Interaction between ductile RC perimeter frames and floor slabs containing precast units

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ABSTRACT: Lateral load tests of reinforced concrete perimeter frames with diaphragms have shown that the addition of a floor slab (diaphragm) can have a major influence on structural performance. At the University of Auckland three moment resisting frames were tested. Two of these frames were tested without a floor slab being attached to the beams, while the remaining frame was tested with the addition of a typical floor slab containing prestressed units. The tests showed that the addition of the floor slab increased the strength of the beams appreciably and as a result the lateral strength of the frame was increased by close to 150 percent. Clearly a strength increase of this order of magnitude is of major concern in seismic design in cases where it is essential to avoid the premature formation of a column sway mechanism. The test results presented together with an analytical study show the origins of this strength increase. Understanding these mechanisms is a first step in establishing a design method for assessing over-strength values in perimeter frames, which contain floors with prestressed units.

1.0 INTRODUCTION

Over the last three and a half decades major advances have been made in our understanding of the seismic response of structures. Detailed testing of structural elements such as moment resisting frames, braced frames and walls, together with extensive dynamic time history analyses has enabled comprehensive rules to be developed and codified for the design of structures to provide ductile lateral force resistance. These have been used in the design of individual components of seismic resistant structures. However, relatively little attention has been placed on the design of diaphragms and their interaction with other structural elements. It is only in the last decade that appreciable effort has been directed at research into this aspect. Ductile frame diaphragm subassemblies have been tested at both Auckland and Canterbury Universities during the last five years. The results have shown just how little we understand about the behaviour of diaphragms and their interaction with other structural elements. It is has been shown that they can have a major influence on the performance of other elements.

In 1998 detailed plans were drawn up to test the performance of diaphragms at both the University of Canterbury and University of Auckland. Close planning ensured that the two sets of tests were complementary to each other, with each test looking at different aspects of behaviour. At Canterbury the emphasis was on the way in which hollowcore units should be supported and how these units would interact with a perimeter frame. At Auckland smaller scale tests were planned for a perimeter frame and diaphragm, but in this case using a floor made of pretensioned stem type units with a cast insitu slab linking these members together. In the experimental phase of the project at Auckland two units representing part of a ductile moment resisting perimeter frame (units 1 and 3) were built and tested. A third identical perimeter frame (unit 2) was built and tested, only in this case a floor (or diaphragm) was added. The difference in performance between the units gave a qualitative measure of the influence of diaphragms on behaviour. The experimental study was followed by an analytical study in which the test units were modelled and then analysed under the same loading conditions as had been applied in the tests. The understanding gained from such tests and analyses provides the first step in establishing design rules.

1.1 Elongation in beams

When reinforced concrete beams with low or no axial load crack as a result of bending, elongation occurs. This situation arises as the strains on the tension side of the member are greater than the corresponding strains on the compression side, consequently the mid-depth of the member elongates. With the formation of plastic hinges, elongation becomes much more significant (Fenwick et al 1993). The restraint that slabs (diaphragms) can apply to this elongation causes the strength to increase. It should be noted that there is a marked difference in the way in which restraint is provided by a slab reinforced with passive reinforcement and a slab that contains either pretensioned or post-tensioned reinforcement, as outlined in the following paragraphs.

Where a slab containing only passive reinforcement is composite with a beam, the reinforcement in the slab adjacent to the flexural tension zone yields when a plastic hinge is formed. The tension force provided by this reinforcement increases the flexural strength of the member. However, when the structural actions are reversed, the reinforcement in the slab, which had previously been yielded in tension, does not yield back and it acts to prop open the crack in the compression zone. Tests have shown that there is little difference in the resultant elongation that arises in a rectangular beam and an identical beam with a composite slab (Fenwick et al 1995). This situation is fairly well covered by criteria in the current Structural Concrete Standard (NZS3101-1995).

Where a slab contains prestressed reinforcement the situation is different. The prestressed reinforcement may be in the form of post-tensioned cables or pretensioned elements within the floor slab. If this prestressed reinforcement is parallel to the beam, its greater force and elastic strain capacity acts to restrain elongation and it does not yield. In this process significant restraint can be applied to elongation and this increases the flexural strength of beams. Investigating this action was one of the principle objectives of the project at Auckland.

2.0 STRUCTURAL TESTS

At the University of Auckland the three test units were designed to represent a level of a ductile moment resisting perimeter frame, with the columns extending between the mid-height of adjacent stories. The units were approximately one-third full scale. The three frames were essentially identical to each other. Each frame comprised two full bays with two cantilever spans. Under seismic actions the lateral load resistance was limited by the flexural strength of the beams with plastic hinges forming against the column faces. To ensure that the plastic hinges were confined to the beams, the columns were designed to be significantly stronger than the beams. The key details of the frames are shown in Fig. 1. The test arrangement is illustrated in Fig. 2, with the lateral forces being introduced by hydraulic jacks acting at the mid storey height in the top level with further jacks resisting lateral forces in the mid-height of the lower storey. In the test small increments of lateral load were applied at each stage with adjustments being made to keep the columns parallel to each other.

With test unit two a floor slab was added as shown in Fig. 3. The frame was identical to that in units 1 and 3 except for the addition of the floor. The floor was made up of prestressed stem units spaced at 450mm centre to centre with a 40mm thick cast-in-situ concrete topping to give composite action. The precast units were supported by three beams, which were transverse to the frame. The central one of the beams framed into the central column in the perimeter frame, while the other two were supported on half hinges located at the ends of the cantilever beams. The half hinge supports acted as pin supports. The transverse beams in turn were supported on PTFE bearings to allow them to slide freely when lateral forces were applied to the columns in the frame. With this arrangement the precast prestressed floor units spanned directly past two of the columns but were supported on the central transverse beam.

