6.1 INTRODUCTION

Units 5 and 6 were cruciform shaped precast concrete subassemblies typical of the upper storeys of a perimeter frame of a multistorey building, where the absence of high axial compression loads in the columns would cause the most unfavourable conditions in the beam-column joint regions. System 1 of precast concrete construction used in New Zealand, and outlined in Chapter 1 (see Fig. 1.2 (a)) was incorporated in Unit 5. The precast concrete beams of this subassembly were partly precast and had the bottom reinforcement anchored in the cast in place beam-column joint region. The precast beams were seated on the cover of the concrete column below and cast in place concrete was placed in the joint region and in the top of the beams after placing and tying the additional beam top bars and the beam-column joint reinforcement (see Fig. 3.7).

Unit 6, on the other hand, had a precast concrete element typical of System 2, outlined in Chapter 1 (see Fig 1.2 (b)), where the beam and the beam-column joint were cast in the same operation. Vertical holes in the beam-column joint region of the beam were preformed using galvanized steel corrugated ducts to allow the vertical column bars protruding from the column below to pass through the precast beam member (see Fig. 3.8). A fluid and rich-in-cement grout was poured by gravity through one of the orifices to make the horizontal joint and to bond the vertical column bars to the beam.

Complete reinforcing details of these two units and the test sequence were presented in Sections 3.2.4 and 3.7 respectively. Both Units were designed for plastic hinging to form in the beams adjacent to the column faces. The shear span/effective depth ratio of the beams was 2.6 for both Units 5 and 6, which corresponds to beams with a clear span/depth ratio equal to 4.5. The beams of Unit 5 had approximately equal top and bottom beam longitudinal reinforcement, while the beams of Unit 6 were symmetrically reinforced. The nominal shear stress in the beams of both Units at an overstrength of 1.25 times the beam flexural strength was \( \nu^o = V^o / bd = 0.23 \sqrt{\gamma} \).

6.2 TEST RESULTS OF UNIT 5

6.2.1 General Behaviour

Unit 5 took a week to be tested. After the completion of loading cycles up to \( \mu_a = \pm 6 \), at an interstorey drift of 3.6%, its lateral load capacity dropped to 80% of the maximum value recorded. The cumulative displacement ductility attained by Unit 5 was \( \Sigma \mu_a = 48 \) equivalent to an available
displacement ductility factor $\mu_a = 6$. At the end of the test crushing of the concrete had occurred in the beams surrounding the top bars at the column faces. An earlier bond failure of these bars in the joint region meant that they were anchored in the beam where they would normally be subjected to compression. Neither the construction joints between precast and cast in place concrete, nor the anchorage of the bottom beam bars inside the beam-column joint region, were observed to have a detrimental effect on the general cyclic performance of Unit 5.

The visible cracking in the region of the beam-column joint of Unit 5 at different stages during the test is shown in Fig. 6.1. In the loading cycles in the elastic range, load runs 1 to 4, cracking spread along the top and bottom of the beams at regular intervals to a distance of 970mm from the column faces. The crack pattern in the beam was unsymmetrical. The top regions of the beams showed smaller cracks, with two cracks diagonally propagating towards the compression zone of the beam at the column faces. The bottom regions of the beams showed many fine cracks. The cracks in the beams at the face of the column were wider in the top region than in the bottom region. The maximum crack width recorded at this stage were 0.2mm and 0.3mm in the web and at the column faces, respectively. Some diagonal cracks propagated through the beam-column joint; the maximum width of these cracks was 0.25mm. The crack pattern in the column was very symmetrical and only the cracks at the horizontal construction joint showed a width comparable to the cracks in the beam. The crack widths in these regions of the column were 0.5mm, measured at the extreme fibres of the tension side of the columns.

In the loading cycles to $\mu_a = \pm 2$, load runs 5 to 8, the main cracks concentrated in the beams at the faces of the column, where they reached a width of 5mm in the extreme tension fibres. It was observed that these cracks actually developed in the precast concrete beams rather than in the vertical construction joints. The crack pattern in the joint panel became denser but the width of the cracks there was small, reaching values of 0.8mm. The crack widths at the horizontal construction joint in the extreme tension fibres of the column were up to 1mm wide. Cracks elsewhere remained very fine.

The unsymmetrical behaviour of the top and bottom regions of the beams of Unit 5 became very obvious in the first cycle to $\mu_a = 4$, load run 9. A 7mm crack concentrated in the south beam at the face of the column. Cracks elsewhere in this beam remained very small. On the other hand, the cracks in the bottom region of the north beam extended from the column face, thus spreading the plastic hinge. The main reason for this dissimilar behaviour was that the top beam bars tended to slip through the beam-column joint region.

In the reverse load run 10 to $\mu_a = -4$, a similar crack pattern in the beams was observed. At this stage the top corners of the beam-column joint had been dislodged from the rest of the panel forming a cone around the beam reinforcement. Also, crushing of the concrete was observed in the top region of the south beam, confirming that the top bars were being anchored in the beam on the other side of the joint rather than in the joint itself. The cracks at the column face commencing at the bottom of the beam remained open, mainly because significant slip of these bars did not occur.
Fig. 6.1 - Cracking Beam-Column Joint Region of Unit 5 at Different Stages During Testing.
In the loading cycles to $\mu_a = \pm 6$, load runs 13 to 16, the compression region in the top of the beams was gradually crushed until in load run 15 a drop in lateral load carrying capacity of 19% was observed.

The main reason for this unexpected behaviour can be attributed to the poor quality, not strength, of the concrete cast in the beam-column joint region, which accentuated the well known "top bar effect". Another factor which may have contributed to this behaviour, was the minimum longitudinal reinforcing steel placed in the top of the precast beams, for holding the stirrups during casting of the concrete, which terminated at the column faces. However, it is believed that this second factor alone could not have caused the rotation of the beam to concentrate at the face of the columns. Tests carried out by Park and Bull (1986) on filled prestressed U-beams showed the plastic hinges did not concentrate at the face of the column despite the strands of the precast U-beam being terminated at this location.

In spite of the vertical construction joints at the ends of the precast beams at the faces of the column, no large relative vertical sliding displacements were observed in this test. This apparent contradiction with the large relative vertical displacements observed by Beattie (1989) stems from the fact that the precast concrete beams were seated on the cover concrete of the column below. Crushing or spalling of the cover concrete of the column was not observed during the tests.

The conditions of the concrete surrounding the beam bars were inspected at the end of the test of Unit 5. Figs. 6.2 (a) and (b) show close up views of the top east D24 bar and the bottom east D28 bar.

Fig. 6.2 - Beam Bars in the Joint Region of Unit 5 after the End of Test.
D28 bar, respectively. These views were obtained by removing the side cover concrete to expose these beam bars. It can be seen that the surface around the top bar is very smooth. Crushed concrete was found to be packed near the deformations of the bars. One of the inner bars was pushed with a hydraulic jack to measure the residual friction. The frictional force measured was very dependent on the rate of displacement applied but, in terms of average bond stress, it did not exceed 0.18MPa.

6.2.2 Load-Displacement Response

The lateral load versus the lateral displacement response at Unit 5 for the loading cycles in the elastic range is shown in Fig. 6.3. The stage at which first cracking appeared in the beams, columns and beam-column joint is also presented in this figure. The joint cracked in the positive direction at a nominal horizontal joint shear stress of $0.39\sqrt{f_c}$ and in the reverse direction at a lower value of $0.33\sqrt{f_c}$. The "elastic" stiffness of this Unit was 17.1kN/mm equivalent to only 42.1% of that predicted in Section 3.2.1. The interstorey drift at the projected first yield displacement was 0.59%, and that at the dependable lateral load capacity of the unit was 0.54%. This value exceeded by a large margin the limitation imposed by the NZS 4203 Loadings Code (1984). As will be shown later in the next section, the large error in the predicted initial stiffness is chiefly due to the fixed-end rotation in the beams and columns as a result of the strain penetration of the longitudinal bars into the joint region, as well as the joint panel shear deformations, both of which were ignored in the theoretical calculations. As it has been observed and discussed with regard to the tests in Units 1 to 3, the second loop was narrower than the first one.

The complete hysteretic response of Unit 5, in terms of the lateral load and displacement, is depicted in Fig. 6.4. The post-elastic stiffnesses defined in the same way that for Units 1, 2 and 3, were 2.3 and 1.3% of the initial "elastic" stiffness for the positive and negative cycles respectively. The positive load runs consistently attained higher loads than the negative load runs.

In the first load run in the inelastic range, the lateral load attained exceeded by 8% the theoretical lateral load, $H_s$. In the reverse load run, load run 6, the lateral load attained was $1.04H_s$.

The hysteretic loops were very stable until the first cycle to $\mu_s = \pm 4$. The energy dissipated was 79 and 55% of that of the ideal bi-linear loops, respectively, in the two cycles to $\mu_s = \pm 2$ and 57% in the first cycle to $\mu_s = \pm 4$. Pinching of the loops commenced in the second cycle to $\mu_s = \pm 4$ mainly due to the commencement of slip of the top beam bars inside the joint region. Consequently, the normalized energy dissipated dropped to 22% in that load cycle.

In the loading cycles to $\mu_s = \pm 6$, the normalized energy dissipation amounted to only 34 and 23% for the first and second cycle, respectively. The maximum lateral load overstrengths were attained in the first loading cycle to $\mu_s = \pm 6$, load runs 13 and 14, and were $1.14H_s$ and $1.08H_s$. The maximum beam shear, in terms of the nominal shear stress $\nu_0 = V_0/bd$, was $0.2\sqrt{f_c}$. 
Fig. 6.3 - Lateral Load-Lateral Displacement Response of Unit 5 During the Cycles in the Elastic Range.

Fig. 6.4 - Lateral Load-Lateral Displacement Response of Unit 5.
The cumulative energy dissipated by the hysteretic response accounting for all the loading cycles in the inelastic range, including that dissipated in load run 16, was 35% of that dissipated by the ideal bi-linear loops.

6.2.3 Decomposition of Lateral Displacements

The components of the total lateral displacement at the peak of each load run is illustrated in Fig. 6.5. In the loading cycles in the elastic range the largest contribution came from the column, which contributed from 34 to 41% of the imposed lateral displacements. However, because the column remained essentially elastic, with minor cracking, its contribution towards the total displacement became less important during the remainder of the test. The fixed-end rotation of the beams made an important contribution to the lateral displacements in the elastic range, and since the top beam bars eventually slipped in the joint this mode of deformation became dominant. Flexure deformations along the beams in the inelastic range arose mainly from the spreading of the plastic hinge regions in the beams during positive bending moment. This flexural contribution was of a similar magnitude to that from the distortion of the joint panel.
Note that the deformations of the column and joint in Fig. 6.5 are not shown beyond the first semi-cycle to $\mu_a = +4x1$, because the instrumentation was disturbed by the loss of the corners of the beam-column joint panel.

6.2.4 Joint Behaviour

6.2.4.1 Strains in the Transverse Reinforcement

The transverse reinforcement in the beam-column joint region was designed in accordance with the current Concrete Design Code [NZS 3101 (1984)], which in the absence of axial compression load in the column required the whole horizontal shear at overstrength of the longitudinal reinforcement to be taken by the transverse reinforcement. This approach has been proved to provide an upper bound and therefore no yielding of the joint hoops was expected to occur.

Fig. 6.6 shows the strains in the hoops at different peak load runs, which were monitored by two 5mm electrical strain gauges placed on opposite sides of the perimeter hoops to cancel out the effects of hoop bending. In general, the central hoop showed the larger strains but the strain distribution throughout the set of hoops was rather uniform.

It can also been seen in Fig. 6.6 that beyond load run 9 to $\mu_a = \pm 4$ the strain in the top hoops decreased. On the other hand the strain of the bottom hoops always increased with the test sequence. This effect was due to the bond conditions of the beam bars. The top bars commenced to slip from load run 9 because of bond degradation, and the most of the shear resistance had to come from the diagonal concrete strut mechanism. In contrast, very little slip occurred on the hooked bottom bars and hence part of the bond forces were transferred by the diagonal concrete strut mechanism and the other part mobilized the truss mechanism. These mechanisms of joint shear resistance will be discussed in detail in Chapter 7.

6.2.4.2 Slip of the Beam Bars

The local bar slip of the top D24 beam bars, at three locations in the beam-column joint, are shown in Fig. 6.7. The procedures for determining, and limitations on the definition of local bar slip, were discussed in Section 3.8.4.

The local bar slip at each of the locations shown progressively increased with the test. In load run 10 to $\mu_a = -4x1$, the amplitude of the slip at the centre line of the column and in the north side exceeded the clear distance of 11mm between the bar deformations. In the following load run, the local bar slip everywhere else in the joint core exceeded this distance indicating that the concrete surrounding the top bars had been completely sheared off and bond could only be transferred by friction and not by mechanical contact between the bar deformations and the surrounding concrete. This significant slippage resulted in concentration of the rotation of the beam at the column face, where a single large crack developed.
Fig. 6.6 - Measured Strains in Joint Hoops of Unit 5.
The instrumentation became unreliable beyond load run 12 when the studs welded to the reinforcing bars were bearing against the walls of the hole provided as clearance.

6.2.4.3 Bar and Bond Stresses of the Beam Longitudinal Reinforcement

Figs. 6.8 and 6.9 show the bar and bond stresses for the top and bottom longitudinal bars of the beam anchored in the beam-column joint region.

Bar stresses were estimated using the cyclic strain-stress model for steel discussed in Chapter 2 and the bar strain history collected from the electrical resistance strain gauges attached at regular intervals to the reinforcing steel. In addition the stresses in the reinforcement at each end of the joint during the loading cycles in the inelastic range were calculated from the data collected from the clip gauges. Electrical resistance strain gauges and clip gauges were attached to different bars. In spite of this, it is assumed that the stress is the same in both bars. This assumption is made because the electrical resistance strain gauges located on the bars at the face of the column failed prematurely in the tests. In addition, in Fig. 6.8 and 6.9 is also assumed that the calculated bar stresses at the level of the clip gauges are the same as the bar stresses measured by electrical resistance strain gauges at the column faces. The bond stresses shown in this figures were obtained dividing the difference of stresses between two consecutive gauged points by their distance and by the nominal length of the circumference of the bar.

In the loading cycles, in the elastic range, the load run numbers 1 and 2 in Fig. 6.8, the top bars were in tension all along the joint and hence they were anchored in the compression region of the
Fig. 6.8 - Bar and Bond Stresses - Top D24 Beam Bar of Unit 5.
Fig. 6.9 - Bar and Bond Stresses - Bottom D28 Beam Bar Unit 5.
opposite beam, where they would normally be considered to be in compression. The bond stresses at this stage of load were fairly uniform for both directions of loading although there was a trend to increase where the concrete compression stress block from the column acted. This trend was accentuated during the loading cycles in the inelastic range, load runs 5 to 9, when the bond stresses peaked at 15.3MPa. In any case the bond stresses for the top bars at the column centreline did not exceed 5MPa. Yield in the reinforcement penetrated to the column centreline, distance equivalent to 12.5d_e. Data could not be collected beyond load run 9 at μ_a = ±4x1 because all the circuits of the electrical resistance strain gauges were damaged when the top bars slipped.

The bar stress distribution at the bottom north D28 hooked beam bar is also shown in Fig. 6.8. In the positive load cycles, when the bar was subjected to tension, part of the tension force was transferred to the joint core by bond and, in the loading cycles in the inelastic range, a large portion of the force was transferred by the hook of the bar. Yield of the bottom bars penetrated into the joint core to about 210mm from the face of the column. This is equivalent to a distance of 7.5d_e. It can be observed that the bond stresses at positive ductilities was very uniform at the beginning of the test, but the bond strength deteriorated with the penetration of yield.

A very different behaviour characterized the bar and bond stress distribution of the bottom hooked bars in the load runs at negative ductilities. Most of the bar compressive force was transferred to the joint core near the face of the column where the concrete compression stress block of the column was acting. Bond forces were, then, higher in this region. The stresses in the bar at the beginning of the loading runs were very small, and even were in tension, indicating some internal build up in stresses in the previous loading cycles when the bars were subjected to tension.

The ratio of forces at the beginning and at the end of the hook of the bottom north bar is depicted in Fig. 6.10. This ratio is only shown for load runs to positive ductilities when this bar was in tension. In the loading cycles in the elastic range, when the stress at the beginning of the hook was about 100MPa and the bar behaved elastically throughout its length, the ratio of forces ranged between 1.4 and 1.1. It can be found that the resultant force R in Fig 6.11 forms an angle to the horizontal between 45° and 55° if the frictional bond resistance along the bend of the hooks is ignored.

