cycles, the compression reinforcement continues to elongate until buckling occurs.

2 PREVIOUS RESEARCH

Two approximately 1/3 scale test units were built and tested in 1995 (Fenwick et al 1995) to assess the influence that composite reinforced concrete slabs had on the seismic performance of internal frame structures. One of these units had a slab while the other was without. The reinforcing details in the main beams and columns were identical for both units. The unit with the composite slab had transverse beams, which cantilevered out from each column. The test arrangement for the two units is shown in Fig. 2. Hinges were fixed to the column bases and lateral forces were applied by reversing pin ended hydraulic actuators fixed to the top of each column as shown in the figure.

The presence of the slab significantly increased the strength and stiffness of the unit. The ductility 1 displacements for the units with and without the slab corresponded to inter-storey drifts of 0.95% and 0.67% respectively. For the unit without the slab the maximum lateral strength was 124kN, while for the unit with the slab the maximum lateral strength was 196kN. It was clear that the reinforcement in the full width of the slab, which was equal to 3.7 beam depths on each side of the web, was effective in contributing to the flexural strength when the inter-storey drift was of the order of 2%. The contribution of the slab to the over-strength of the beam was considerably greater than that indicated by the Concrete Standard (Standards New Zealand 1995).

For the unit without the slab, it was found that an average elongation of 2.3% of beam depth occurred per plastic hinge. For the composite slab unit the average elongation, when the elongation of the unit was a maximum, was an average of 1.9% of the beam depth per plastic hinge. There was little difference, in terms of elongation, between the units, indicating that the slab did not have a significant influence on this factor.

The test described above shows that composite reinforced concrete slabs do not significantly restrain elongation. When the slab reinforcement has been yielded it acts to prop open the cracks. However, where prestressed reinforcement is located in the slab very different actions can be anticipated with considerably restraint being provided to elongation. This effect could be expected to considerably enhance the flexural strength of the beams. Clearly it is important to assess the likely strength enhancement of the beams due to the restraint forces from the diaphragm. An underestimate could lead to non-ductile failure modes, such as shear failure in beams or forcing plastic hinges into columns, leading to possible column-sway failure modes in some situations.

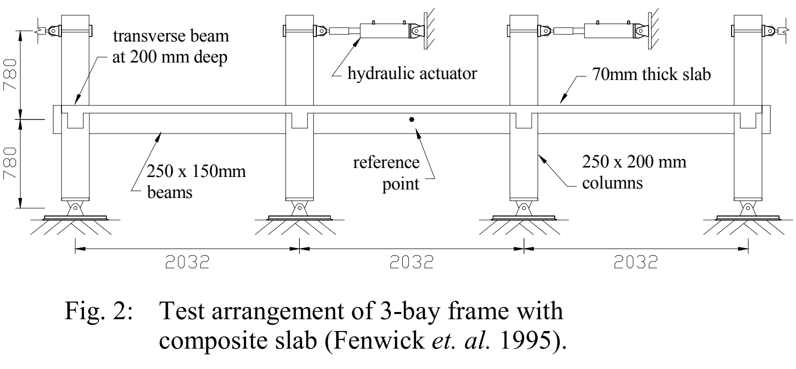


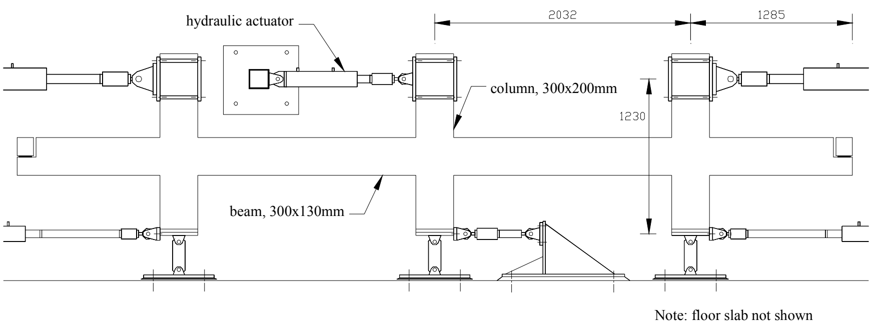
Fig. 2: Test arrangement of 3-bay frame with composite slab (Fenwick *et. al.* 1995).