The three units were each subjected to cyclic loading, but for a number of reasons detailed in a previous report (Lau 2001) different load cycles were applied to each unit. However, a comparison of the lateral strengths of the units at different drift levels, as shown in Fig. 4, shows very clearly the importance of the slab on the lateral strength. In units 1 and 3 only four plastic hinges could develop under cyclic loading. However, in the second unit, which had the floor slab acting compositely with

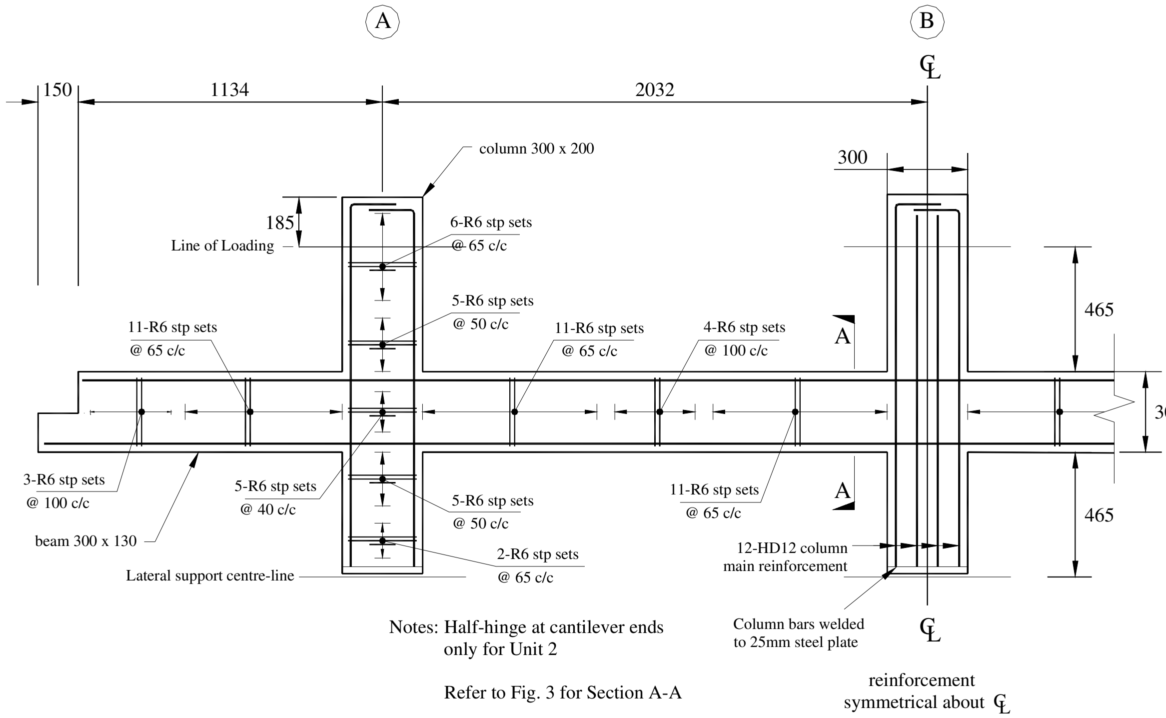

Fig. 1: Details of reinforcement in all three frames.

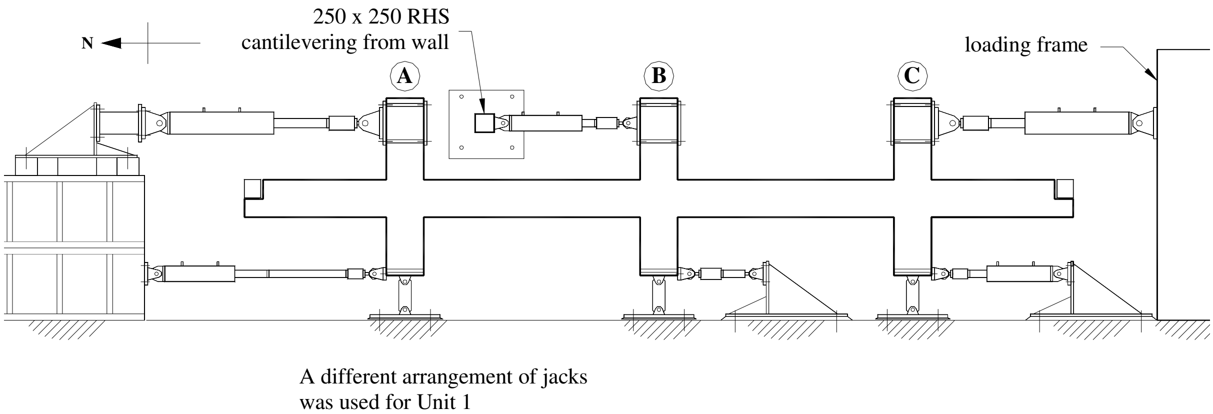

Fig. 2: Schematic of test arrangement for Units 2 and 3.

the beams in the perimeter frame, the flexural stiffness of the transverse beams combined with the slab enabled two additional plastic hinges to form, giving a 50 percent increase in the number of plastic hinges. For this reason in Fig. 4 the average lateral force resistance of units 1 and 3 has been increased by 50 percent to enable comparisons to be made with unit 2. However, even with this adjustment it can be seen that the slab has increased the average flexural strength of the plastic hinges by close to 80 percent.

2.1 Strength increase due to addition of slab where the precast units span past the columns

The restraint that the floor slab provides to elongation is equivalent to the slab providing an additional tension force capacity, T_{slab}, to the beams. This is illustrated in Fig. 5. The tension force is of similar magnitude on each side of a column, and it acts at a level close to the centre of the insitu slab. With reference to Fig. 5 (c) it can be seen that on the positive moment side of the column the beam sustains the tension force carried by the slab in the compression zone, together with a tension force in the bottom reinforcement in the beam. These two forces are balanced by a compression force in the concrete at the top of the beam. As the insitu concrete in the slab acts with the beam the resultant