In the loading cycles in the inelastic range the ratio of forces at the beginning and at the end of the hook dropped considerably and ranged from 0.4 to 0.5. This low ratio implies a resultant force, R, having an unreasonable angle of inclination to the horizontal of between 22° and 29°, if the bond stress around the hook of the bars is neglected. What appears to happen is that the frictional bond stress around the concave side of the bend plays an important role and cannot be ignored. This effect is depicted in Fig. 6.11.

A common characteristic of the behaviour of top and bottom longitudinal beam reinforcement is that the compressive force carried by the bars at the face of the column was smaller than the tension force at the other face. The compression stress is always below yield, in spite of the beams being approximately equally reinforced. A similar observation was also made in the test of Unit 2 although in that test the recorded strains were not made at the face of the columns but farther away along the span.
Fig. 6.10 - Ratio Between Forces at the Beginning and End of the Hook in Bottom D28 Beam Bar of Unit 5.

Fig. 6.11 - Effect of Bond Stresses Acting on Hooked Anchorage of Unit 5.
of the beam. Fig. 6.12 plots the predicted bar force in the compression zone of the beam and the estimated total compression force against the measured bar strain. The bar force was determined from the measured bar strain assuming that all the reinforcement was subjected to an equal strain history. The total compression force was found from the shear force in the beams, measured at the beam ends by load cells. It is evident that when the bars go into compression they supply only a fraction of the total compression force with the remainder being provided by compression in the concrete. One question that may arise is: How can the concrete transfer any force if it is widely cracked around the bars? An answer to it is that in this region the hypothesis of plane sections remain plane after bending is not valid. That is, no compatibility of deformations can be assumed on this basis. Instead, a truss model is more applicable. Also, shear deformations and debris cause cracks to transfer compression stresses before they close. A large percentage of the shear in the beam enters the joint region by a diagonal compression strut, which provides the remaining compressive force in the beam. This mechanism of shear transfer in the beam plastic hinges will be discussed in Chapter 7.

6.2.4.4 Bar and Bond Stress of the Column Vertical Reinforcement

Figs. 6.13 and 6.14 show the bar and bond stresses of the strain gauged vertical column bars of Unit 5. Fig. 6.13 plots the results obtained from the south-west corner bar. In the loading cycles in the elastic range, load runs 1 and 2, the bars were in tension over a distance of 24 in. from the beam faces. This strain penetration led to the wide crack observed to develop at the vertical construction joints. In the loading cycles in the inelastic range the bars yielded in tension. In load run 6 it appears that yield penetrated 270 mm into the joint. Unfortunately the strain gauge CE4 at this location failed prematurely after this load run. A simple elastic theory analysis indicates that yield in tension of the vertical column bars at the level of the beam longitudinal reinforcement would commence at a lateral load of \( 1.12H_{\text{e}} \), a value which was only attained at the final cycles of the test.

Beckingsale (1980) observed a similar effect in the testing of two interior beam-column joint units and postulated the cause to be disturbances resulting from the bond forces of the beam bars. It is believed in the present study that the increase in stress in the outer column bars in tension results of a combination of two different sources. First, the intermediate column bars are required to provide part of the column flexural resistance and the vertical resistance of the inclined truss in the joint panel. When these bars yield due to the combined action, additional flexural resistance cannot be provided and consequently higher tensile stresses are necessary to develop in the outer column bars in tension. Second, it has been observed in this, and other studies, that after few reversal cycles in the inelastic range the column cover along the longitudinal beam bars is dislodged and cannot be fully used to transfer compressive stress in a reduction of the column lever arm and in an increase of the stresses in the tension reinforcement, in order to maintain the equilibrium of the column as a free body with a lateral load directly applied at the end of it.

The bond stresses of the corner bars of the column plotted in Fig. 6.13 indicate that in the loading cycles in the elastic range the bond stresses, as for the beam bars, increased towards the region where the concrete compression stress block of the beams acted. Unfortunately, it was not possible to
Fig. 6.12 - Predicted Bar Forces versus Measured Total Force in the Beam 78mm from the Column Face of Unit 5.
Fig. 6.13 - Bar and Bond Stresses - South-West Corner Column Bar of Unit 5.
Fig. 6.14 - Bar and Bond Stresses - South-West Interior Column Bar of Unit 5.
get conclusive trends of the bond stresses in the loading cycles in the inelastic range because of the failure of the strain gauge CE4 (see Fig. 6.13).

The bar and bond stresses of the inner south-west column vertical bar are plotted in Fig. 6.14. This bar was always subjected to tension at each end of the joint, and, as expected, no large bond forces were recorded.

6.2.5 Column Behaviour

From the visual observations made during the test, the column remained essentially elastic with small cracks distributed at regular intervals up to the end pins. The vertical sliding of the beams along the vertical construction joint at the column faces was very small, as indicated by the chart shown in Fig. 6.15.

6.2.6 Beam Behaviour

6.2.6.1 Curvature and Rotational Ductility Factors

The beam curvature ductility factors shown in Fig. 6.16 were calculated from the second set of linear potentiometers along the north and south beams. The projected curvature at first yield indicated a value lower than that predicted by moment-curvature analysis. The large curvature ductility
factors in the cycles to $\mu_A = \pm 6$ were influenced by the slippage through the beam-column joint of the beam top reinforcement. That reinforcement when anchoring in tension in the beam at the opposite side of the column induce large compressive strains in the compression region of the beam.

Fig. 6.17 illustrates the rotational ductility factors of the north and south beams of Unit 5. Since the components of the total lateral displacement of Unit 5 were controlled by the fixed-end rotation of the beams and the flexural response, the predicted rotational ductility demand was close to that estimated from the test results.

6.2.6.2 Beam Strain Profiles at the Level of the Longitudinal Reinforcement

Figs. 6.18 and 6.19 depict the estimated member strains at the level of the beam top and bottom reinforcement respectively. It is evident in Fig. 6.17 that the slippage of the top reinforcement in load run 9 had a large effect on the distribution of strains along the beam. Beyond this load run the concrete surrounding the top bars in the beam near the column face subjected to compression was subjected to very large compressive strains. As it was mentioned in Section 6.2.1, the loss of the concrete due to crushing in this region was the final cause for the reduction in load carrying capacity of the test unit.

Contrary to the behaviour of the top reinforcement of the beam the strains calculated in the bottom reinforcement of the beam depicted in Fig. 6.19, showed the spreading of the plastic hinges along the north and south beams. From the data reduced it is possible to estimate that the plastic hinges extended a distance close to the effective depth of the beam, d.

6.2.6.3 Beam Elongation

The total measured elongation of the beams of Unit 5 is plotted in Fig. 6.20 against the measured storey shear. The elongation of the beam followed the same pattern as for the other units tested in this programme. However, if Fig. 6.20 is studied together with Figs. 6.18 and 6.19 it is concluded that in Unit 5 the elongation of the beams was caused by the residual tensile strains in the bottom reinforcement of the beams and by the effect of the top bars slipping through the joint, instead by residual tensile strains in the top bars of the beams. The elongation of the beam caused by the slippage of the bars results in a comparatively smaller cumulative elongation of the beam. Hence, it would be expected that the lengthening of the beam of Unit 5 would have been larger had the bond failure in the top bars been precluded.

In the beam of Unit 5 the elongation took place mainly in the first complete loading cycle to a new displacement ductility. The second loading cycles did not increase the elongation of the beam as much as noted in the other units tested in this project, because in the second cycles the Unit did not develop its flexural capacity as a consequence of the loss of stiffness caused by the bond failure of the top reinforcement of the beam.
\[ \phi_y = 0.0017 \text{ rads/m} \]

**Fig. 6.16 - Beam Curvature Ductility Factors of Unit 5.**

\[ \theta_y = 0.0035 \text{ rads} \]

**Fig. 6.17 - Beam Rotational Ductility Factors of Unit 5.**
Fig. 6.18 - Beam Strain Profiles at the Level of the Top Reinforcement of Unit 5.
Fig. 6.19 - Beam Strain Profiles at the Level of the Bottom Reinforcement of Unit 5.
In Fig. 6.20 it can also be observed that after load run 13, the first cycle to $\mu_a = 6$, the measured beam elongation became erratic and lost its typical pattern. It is not known what caused this pattern. The lengthening of the beam calculated from the linear potentiometer measurements along the regions of the beams showed a similar trend, which means that an error was not made in the measurement procedure.

6.3 TEST RESULTS OF UNIT 6

6.3.1 General Behaviour

Fig. 6.21 shows the beam-column joint region of Unit 6 at different stages during the test. Plastic hinges developed in the adjacent beams, as expected, and minor and well distributed cracking was observed to appear elsewhere in the assemblage. The test of Unit 6 took one week to complete and ended after cycles up to a $\mu_a = 7.3$ at an interstorey drift of 3.6% (see Fig. 6.23). The cumulative displacement ductility before losing its lateral capacity by more of 20% of the maximum lateral load measured was $\Sigma \mu_a = 88$ implying, based on the method discussed in Section 3.7, an available displacement ductility factor of $\mu_a = 8$.

At the end of the test the concrete in the plastic hinge regions of both beams appeared damaged as a consequence of the large shear deformations that occurred in these regions. The test unit performed as if of monolithic construction. No adverse effects were observed to occur in the grouted vertical sleeves where the column longitudinal reinforcement was anchored. Some sliding along the
(a) At $\mu_\Delta = -4.9 \times 1$

(b) At $\mu_\Delta = -7.3 \times 1$

(c) At $\mu_\Delta = -7.3 \times 2$

Fig. 6.21 - Cracking of Beam-Column Joint Region of Unit 6 at Different Stages During the Test.
lower horizontal construction joint between the precast concrete member and the column below was recorded but, relative to the total lateral displacement imposed, it was insignificant.

In the loading cycles in the elastic range cracking in the beams spread to a distance equal the depth of the beam from the column faces and developed at regular intervals. However, crack widths were significantly larger in the top of the beams where a maximum crack of 0.4mm was observed. Cracks in the beam at the column faces ran continuously along the column interface but their widths never exceeded 0.2mm. Nonetheless, crack widths in the columns at the face of the beams were larger with values up to 0.4mm recorded. Diagonal cracking in the joint panel was in general well distributed and the width of the cracks did not exceed 0.2mm.

In the first loading cycles in the inelastic range to $\mu_a = \pm 2.4$, load runs 5 to 8, the main cracks developed in the beams at the face of the columns as a result of yielding of the beam longitudinal reinforcement penetrating into the beam-column joint region. These cracks were up to 4mm in width. Some of the diagonal cracks in the web of the beams grew in the second cycle to $\mu_a = \pm 2.4$ but their width never exceeded 1mm. Elsewhere in the subassemblage the crack pattern became denser but their width did not exceed 0.5mm.

Plastic hinges spread along the beams in the loading cycles to $\mu_a = \pm 4.9$. Crack widths of the diagonal cracks, which radiated from the compression region in the beams at the face of the columns, increased with values of 2mm recorded at the mid-depth of the beam. Some relative sliding action, grinding and spalling was also observed to occur along the cracked concrete in the web of the beam near the column faces. The cracks in the columns at the faces of the beams also widened to 1.6mm at the extreme fibres in tension. The main cracks in the joint panel were those crossing the diagonals of the joint. Their width did not exceed 0.6mm.

The concrete in the plastic hinge regions of the beams suffered further deterioration in the loading cycles to $\mu_a = \pm 7.3$, load runs 13 to 20, because of the grinding action caused by sliding shear. A final push towards larger displacements showed the ability of the unit to maintain levels of strength at least equal to the theoretical load. The lateral load finally dropped when the top longitudinal bars of the beams buckled. At the end of the test the beam-column joint region remained in solid condition with small cracks. One of the corrugated ducts was removed and sectioned for inspection and the grout inside was found to be in sound condition. Only radial cracks were detected in the grout at the ends of the ducts. Also, a small gap between the grout and the embedded reinforcing bar was observed at the ends which was likely to have been caused by crushing of the grout due to dowel action of the bar.

6.3.2 Load-Displacement Response

Figure 6.22 depicts the lateral load versus lateral displacement response of Unit 6 in the first load runs where the load was cycled to 75% of the theoretical lateral load capacity, $H_t$. The initial "elastic" stiffness determined following the procedure discussed in Section 3.7 was only 49% of that predicted based on the assumptions made in Section 3.2.1. The interstorey drift at the projected first yield displacement was 0.49% and that at the dependable lateral load capacity was 0.46%, which was
well in excess of the limit prescribed by the NZS 4203 Loadings Code (1984). The apparent difference between measured and predicted stiffness lies on the additional flexibility due to the fixed-end rotation in the beams and the columns caused by the strain penetration of the reinforcement inside the joint. In contrast with Unit 5, the shear deformations in the beam-column joint panel did not have an important effect on the initial "elastic" stiffness, as it will be explained in the next section.

Also shown in Fig. 6.22 are the load levels at which first cracking in the beams, column and beam-column joint were observed. In the first load run the beam-column joint panel cracked at a nominal horizontal joint shear stress of $0.39\sqrt{\gamma}$, which in terms of the column shear is equivalent at a load of $0.75H$. That is, the joint remained uncracked until the last increment before reaching the peak load and hence its contribution towards the lateral displacement was minimal. In the reverse load run the joint panel cracked at a lower joint shear stress of $0.32\sqrt{\gamma}$, following the same trend observed in Unit 5 and which possibly can be related to the notch effect caused the jagged shape of the first diagonal crack.

Fig. 6.23 plots the hysteretic load-displacement response of Unit 6. The displacement controlled cycles followed the displacement sequence applied to Unit 5 and hence the actual displacement ductility factors to which Unit 6 was subjected did not follow the conventional pattern of $\mu = \pm 2, \pm 4$ and $\pm 6$ but they followed a multiple of them.

The post-elastic stiffnesses of the hysteretic response of Unit 6 were 3.8 and 3.7% of the initial "elastic" stiffness for the positive and negative cycles, respectively. The average of these stiffnesses plus the initial "elastic" stiffness were used to define an ideal bi-linear loop in order to normalize the energy dissipated by the actual hysteresis loops measured for the unit.

In the first loading cycle in the inelastic range to $\mu = \pm 2.4$, load runs 5 and 6 in Fig. 6.23, the lateral load attained exceeded the theoretical load $H$ by 9% and 2%, respectively. The second cycle to the same ductility level was very stable and the loads attained were only slightly lower. The normalized energy dissipated in these cycles was 76% and 48%, respectively.

The loading cycles to $\mu = \pm 4.9$, load runs 9 to 12, showed a steady increase in the lateral load attained and only in load run 12 was some pinching of the measured hysteretic loop observed. The main source of pinching at this stage was due to shear deformations in the plastic hinge regions of the beams. The normalized energy dissipated in these cycles was 65 to 45%, respectively.

The maximum lateral load strength of $1.24H$ attained by Unit 6 was observed in load run 13 to $\mu = \pm 7.3$. A very similar value of $1.23H$ was attained in the reverse load run. The maximum shear in the beams was recorded in load run 13 in the north end. Its value, expressed in terms of the nominal shear stress was $V_0 = V/\beta d = 0.20\sqrt{\gamma}$. The second cycles to $\mu = 7.3$ were pinched and there was a gradual loss of stiffness, which affected the lateral load attained at the given lateral displacement. The load attained in load run 20 was 21% below that previously attained in load run 14. The normalized energy dissipated also reflects the gradual degradation of the response at this stage, being 32, 22 and 20% from the first to the fourth cycle to $\mu = 7.3$, respectively.
Fig. 6.22 - Lateral Load-Lateral Displacement Response of Unit 6 During the Cycles in the Elastic Region.

Fig. 6.23 - Lateral Load-Lateral Displacement Response of Unit 6.
The ratio between the cumulative energy dissipated by the loops of Unit 6 and that of the ideal bi-linear loop was 48% at the end of load run 16 and 38% at the end of the test in load run 20. This index indicates that the hysteretic performance of Unit 6 was superior to that of Unit 5.

6.3.3 Decomposition of the Lateral Displacement

The components of the lateral displacement imposed on Unit 6 in the tests are depicted in Fig. 6.24. These components are presented as a percentage of the measured applied lateral displacement.