3 DESCRIPTION OF TEST UNITS

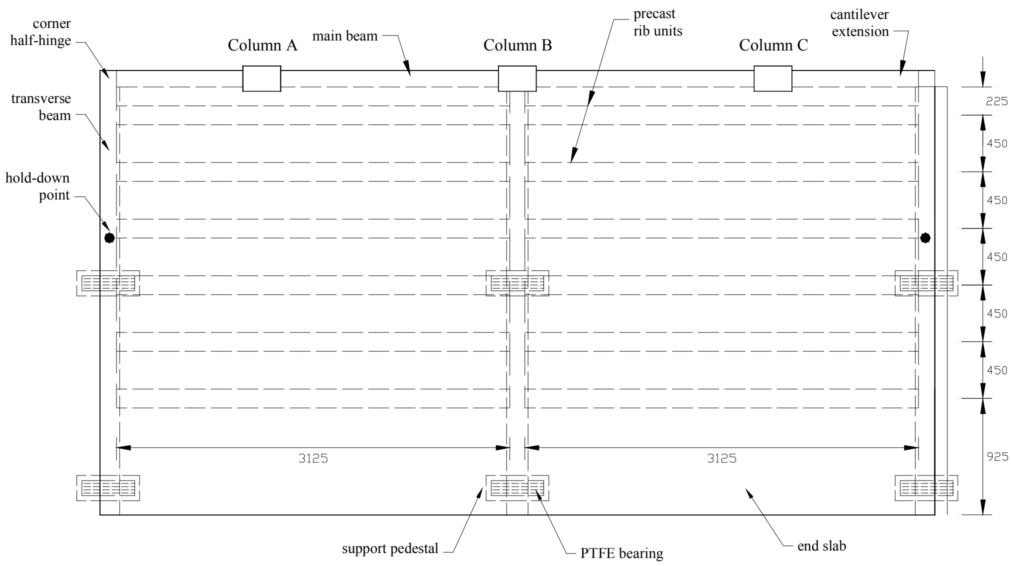
The primary objective of this project was to investigate the influence of a precast-prestressed flooring system on the structural performance of a perimeter frame. To achieve this, three beam-column frame subassemblies were tested. The first and third of these, Units 1 and 3, involved a level of a 2-bay frame with cantilever extensions on each end. The second test, Unit 2, contained the same frame as in the others, but with the addition of floor slabs, built incorporating precast-prestressed units, which spanned between transverse beams located at the central column and at the ends of the cantilever beams. The units and test arrangement is shown in Fig. 3.

The test frame was designed to represent one storey of a ductile-moment resisting perimeter frame of a multi-storey building, and as such, it would be expected to form reversing plastic hinges in the beams in the event of a design level earthquake. In the test units, cyclic lateral forces were applied to the top and bottom of the columns in order to simulate seismic loading. These positions represent the mid-height of a storey in a frame building, where the points of inflexion in the columns are expected.

The test units were detailed in accordance with the New Zealand Concrete Structures Standard (Standards New Zealand 1995). The test units were scaled to approximately 1/3 full scale, in order to accommodate the space in the test hall.



(a) Elevation view



(b) Plan view (Unit 2)

Fig. 3: Arrangement for structural tests.