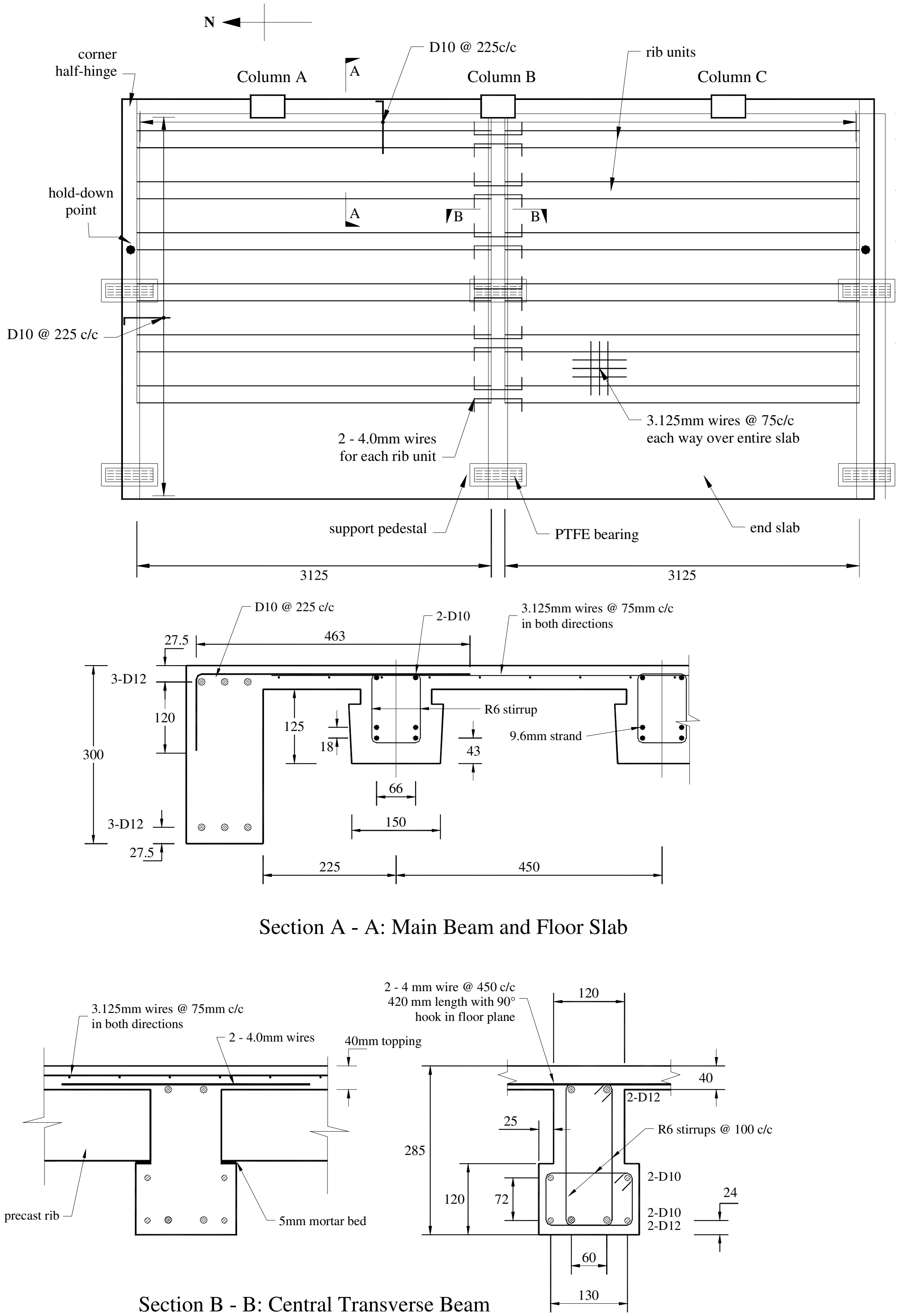

Fig. 3: Details of unit 2 (note that the section is the same one in each view but with different information)

compression force can spread into the slab. For the purpose of calculation it has been assumed that the insitu concrete within a distance equal to 4 times the slab thickness of the beam (see Fig. 5 c) acts as the compression zone. Hence the tension force in the slab, T_{slab}, is effectively cancelled out by an equal (but opposite) compression force in the concrete. The net result is that the tension force in the slab makes little difference to the moment capacity on the positive moment side of the column.

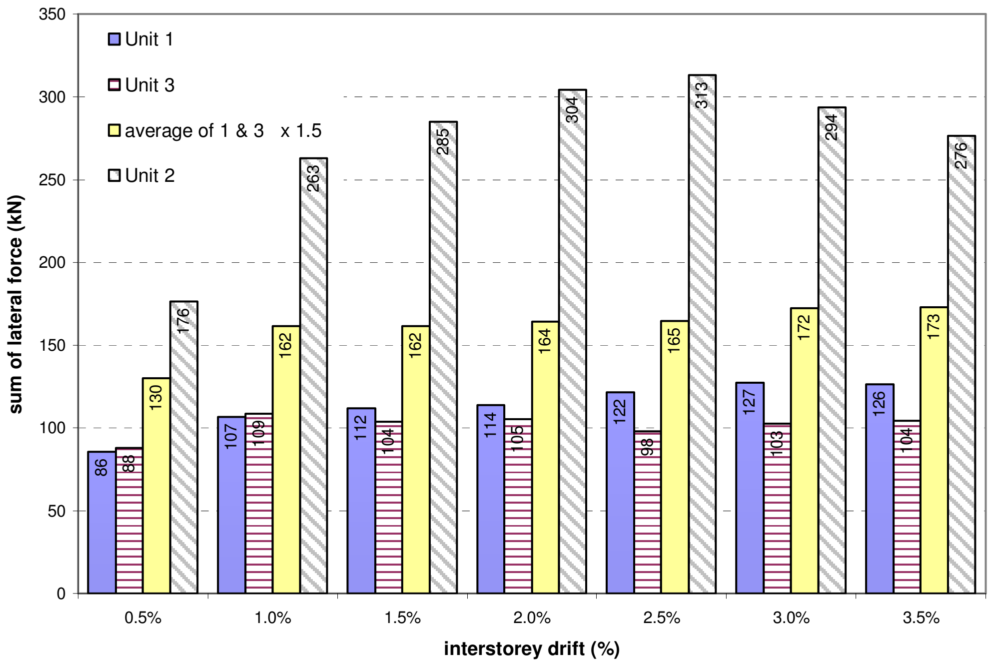

Fig. 4: Lateral shear resisted by frames at peak displacements in positive direction.