The deformations of the column during the loading cycles in the elastic range contributed to 32% to 35% of the total lateral displacement and was the largest source. Contrary to the observations made in Unit 5, the distortions of the beam-column joint did not make a large contribution to the deformations in the elastic range. The deformations in the beam, due to flexure, fixed-end rotation and shear, were of similar magnitude to those measured in Unit 5. It appears that the main reason for the difference in stiffness between Unit 5 and 6 was the smaller deformation of the joint region of Unit 6.
In the initial loading cycles in the inelastic range Unit 6 showed a flexural response with most of the displacements being caused by flexure and fixed-end rotation of the beams. However, at the end of the test the flexural response diminished. Instead, shear deformations in the plastic hinge regions became the predominant source of deformation. Unfortunately the devices monitoring the shear displacement of the beam-column joint had to be removed in the first loading cycle to $\mu_a = 7.3$ because the corners of the joint, where they were attached, became dislodged by the longitudinal beam bars. However, if the closure error at the end of the test is assigned to deformations within the joint panel, it is possible to conclude that the joint distortion contributed approximately 20% of the total lateral displacement in the final loading cycles near the end of the test.

6.3.4 Joint Behaviour

6.3.4.1 Strains in the Transverse Reinforcement

The measured hoop strains in the joint recorded during the test of Unit 6 are plotted in Fig. 6.25. Bar strains were measured by double 5mm electrical strain gauges on one leg of the perimeter hoops. The average of the two strain readings is presented in Fig. 6.25, except for the lower hoop where strain gauge HSB failed to work. The strains recorded show in general an increase as the test progressed, implying a gradual transition between a strut and a truss mechanism. They were very small in the loading cycles in the elastic range mainly because cracking was confined to few hairline cracks. The strains were in general very uniform and in the first loading cycle to $\mu_a = \pm 7.3$, load runs 13 and 14, they approached or reached yielding. In the repetitive cycles to $\mu_a = \pm 7.3$ the recorded strains reduced since the lateral load attained by the test specimen diminished.

If it is assumed that the interior and other perimeter legs of the hoops were equally stressed, the force in the transverse reinforcement would be equal to 97% of the total horizontal joint shear in load run 13.

6.3.4.2 Slip of the Beam Bars

The local bar slip of the top and bottom D24 bars in the joint core of Unit 6 are plotted in Fig. 6.26 and 6.27, respectively, for three locations in the joint. The bar slip measured at the centreline of the column was directly monitored by a linear potentiometer mounted on the concrete surface. Strictly, this measurement will be affected by the deformations of the concrete in this region. However, these deformations were considered negligible relative to the magnitude of the local bar slip. The local bar slip at the north and south ends, which correspond to the local bar slip at the level of the outer column bars, were estimated from the measured slip at a target located on the reinforcement at 10mm from the column faces extrapolated assuming that the bar strain was equal to that directly measured on the beam bar at the face of the column.

The local bar slip of the top bars was larger than that of the bottom bars, indicating the influence of the direction of casting of the concrete, commonly referred to as the "top bar effect". This effect is evident when the measurements at the column centreline in Figs. 6.26 and 6.27 are compared.
Fig. 6.25 - Measured Strains in Joint Hoops of Unit 6.
Fig. 6.26 - Measured Slip of Top Beam Bars of Unit 6.

Fig. 6.27 - Measured Slip of Bottom Beam Bars of Unit 6.
For instance, the maximum absolute bar slip of the top D24 bar was 8.3mm and was recorded in the last load cycle to $\mu_\Delta = \pm 7.3$. This value is 51% larger than the maximum absolute local bar slip of 5.5mm recorded in the same cycle at the bottom bar level.

It is obvious that the anchorage conditions for the bars in the beam-column joint core of Unit 6 were significantly better than in Unit 5. However, the better anchorage conditions in Unit 6 implied that the transfer of the joint shear relied more on a truss mechanism while for Unit 5 it relied, in the later stages, on the concrete strut mechanism. The level of strain in the joint hoops in both units, shown in Figs. 6.6 and 6.25, indicate that in the first loading cycles in the inelastic range, when local bar slip was controlled, the hoops were equally stressed down the depth at each load run. Even the hoop stresses in load run 9 were very similar, but beyond this load run, when the top bars of Unit 5 were sliding along the joint core, the hoops were unequally stressed.

**6.3.4.3 Bond and Bar Stresses of the Beam Longitudinal Reinforcement**

The bar stresses and corresponding bond stresses of the top and bottom beam reinforcement are illustrated in Figs. 6.27 and 6.28, respectively. The method and limitations for obtaining these values have already been discussed in Section 6.2.4.3.

In the loading cycles in the elastic range, load runs 1 to 5, the bond distribution in the top and bottom bars was rather uniform and the stresses in the bars decreased in an almost perfect linear fashion. The top bars were in tension throughout the beam-column joint while the tensile stress in the bottom bars penetrated inside the joint an average distance of 420mm or 17.6$\ell$. Eventually, with further loading runs, yield in the reinforcement penetrated into the joint core to a maximum distance of 6.5$\ell$ from the face of the column.

The bond stresses in the longitudinal beam bars estimated from measurements show a trend of increasing in the region of the compression zone of the column, which exerted pressure on the bars and clamped them. It is evident that the bond stresses along the length of these bars are far from being uniform. Two different mechanisms can be postulated as to contributing towards the development of bond.

In the first mechanism the bond relies on the mechanical contact between the bar deformations and the surrounding concrete. This is the main source of bond in the region unaffected by the compression zone of the column. The second mechanism is the frictional bond provided by the clamping action caused by the compression region of the column.

It is also evident in Figs. 6.28 and 6.29 that the compressive forces carried by the longitudinal reinforcement were smaller than the tension forces, following the same trend observed in the beam longitudinal reinforcement of Unit 5. This observation disagrees with the concept that for equally reinforced members the reinforcement in compression yields to balance the tension force. The compression force in the beam calculated from the measured shear is shown in Fig. 6.30 compared with the predicted force in the reinforcement at the column face of the top south chord.
Fig. 6.28 - Bar and Bond Stresses - Top D24 Beam Bar of Unit 6.
Fig. 6.29 - Bar and Bond Stresses - Bottom D24 Beam Bar of Unit 6.
As in Fig. 6.11 the compression force provided by the steel is always smaller than the total compression force estimated from the measured beam shear. The main reasons for this disagreement have already been discussed in Section 6.2.4.3.

6.3.4.4 Bar and Bond Stresses of the Column Vertical Reinforcement

Fig. 6.31 shows the bar stress and bond distribution of the north-west corner bar of the column of Unit 6. In the initial loading cycle in the elastic range, load runs 1 and 2 in Fig. 6.31, the tensile strain penetrated to a distance of 17.7d₀ from the face of the beams. The level of stresses increased as the test progressed and apparently yielding of the reinforcement occurred at the ends of the corrugated ducting. It is believed that this apparent yielding was caused by a malfunction of the strain gauges there that were probably disrupted by the dowel action of the bars due to sliding shear taking place at the horizontal construction joint.

As in Unit 5, a simple elastic analysis, assuming the critical region in the columns to be at the level of the longitudinal beam reinforcement, predicted that the corner bar would be subjected to a stress of 0.80fₑ at the maximum recorded load. If the composite action including the duct and grout is accounted for at 80mm from the end of the duct, the maximum stress level in the reinforcement is estimated to be only 0.72fₑ. However, the stress distribution indicates that this may not be the actual case and that it would be appropriate to ignore such composite action.
Fig. 6.31 - Bar and Bond Stresses of the North-West Corner Column Bar of Unit 6.
The bond stress distribution along the corner vertical bar in Fig. 6.31 indicates that the bar was mainly anchored in the middle third of the column. This distribution is in full agreement with earlier observations reported by Beckingsale (1980) and Cheung (1991). In addition, no detrimental effects were found to occur because of the lack of steel to steel contact between the longitudinal bar and the transverse hoops bent around the corrugated ducting. The grouted duct appears to act as an effective media through which the tie forces required for equilibrium at the nodes of the truss mechanism can be transferred.

Fig. 6.32 depicts the bar stress and bond distribution of the interior north-west column vertical reinforcement. Again, the apparent yielding at the duct ends was probably caused by the dowel action of the reinforcement due to horizontal sliding shear displacements along the construction joint. The general trend was for the bar stresses to increase with the advancement of the test. This would have been because when the truss mechanism gradually developed the longitudinal reinforcement was required to provide the vertical joint shear resistance in the joint.

The stress distribution in the loading cycles in the elastic range peaked at the ends of the joint, reflecting that the stresses at this stage were due only to flexure of the column, because at this stage the truss mechanism had not developed. In the loading cycles in the inelastic range the combination of both mechanisms (flexure and truss) implied that the bars were subjected to a rather uniform field of stresses along the joint panel. This uniform distribution of bar stress has been observed by other researchers as well.

In summary it can be concluded that the grouted corrugated ducting allowed the development of the truss mechanism, which enabled the transfer of a significant portion of the joint shear through this mechanism.

The bond between the bar and the grout was apparently not degraded at any stage during the testing. Nor was the bond degraded between the corrugated ducting and the concrete of the precast concrete unit.

6.3.5 Column Behaviour

The behaviour of the column during the test was essentially elastic. Cracking developed symmetrically above and below the beams and extended to the end pins. The size of the cracks remained always very small except at the construction joints where the horizontal cracks reached 2.5mm wide at the extreme fibre in tension.

It was also noticed that sliding shear, especially along the lower horizontal construction joint, was one for the main sources of deformation of the column. This displacement was monitored by DEMEC gauges at each construction joint. Fig. 6.33 shows the reduced data. Sliding shear, although small in terms of the overall lateral displacement imposed, accounted for a large proportion of the deformation of the column. The main reason for this source of flexibility was that the surfaces of the construction joints in the precast member were deliberately left smooth. No scrubbing nor water-jetting
Fig. 6.32 - Bar Bond Stresses of North-West Interior Column Bar of Unit 6.
was applied to roughen the surface. The results of Fig. 6.33 indicate roughening of the interface surfaces should be recommended in practice, particularly since roughness can be achieved by a simple procedure.

6.3.6 Beam Behaviour

6.3.6.1 Curvature and Rotational Ductility Factors

Beam curvature ductility factors obtained over a gauge length of 170mm, using the second set of linear potentiometers along the beam and close to the column faces, are shown in Fig. 6.34. As in all previous results, the beam curvature ductility factors so found have a considerable scatter.

The rotational ductility factors plotted in Fig. 6.35 show a more predictable array. The predicted values always overestimated the measured ones because in the analysis it was assumed that deformations occurred mainly in the beams.

6.3.6.2 Beam Strain Profiles at the Level of the Longitudinal Reinforcement

The strain profiles illustrated in Figs. 6.36 and 6.37 were estimated from the readings taken with linear potentiometers attached to the beam and following the procedure described in Section 3.8.3.2.
Fig. 6.34 - Beam Curvature Ductility Factors of Unit 6.

Fig. 6.35 - Beam Rotational Ductility Factors of Unit 6.
Fig. 6.36 - Beam Strain Profiles at the Level of the Top Reinforcement of Unit 6.
Fig. 6.37 - Beam Strain Profiles at the Level of the Bottom Reinforcement of Unit 6.
In the initial loading cycles in the inelastic range yielding spread along the beam from the column faces and then gradually extended over a distance larger than \( d \). However, the instrumentation placed did not permit determination with certainty of whether yield of the reinforcement actually occurred beyond a distance \( d \) from the column faces.

The strain distribution as estimated from linear potentiometer measurements followed the same trends that those observed in the beams of Units 1, 2 and 3 as well as in the bottom of the beams of Unit 5. That is, the measurements taken near the faces of the columns were very sensitive to the formation of cracks in the beams in that region. Cracks tended to cross the holes provided for clearance around the steel rods embedded in the concrete and used to hold the brackets for fastening the displacement transducers. This is why the strain distributions in the negative displacement ductilities of the north top of the beam (Fig. 6.36) and in the positive displacement ductilities of the north bottom of the beam (Fig. 6.37) did not indicate maximum strain at the faces of the column.

### 6.3.6.3 Beam Elongation

The total measured elongation of the beams of Unit 6 plotted against the lateral load is depicted in Fig. 6.38. Some residual beam elongation remained after the loading cycles in the elastic range due to the imperfect closure of the initial cracking. In the loading cycles in the inelastic range the cumulative elongation of the beam increased until in load run 15 the flexural capacity of the unit was no longer attainable due to the considerable shear deformations in the plastic hinges. At this stage the total elongation had reached 45mm, a value much larger than that recorded in Unit 5. The main reason for the difference between the two results is the good anchorage conditions in the joint of Unit 6, which permitted the plastic hinges to spread along the beam from the columns faces.
6.4 CONCLUSIONS

1. The overall performance of the units tested was very satisfactory in terms of strength and ductility. The connection details between the precast elements were observed not to have a detrimental effect on the seismic behaviour of the beam-column systems tested. Hence behaviour of the units as if of monolithic constructions was achieved.

2. Unit 5, which had the lower part of the beams precast and the bottom beam bars anchored inside the cast in place concrete beam-column joint core showed that this type of detail can be effectively used in the construction of ductile moment resisting frames. Sliding along the vertical construction joints at the ends of the precast concrete beams at the face of the column was kept to a minimum despite the smooth surface of the construction joints. It appears that by seating the precast concrete beams on the cover of the column below vertical sliding shear is controlled and it is not necessary to provide shear keys along the construction joints. However some surface roughness of the ends of the beams is recommended.

3. From the observation made in Units 5 and 6, it can be concluded that the current recommendations of the Concrete Design Code [NZS 3101 (1982)] for limiting the diameter of the longitudinal bars passing through the joint are adequate but not necessarily conservative. In exceptional cases such as with the quality of the concrete cast in place in the beam-column joint core of Unit 5, bleeding of the concrete may result in bond failure of top bars during severe earthquake loading.

   It is therefore recommended that any revision to the current provisions considers the "top bar effect".

4. In Unit 6 the effect of the grouted vertical ducting for the column bars did not alter the behaviour of the Unit as a whole. The forces developed in the transverse reinforcement in the beam-column joint region were effectively transferred through the grout to provide the node required to mobilise the truss mechanism for shear resistance. The smooth horizontal construction joints at the bottom of the precast concrete beam member allowed some horizontal sliding action to occur, which was not important in terms of the overall lateral displacement imposed but was large in terms of the deformations of the columns. Again some surface roughness at the bottom of the precast beam at the joint is recommended.

5. From the data gathered in this study it is concluded that in short beams typical of perimeter frames with clear span/depth ratios lower than 5, shear deformations will be expected to govern the response of the beam at large displacement ductility factors even if the nominal vertical shear stresses in the plastic hinges of symmetrically reinforced beams are kept as low as $0.20\sqrt{E}$. The main factor determining this behaviour is the aspect ratio of the member.

6. The measured stiffness of the cruciform components tested indicate that the normal design office procedures used to estimate this value need more refinement.
7.1 INTRODUCTION

This chapter deals with the design of the connections between precast concrete members using the reinforcement details of the test programme. Truss models and the concept of shear friction are used to explain the distribution of forces and to find the amount of necessary reinforcement. An analysis of the connections at midspan is made in the first part. In the second part the connections at the beam-column joint region are considered. The effects of the elongation of the beam plastic hinges are also discussed in this chapter.

7.2 THE DESIGN OF MIDSPAN CONNECTIONS BETWEEN SHORT BEAMS OF PERIMETER FRAMES

7.2.1 Determination of the Tension Force in the Longitudinal Reinforcement of a Beam

It has long been recognized that in cracked reinforced concrete beams the tension force in the longitudinal reinforcement cannot be directly estimated from the M/\(M_{zd}\) diagram due to the interaction between flexure and shear [Paulay (1969), Park and Paulay (1975)].

Following experimental work on short coupling beams between walls, Paulay (1969) found that the measured tension force in the longitudinal reinforcement was reasonable parallel to the M/\(M_{zd}\) diagram but with a permanent shift. He concluded that due to this effect the top and bottom longitudinal reinforcement in a beam will be subjected to equal tension forces in its point of contraflexure and, in extreme circumstances such as very deep beams, all longitudinal reinforcement could eventually be subjected to tension all along the beam.

The effects of the "tension shift" are illustrated in Fig. 7.1, where a typical bending moment diagram for a beam of a perimeter frame of a building is shown. In general in short beams of perimeter frames gravity loading does not have a significant influence and therefore it can be assumed that the point of contraflexure in the beam coincides with its midspan.