The beams were 300mm deep and 130mm wide. They were reinforced with equal top and bottom longitudinal reinforcement consisting of three 12mm deformed grade 300 bars. The six bars ran the full length of the frame unit. The columns were 300 deep and 200mm wide. They were each reinforced with twelve 12mm deformed Grade 430 longitudinal running along the full height of each column. The columns were deliberately over-designed to ensure that they remained elastic under the anticipated strength enhancement due to the floor diaphragm in Unit 2. As shown in Fig. 3, the floor slab of Unit 2 consisted of two spans of precast 'rib and infill' units with 40mm of insitu concrete topping. Each precast unit spanned 3125mm from the outer transverse beam adjoined to the end of the cantilever in the main beam, to the central transverse beam extending from the central column. The first precast unit was spaced at 225mm from the face of the frame beam, and five subsequent units were spaced at 450mm centre-to-centre. At the sixth unit the insitu slab was increased in depth to 165mm and it was reinforced with six, high tensile 20mm reinforcing bars on each side. This zone represented the strength and stiffness of the slab, which for space reasons could not be added to the test unit. The floor slab was supported at two locations for each transverse beam by pedestals bolted to the strong floor. PTFE bearings were used to allow the floor to slide over the pedestals during testing.

The insitu concrete above the ribs was reinforced with mesh, which consisted of 3.125mm wires spaced at 75mm centre-to-centre in both directions. Joining the frame beam to the floor were 10mm deformed starter bars with centre-to-centre spacing of 225mm. The continuity reinforcing between each of the ribs and the central transverse beam was provided by two 4.0mm wires. A seating width of 25mm was provided for the ribs along the transverse beams.

Lateral loads were applied at the top and bottom of columns to induce moments into the beams. 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).

4 DISCUSSION OF TEST RESULTS

Unit 1 was loaded to ± 3/4 of the theoretical yield strength for two complete cycles. By linear interpolation of the displacement at this point, the ductility 1 displacement was determined. Two complete load cycles were to be applied at displacement ductilities of 2, 4 and 6.

For Unit 2 the ductility 1 displacement could not be accurately calculated as the yield strength of the unit could not be found due to uncertainty about the interaction of the frame and diaphragm. It was decided that this unit was to be displaced to predetermined inter-storey drift levels. For the initial elastic cycles, the unit was displaced to 0.1% inter-storey drift for at least two cycles, and then up to 0.2% for another two cycles. It was then displaced up to 0.5% inter-storey drift. Two displacement cycles for each target drift levels were taken and increments of 0.5% drift were taken thereafter until there was substantial falling off in performance. Unit 3 was tested in a similar fashion. Therefore the performance of Units 2 and 3 are more appropriate for comparison purposes.

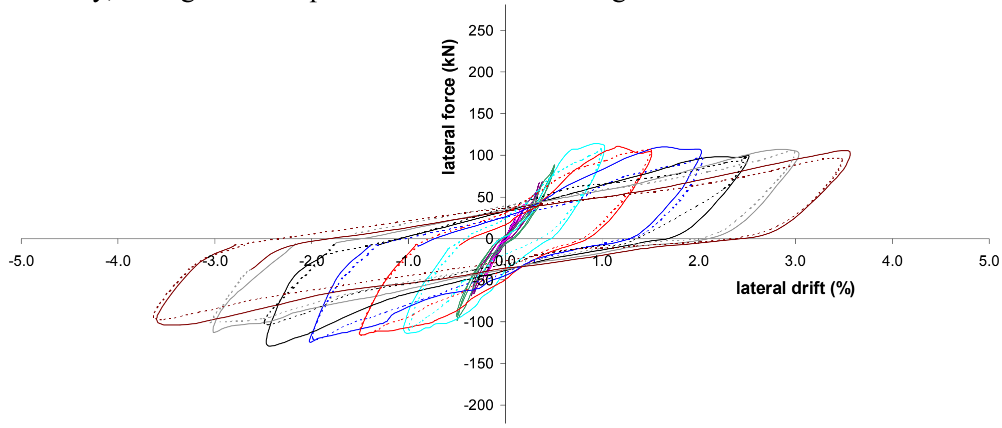
4.1 *Comparison of lateral strength*

As expected, there was a substantial increase in strength due to the presence of the floor slab. Fig. 4 shows the lateral force versus inter-storey drift relationship for units 2 and 3. Fig. 5 summarises the lateral forces resisted by the units at nominated inter-story drifts. For Unit 1, the lateral forces indicated were taken from the load cycle at which the nominated inter-storey drift level was first reached, while for Units 2 and 3 the values indicated are at peak displacement cycles. In Units 1 and 3, only the beam bays were effective in contributing to the lateral strength. On inspection of the strains in the longitudinal reinforcement of the cantilever ends near the column faces of Unit 2 indicated that the reinforcement was yielding from 1.0% drift onwards. Therefore for comparison purposes the average values for Units 1 and 3 were multiplied by 1.5, with the result being shown by the 3rd column in the charts.