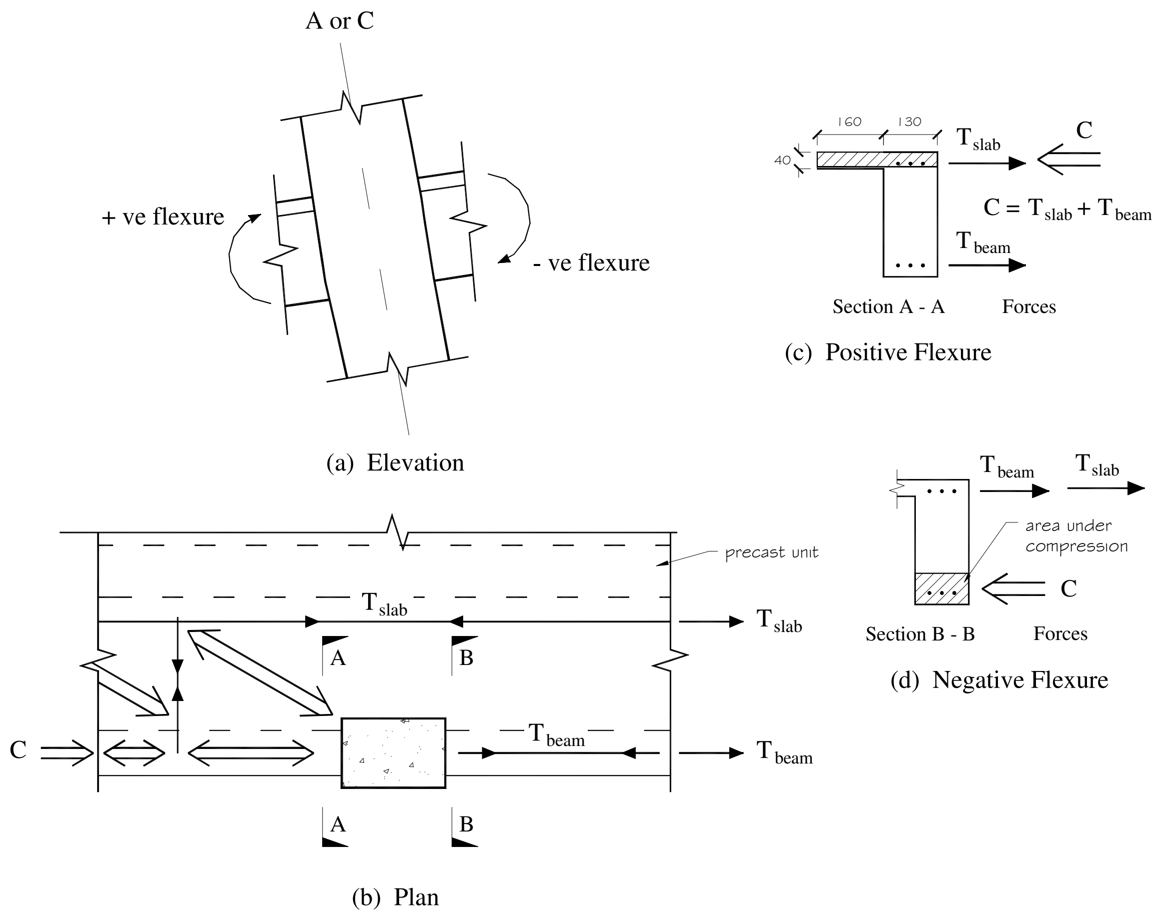

Fig. 5: Strength increase for beam at columns A & C.

The situation is different on the negative moment side. In this case the effective tension force in the slab (T_{slab}) acts directly with the tension force carried by the reinforcement in the beam, with these two forces being balanced by a compression force in the bottom of the beam. The moment capacity is directly increased as is illustrated in Fig. 5(d).

To assess the magnitudes of the tension force resisted by the slab in unit 2 at columns A and C, at different load stages, the following steps were undertaken.

1 The sum of the bending moments resisted by the column at different load stages was calculated from the jacking forces acting on the columns. In addition the corresponding axial forces acting on the beams were determined.

2 The sections on each side of the column were analysed when subjected to the axial forces calculated in 1 above and with an arbitrary effective tension force sustained by the slab, T_{slab}, acting at the mid-height of the 40mm thick in-situ-concrete. This arbitrary force was varied until the sum of the moment capacities on each side of the column was equal to the experimentally measured value in the test at the load stage being considered. This gave the effective tension force resisted by the slab (T_{slab}).

The theoretical strength of each beam calculated in step 2, was found using the standard rectangular stress block described in NZS 3101-1995 with 30MPa concrete and with a bilinear stress strain model for the reinforcement. Based on tensile tests of the reinforcement the yield-stress was taken as 320MPa and a stress of 420MPa at a strain of 10 percent. The concrete strength in all the units was very close to 30MPa. To assess the reliability of the predicted strengths the approach was used to predict the strengths of units 1 and 3, which were plane frames. These predictions were compared with experimentally measured values and a summary is given in Table 1. With this approach one would expect the predicted strengths to be within a few percent of the experimental values. However, two problems reduced the accuracy. Firstly, there was a problem with the data logger and secondly in unit 1 the forces acting on the individual columns had to be found by taking the difference of several loads in the jacks. This reduced the accuracy of the test results for this unit. In spite of the problems the average predicted strength in the load cycles to a given displacement value is in reasonable agreement with the experimental value. The approach appeared to be sufficiently reliable for it to be used on unit 2 to assess the effective tension forces resisted by the slab, T_{slab}, though the effective slab tension forces may be high by a few percent.