The tensile forces in the longitudinal steel of a conventionally reinforced beam can be estimated from Fig. 7.1 (c) by accounting for the "tension shift". If the effect of gravity loading is ignored, the maximum probable tension force \(T'\) at a distance \(x\) from the point of contraflexure is
\[
T' = V^o \left( \frac{x}{jd} + \frac{1}{2 \tan \theta} \right) \quad \text{for} \quad \frac{l_b'}{2} - \frac{jd}{2 \tan \theta} \leq x \leq \frac{jd}{2 \tan \theta}
\]

or

\[
T' = V^o \frac{l_b'}{2jd} = T^o \quad \text{for} \quad \frac{l_b'}{2} \leq x \leq \frac{l_b'}{2} - \frac{jd}{2 \tan \theta} \quad \text{(7.1)}
\]

where \( V^o \) is the shear at the development of the flexural overstrength of the beam, \( jd \) is the internal lever arm, \( \theta \) is the angle of inclination of the compression field, \( l_b' \) is the beam clear span, and \( T^o \) is the tension force in the longitudinal beam reinforcement at the steel overstrength.

Fig. 7.1 - A Conventionally Reinforced Concrete Beam of a Perimeter Frame.
The tension force $T'$ at midspan can be obtained from Eq. 7.1 substituting $x = 0$,

$$
T' = \frac{V^2}{2\tan\theta}
$$

(7.2)

It is evident from Eq. 7.2 that any connection located at the point of contraflexure of a short beam will have to resist a moderate set of forces $T'$ without adversely affecting the capacity nor the ductility of the connecting members. It can also be seen in Eq. 7.2 that larger values of $T'$ are expected in beams carrying larger shear forces and that the flatter the angle of inclination $\theta$ of the diagonal compression field, the larger the effects of the "tension shift".

Park and Paulay (1975) made a theoretical evaluation of the "tension shift" effect based on considerations of equilibrium assuming that the shear in the beam is carried by two different mechanisms, involving the concrete (through aggregate interlock) and the transverse steel. They found a family of solutions for different combinations on the amount of shear carried by aggregate interlock, the inclination of the cracks and the angle of inclination of the stirrups. They recommended that a tension shift $e_v = d$, where $d$ is the effective depth of the beam, would be a simple value to be used in a design procedure. This value has been implicitly adopted in the ACI-318 Concrete Design Code (1989) and in the NZS 3101 Code (1982) in the curtailment of the flexural reinforcement of beams.

The magnitude of the "tension shift" can also be directly evaluated by modelling the cracked reinforced concrete beam as a truss member with the diagonal compression struts inclined at an angle $\theta$ to the horizontal. This is known as the variable angle truss model [Collins and Mitchell (1991)]. In an attempt to quantify the angle of inclination of the struts, the plasticity theory [Nielsen et al (1978), Thürlimann (1978)] assumes that both concrete and steel will reach ultimate conditions at the same time. An effective or reduced concrete strength is accounted for in the equilibrium conditions and some empirical concrete contribution is needed to satisfy the solution for lightly reinforced beams. The variable truss model has been used in the CEB-FIP Concrete Design Code (1978) where the designer chooses an angle of inclination $31^\circ \leq \theta \leq 59^\circ$. The corresponding "tension shift" is from geometry in Fig. 7.1 (c) $e_v = jd/2\tan\theta$, where $jd$ is the lever arm in the beam. If $j$ is assumed equal to 0.9 the tension shift is limited to $0.27d \leq e_v \leq 0.75d$, which are smaller values than those implicitly adopted by the ACI-318 and the NZS 3101 Concrete Design Codes. In terms of the angle of inclination of the diagonal compression field these two codes assume $\theta = 24^\circ$.

A refined procedure to find the values of the angles of inclination of the diagonal struts has been presented by Collins and Mitchell, which considers conditions of equilibrium, compatibility and assuming a stress-strain relationship for the materials. This procedure has been shown to be accurate but can be lengthy for use in practice.

The "tension shift", $e_v$, measured at the beginning of the hooks in Units 1 and 2 and at midspan in Unit 3 was $0.50d$, $0.23d$ and $0.56d$, respectively. These values are closer to the tension shift
predicted by the variable angle truss model than that of \( \varepsilon, = d \), implicit in NZS 3101. Note, however, that the above values were determined from calculated stresses in the reinforcement, which in turn were estimated from average strains in the reinforcement. As a result, the total force in the section may be underpredicted because the concrete carries tensile forces between cracks. However, this contribution is unlikely to compensate for the difference observed between \( \varepsilon, = d \) and that measured.

It is believed that the New Zealand code provisions could be made more realistic. The transverse reinforcement in the critical region of members designed in accordance with NZS 3101 (1982) can be governed either by the shear or by the anti-buckling requirements if the level of axial load is small as it is the case of beams or cantilever structural walls. The design for shear is based on a 45° truss model. The contribution of the concrete in carrying the shear is nil for zero axial load level and increases with the axial compressive load. Outside of the plastic hinge region the contribution of the concrete in carrying the shear is significantly increased in recognition of the shear transfer through aggregate interlock. That is, the resulting diagonal compression field will be flatter in this region. It is estimated that an angle \( \theta = 35° \) will still provide a conservative envelope in members designed to NZS 3101:1982. This angle of inclination of the diagonal compression field will result in \( \varepsilon, = 0.7jd = 0.65d \), which could have an important effect on the curtailment of the reinforcement of structural concrete and masonry walls and to a lesser extent in the curtailment of the beam reinforcement. This angle will be adopted in the derivation of the design recommendations for the connection between precast concrete members presented in the following sections.

### 7.2.2 Design of Midspan Connections with Overlapping Hooks

The use of overlapping hooks for connecting precast concrete beams is particularly useful when these beams possess a rather low aspect ratio and have space limitations for the connection. This section deals with the design of the connection details using overlapping hooks at the point of contraflexure such as those that were tested in this study.

In the test of Units 1 and 2 it was indicated that the connection detail at the beam midspan did not fulfil a lapping action since the tension bars were well anchored by the hooked ends.

An equilibrium solution for the connecting detail used in Unit 1 that satisfies the experimental observations is illustrated in Fig. 7.2. Since there is no continuity between the top overlapping bars and the hooked bars protruding from the precast concrete unit to the right in Fig. 7.2 the whole beam shear needs to be locally transferred by a tension tie between the top and bottom connecting details. In the detail using overlapping hooks the compression field will be fanned in a very similar way to that of a simply supported beam. The main difference between this detail and the seating of a simply supported beam is the way how the forces enter the node. With regard to Fig. 7.3, the vertical force in the connecting detail comes from a tie in tension while in the simply supported beam it comes from the compression against the seating.
Fig. 7.2 - Truss Model for Midspan Connection with Overlapping Hooks.

Fig. 7.3 - Comparison of Boundary Conditions between a Simple Supported Beam and a Midspan Connection.
It can be argued that transferring the whole shear at the connection region by a tension steel tie ignores any contribution by tension in the concrete, especially in beams with a nominal shear stress below the diagonal tension cracking stress of about $0.33\sqrt{f_c}$ as stipulated by the New Zealand Concrete Design Code. However, for this type of short splice a large difference in the amount of tie reinforcement required will not bring large savings to the construction itself.

Splitting of the concrete caused by the radial components of the hooked anchorages can eventually have a detrimental effect on the compressive strength of the concrete strut in the connecting detail. This is why the detailing of the reinforcement in this region is of primary importance. Transverse rods will help in the distribution of the stresses as illustrated in Fig. 7.4. In addition, closed stirrups around the hooked connection will also be required to avoid a premature failure due to splitting.

The bend radii of the hooked bars shall be such to avoid a premature compression failure of the concrete strut at that point. For this, the calculated stresses should be limited to a value related to the unconfined cylinder strength of the concrete, $f_c$. From Fig. 7.2

$$D^o = \sqrt{V^o^2 + T^2} \quad (7.3)$$

where $D^o$ is the diagonal force carrying the beam shear $V^o$. Both forces are associated with the beam flexural overstrength.

Substituting $\theta = 35^\circ$ in Eq. 7.2 gives
and substituting $T'$ from Eq. 7.4 (a) into Eq. 7.3 gives

$$D^\circ = 1.23 V^\circ$$  \hspace{1cm} \text{(7.4 b)}$$

The above force $T'$ needs to be resisted by the longitudinal reinforcement being overlapped at midspan.

The bearing stresses, $f_b$, induced by the diagonal strut acting upon the hooked bar (see Fig. 7.2) can be estimated as

$$f_b = \frac{D^\circ}{\zeta d_b (b_w - 2c_o)}$$  \hspace{1cm} \text{(7.5)}$$

where $\zeta$ is the ratio between the diameter of the bend and the diameter of the bar $d_b$, $b_w$ is the width of the beam and $c_o$ is the concrete cover to the reinforcement.

Substituting Eq. 7.4 (b) in Eq. 7.5, assuming $b_w - 2c_o = 0.75b_w$ and rearranging for $\zeta$ gives

$$\zeta = \frac{1.64V^\circ}{d_b b_w f_b}$$  \hspace{1cm} \text{(7.6)}$$

According to Schlaich et al. (1987) an appropriate value for the maximum bearing stress in a node of the type shown in Fig. 7.3 (b) in terms of the concrete cylinder strength is

$$f_b = 0.6 \times 0.85 f'_c = 0.51f'_c$$  \hspace{1cm} \text{(7.7)}$$

where a strength reduction factor $\phi = 1$ has been taken for a capacity designed element [NZS 3101 (1982)].

Then substituting Eq. 7.7 in Eq. 7.6 gives the minimum value permitted for $\zeta$

$$\zeta = \frac{3.2V^\circ}{d_b b_w f'_c}$$  \hspace{1cm} \text{(7.8)}$$

According to the recent amendment to the New Zealand Concrete Design Code, the factor $\zeta$ shall not be less than 5 for bars of 6-20mm in diameter or less than 6 for bars of 24-40mm in diameter.

The connection detail using overlapping "drop in" double hooked bars can be considered an extension of the previous case. A truss model based on the experimental evidence is illustrated in
Fig. 7.5 - Truss Model for Midspan Connection Using "Drop In" Double Hooked Bars.

Fig. 7.5. The compression field fan will spread through the splice at an angle varying from $\theta$ to $\theta''$ as shown in Fig. 7.5. The resultant diagonal force $D^\circ$ will be at an angle $\theta'$ which lies between $\theta$ and $\theta''$. For simplicity it can be assumed that the diagonal compression field will spread from the connection at this angle $\theta'$. Therefore the force $T''$ in the "drop in" bars can be estimated from Fig. 7.5 to be

$$T'' = T' - \frac{\Delta V^\circ}{\tan \theta'} \quad (7.9)$$

where $\tan \theta' = V^\circ/T'$ and $\Delta V^\circ$ is the fraction of the beam shear force at overstrength transferred by the set of stirrups in the overlapping region.

A practical lay-out of the transverse reinforcement in this region is to have five sets of stirrups arranged as shown in Fig. 7.5. This arrangement implies that $\Delta V^\circ = 2/5V^\circ$ will be transferred in the overlapping region.

$$T'' = T' - \frac{2}{3} V^\circ \frac{1}{V^\circ/T'} = \frac{3}{5} T' \quad (7.10)$$

The force $T'$ at the beginning of the hooks can be calculated assuming that the offset distance to the point of contraflexure is $0.2jd$. This is a reasonable assumption for the dimensions normally used
in perimeter beams. Hence, substituting Eq. 7.1 in Eq. 7.10 and $x$ by the approximate value results in

$$\theta = 35^\circ,$$

$$T' = 0.91 V^\circ,$$  \hspace{1cm} \text{(7.11a)}

and

$$T'' = 0.55 V^\circ.$$  \hspace{1cm} \text{(7.11b)}

Enough longitudinal reinforcement should be present in the overlapping region to ensure that the force $T'$ can actually be developed. Besides, the above force $T''$ needs to be transferred to the "drop in" bars from the protruding hooked bars through the concrete. The shear transfer mechanism can be designed using the shear friction concept. With reference to Fig. 7.6, the area of transverse reinforcement required, $A_{st}$, is

$$A_{st} = \frac{T''/n}{\phi \mu_t f_{yt}}$$  \hspace{1cm} \text{(7.12)}

where $f_{yt}$ is the yield strength of the transverse steel, $\mu_t$ is the coefficient of friction taken as 1.4 for monolithic concrete, $n$ is a factor that relates the maximum force to be transferred between the lapped bars estimated as shown in Fig. 7.7, and $\phi$ is the strength reduction factor taken as 1 for capacity designed members.
Then combining Eq. 7.11 (b) and Eq. 7.12 gives

\[ A_{s1} = 0.39 \frac{V^n}{n f_y} = \frac{V^n}{2.5 n f_y} \]  

(7.13)

It is now necessary to verify that the force \( T'' \) can be transferred without premature crushing of concrete. The current New Zealand Concrete Design Code limits the shear stresses in the concrete, \( \tau_r \), at the development of the shear friction mechanism to a maximum of 0.2\( f'_c \) or 6MPa whichever is smaller. The effective friction surface \( A_r \), where the lapping action between the hooks occurs can be estimated as depicted in Fig. 7.6,

\[ A_r = 4.5 \eta d_b^2 \]  

(7.14)

where \( \eta d_b \) is the lapping distance measured between the centrelines of the extension lengths of the hooks. In no case will \( \eta \) be less than the minimum diameter of bend plus one bar diameter. That is \( \eta \geq 6 \) for bars of 6-20mm in diameter and \( \eta \geq 7 \) for bars between 24-40mm in diameter.

Therefore the average shear stress, \( \tau_r \), is equal to the force being transferred, \( T''/n \) divided by the area \( A_r \), resulting in

\[ \frac{T''}{n A_r} = \frac{2(A_{s1}+A_{s2}+A_{s3})}{A_{s1}+A_{s3}} \text{ or } \frac{2(A_{s1}+A_{s2})}{A_{s1}} \text{ or } \frac{2(A_{s1}+A_{s2})}{A_{s2}} \]

Fig. 7.7 - Criteria for Determining Factor \( n \).
Combining Eqs. 7.11 (b) and 7.14 in Eq. 7.15 and rearranging for $\eta$ gives

$$\eta = \frac{0.12 V^o}{n \tau_f d_b^2}$$

(7.16)

Now substituting $\tau_f$ by the limiting values and summarizing results in

$$\eta \geq \begin{cases} 
0.6 \frac{V^o}{n f_c' d_b^2} \\
\text{or} \frac{V^o}{45 n d_b^2}, \\
\text{or 6 for } 6 \leq d_b \leq 20, \\
\text{or 7 for } 24 \leq d_b \leq 40.
\end{cases}$$

(7.17a) (7.17b) (7.17c) (7.17d)

7.2.3 Design of Midspan Connections Using Straight Bar Splices

Straight bar splices offer a very simple and effective way for connecting precast elements together. The straight bars protruding from the precast concrete beams may or may not overlap the bars from the connecting element. In the first case a non-contact lap splice is normally used while in the second case a double straight lap involving additional reinforcement is used.

The design procedure for finding the length of the splice as well as the transverse reinforcement around it can be derived using the recommendations of the New Zealand Concrete Design Code. In evaluating the code required splice length, which is considered equal to the development length, $l_d$, all modification factors may be considered. This section will present an alternative design procedure to determine the splice length and the transverse reinforcement based on the shear friction concept, which is similar to that presented by Paulay (1982) (see also Paulay and Priestley (1992)).

Fig. 7.8 shows a non-contact horizontal lap splice that is used to connect two precast concrete elements at the beam midspan. Also shown in Fig. 7.8 are two diagonal compression fields, one of them in the vertical plane which is required to balance the shear forces and the force $T - T'$. 
The other diagonal compression field, acting on the horizontal plane, transfers the force $T'/2$ between each pair of bars. For a lapped connection symmetrically located at the point of contraflexure it is convenient, and conservative, to assume that the force $T'$ at the end of the splice is equal to that derived from the shifted tension force diagram at midspan $T''$. This assumption has the advantage of uncoupling $T'$ from the length of the splice and also forms the particular solution for a double straight lap.

Therefore, $T''$ is found assuming $\theta = 35^\circ$ in Eq. 7.2.

$$T'' = 0.71 V^o$$ (7.18)

The amount of reinforcement required to cross the potential crack along the spliced bars can be estimated using the shear friction concept as in Eq. 7.12. Hence substituting Eq. 7.4 in Eq. 7.12 and replacing $\phi = 1$ and $\mu = 1.4$,

$$A_{st} = 0.51 \frac{V^o}{nf_{yt}} = \frac{V^o}{2nf_{yt}}$$ (7.19)
The above expression can also be applied to splices where the shear friction mechanism will develop in the vertical plane.