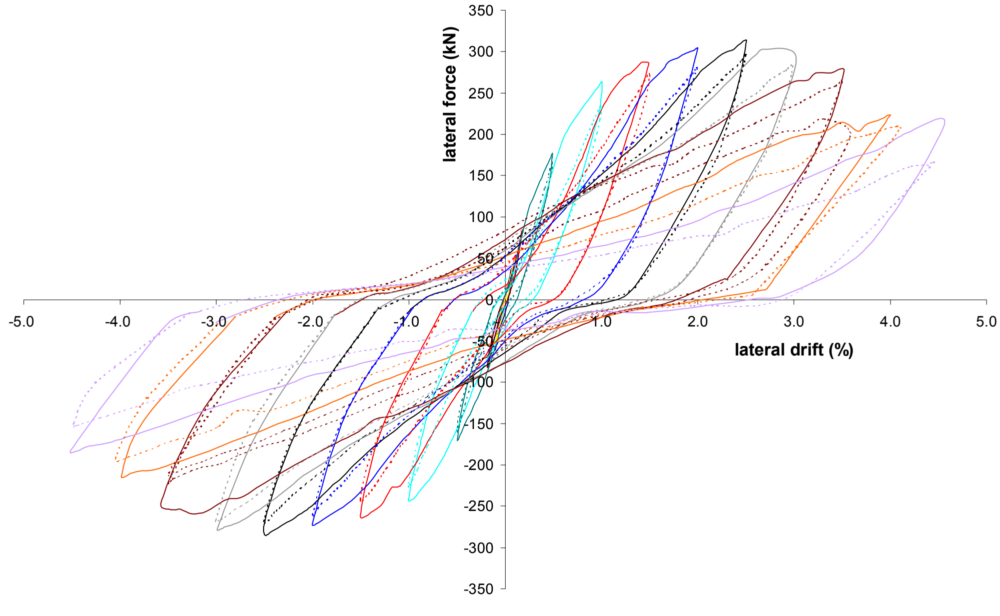
It can be seen from Fig. 5 that the addition of the slab had a major influence on the lateral force sustained particularly in the cycles with maximum drifts between 1.0% to 3.0%. In this range the peak lateral forces varied from 2.3 to 2.7 times the value of Unit 1 and 2.4 to 3.2 times the corresponding values of Unit 3. Allowing for the difference in the number of plastic hinges due to the cantilever extensions, the corresponding average strength enhancement per plastic hinge is 1.78.

The strength enhancement can be attributed to two main features.

1. Firstly, the restraint provided by the stiff pretensioned units to the elongating beams induced high shear forces at the interface between the slab and the beam. This led to significant increase of the negative moment flexural strength in the beam.
2. Secondly, elongation of perimeter frame beams generated wide cracks in the topping



(a) Unit 3



(b) Unit 2.

Fig. 4: Load-displacement response of test units.