Table 1: Ratio of measured divided by predicted strengths for columns in units 1 and 3

Sway	Ratio of measured moment input to columns to theoretical value	
	Unit 1	Unit 3
± 1 %	0.85	0.95
± 2 %	1.07	1.02
±2.5 %	1.12	1.05
± 3 %	1.09	0.98

The approach described above was applied to the columns A and C in unit 2. The average predicted equivalent tension forces sustained by the slab in the positive and negative peak displacements for the two columns are given in Table 2. It can be seen that the slab made an appreciable contribution to strength. The tension force capacity of the reinforcement in the beam at yield was 109kN. In the last three displacement cycles reported in Table 2 the average equivalent tension force sustained by the slab was also equal to 109kN, hence approximately doubling the strength of the beam. It should be noted that there was appreciable variation in the individual values for each column and in the different cycles to the same ± displacement. Much of the variation can be simply explained.

Table 2: Equivalent tension force resisted by slab at columns A and C in unit 2

Sway	Average equivalent slab tension force (kN)
± 1 %	38
± 2 %	102
± 2.5 %	121
± 3 %	104

The question arises as to how this corresponds to values, which might be calculated by practising structural designers. The current structural concrete Standard indicates that the strength of the slab within a width a quarter of the span of the beam (508mm) from the centre-line of the column contributes to the strength of the beam. There are several ways that practising engineers might set out to assess this, as set out below.

1 The slab reinforcement located in the 40mm topping concrete within this distance of the beam was equal to six 3.125mm and two 10mm bars. Using the measured yield stresses of 408MPa and 312MPa for the 3.125mm and 10mm bars respectively the tension force at yield is equal to 68kN, or at 6% strain, which approximately corresponds to the value at over-strength, the corresponding force is 77kN.

2 The strength of the precast prestressed stem might be included in this calculation. As the tension force acts at the mid-height of the slab it is eccentric to the prestressed section and in calculating the strength of the stem allowance must be made for the gravity load bending moments which act on it. Within the width of 508mm from the beam centre-line there is only one stem. In terms of the Standard this stem is close enough to the column for it to be effectively anchored (see NZS3101-1995, Fig. C8.10). An ultimate strength analysis indicates that provided that full gravity load acts, each stem can resist an eccentric force of 72kN. Adding this value to the 77kN sustained by the passive reinforcement in the slab gives a tension force capacity of 149kN. However, for the stem to sustain the 72kN extensive cracking should have developed across the top surface of the stem, and such cracking was not observed.

3 Another option might be to see what the pretensioned concrete stem and insitu concrete might resist without cracking on the top surface. This gives an answer of 44kN per unit for a cracking stress of 2.9MPa in the topping concrete. This calculation ignores any existing stresses in this concrete due to creep or shrinkage of the concrete. With the assumption of un-cracked concrete the tensile capacity of the reinforcement in the insitu concrete cannot be added in.

The 40mm thick insitu slab connecting the beam to the first pretensioned stem unit is referred to as the linking slab. In the displacement cycles to ±2 % drift appreciable spalling occurred in the linking slab adjacent to the columns. In this region relative vertical displacements developed between the beam and the first stem. It appears that this spalling was a result of the combined actions of local bending in the “linking” slab, due to this vertical movement, and the shear transfer induced by the restraining action of the slab to elongation of the beams. In the displacement cycles to ±2.5 percent drift this spalling extended to approximately 300mm from the column faces, while in the ±3 percent cycles it extended in one case to the middle of one of the beam spans. This spalling must have reduced the confinement that the diaphragm provided to the beam plastic hinges adjacent to columns A and C, and hence reduced the effective tensile force in the slab (T_{slab}) and flexural strength of the beams.

2.2 Strength increase where prestressed units are supported on transverse beam at the column

In this case, which is represented by column B in unit 2, the pretensioned units are supported on the transverse beam, which framed into the central column (B) of the perimeter frame. When plastic hinges form in the perimeter beams at this column elongation occurs, as illustrated in Fig. 6, and compression is induced in the beams and a tension force in the slab. This movement causes cracks to develop in the insitu concrete between the transverse beam and the ends of the prestressed units. The compression force in the beams is balanced by an effective tension force in the slab. This tension force, T_{slab}, is made up of-

- tension forces carried by the reinforcement in the insitu concrete connecting the two halves of the slab at the central transverse beam,
- shear force resisted by bending action of the floor slab, which acts as a deep beam.

These two actions are illustrated in Fig. 6.

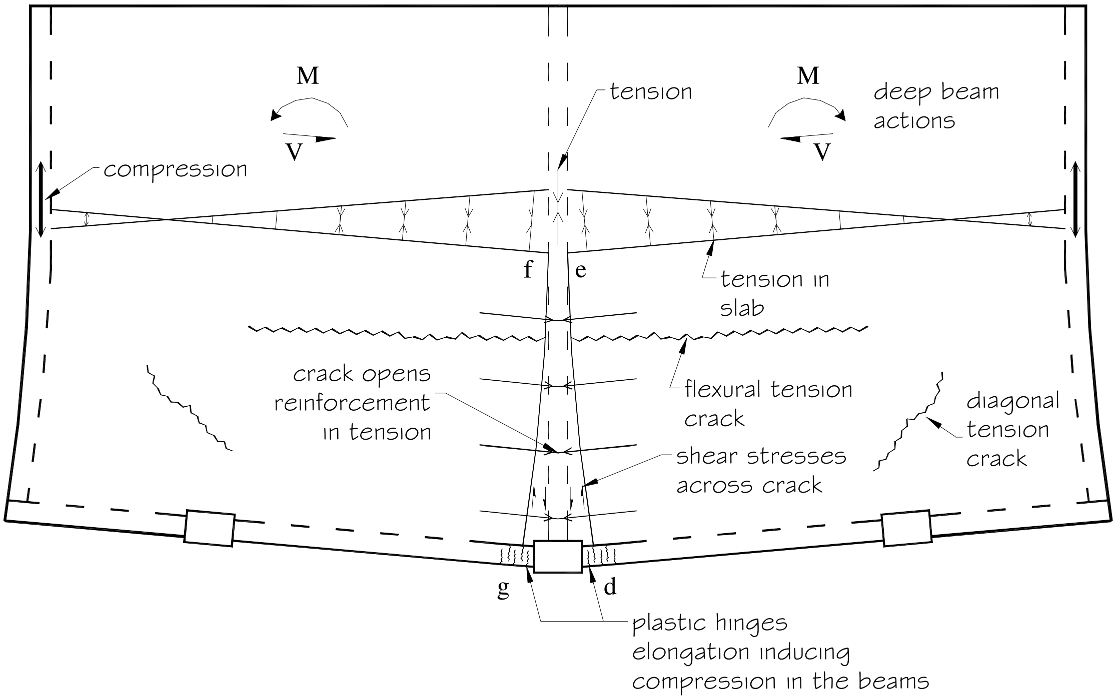

Fig. 6: Floor slab acting as a deep beam due to elongation in plastic hinge regions.