To enable an efficient use of the transverse reinforcement given by Eq. 7.19, no splicing bars should be at more than 100mm apart from an adjacent tie (see Fig. 7.7).

The force $T'/n$ in each lapping bar will be transferred over the length of the splice, $l_s$ and a width of $3d_b$ as suggested by Paulay (1982). Thus the effective friction surface, $A_e$, is given by

$$A_e = 3d_b l_s$$  \hspace{1cm} (7.20)

and the average shear stress, $\tau_f$ is

$$\tau_f = \frac{T'}{n} \frac{1}{A_e}$$  \hspace{1cm} (7.21)

Substituting Eqs. 7.18 and 7.20 in Eq. 7.21 and simplifying

$$\tau_f = 0.24 \frac{V^o}{n d_b l_s}$$  \hspace{1cm} (7.22)

Then the length of the splice shall be such as to satisfy the two limits imposed to avoid a crushing failure at the development of the shear friction mechanisms. That is, substituting 6MPa or $0.2f'_c$ for $\tau_f$ Eq. 7.22 and rearranging for $l_s$ gives in summary

$$l_s \geq \begin{cases} 
\frac{1}{25n} \frac{V^o}{d_b} \\
\frac{1.2}{n} \frac{V^o}{d_b f'_c} \\
or \frac{l_{db}}{}
\end{cases}$$

### Chapter 5 was concerned with the test of a precast concrete subassemblage with diagonal reinforcement and connected at midspan with bolted steel plates. The test unit had been designed to concentrate all the inelastic deformations within the central part of the beam. Thus, the plastic hinge
regions were relocated from the beam ends using extra reinforcing steel as required by the simple truss model shown in Fig. 3.9. The initial design overlooked the fact that the additional bars used to relocate the beam plastic hinges and the bars that were diagonally bent and welded to the midspan steel plates were in different planes. As a result there were significant out of plane forces that caused splitting cracks between the layers of reinforcement. Consequently the nodes of the simple truss model were destroyed and the test unit displayed a limited ductile response.

The damaged unit was repaired taking into account the three dimensional effects and detailing transverse reinforcement in the critical regions at the bend of the diagonal bars. This test displayed an excellent ductile behaviour.

The experimental results showed an adequate performance of the bolted connection at the midspan of the beams. No further discussion regarding the bolted steel connections is made in this study since the design of these types of connections is found in standard structural steel textbooks. Therefore, the remainder of this section will be limited to the design of the transverse reinforcement around the outer bars in the critical region at the bend of the diagonal bars.

With regard to Fig. 5.6, the force $R_H$ required for equilibrium on the outer bars in the strong ends of the beam was found in Eq. 5.2 in terms of the different forces acting upon the critical region at the bend of the diagonal bars. For a capacity designed member it will be expected that the sum of forces $T_v + S_T$ should be equal to the beam shear. Also, for the inclination of the diagonal reinforcement in normal practice it can be considered that the difference between force $T_H$ and $T$ is sufficiently small that these forces can be ignored. Hence Eq. 5.2 can be rewritten in terms of the beam shear force at the flexural overstrength, $V^o$, and the angle of inclination of the diagonal strut, $\beta$, as

$$R_H = \frac{V^o}{\tan \beta} \quad \text{(7.24)}$$

The beam shear force can be found in terms of the force in the diagonal bars as

$$V^o = \frac{2A_{sdg} \lambda_o f_y}{\sqrt{1 + \left(\frac{l_{sg}}{d-d'}\right)^2}} \quad \text{(7.25)}$$

where $A_{sdg}$ is the area of reinforcement in each of the diagonals, $\lambda_o$ is the steel overstrength factor, $l_{sg}$ is the length of the beam between the critical regions and $d-d'$ is the distance between the centroids of the beam longitudinal reinforcement.

The inclination of the diagonal strut can also be found in terms of the geometry of the beam as shown in Fig. 7.9

$$\tan \beta = \frac{d-d'}{l_\pi} \leq 0.75 \quad \text{(7.26)}$$
Fig. 7.9 - Diagonally Reinforced Beam with Relocated Plastic Hinges.

where \( l_a \) is the distance between the face of the column and the critical region at the bend of the diagonal reinforcement. It is suggested that the angle \( \beta \) should not be taken as less than 37\(^\circ\) since it is likely that for plastic hinges relocated at a distance larger than 1.33 (\( d - d' \)) from the column face, the diagonal strut at the bend of the diagonal reinforcement will not converge at the column face.

Thus, substituting Eqs. 7.25 and 7.26 into Eq. 7.24 gives

\[
R_H = 2 \frac{l_{at}}{d - d'} \frac{A_{sdg} \lambda_o f_y}{\sqrt{1 + \left( \frac{l_{dg}}{d - d'} \right)^2}} \quad (7.27\ a)
\]

or

\[
R_H = \frac{8}{3} \frac{A_{sdg} \lambda_o f_y}{\sqrt{1 + \left( \frac{l_{dg}}{d - d'} \right)^2}} \quad (7.27\ b)
\]

whichever is smaller.

The clamping force required to transfer one half of the force \( R_H \) to each side of the beam is found from the shear friction concept

\[
A_{st} f_{st} = \frac{R_H/2}{\phi \mu_f} \quad (7.28)
\]
where $A_n$ is the area of transverse reinforcement and $f_{yt}$ is its yield strength, $\phi$ is the strength reduction factor taken equal to 1 for capacity designed elements, and $\mu_r$ is the coefficient of friction taken equal to 1.4 for monolithic concrete.

Substituting Eq. 7.27 in Eq. 7.28, assuming $\lambda_n = 1.25$ and rearranging for $A_n$ gives

$$A_n = 0.9 \frac{1}{d-d'} \frac{1}{f_{yt} A_{sdg}} \frac{f_y}{f_{yt}} A_{sdg}$$

(7.29 a)

or

$$A_n = 1.2 \frac{1}{d-d'} \frac{1}{f_{yt} A_{sdg}} \frac{f_y}{f_{yt}} A_{sdg}$$

(7.29 b)

whichever is smaller.

The effective shear friction surface, $A_n$, can be approximated to $64d_b^2$ as suggested in Fig. 7.10 where $d_b$ is the diameter of the additional bars placed in the strong ends of the beam. This

Fig. 7.10 - Effective Shear Area in the Critical Region of a Diagonally Reinforced Beam with Relocated Plastic Hinges.
area is considered to be larger than the unreinforced one shown in Fig. 5.6 (c) due to the extended influence of the clamping forces. Thus the average shear stress is given by

$$\tau_f = \frac{R_H/2}{A_T} = \frac{R_{H}}{128d_b^2}$$  \hspace{1cm} (7.30)

where \(\tau_f\) is to be kept to below the smaller of 0.2\(f'_c\) or 6MPa.

7.2.5 Example Illustrating the Design of Connections at the Beam Midspan

The precast concrete beams of a perimeter frame are to be connected at midspan in a cast in place concrete joint. Evaluate three different alternatives using the following information,

\(A_{st} = A_{st} = 6\) HD24
\(f'_y = 430\)MPa
\(f'_c = 30\)MPa
\(d = 900\)mm
\(b_w = 500\)mm
\(V^o = 605\)kN

a) Overlapping Hooks (see Fig. 7.11 (a))

Step 1 - Splice 4 HD24 bars

\(T_{\text{provided}} = 4 \times 452 \times 430 / 1000 = 777\)kN

Check that the amount of steel provided is greater than that required

\(T_{\text{required}} = 0.71 \times 605 = 777\)kN \hspace{1cm} [Eq. 7.4 (a)]

\(\frac{T_{\text{required}}}{T_{\text{provided}}} = 0.55, \text{ satisfactory}\)

Step 2 - Check the diameter of bend

$$\zeta \geq \begin{cases} 5.4 \hspace{1cm} \text{[Eq. 7.8]} \vspace{0.5cm} \\ 6 \hspace{1cm} \text{[NZS 3101 - Table 5.1]} \end{cases}$$

Use a diameter of bend of at least 150mm (\(\zeta = 6.3\)).
Use 180° standard hooks and detail HD24 transverse rods in contact with the concave sides of the hooks.

**Step 3**

Design the vertical transverse reinforcement in the connection region.

This reinforcement is to carry all the shear force at overstrength.

\[
A_{st} = \frac{605,000}{430} = 1,407 \text{mm}^2
\]

Use 2-4 HD16 closed stirrups \((A_{st} = 1,608 \text{mm}^2)\)

b) "Drop in" Double Hooked Connection (see Fig. 7.11 (b))

**Step 1**

Overlap 4 HD24 bars

\[
\frac{T_{\text{required}}}{T_{\text{provided}}} = 0.71, \text{ satisfactory}
\]

**Step 2**

Design the vertical reinforcement at the connection to take all the shear force at overstrength

\[
A_{st} = 1,407 \text{mm}^2
\]

Use 5-4 HD10 closed stirrups \((A_{st} = 1,570 \text{mm}^2)\)

**Step 3**

Design the transverse reinforcement at the overlapping of the hooks to enable the development of the shear friction mechanism.

Only the legs of the outer stirrups are effective for this purpose. There are 2 legs along each splice. Hence

\[
A_{st \text{ provided}} = 2 \times 78.5 = 157 \text{mm}^2
\]
\[n = 4 \text{ (four bars spliced in different vertical planes, see Fig. 7.7)}\]

\[
A_{st \text{ required}} = 141 \text{mm}^2
\]

**Step 4**

Check the overlapping distance

Overlap the bars a distance of 174mm between centres of the extension lengths \((\eta = 7.25)\).
c) Non-contact splices (see Fig. 7.11 (c))

Step 1 - Lap 4 HD24 bars

\[
T_{\text{provided}} = 777 \text{kN}
\]
\[
T_{\text{required}} = 0.71 \times 605 = 430 \text{kN}
\]

\[
\frac{T_{\text{required}}}{T_{\text{provided}}} = 0.55, \text{ satisfactory}
\]

Step 2 - Find the length of the splice according to the New Zealand Concrete Design Code.

Basic development length,

\[
l_{db} = \frac{380 \times 452}{52 \times \sqrt{30}} = 603 \text{ mm}
\]

Modification factors:

Different yield strength factor, \( m_1 = \frac{430}{300} = 1.43 \)

Top bar effect factor, \( m_2 = 1.3 \)

Surplus of reinforcement factor, \( m_3 = 0.55 \)

Transverse reinforcement factor, \( m_4 = \frac{62}{62 + 24} = 0.72 \)

Hence, \( l_d = m_1 m_2 m_3 l_{db} = 0.74 l_{db} = 446 \text{ mm} \)

\( l_d \geq 300 \text{ mm}, \text{satisfactory} \)

Find the length of the splice using the alternative procedure from Figs. 7.7 and 7.11 (c),
n = 2

\[
l_s \geq \begin{cases} 
548 \text{mm} & \text{[Eq. 7.23 a]} \\
504 \text{mm} & \text{[Eq. 7.23 b]} \\
446 \text{mm} & \text{[Eq. 7.23 c]}
\end{cases}
\]

Use \( l_s = 600 \text{mm} \) (25\(d_b\))

**Step 3 - Design for shear at midspan**

\( v_t = 1.34 \text{ MPa} \) \hspace{1cm} \text{[NZS 3101:1982, Eq. 7-2]}

\( v_c = 0.11 \sqrt{\frac{f_{c}}{f_{c}}} = 0.60 \text{ MPa} \) \hspace{1cm} \text{[NZS 3101:1982, Eq. 7-4]}

\( v_s = 1.34 - 0.60 = 0.74 > 0.35 \) \hspace{1cm} \text{[NZS 3101:1982, Eq. 7-12]}

\[
\frac{A_v}{s} = 0.74 \times \frac{500}{430} = 0.86 \frac{\text{mm}^2}{\text{mm}}
\] \hspace{1cm} \text{[NZS 3101:1982, Eq. 7-14]}

**Step 4 - Find the transverse reinforcement required to develop the shear friction mechanism along the spliced bars.**

\[
A_v = \frac{605,000}{2 \times 2 \times 430} = 352 \text{mm}^2
\] \hspace{1cm} \text{[Eq. 7.19]}

that is, \( \frac{A_v}{l_s} = \frac{352}{600} = 0.59 \frac{\text{mm}^2}{\text{mm}} \)

Therefore the shear friction mechanism controls the design, use 5 HD10 closed stirrups @ 130mm centre to centre, (\( A_{v, \text{provided}}/l_s = 1.21 \text{mm}^2/\text{mm}, \ A_{v, \text{provided}}/l_s = 0.6 \text{mm}^2/\text{mm} \)).

**7.3 DESIGN OF CONNECTION AT THE BEAM-COLUMN JOINT REGION**

**7.3.1 General**

This section deals with the design of connections between precast concrete members at the beam-column joint region. The experimental programme showed that connections of Systems 1 and 2
Fig. 7.11 - Design Example of Midspan Connection Details.
behaved as conventional monolithic construction. Hence, the design of interior beam-column joints will be reviewed in relation to the bond and shear transfer mechanisms.

7.3.2 Input Actions in Interior Beam-Column Joints

Fig. 7.12 shows the input forces at an interior beam-column joint once plastic hinges have developed in the beams at the faces of the column.

A reasonable approximation is to assume that the beam shear forces are carried to the boundaries of the joint by a set of diagonal struts with forces $D_B$ and $D_T$. The inclination of these diagonal struts can be estimated from the fan-shaped crack pattern at the plastic hinges of the beams and assuming that the stirrups in those regions are equally stressed. From Fig. 7.13 where the top plastic hinge of the right beam is analyzed

$$\tan \theta_t = \frac{l_{pf} (d - d')} { \int_0^{l_{pf}} x \, dx} = 2 \left( \frac{d - d'} {l_{pf}} \right)$$

(7.31)

The length of $l_{pf}$ where the beam shear is transferred through the stirrups is expected to be no more than $(d - d')$ for beams designed according to the New Zealand Concrete Design Code because the transverse reinforcement placed can be governed by the shear requirements, assuming a $45^\circ$ truss model, or by the need to prevent premature buckling of the longitudinal beam reinforcement. For simplicity it will be assumed

$$l_{pf} = (d - d')$$

(7.32)

therefore the horizontal component of the diagonal force $D_T$ is

$$V_{HT} = D_T \cos \theta_t = \frac{V_T \tan \theta_t} {\tan \theta_t}$$

(7.33)

now substituting Eqs. 7.31 and 7.32 in Eq. 7.33 gives

$$V_{HT} = \frac{V_T} {2}$$

(7.34)

Equilibrium considerations in the right beam of the column illustrated in Fig. 7.12 require that

$$C_{SB} + C_{CB} = T_T + T_s - D_T \cos \theta_T = T_T + T_s - V_{HT}$$

(7.35)

where $T_T$ is the actual tension force in the top beam longitudinal reinforcement, $T_s$, is the tension force due to the effective reinforcement in the slab, $C_{SB}$ is the compression force in the bottom longitudinal reinforcement and $C_{CB}$ is the compression force carried through the concrete.
Fig. 7.12 - Forces Acting Upon a Concrete Column.

Fig. 7.13 - Shear Transfer in the Plastic Hinge Region of a Beam.
The resultant force $T_T + T_s - V_{HT}$ in Fig. 7.12 is largely carried by the concrete in beams of normal dimensions behaving elastically. Nevertheless, the force $C_{CB}$ will gradually decrease, and eventually become zero, once the longitudinal reinforcement at this point has previously yielded in tension, unless the total compression force is such that the reinforcement yields back in compression, or that the bond in the beam-column joint has deteriorated to an extent where the local slip of the bars is such that this mechanism is unable to provide most of the reaction required, or that the compression bars have buckled. The extreme case of bond deterioration is when a bond failure has occurred and the reinforcement in the compression side is in tension and needs to be anchored in the beam. It is evident that all these cases imply a closure of the crack at the interface between the beam and the column.

Bond deterioration and bar buckling are unlikely to significantly affect the distribution of the resultant $T_T + T_s - V_{HT}$ between forces $C_{SB}$ and $C_{CB}$ in well detailed beams at moderate levels of ductility, since an objective of the design philosophy is to delay these two sources of failure. Hence, these two cases will not be considered in this study.