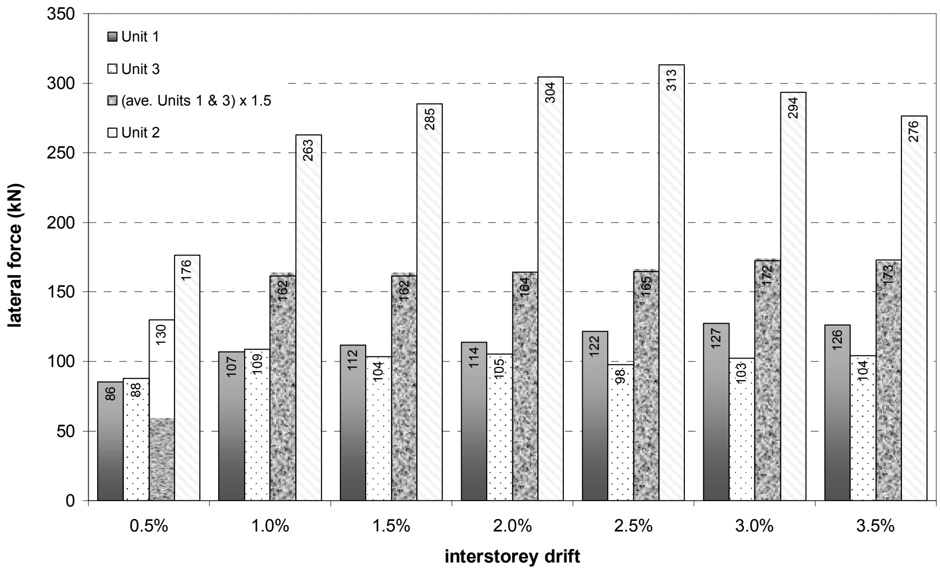


Fig. 5: Lateral strength comparison of test units.

concrete on either side of the 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 slab acts as a deep beam, with bending moment and shear forces being sustained. These actions together with the force transmitted across the cracks applied an axial force to the beams in the frame, which increased the flexural strength substantially. These actions are described in more detail in reference (Fenwick et at 1999).

4.2 *Elongation of beams*

Beam elongation measured in the tests is shown in Fig. 6. The values given are in terms of average elongation per plastic hinge as a percentage of beam depth (300mm). The maximum value of 3.2 percent was reached at 4.0% inter-storey drift (approximately displacement ductility 8). At 3.0% inter-storey drift (approx. displacement ductility 6), the average elongation for Units 1 and 3 was 2.4% of beam depth and was 1.3% for Unit 2. Clearly the floor slab restrained the beam elongation in Unit 2. As indicated by the figure, the elongation in Unit 2 rapidly increased from 3.0% inter-storey drift onwards, as greater damage occurred at the interface of the slab and the beams.

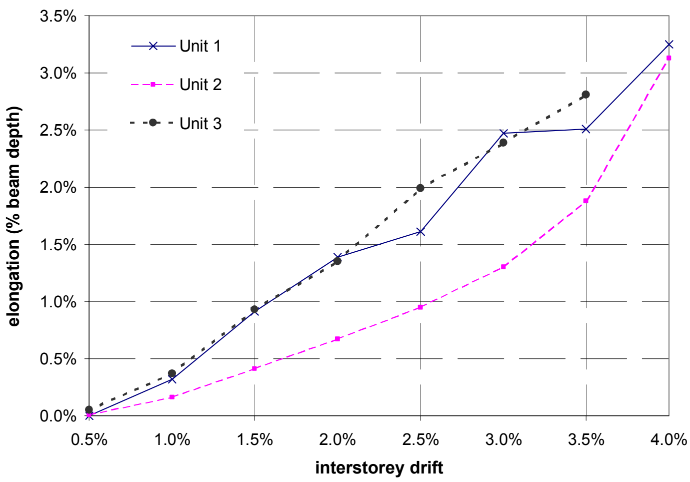


Fig. 6: Elongation of beams.

Elongation at 1.3% of beam depth per plastic hinge (when effectively restrained) at 3.0% inter-storey drift could still be significant when designing for seating widths of floor slabs, particularly when the slabs span across several beam spans. For example, for a 900mm deep beam and a floor slab unit spanning across three beam bays, up to 70mm could be expected.

4.3 *Relative stiffness of frames*

The test units were back analysed to find the proportion of the gross section properties that should be used to predict the ductility 1 displacement. The elastic modulus of the concrete was calculated from the equation given in the Concrete Standard and the effective section stiffness was doubled in the joint zones to represent the local stiffening in these regions. For Unit 1, the sum of lateral force corresponding to the nominal flexural strengths of the beam was 102.6kN. The average extrapolated ductility 1 displacement was 6.4mm, though there is uncertainty with the ductility 1 displacement determined from the test as the test unit was pre-cracked due to equipment failure at the initial stages of the test. However, using this value the gross section properties have to be multiplied by 0.32 to obtain the ductility 1 displacement. This value is reasonable in comparison with other tests. The values were 0.30 and 0.31 for the 3-bay frame with and without slab respectively (Fenwick et al 1995).