The deep beam action referred to above induces shear, hence diagonal tension and diagonal compression as well as flexural stresses in the floor. Fig. 7 shows the crack pattern sustained in the floor at a drift of 3 percent. It can be seen that the slab contains both flexural tension and diagonal tension cracks consistent with the deep beam mechanism, with the extent of these cracks indicating that significant force was resisted by this action.

To find the contribution of the slab to the flexural strength of the beams the same process that was used for columns A and C was followed. That is the axial load applied to the beams at different loading stages was found from the jacking forces applied to the columns. Using these values the theoretical strengths of the beams were determined assuming an arbitrary tension force, T_{slab}, was resisted by the floor at the mid-height of the insitu concrete. This arbitrary force was changed on a trial and error basis until the sum of the bending moments resisted by the beams equalled the measured sum of the bending moments in the columns. Having obtained the T_{slab} values for different loading stages the next step was to determining what proportion of this force was carried by the reinforcement crossing the central cracks, d - e and g – f on Fig. 6. The remaining force is resisted by deep beam action of the slab.

The forces sustained by the reinforcement crossing the central cracks, d – e and f - g on Fig. 6, were assessed from crack width measurements made during the test and the stress strain response of the bars. There were two different sets of reinforcement involved. The first consisted of a welded mesh of ductile plain 3.125mm bars at 75mm centres. This was welded up in the laboratory and it was cast into the 40mm thick insitu topping concrete. These 3.125mm bars had a yield stress of 408MPa and an ultimate stress of 480MPa at a strain of about 12.5 percent. The second set of reinforcement consisted of two 4mm diameter deformed bars located directly above each precast concrete stem unit. These bars, which are shown in Fig. 3, were 420mm in length, and they had yield strength of 430MPa with an ultimate stress of 492MPa at a strain of close to 12 percent. The strains in the reinforcement at the cracks were assessed from the crack widths by assuming effective lengths of 75mm for the mesh bars and 120mm for the 4mm bars. The change in strain between the no load stage and peak displacement was used together with the measured monotonic stress strain response of the reinforcement to find the forces, as illustrated in Fig. 8. As the majority of the predicted strains were well above the yield point the strain levels did not have to be known accurately to obtain reasonable estimates of the forces at the different load stages.

A summary of the results of the analysis is given in Table 3. The average values are given for the positive and negative peak load displacements to a set displacement level. As with columns A and C there was appreciable variation in the individual values at each peak displacement. The bending moments resisted by the slabs, acting as deep beams, have been calculated for a section located 1,940mm from the centre-line of the perimeter frame. This location was chosen as it is close to the end of the cracks, which formed along the central beam, at a drift of 3 percent. To find the effective contribution of the forces carried by the reinforcement across the crack, T_{rein}, moments were taken about this section.

Table 3: Contribution of slab to tension force on column B of unit 2.

Drift (%)	T_{slab} (kN)	T_{rein} * (kN)	Sum reinforcement forces (kN)	Actions resisted by slab as deep beam: Shear force# (kN)	Actions resisted by slab as deep beam: Bending moment (kNm)
1	165	16	22	165-143	290
2	208	57	93	208-115	292
2.5	220	60	101	220-119	308
3	205	62	104	205-101	278

* T_{rein} is the equivalent tension force from the slab reinforcement acting with the perimeter beams.

The higher value is the shear at the face of the perimeter beam and the lower value is at 1940mm from the centre-line of the perimeter frame.

From the table it can be seen that about one quarter of the increase in strength due to the addition of the slab comes from the reinforcement in the topping concrete. The remainder comes from deep beam action of the slab. If the criteria in NZS3101-1995 was used the tension force capacity of the slab reinforcement would have been taken as 21kN. This value is based on the assumption that strain-hardening increases the stress to 1.1 times the yield stress. This value is only a small fraction of the experimentally determined tension force.

The high contribution of deep beam action in the slab to the strength of the beams adjacent to the central column is supported by the crack pattern (see Fig. 7). The shear force in the slab averages about 210kN in the 2, 2.5 and 3 percent drift cycles. This corresponds to an average shear stress of 1.7MPa, which is more than sufficient to account for the diagonal tension cracks that can be observed in Fig. 7. The average bending moment at the section 1940mm from the frame centre-line is close to 290kNm. This magnitude of moment can be sustained provided that the longitudinal reinforcement in the central transverse beam can act in tension. This may occur provided some shear stresses are transmitted across the cracks (e – f and g – h in Fig. 6) by aggregate interlock and dowel action of the reinforcement. If this shear transfer is fully effective the moment capacity is close to 570kNm, which is well in excess of the calculated moment of 290kNm, which acted on this section.