The compression force in the longitudinal steel after few reversal cycles in the inelastic range can be estimated by combining Eqs. 7.34 and 7.35, rearranging for $C_{SB}$ and assuming $C_{CB} = 0$ results in

$$C_{SB} = T_T + T_s - \frac{V_T}{2} \quad \text{and} \quad C_{SB} \leq A_{SB} \frac{\lambda_o f_y}{2} \quad (7.36 \text{a})$$

or in terms of the reinforcement areas and stresses,

$$A_{SB} f_s = (A_{ST} + A_{SS}) \lambda_o f_y - \frac{V_T}{2} \quad \text{and} \quad f_s \leq \lambda_o f_y \quad (7.36 \text{b})$$

where $\lambda_o$ is the steel overstrength factor, $A_{SB}$ is the bottom reinforcement in the beam, $A_{ST}$ is the top reinforcement in the beam, $A_{SS}$ is the effective reinforcement in the slab and $f_s$ and $f_y$ are the stresses in the reinforcement.

The beam shear $V_T$ can be estimated as a function of the steel in tension as

$$V_T = (A_{ST} + A_{SS}) \lambda_o f_y \frac{jd}{l_c} \quad (7.37)$$

where $jd$ is the internal lever arm and $l_c$ is the distance between the point of contraflexure and the critical region.

Eq. 7.37 can be further simplified assuming a value for $jd/l_c = 0.4$, which is a typical value in beams of perimeter frames and of the negative moment in gravity dominated moment resisting frames. Thus
Now combining Eqs. 7.38 and 7.36 (b) gives

\[ A_{SB}f_s = 0.8(A_{ST} + A_{SS})\lambda_o f_y \quad \text{and} \quad f_s \leq \lambda_o f_y \]  

(7.39 a)

a similar analysis on the left beam of Fig. 7.12 yields

\[ A_{ST}f_s = 0.8A_{SB}\lambda_o f_y \quad \text{and} \quad f_s \leq \lambda_o f_y \]  

(7.39 b)

Note that in Eq. 7.39 (b) the slab reinforcement has not been taken into account since at the column face this reinforcement will be anchored and cannot participate in transferring the compression forces, as it is evident in the analysis of the free body to the left of Section A-A in Fig. 7.14.

The critical ratios \( A_{SB}/(A_{ST} + A_s) \) and \( A_{ST}/A_{SB} \) in Eqs. 7.39 (a) and (b) occur when \( f_s = \lambda_o f_y \), that is,

\[ \frac{A_{SB}}{A_{ST} + A_{SS}} = 0.8 \]  

and

\[ \frac{A_{ST}}{A_{SB}} = 0.8 \]  

(7.40)

![Diagram](Fig. 7.14 - Transfer of Forces to a Beam-Column Joint in the Horizontal Plane Including the Slab Reinforcement.)
These two expressions explain why in symmetrically reinforced beams, without slab, where \( A_{SB} = A_{ST} \) the compression steel consistently shows a lower stress level than the reinforcement in tension. This trend was observed in the tests of Units 5 and 6 and discussed in Chapter 6. The level of stress predicted by Eqs. 7.39 (a) and 7.39 (b) agrees with the observed behaviour shown in Figs. 6.12 and 6.30. The stress level of the hooked beam bars anchored in the beam-column joint of Unit 5 and shown in Fig. 6.12 (b) also follows the same trend. Hence, the lower stress level cannot be attributed to bond deterioration as suggested by Cheung (1991) and Paulay and Priestley (1992).

### 7.3.3 Distribution of Internal Forces in the Joint Panel

The bending moment and shear force diagrams of the column shown in Fig. 7.12 indicate that the shear forces in the joint panel are significantly larger than elsewhere in the column. The horizontal shear force \( V_{jh} \) can be estimated from equilibrium of horizontal forces to be

\[
V_{jh} = T_T + T_S + C_{ST} + C_{CT} + D_B \cos \theta_B - (V_{col} + F_{EQ}) \tag{7.41 a}
\]

or similarly

\[
V_{jh} = T_B + C_{SB} + C_{CB} + D_T \cos \theta_T - (V_{col} + F_{EQ}) \tag{7.41 b}
\]

or simply as

\[
V_{jh} = T_T + T_S + T_B - (V_{col} + F_{EQ}) \tag{7.41 c}
\]

where \( F_{EQ} \) is the inertial force induced by the earthquake at the level of the slab that is carried by the column.

The consideration of equilibrium of vertical forces in the joint panel will lead to a similar expression for estimating the vertical shear force, \( V_{jv} \). However, a relatively lengthy procedure is often required to find the position of the resultant of the compression force of a column with axial load and the forces on each layer of reinforcement. This calculation often involves a tedious calculation. A simple and quite accurate estimate of \( V_{jv} \) can be obtained as

\[
V_{jv} = \frac{h_e}{h_c} V_{jh} \tag{7.42}
\]

Now the shear stress in the joint, \( \sigma_j \), which is uniquely defined in both the horizontal and vertical directions, can be found from

\[
\sigma_j = \frac{V_{jv}}{b_j h_c} = \frac{V_{jh}}{b_j h_b} \tag{7.43}
\]
where $b_j$ is the effective joint width defined as shown in Fig. 7.15 and $h_c$ and $h_b$ are the column and beam depths, respectively.

Uncracked beam-column joints of normal dimensions, where bond of the beam and column bars has not deteriorated due to yield penetration or to excessive bond stresses, can be represented by an elastic body. The first diagonal crack will appear at a joint shear stress $\tau_j$ once the diagonal tensile strength of the concrete, $f'_{\tau}$, is exceeded. This value can be found from the Mohr circle of stresses as

$$\tau_j = \sqrt{f'_{\tau} \left( \frac{f'_{\tau} + \frac{P_a}{A_g}}{A_g} \right)} \quad (7.44)$$

where $P_a$ is the axial load acting over the horizontal column gross area $A_g$. $P_a$ is taken positive in compression.

Table 7.1 illustrates the values of the joint shear stress at which first visible cracking occurred in different tests reported in New Zealand. The mean $f'_{\tau}$ obtained from the tests was $0.37\sqrt{f'_{\tau}}$. Priestley and Calvi (1991) and Priestley (1992) have suggested that diagonal cracking in beam-column joints can be assumed to occur once the diagonal tension stress exceeds $0.29\sqrt{f'_{\tau}}$ and $0.30\sqrt{f'_{\tau}}$, respectively.

Before cracking in the joint, the reinforcement in the beam-column joint panel remains nearly unstressed. However, after cracking there is significant redistribution of internal forces and the reinforcement is activated by the mobilization of a diagonal compression field mechanism of shear transfer. A review of the allocation of forces by different researchers in New Zealand in the interior beam-column joints after cracking is presented below.
Table 7.1
Principal Tensile Stress at First Visible Cracking in Beam-Column Joints

<table>
<thead>
<tr>
<th>Test</th>
<th>Interior</th>
<th>Unit 1</th>
<th>Unit 2</th>
<th>Unit 3</th>
<th>Unit 4</th>
<th>Unit 5</th>
<th>Unit 6</th>
<th>Unit 5</th>
<th>Unit 6</th>
<th>Mean</th>
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<td>(MPa)</td>
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The well known concrete strut and truss mechanism proposed by Park and Paulay (1975), based on an admissible state of equilibrium, depends primarily on the bond distribution along the longitudinal reinforcement of the members that may be designed to form plastic hinges and dissipate energy during an earthquake. This shear transfer mechanism is illustrated in Fig. 7.16. Some part of the joint shear forces can be directly transferred by a diagonal concrete strut without the need of any reinforcement. An additive truss mechanism, acting with a compression field parallel to the diagonal concrete strut, transfers the remainder of the shear forces originated by the bond of the outer beam and column reinforcement. Evidently this mechanism requires vertical and horizontal reinforcement.

According to this model the shear forces in a joint panel can be mathematically expressed as the combination of the two mechanisms

\[ V_{jh} = V_{ch} + V_{sh} \] (7.45 a)

and

\[ V_{js} = V_{cv} + V_{sv} \] (7.45 b)

in which \( V_{ch} \) and \( V_{cv} \) are the contribution of the diagonal concrete strut in the horizontal and vertical directions, respectively and \( V_{sh} \) and \( V_{sv} \) show the contribution of the truss mechanism.

A lower bound approach was initially presented by Blakeley (1977) for the design of "inelastic" interior beam-column joints where beam plastic hinges were expected to form at the column faces. In this approach the whole horizontal joint shear at the flexural overstrength was to be taken by the truss mechanism if the axial compressive load in the column was below \( 0.1 \lambda f'c \). Some of the joint shear was allowed to be transferred through the concrete strut mechanism for higher axial compressive loads. This approach was justified by Paulay et al (1978) because it was believed that after few reversed cycles of loading in the inelastic range bond forces would concentrate towards the centre of the column due to the effects of yield penetration. Fig. 7.17 (a) shows the bond distribution postulated...
Fig. 7.16 - Traditional Truss and Diagonal Strut Mechanism for the Transfer of the Joint Shear [Park and Paulay (1975)].

(a) Diagonal strut mechanism
(b) Truss mechanism

Fig. 7.17 - Distribution of Bond Forces along the Beam Bars in Interior Beam-Column Joints after Several Inelastic Reversed Cycles.

(a) Paulay et al (1978)
(b) Cheung et al (1991)
(c) Paulay and Priestley (1992)
by Paulay et al. The requirements for estimating the vertical shear reinforcement were not as severe owing to the expected elastic behaviour of the columns. The proposal presented by Blakeley was incorporated into the New Zealand Concrete Design Code [NZS 3101 (1982)] with slight modifications. In this standard the diameter of the bars passing through the joints was also limited to prevent their premature global slip. It was recommended that the ratio between the column depth \( h_c \) and the diameter of the bars yielding at the column faces should not exceed

\[
\frac{h_c}{d_b} = \frac{f_y}{11} \tag{7.46}
\]

The use of the code recommendations often led to rather congested designs and the placing of the fresh concrete became a very important consideration. An alternative design, utilized to decongest the joint region, was to locate the plastic hinges in the beams in a region away from the column faces to avoid yielding of the beam longitudinal reinforcement penetrating into the joint core. This would eliminate bond deterioration taking place in this region. Such joints were termed "elastic" joints and a large participation of the concrete strut mechanism in carrying the horizontal joint shear was allowed, easing the amount of transverse reinforcement required. Research work conducted by Priestley (1975), Yeoh (1978), Blakeley et al (1979), Fenwick and Nguyen (1981), Beckingsale et al (1980) was used as the basis of the guidelines or as confirmation tests as it was the case of the experimental work carried out by Milburn and Park (1982).

Further experimental work conducted more recently by Park and Dai (1988) showed that the cyclic load performance of interior beam-column joints designed with a fraction of the horizontal and vertical shear reinforcement required by the New Zealand Concrete Design Code was not greatly affected and as a result they concluded that code requirements could be relaxed without reducing the level of seismic performance. They suggested that for axial column load levels below \( 0.1A_f f'_c \) the concrete strut mechanism could, in "inelastic" interior beam-column joints of structures designed for full ductility, take about 40% of the horizontal joint shear and 70% of the vertical joint shear. It was reasoned that the bond forces at the extremities of the beam bars in the beam-column joint were transferred across the joint by the strut mechanism. For "inelastic" interior beam-column joints of structures designed for limited ductility, they recommended allocating 60% of the horizontal joint shear to the concrete strut mechanism, and place at least one intermediate column bar at each side of the joint.

Park and Dai also observed that the code requirements concerning the limitation of the diameter of the inelastic longitudinal bars passing through the joint should recognize the case of beams with different amounts of top and bottom reinforcement and different concrete compressive strengths \( f'_c \). Based on their experimental work it was proposed that for axial load levels below \( 0.1A_f f'_c \):

\[
\frac{h_c}{d_b} \leq \frac{(1 + \beta) f_y}{5 \sqrt{f'_c}} \tag{7.47}
\]
where $\beta$ is the ratio between the amounts of bottom and top reinforcement and $\beta \leq 1$. The above formulation was made on the assumption that an average bond stress $\mu = 6.88\text{MPa}$ over the column depth could be sustained in the joint region when $f_c'$ was 20MPa and that the bond strength was a function of $\sqrt{f_c}$. For structures designed for limited ductility they concluded that no limits were needed.

Cheung et al (1991a) proposed a relaxation to the current code requirements by assuming the trapezoidal bond distribution depicted in Fig. 7.17 (b). They allocated the bond forces within the neutral axis depth of the column to the strut mechanism. They also made an estimate of the possible forces carried by the compression longitudinal beam reinforcement at the column face. The forces carried by the truss mechanism were allocated based on the remaining bond forces and considering the variation of the neutral axis depth of the column with the axial load. The formulation for the vertical shear reinforcement was also reviewed and modified using equilibrium considerations. They also demonstrated that the enhancement of the flexural capacity of the beams due to the reinforcement of the slab induces an extra joint shear force that is carried by the diagonal concrete strut without the need of additional shear reinforcement in the joint panel. Thus, the following expressions were recommended.

$$V_{ch} = 0.3 \left(1 + 3.5 \frac{P_u}{A_g f_c'} \right) V_{jv} \quad (7.48 \text{ a})$$

and

$$V_{rv} = 0.5 V_{jv} + P_u \quad (7.48 \text{ b})$$

In a study of the possible maximum bar diameters that could pass across an "inelastic" interior beam-column joint, Cheung et al furthered the work of Park and Dai and presented the following expression

$$\frac{h_b}{d_b} \geq \frac{1}{1.9 \xi_m \xi_p \xi_f \xi_t} \frac{f_y}{\sqrt{f_c'}} \quad (7.49)$$

in which $\xi_m$ is a factor that accounts for the compression forces in the beam bar, $\xi_p$ is a factor to account for the effect of the axial compression load, $\xi_f$ is a factor that accounts for the detrimental effects on the bond strength of joints with beams hinging at the four faces of the joint, and $\xi_t$ is a factor to account for the "top bar" effect where more than 300mm of fresh concrete is cast underneath the bars.

The following values were recommended for the different factors:

$$\xi_m = 1.3 \text{ for top bars,}$$

or

$$\xi_m = 1 + \beta \geq 1.1 \text{ for bottom bars,}$$

$$\xi_p = 1 \quad \text{if} \ P_u/A_g f_c' \leq 0.2,$$
Paulay and Priestley (1992) followed the same procedure presented by Cheung et al. They
used the idealized bond distribution shown in Fig. 7.17 (c) and made some simplifications to derive the
following expression for determining the amount of horizontal shear force carried by the truss
mechanism in "inelastic" interior beam column joints:

$$V_{sh} = \left( 1.15 - 1.3 \frac{P_u}{A_s f'_c} \right) T$$  \hspace{1cm} (7.50)

where $T$ is the tensile force in the top beam reinforcement at overstrength.

For the vertical joint shear reinforcement they adopted the expression proposed by Cheung
et al (see Eq. 7.48(b)).

Paulay and Priestley also presented the following expression for the contribution of the
diagonal strut in carrying the joint horizontal shear in "elastic" joints:

$$V_{ch} = 0.5 \left( \beta + 1.6 \frac{P_u}{A_s f'_c} \right) V_{jh}$$  \hspace{1cm} (7.51)

in which $\beta$ is the ratio between the amounts of bottom and top reinforcement.

They recommended Eq. 7.48(b) for finding the vertical joint shear reinforcement in "elastic"
joints.

Concerning the maximum bar diameter passing through an "inelastic" interior beam-column
joint they derived the following expression

$$\frac{h_s}{d_b} \geq \frac{\xi_p \lambda_a f_y}{5.4 \xi_p \xi_t} \sqrt{f'_c}$$  \hspace{1cm} (7.52)

where
\[ \xi_m = 2.55 - \beta \leq 1.8, \]
\[ \xi_p = P_d/(2 A_s f'_s) \quad \text{and} \quad 1 \leq \xi_p \leq 1.25, \]
\[ \xi_t = 0.8 \quad \text{for top bars with more than 300mm of fresh concrete cast underneath,} \]
\[ \text{or} \quad \xi_t = 1.0 \quad \text{for bottom bars,} \]
\[ \xi_t = 0.9 \quad \text{if plastic hinges form in all four faces of the interior beam-column joint,} \]
\[ \text{or} \quad \xi_t = 1.0 \quad \text{in all other cases.} \]

It was also recommended that Eq. 7.52 could be used in the design of elastic joints considering a reduction of the factor \( \xi_m \) as follows:
\[ \xi_m = \frac{f_s + f'_s}{\lambda_s f_y} \leq 1.2 \] (7.53)

where \( f_s \) and \( f'_s \) are the estimated tensile and compressive stresses in the beam bars at the faces of the column.