For Unit 3, the lateral force corresponding to the nominal flexural strengths of the beam was 103.6kN and the corresponding ductility 1 displacement was 6.7mm at an inter-storey drift of 0.55%. To obtain this value analytically the gross section properties are multiplied by 0.32.

For Unit 2, the ductility 1 displacement could not be determined since the yield strength of the unit could not be accurately calculated due to uncertainty concerning the interaction of the frame and the diaphragm. The 0.5% drift displacements (about ductility 1 displacement for Unit 3) and the sum of lateral forces recorded from the test to reach this level was used for the analysis. At this level, the average displacement was 6.1mm and the sum of lateral force was 172.8kN. To obtain this value the gross section properties were multiplied by 0.45. For this analysis, 160mm width of slab was included.

4.4 *Relative vertical movement of floor slab*

Vertical movement of the floor relative to the beams caused wide cracks to form with the slab separating from the beam over an appreciable length. This is shown by Fig. 7 and illustrated by Fig. 8. As the frame is displaced laterally, the beams rotate about the ends (positive rotation one end and negative rotation on the other). However, due to the stiff prestressed ribs, the slab had the tendency to remain straight. This appears to have led to failure of the slab connecting the diaphragm to the beam.

Figure 7: Relative vertical movement of slab and beam.

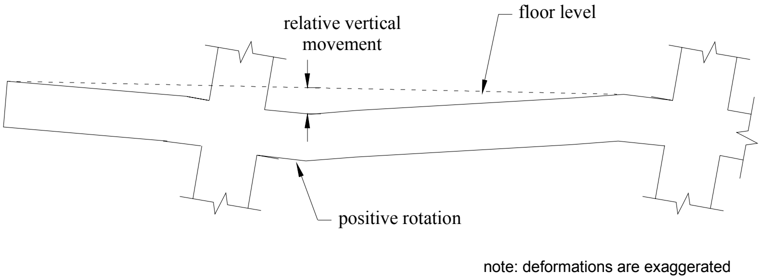


Figure 8: Illustration of vertical movement between floor and beam.

The relative vertical movement between the beam and the precast floor units could well be more important in cases where stiff precast units are used and where the first unit is close to the beam. In the test of Unit 2 there was a relatively flexible link, consisting of a 40mm thick slab with a clear span of 150mm between the beam and the first precast unit. If this link had been shorter, failure between the slab and beam would probably have occurred at an earlier stage. This could have important implications for the design of flooring systems.

5 CONCLUSIONS

1 With perimeter frame buildings the floor slab is supported on beams in such a way that that the slab spans directly past a number of columns in the perimeter frame.

2 The inclusion of a slab containing precast prestressed units was found to restrain the elongation, which developed in plastic hinge zones in the perimeter frames. This led to a very significant increase in strength. In these tests the addition of the slab to the perimeter frame was found to more than double the strength of the frame. It is clearly important to allow for this strength increase in the beams as an under-estimate could in some cases result in an unintended failure mechanism, such as a column sway mode, developing in preference to the intended ductile failure mode.

3 The perimeter frames that were tested had cantilever end spans, an arrangement that is frequently used to avoid problems with corner columns. In the test where there was no slab, bending moments were not induced in these cantilevers by the lateral forces. However, in the test with the slab the slab stiffness enabled plastic hinges to form in the cantilevers.

4 In the unit with the floor slab the precast units were found to remain straight between their support points. The beams deformed due to the formation of plastic hinges and this led to differential vertical deflection between the beams and the precast units. It was this movement which caused failure to occur.

6 ACKNOWLEDGEMENTS

This project is supported by the Earthquake Commission and the University of Auckland. Materials for the project were donated by Stahlton Prestressed Flooring Limited, Pacific Steel and Stevenson Concrete. These contributions are gratefully acknowledged.

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