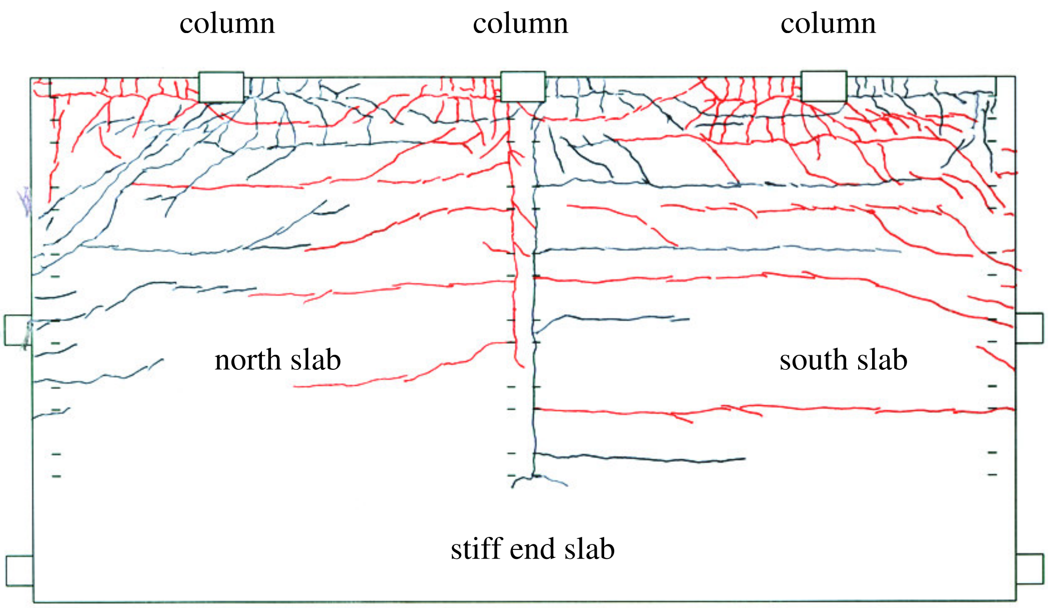

Fig. 7: Crack pattern in floor slab at 3% drift.

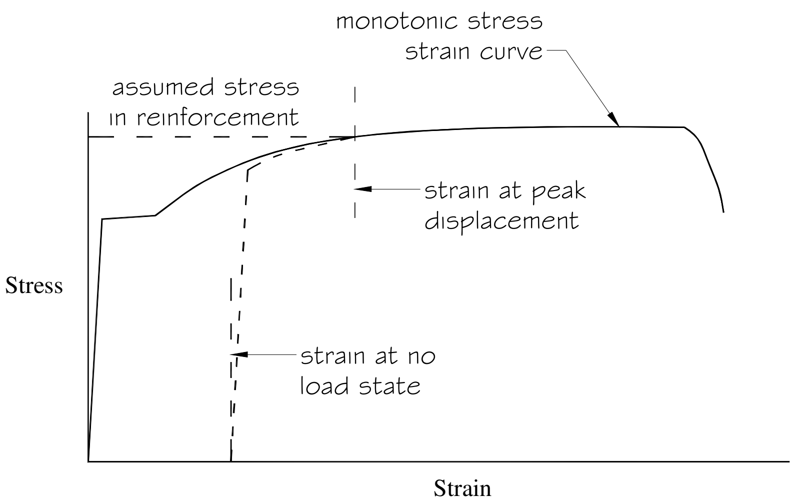

Fig. 8: Assessing stress levels in reinforcement crossing cracks (d – e and f – g)

3 NUMERICAL MODEL

Finite element models have been developed to help explore the mechanics of the behaviour of units 2 and 3. These models have been constructed using the computer program SAP2000. This program was chosen as it is well known and readily available to structural engineers. It has an added advantage in that it has a good graphical interface, which allows easy checking of the model. The essence of the model was the development of a substructure to represent the elongation effect of the beam plastic hinges. Elongation of a plastic hinge is dependent on the rotation and shear force history of the hinge together with the axial force. The hinge substructure model used here has been described previously (Lau et al 2003), and it provides a reasonable approximation to the hinge behaviour. It does however fail to adequately describe the observed degradation of stiffness as a consequence of there not being suitable elements in the SAP2000 library. Consequently the analytical model under-estimates pinching in the load deflection relationship.

Figure 9 illustrates the SAP2000 model for unit 3. Frame elements were used to model the columns, the perimeter beams away from the hinge zones, the edge beams and the central beam. Frame elements were also used to model the prestressed floor rib units. Shell elements were used to model the floor slab. A large amount of effort was made to include in the model the difference in elevation of the centroids of the different beams and the floor slab. In unit 2 non–linear connections were made to the model between the floor slab and the central transverse beam. In addition non-linear elements were used to model the linking slab (between the perimeter beams and first stem in the floor).

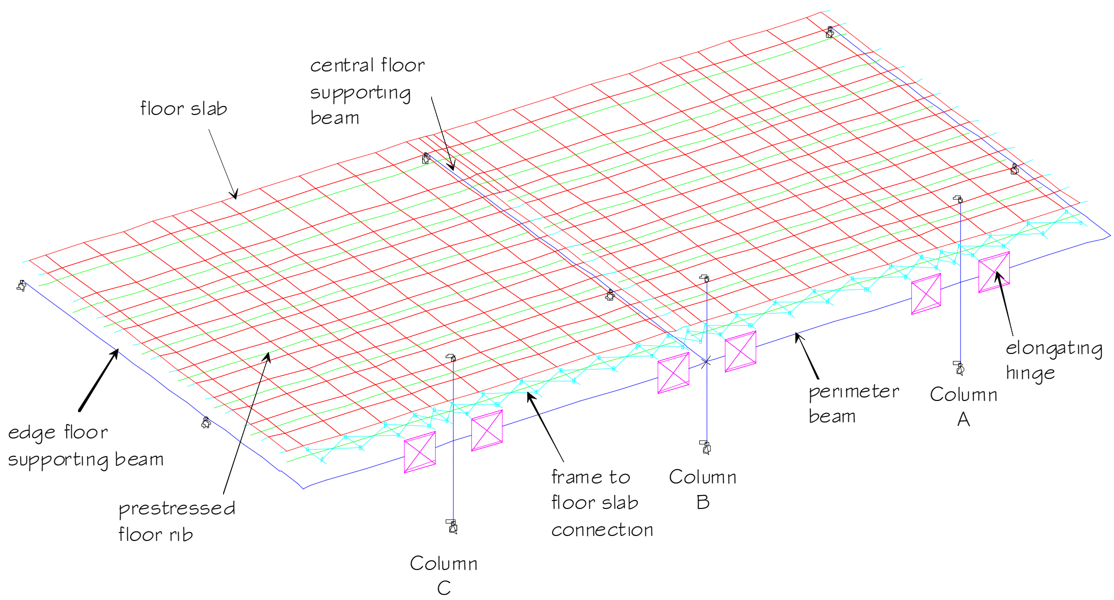

Fig. 9: SAP2000 Model of unit 2

This latter group can be seen as a series of edge triangles in Fig. 9. Each of the diagonal members in this group was built up of three vertically spaced gapping elements spaced vertically within the 40mm thickness of the member at each end of the diagonal member. With this arrangement the diagonals could not sustain any tension, that is they were compression members only. By having three gapping elements through the thickness at each end of the member they could model flexural cracking in the