An additional formulation to determine the anchorage of beam bars passing through interior beam-column joints with plastic hinges forming in the beams at the face of the columns has been presented by Xin (1992). He based his proposal on the initial formulation postulated by Park and Dai for which he used the results of six interior beam-column joint units. The main variables in this research project were the concrete strength, the ratio top to bottom beam reinforcement and the ratio \( h_d/d_b \). Grade 430 steel was used as main beam and column reinforcement. The columns were not subjected to axial load.
\[ \frac{d_b}{h_c} = \frac{3.84 \xi_t k_3 \sqrt{f'_c}}{(1 + 0.56 \xi_m \beta) f_y} \] (7.54)

in which \( \beta \) is the ratio between the amounts of bottom and top reinforcement,
\[ \xi_t = 1.0 \quad \text{for top bars,} \]
\[ \text{or} \quad \xi_t = 1.1 \quad \text{for bottom bars,} \]
\[ \xi_m = 1.3 \quad \text{for top bars with} \quad \beta \leq 0.75, \]
\[ \text{or} \quad \xi_m = 1.0 \quad \text{for top bars with} \quad 0.75 < \beta \leq 1.0, \]
\[ \xi_m = 1.2 \quad \text{for bottom bars with} \quad \beta \leq 0.8, \]
\[ \text{or} \quad \xi_m = 1.0 \quad \text{for bottom bars with} \quad 0.8 < \beta \leq 1, \]
\[ \text{and} \quad k_3 = 1.0 \quad \text{for top bars,} \]
\[ \text{or} \quad k_3 = \sqrt{\beta} \quad \text{for bottom bars.} \]
7.3.4 Alternative Formulation for the Design of Interior Beam-Column Joints

7.3.4.1 Background

A brief review of the evolution of the methods for the design of interior beam-column joints was made in previous section. This section concerns with an alternative formulation of the bond and shear transfer mechanisms.

The design criteria stated by Paulay et al (1978) for the design of beam-column joints is adopted in the formulation:

(a) The strength of a joint should not be less than the maximum strength of the weakest member it connects, to eliminate the need for repair in a relatively inaccessible region and to prevent the need for energy dissipation by mechanisms that undergo strength and stiffness degradation when subjected to cyclic loading in the inelastic range.

(b) The capacity of a column should not be jeopardised by possible strength degradation within the joint.

(c) During moderate seismic disturbances a joint should preferably respond within the elastic range.

7.3.4.2 Mechanisms of Bond Strength

In Chapter 6 it was discussed that perhaps two different mechanisms of bond strength act along the longitudinal bars passing through a beam-column joint. Rather large bond stresses, concentrated towards the corner of the joint panel where the compression forces from the column and the beam meet. By observing Figs. 6.7 and 6.8 as well as Figs. 6.26 to 6.29 it is evident that any additional force in the beam bars entering the joint region after yielding in tension has taken place in the beam longitudinal reinforcement at the column faces is resisted by bond within the corner in compression. This also associated with an initiation of the global bar slip, which can be related to the local bar slip at the centreline of the column.

It is also obvious that the conditions for the development of bond forces are much better towards the corner where the compression forces from the column act than on the other corner at the same level of the beam bars. Two main reasons explain this phenomena. First, the effect of bending in the column inducing tension in one side and compression on the other. In the tension side (see right corner of the section shown in Fig. 7.18) there are splitting cracks along the bar reinforcing. It is quite likely that these cracks propagate from the column interface crack at the beam faces and will not induce a splitting failure due to the presence of the column reinforcement. On the other hand, the splitting cracks will close in the presence of the compressive stresses arising within the column. The second effect is that the intensity of the shear forces entering the joint is likely to be much larger towards the compression region than towards the region in tension. Sections A-A and B-B in Fig. 7.18 depict the probable distribution of the bond forces along the surface of the beam reinforcement.
The upper bond mechanism illustrated in Fig. 7.18 acts only within the joint core along the beam longitudinal reinforcement. Its gradient reflects the effects of bending of the column adjacent to the joint. It basically assumes an elasto-plastic cyclic bond-bar slip behaviour, which makes possible simplifications and design guidelines to be proposed, instead of more complex models. An analysis of tests conducted in New Zealand showed that a peak bond of \( 2.2 \sqrt{f'_c} \) can be sustained during several excursions in the beam bars in the plastic range without showing signs of significant global bar slip. Factor \( \xi_t \) recognizes that in certain circumstances the effects of sedimentation, bleeding and porosity in the local concrete under the bars can significantly make it very different from that tested to determine the concrete compressive strength. A value of \( \xi_t = 0.70 \) is recommended if 300mm or more of fresh concrete is cast underneath the bars in the same operation. In other cases a value of \( \xi_t = 1.00 \) is
recommended. Also, this mechanism is associated with the tensile strength of the concrete, as a function of $\sqrt{\frac{f_t}{f_c}}$, based on the results obtained by Eligehausen et al (1983).

Another mechanism will be mobilized when the input forces from the bars are such that the first mechanism cannot provide the full bond resistance. Global slip of the bars is likely to commence at this stage without necessarily implying an imminent bond failure. Bond failure in the beam longitudinal bars is defined here as when the local bar slip at the column centreline exceeds the clear distance between bar deformations. This mechanism acts only over the compression zone of the column and it appears to be caused by the crushed concrete around the bar deformations that dilates and permits a frictional mechanism to be developed. Therefore this mechanism is associated with the concrete compressive strength.

The maximum bar diameter that can pass through the beam-column joint can be determined combining these two mechanisms. For this it is necessary to know the frictional factor $\alpha$. This factor can be related, using the existing data, to the energy dissipated in a similar way as Eligehausen et al (1983) determined the reduced envelope of the cyclic bond-slip relationship for deformed bars. Alternatively, the cumulative displacement ductility factor attained until bond failure was observed can also be used. This second alternative is chosen in this study.

Fig. 7.19 plots the cumulative displacement ductility at which bond failure was reported versus the calculated frictional factor $\alpha$ for all tests that have displayed this type of failure at the University of Canterbury.

Although scattered, this data shows the expected trend for factor $\alpha$ decreasing with the cumulative displacement ductility factor. The "admissible" limit line depicted in Fig. 7.19 demarcates the region where, for a given cumulative displacement ductility factor, $\alpha$ can be attained without

![Fig. 7.19 - Tests with Bond Failure Reported at the University of Canterbury.](image-url)
implying a bond failure. This line has been arbitrarily traced and below it there are three tests that presented this type of failure. One of these tests refers to Unit B11 tested by Beckingsale et al. (1980) which had perforations in the concrete cover at 102mm intervals in both sides of the joint, required for instrumentation of the longitudinal bars. It is likely that these perforations could have disrupted the bond of the bars. The other two cases were for Units 1 and 4 tested by Xin (1992) that had no abnormal construction procedures.

The next step consists in quantifying an acceptable performance where bond failure is not necessarily precluded but delayed. It is assumed here that the acceptable performance is attained when bond failure is prevented from occurring before half of the expected available displacement ductility factor is reached. Using the definition of available ductility factor in terms of the cumulative displacement ductility factor given by Park (1989)

\[ \mu_s = \sum \mu_a \]

where for a full ductile member \( \mu_d/2 = 3 \) and for a member designed for limited ductility \( \mu_c/2 = 1.25 \), the corresponding cumulative displacement ductility factors are 24 and 10, respectively. According to the criteria of the "admissible" line in Fig. 7.19 the frictional factor \( \alpha \) for each case is 0.46 and 0.60.

### 7.3.4.3 Available Bond Strength in Interior Beam-Column Joints

An assessment of the maximum bond strength in interior beam-column joints is needed to determine the maximum diameter of bars allowed to pass through the joint.

The available bond strength, which is given as addition of the two mechanisms of bond resistance shown in Fig. 7.18 should in every case be larger than the demand of the input forces in the longitudinal bars at the faces of the joint. That is

\[ \left( \alpha f_y \sqrt{\frac{k_{hc}}{h_c}} + 2.2 \sqrt{\frac{f_{yc}}{2}} gh_c \right) \frac{\pi}{2} d_b \geq T + C \]  \hspace{1cm} (7.55)

where \( k_{hc} \) is the depth of the neutral axis of the column, \( gh_c \) is the distance between the column outer layer of bars, \( d_b \) is the diameter of the bar passing through the joint, and \( T \) and \( C \) are the input tension and compression forces in the bar at the faces of the joint.

Eq. 7.55 is a general formulation that can be used under different conditions. This equation will be first simplified to obtain an expression that permits the evaluation of the minimum ratio \( h_c/d_b \) in an interior beam column-joint with hinges at the faces of the joint and then when plastic hinges are, deliberately or not, located away from the faces of the columns.

A simple expression for the neutral axis depth of a column has been given by Paulay and Priestley (1992)
Combining Eq. 7.55 in Eq. 7.56 and assuming $g = 0.8$

$$
kh_e = \left(0.25 + 0.85 \frac{P_u}{A_g f'_c}\right) h_c \quad (7.56)
$$

Combining Eq. 7.55 in Eq. 7.56 and assuming $g = 0.8$

$$
\left[\alpha f'_c \left(0.25 + 0.85 \frac{P_u}{A_g f'_c}\right) + 1.76\sqrt{\frac{f'_c}{\sigma}}\right] \frac{\pi}{2} d_b h_e \geq T + C \quad (7.57)
$$

Analyzing the term at the right hand side of Eq. 7.57 when the beam top reinforcement reaches its maximum likely stress,

$$
T = T_T = A_{ST} \lambda_o f_y \quad (7.58\text{ a})
$$

and from Eq. 7.39 (b)

$$
C = C_{ST} = 0.8 A_{SB} \lambda_o f_y \quad \text{and} \quad C_{ST} \leq A_{ST} \lambda_o f_y \quad (7.58\text{ b})
$$

Combining Eqs. 7.58(a) and 7.58(b) gives

$$
T_T + C_{ST} = (A_{ST} + 0.8 A_{SB}) \lambda_o f_y \leq 2 A_{ST} \lambda_o f_y \quad (7.59)
$$

A similar expression can be obtained for the bar forces entering the joint through the bottom reinforcement

$$
T = T_B = A_{SB} \lambda_o f_y \quad (7.60\text{ a})
$$

and from Eq. 7.39 (a)

$$
C = C_{SB} = 0.8 (A_{ST} + A_{SS}) \lambda_o f_y \quad \text{and} \quad C_{SB} \leq A_{SB} f_y \quad (7.60\text{ b})
$$

Then combining Eqs. 7.60 (a) and 7.60 (b)

$$
T_B + C_{SB} = (A_{SB} + 0.8(A_{ST} + A_{SS})) \lambda_o f_y \leq 2 A_{SB} \lambda_o f_y \quad (7.61)
$$

Eqs. 7.59 and 7.61 can be reduced to one general equation
or in terms of the bar diameter and steel stress as

\[ T + C = (1 + 0.8\kappa) \frac{\pi}{4} d_b^2 \sigma_y \]  
and  
\[ (1 + 0.8\kappa) \leq 2 \]  

(7.62 b)

where \( \kappa \) is the ratio between the area of steel in compression and the steel in tension. In other words, for top bars, \( \kappa = A_S/A_{ST} = T_B/T_T \) and for bottom bars \( \kappa = (A_{ST} + A_S)/A_{SB} = (T_T + T_S)/T_B \).

Factor \( \kappa \) is very similar to factor \( \beta \) defined by Park and Dai (1988), Cheung et al (1991, 1991a), Paulay and Priestley (1992), and Xin (1992) (see Section 7.3.3). Nevertheless, the role of the slab bars carrying or not carrying compressive forces is not fully addressed by these researchers. It appears that for the evaluation of the bond condition of the top beam bars the factor \( \beta \), unlike factor \( \kappa \), may include both the top beam reinforcement and the effective reinforcement of the slab.

The ratio \( h_c/d_b \) can now be obtained by substituting Eq. 7.62 (b) in Eq. 7.57 and rearranging as

\[ \frac{h_c}{d_b} \geq \frac{\lambda_o f_y \xi_m \xi_f}{2 \left( \alpha f'_c \left( 0.25 + 0.85 \frac{P_u}{A_g f'_c} \right) + 1.76 \sqrt{\xi_t f'_c} \right)} \]  

(7.63)

where \( \xi_m = (1 + 0.8\kappa) \) and \( \xi_m \leq 2 \). The multiplier factor \( \xi_t \) has been added in Eq. 7.63 to recognize the detrimental effect of the simultaneous formation of plastic hinges in all four faces of the joint [Cheung (1991)] in which case \( \xi_t = 1.1 \) is recommended. \( \xi_t = 1 \) is to be used in all other cases.

Eq. 7.63 has been plotted against Eq. 7.52 in Fig. 7.20 for the top bars of a beam without slab but with equal top and bottom Grade 430 steel and for two cases of ductility: (a) full ductility with \( \lambda_o = 1.25 \) and \( \alpha = 0.46 \), and, (b) limited ductility with \( \lambda_o = 1.15 \) and \( \alpha = 0.60 \).

The term \( T + C \) in Eq. 7.57 can also be simplified if the positive or negative or both plastic hinge regions form in the beam at a distance of at least one effective beam depth from the column faces. This is the case of relocated plastic hinges or when the gravity actions are such that the positive plastic hinge does not form at the column face. In these cases limited yield of the reinforcement may be expected in the beam longitudinal reinforcement at the face of the column and

\[ T + C = 1.3T = 1.3 \frac{\pi}{4} d_b^2 f_y \]  

(7.64)
Fig. 7.20 - Ratio $h_c/d_b$ for Top Bars of Symmetrically Reinforced Beams.
Now the maximum bar diameter for this case is found substituting Eq. 7.64 in Eq. 7.57 and rearranging for $h_e/d_b$:

$$
\frac{h_e}{d_b} > \frac{0.65f'_c}{\alpha f'_c \left(0.25 + 0.85 \frac{P_u}{A_g f'_c} \right) + 1.76 \sqrt{\xi \xi_c}}
$$

(7.65)

which with $\xi_m = 1.3/\lambda_c$ can be expressed in the same form as Eq. 7.63.

Due to the lack of test data using high axial compressive load in the columns and high strength concrete, it is recommended that in Eqs. 7.63 and 7.65 $Pu/A_g f'_c$ should not be taken larger than 0.45 and $f'_c$ should be limited to 55MPa or less.

### 7.3.4.4 The Diagonal Strut and Variable Angle Truss Mechanisms of Joint Shear Transfer

The bond mechanisms of bars passing through an interior beam-column joint were already discussed in Section 7.3.4.2, where two different mechanisms were postulated to act. If it is accepted that the horizontal and vertical joint shear forces are carried by a diagonal strut and truss models, the horizontal shear force, $V_{sh}$, transferred through the truss mechanism by the beam reinforcement can be readily estimated from the first of the bond mechanisms depicted in Fig. 7.18 as

$$
V_{sh} = \frac{2.2}{2} \sqrt{\xi \xi_c} \frac{[(1+g)/2 - k) h_e^2]}{g h_c} \pi \Sigma d_b
$$

(7.66 a)

or

$$
V_{sh} = (T + C) \frac{[(1+g)/2 - k) h_e^2]}{(g h_c)^2}
$$

(7.66 b)

whichever is smaller.

Eq. 7.66 reflects the case when the bond strength is provided by both mechanisms shown in Fig. 7.18 whereas Eq. 7.66(b) applies when the input bar forces are small enough that the frictional bond mechanism needs not to be mobilized.

The above equations can be simplified assuming $g = 0.8$ and substituting Eq. 7.56 into them. Thus $V_{sh}$ is the smaller of

$$
V_{sh} = \frac{\sqrt{\xi \xi_c}}{550} \xi_p h_c \Sigma d_b
$$

(7.67 a)
or

\[ V_{sh} = \xi_p \frac{(T + C)}{1.5} \quad (7.67\ b) \]

where

\[ \xi_p = \left( 1 - 1.3 \frac{P_u}{A_b f'_c} \right)^2 \quad (7.67\ c) \]

In Eq. 7.67 (a) \( V_{sh} \) is given in kN, \( f'_c \) in MPa and \( h_s \) and \( d_b \) in mm.