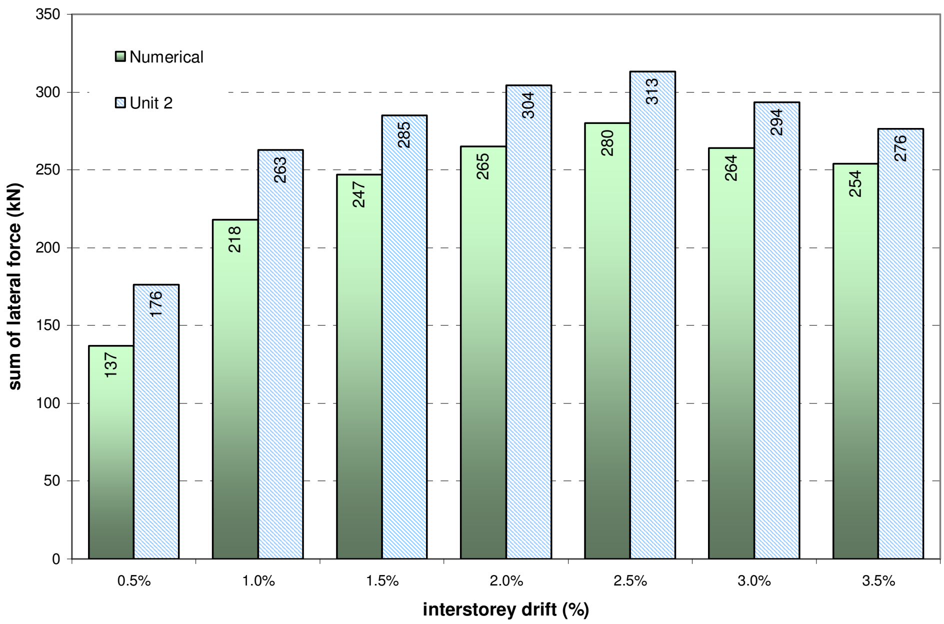

Fig. 10: Comparison of Experimental and Numerical Force versus Deflection Response

linking slab and relate this to the relative vertical displacement between the beam and first pretensioned stem. A fuller description of the numerical model can be found in (Lau 2005). The model for units 1 and 3 used only the perimeter frame components, including the plastic hinge substructures, of the model for unit 3.

As mentioned above, the numerical model has been developed to allow an investigation into the mechanisms of this complex structure. The advantage of the model is that it allows one to determine the magnitude of different actions, which arise with different design configurations.

Fig. 10 compares the average measured lateral force sustained in the test of unit 2 at different levels of displacement with the corresponding values predicted by the numerical analyses. The agreement between to analytical and experiment results appears to be reasonable.

To gain a further insight into the accuracy of the model and to validate the magnitude of the effective tension forces resisted by the floor Table 4 lists the T_{slab} values as calculated from the model and the experimental results. Again average values have been listed for the numerical results for the cycles to the same limiting displacement. As with the corresponding experimental results there were considerable differences in the individual values. These differences arise as the actions are a function of the loading history and the degradation, which occurs as the loading history is applied.

Table 4. Comparison of effective tension force resisted by slab in unit 2

Drift (%)	Average compression force at Columns A and C T_{slab} (kN)		Average compression force at Column B T_{slab} (kN)	
	Expt.	Num.	Expt.	Num.
1	38	93	165	91
2	102	170	208	109
2.5	121	171	220	145
3	104	117	205	184

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The numerical analyses have over predicted the strength of the plastic hinge zones adjacent to columns A and C in unit 2. It is likely that this over-estimate is a result of adopting too high a rotational limit for the linking slab before failure was assumed to occur by spalling. Once spalling occurs the horizontal shear resistance is lost. This observation is supported by the appreciable extent of spalling observed in the tests in load cycles to displacements greater than 1.5%

The under-estimate of the strength enhancement of the plastic hinge zones adjacent to column B in the numerical analyses probably occurs due to the inadequate modelling of the shear transfer from the central transverse beam to the floor slab. In particular this should have included shear across the cracks marked as e - f and g - h in Fig. 6 by dowel and aggregate interlock actions. This shear transfer would enable the central beam to contribute the effective strength and stiffness of the deep beam actions in the floor slab.

4 DISCUSSION AND CONCLUSIONS

1 The experimental and analytical results show that a slab containing prestressed components can have a major influence on the strength of ductile moment resisting frame structures. Current criteria contained in the Structural Concrete Standard [New Zealand Standards 1995] can lead to an under-estimate of the over-strength of the beams in a perimeter frame. This raises the possibility of undesirable failure modes, such as a column mechanism developing instead of the intended ductile beam sway failure mode, in the event of a major earthquake.

2 The tests and analytical work show that considerable strength enhancement can arise due to deep

beam action in slabs. This mechanism of strength enhancement in diaphragms has not been considered before, but it appears that it can be of major importance in many practical situations.

3 Some practical analytical method of analysis is required to establish design rules for the interaction between perimeter frames and floor slabs. The number of practical arrangements of floor slabs containing prestressed components with frame structures is too great for design criteria to be developed from testing alone. In this project a first tentative step has been made in developing such an analytical method of analysis. However, considerable further refinement is required.

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