Eq. 7.67 (b) can be simplified by relating the force \( C \) as a function of \( T \) following the same procedure used to find the limiting bar sizes allowed to pass through an interior beam-column joint. First, for plastic hinges forming in the beams at the column faces Eq. 7.62 (a) is substituted into Eq. 7.67 (b). Second, Eq. 7.64 is substituted in Eq. 7.67 (b) for the case when the plastic hinges form in the beams at a distance of at least \( d \) away from the column faces

\[ V_{sh} = \xi_m \frac{A_a \lambda_s f_y}{1.5} \quad (7.68) \]

in which \( T = A_a \lambda_s f_y, A_a \) is the largest area between the top and bottom beam reinforcement passing through the joint, \( \xi_m = (1 + 0.8 \kappa) \) for the case of plastic hinges forming at the column faces, and, \( \xi_m = 1.3/\xi_s \) for the case when plastic hinges forming at a distance of at least \( d \) from the column face.

When Eqs. 7.67 (a) and 7.68 are analyzed, it becomes obvious that the shear force transferred by bond is often different for the top and bottom reinforcement.

The use of Eqs. 7.67 (a) and 7.68 will lead to rather small values of \( V_{sh} \) when compared with those currently recommended by the New Zealand Concrete Design Code [NZS 3101 (1982)] and it may, in some cases, be also smaller than those estimated from Eqs. 7.48(a), 7.50 and 7.51 proposed by other researchers.

7.3.4.5 The Joint Horizontal Shear Reinforcement

It appears to be contradictory that most tests carried out in New Zealand in interior beam-column joint subassemblies show that the total horizontal force carried by the hoops is significantly larger than the value of \( V_{sh} \) either calculated from test results or from the proposal just discussed. These forces should be of the same order according to the traditional truss mechanism. Take for instance the results from Unit 6. Fig. 6.25 shows that the strains in all hoops in run 13 at \( \mu_s \) = 6x1 are near or past yield and the corresponding force estimated at this stage is 958kN = 0.90 \( V_{sh} \). However, the estimated values of the horizontal shear transferred to the truss mechanism based on the calculated bond forces
in run 13, were 559 and 484kN for the top and bottom bars respectively (see Figs. 6.28 and 6.29). These differences are often too large and cannot be explained using the conventional truss mechanism.

The beam-column joint depicted in Fig. 7.21 (a) is used to explain the above differences noted. In this beam-column joint the area of top reinforcement is larger than that of the bottom reinforcement. The compression field shown in Fig. 7.21 (b) results when the beam-column joint reinforcement has been designed for the “exact” horizontal shear transferred by bond from the top bars, $V_{bh,t}$, arising in the region of the column not subjected to compression. In this example, and following observations made by previous researchers, it can be assumed that after few reversal cycles all the hoops will reach their yield strength and therefore the equivalent transverse pressure from the hoops is constant. On the other hand the horizontal shear transferred by bond by the bottom beam bars, $V_{bh,b}$ is smaller than $V_{bh,t}$ and therefore it cannot balance the total horizontal hoop force as required in the right hand side of the joint of Fig. 7.21 (b). In this approach it is postulated that the remainder of the horizontal force is balance by the compression field marked (b) in Fig. 7.21 (b) that originates in the corner subjected to the column compressive forces. The rest of the horizontal shear is carried by a direct compression strut marked (c) in Fig. 7.21 (b) that runs diagonally through the corners of the joint panel.

The vertical transverse pressure required to equilibrate the compression field is non-linear due to the non-linearity of the bond forces from the beam reinforcement and to the non-uniformity of the compression field marked (a) in Fig. 7.21 (b).

Since there are no external vertical forces acting upon the whole depth of the column, this vertical pressure needs to be provided by vertical reinforcement. The resultant vertical forces from the top and bottom vertical pressure are in most circumstances different in magnitude and generally they are non-collinear. Hence, unlike the horizontal forces, the vertical ones are not self-balanced and require of bond forces on the diagonal compression fields marked (b) and (c) in Fig. 7.21, deviating them from a straight diagonal compression field.

A more general situation arises when the amount of horizontal transverse reinforcement provided exceeds the demand from the bond forces $V_{bh,t}$ and $V_{bh,b}$. This is the case of the interior beam-column joints designed according to the New Zealand Concrete Design Code [NZS 3101 (1982)]. The hoops will reach their yield strength or will be near it unless the amount of transverse reinforcement provided exceeds the horizontal joint shear. In the latter the joint hoops will reach a force smaller than that developed at their yield stress and of similar magnitude to the total horizontal shear force. One example that illustrates the above statement is the interior beam-column joint tested by Priestley (1975). The capacity of the hoops at their yield strength was $2.65V_{jh}$ and at $\mu_a = 6$ the horizontal force in the hoops, determined from averaged strain gauge measurements, was $0.87V_{jh}$. This behaviour is explained using the diagonal compression field illustrated in Fig. 7.21 (c). There, the diagonal compression field marked (b), originating in the corners where the compressive forces from the column act, will be required to balance the hoop pressure in conjunction with the horizontal shear forces $V_{bh,t}$ and $V_{bh,b}$. One consequence of placing more horizontal transverse reinforcement in the joint panel is that the angle of
Fig. 7.21 - Forces Acting at the Boundaries of an Interior Beam-Column Joint and Associated Compression Fields.
inclination to the horizontal of the diagonal compression field marked (a) will decrease and as a result the vertical forces necessary to balance this diagonal compression field will also diminish.

Hence the maximum horizontal reinforcement in the joint, \( V_{tr,h} \), required to sustain the compression field arising from bond in the bars in the region away from the column compression zone is

\[
V_{tr,h} = A_{sh} f_{yh} \geq V_{sh} \quad (7.69)
\]

where \( A_{sh} \) is the area of horizontal transverse reinforcement of yield strength \( f_{yh} \), \( V_{sh} \) is the largest of \( V_{sh,1} \) and \( V_{sh,2} \), which are calculated independently as the smaller of the values given by Eqs. 7.67 (a) and 7.68.

Only the horizontal transverse steel in the joint in the direction considered that extends beyond \( h/4 \) from the column centreline shall be deemed effective in resisting the horizontal shear. Many tests at the University of Canterbury have also shown that those hoops placed next to the beam longitudinal reinforcement are not effective in resisting the joint horizontal shear. Hence, it is also recommended that this reinforcement be placed within \( h/4 \) from each side of the joint mid-depth. Fig. 7.22 depicts the latter definition.

### 7.3.4.6 The Joint Vertical Shear Reinforcement

The reinforcement required to balance the vertical component of the diagonal compression field, and the flexural demand due to bending in the column when the joint reinforcement pass the joint and forms part of the column longitudinal reinforcement, can be expressed as

\[
V_{u,v} = V_{u,v}^* + T_{e2} \quad (7.70)
\]

where \( V_{u,v}^* \) in Eq. 7.55 can be evaluated by lumping the diagonal compression field marked (a) in Figs. 7.21 (a) and (b) in an equivalent strut acting at the centroids of the horizontal and vertical internal field of stresses. For simplicity it will be assumed that the vertical pressure required by the linear bond distribution is linear. With reference to Fig. 7.23.

\[
V_{u,v}^* = V_{sh} \frac{y_u}{x_u} \quad (7.71)
\]

where

\[
x_u = \frac{2}{3} \left( h_c - (1-g) \frac{h_c}{2} - kh_c \right) \quad (7.72)
\]
Fig. 7.22 - Definition of Joint Effective Reinforcement.

Fig. 7.23 - Vertical Joint Reinforcement Required to Sustain the Diagonal Compression Field Originated by Bond.
and

\[ y_\tau = \left( g - \omega \left( 1 - \frac{V_{ub}}{V_{ub,h}} \right) \right) h_b / 2 \]  \hspace{1cm} (7.73)

Now assuming \( g = 0.8, \omega = 2/3 \) and substituting Eq. 7.43 in Eq. 7.57

\[ x_\tau = \frac{2}{3} \left( 0.65 - 0.85 \frac{P_u}{A_g f_c'} \right) h_c \]  \hspace{1cm} (7.74)

and

\[ y_\tau = \left( 0.13 + \frac{2}{3} \frac{V_{ub}}{V_{ub,h}} \right) h_b / 2 \]  \hspace{1cm} (7.75)

Therefore

\[ V_{u,v} = \frac{3}{4} \left( 0.13 + 0.67 \frac{V_{ub}/V_{ub,h}}{V_{ub,h}} \right) h_b V_{ub,h} \left( 0.65 - 0.85 \frac{P_u}{A_g f_c'} \right) h_c \]  \hspace{1cm} (7.76)

The actual value of the term \( T_{el} \) in Eq. 7.70 is very difficulty to quantify owing to the influence of parameters such as the amount and arrangement of the longitudinal reinforcement and, most important of all, the deviation from the values determined from the code recommended static loading of the bending moment pattern and axial load on the columns due to the dynamic effects on the structure, which are caused by the higher modes of vibration and plastification of parts of the frame. Fig. 7.24, reproduced from Paulay (1988), shows one of such cases. It can be seen that the column end

![Diagram of column end](image)

Fig. 7.24 - Dynamic Effects on the Bending Moment of Columns [Paulay (1988)].
moments in the intermediate floors can eventually become much larger than the code calculated moments due to the variation of the point of contraflexure. The New Zealand Concrete Design Code recognizes these dynamic effects and recommends the use of a moment magnification factor. The main aim of the code provisions is to avoid the formation of plastic hinges in the columns located in intermediate floors of the structure and enforce, according to the capacity design philosophy, a weak-beam strong column collapse mechanism. Note, however, that the given provisions do not attempt to avoid yielding of the column longitudinal reinforcement under the worst situation [Paulay and Priestley (1992)].

It is obvious that the critical loading condition for the design of a beam-column joint is in the presence of lower axial compression loads, below the balance point. Under this load condition it could be expected that, due to the dynamic moment demand, the column outer bars will yield in tension and, in the absence of axial load, the intermediate column longitudinal bars will be stressed to about 40% of their yield strength due to flexure. Therefore the remaining 60% can be fully utilized to resist the vertical joint forces generated by the diagonal compression field marked (a) in Figs. 7.21 (b) and (c). It can also be demonstrated that the intermediate column longitudinal bars are not stressed as a result of flexure only at a column compressive axial load between 0.25A\(_g\)f\(_c\)' and 0.30A\(_g\)f\(_c\)'. Consequently, the entire intermediate column longitudinal reinforcement can be used to resist the vertical forces of the compression field marked (a) in Figs. 7.21 (a) and (b).

The above rationale can be taken into account by rewriting Eq. 7.70 as

\[ V_{n,v} = \xi_c V_{n,v} \]  

(7.77)

where \( \xi_c \) is a factor that recognizes the combined flexure and joint vertical forces acting simultaneously in the column intermediate bars. A simple expression for \( \xi_c \) is

\[ \xi_c = 2.3 \left( 0.65 - 0.85 \frac{P_u}{A_g f_c'} \right) \]  

(7.78)

Substituting Eqs. 7.76 and 7.78 in Eq. 7.77 and simplifying

\[ V_{n,v} = 1.4 \xi_v \frac{h_b}{h_c} V_{sh} \]  

(7.79)

in which \( \xi_v \) is a factor that considers the inclination of the compression field between beam and column bars

\[ \xi_v = \left( 0.17 + 0.83 \frac{V_{sh}}{V_{sh,b}} \right) \geq 0.58 \]  

(7.80)
The diagonal strut shown in Fig. 7.23 suggests that the required vertical reinforcement, \( V_{w,r} \), be located near the centreline of the column. Hence, it is recommended that this reinforcement be placed within \( h_c/4 \) from each side of the column centreline as depicted in Fig. 7.22 (b).

### 7.3.4.7 Strength of the Diagonal Compression Field

At the beginning of Section 7.3.4.1 it was stated that the strength of the joint cannot be less than the strength of the weakest member. It is very important to recognize that the carrying capacity of a column cannot be jeopardized by a joint failure. Besides, there are great difficulties in repairing a structure in which the damage has concentrated at the beam-column joints.

In an attempt to avoid degradation of the compression field of a beam-column joint, the New Zealand Concrete Design Code [NZS 3101 (1982)] recommends that the shear stresses \( \sigma_i \) calculated using Eq. 7.32 should not exceed \( 1.5\sqrt{f_c}' \), where \( f_c' \) is the concrete cylinder compressive strength. The above limit was justified by Paulay and Park (1984) based on an analysis carried out using the compression field theory. Then it was pointed out that no beam-column joints having a shear stress below \( 0.2f_c' \) had shown a diagonal compression failure. Paulay and Park also recognized the influence of yielding of the transverse reinforcement on decreasing the strength of the compression field as well as the possible detrimental effects of reversed cyclic loading in developing cracks in the joint in two directions.

The effects of yielding of the reinforcement on the strength of a diagonal compression field are now well known [Collins and Mitchell (1991)]. More recently, Stevens et al (1991) have studied the effects of reverse cyclic loading and yielding of the reinforcement in large reinforced concrete panels subjected to pure shear or pure shear combined with biaxial compression. They concluded that the strength of the compression field is further reduced as a result of cyclic reverse loading if the reinforcement is allowed to yield.

The current trends on relaxing the horizontal reinforcement in the joint panel will lead to yielding of the hoops at an earlier stage than for the joints designed with the current code provisions and consequently the strength of the compression field will be expected to decrease more rapidly.

For instance, a recent interior beam-column joint tested by Xin (1992), Unit 1, did not display a full ductile response due to a diagonal compression failure caused by excessive yielding of the joint hoops. The capacity of the joint hoops at their yield strength was 74% of the horizontal joint shear, \( V_{jh} \), calculated at an overstrength of 1.25. The joint shear stresses were expected to be of the order of \( 0.16\sqrt{f_c}' = 0.91\sqrt{f_c}' \), a value well below of the code limiting value. The joint horizontal reinforcement consisted on 5 sets of hoops but the top and bottom hoops were placed next to the beam bars, rendering them ineffective. This means that the effective joint horizontal reinforcement was of the order of \( 0.44V_{jh} \). Beckingsale (1980) tested a similar interior beam-column joint. The joint shear stress was \( 0.15\sqrt{f_c}' = 0.87\sqrt{f_c}' \). In both tests the beams were symmetrically reinforced and the ratio \( A_t f_c'/h_c E_d \sqrt{f_c}' \) was almost identical. The main difference was that in Beckingsale's test, the effective joint horizontal reinforcement, in which the first and last set of hoops in contact with the beam bars...
were disregarded, was able to take up to 0.94V_{\text{ph}}. Other differences in these tests were that a minimum axial compression load in the column of 0.044A_{c}f'_{c} was present in Beckingsale's test and the ratio h_{j}/h_{b} was slightly different. The reinforcement required to sustain the bond forces from the beam bars V_{sh} as given by Eq. 7.69 is 0.52V_{\text{ph}} for Xin's unit and 0.47V_{\text{ph}} for Beckingsale's unit. Beckingsale observed no detrimental effects in the concrete in the joint core.

In another series of tests Birss (1978) studied the behaviour of elastic interior beam-column joints. The main variables were the joint shear reinforcement and the column axial load. The first unit was tested under minimum axial compressive load of 0.05A_{c}f'_{c} and the joint horizontal steel detailed was capable of providing 0.45V_{\text{ph}}. According to Eq. 7.69 a minimum transverse reinforcement equal to 0.40V_{\text{ph}} will be required. In the second test the axial load was increased to 0.44A_{c}f'_{c}. A minimum joint horizontal reinforcement of 0.14V_{\text{ph}} was provided, while that required by Eq. 7.69 is 0.12V_{\text{ph}}. Both units failed in the initial cycles of loading in the inelastic range after excessive yielding of the joint hoops. The units attained their theoretical lateral load capacity in the first cycle in the inelastic range but they were unable to maintain it in further cycles. The maximum attained joint shear stress was of the order of $0.2f'_{c} = 1.12\sqrt{f_{c}}$. Both units displayed a diagonal compression failure in the joint region.

The above test results suggest that, in simple terms, the maximum joint shear stress, $v_{j}$, that can be sustained without significant degradation of the strength of the compression field may be considered as a function of the ratio between the amount of horizontal reinforcement provided, $V_{\text{tr,h}}$, and the joint horizontal shear, $V_{\text{jh}}$. The following expression is recommended for joints of moment resisting frames designed for full ductility

$$v_{j} \leq \frac{f_{c}}{\xi_{u}} \frac{V_{\text{tr,h}}}{V_{\text{jh}}} \quad (7.81)$$

where factor $\xi_{u}$ depends on the level of ductility and $v_{j}$ is found using Eq. 7.43. In no case shall $v_{j}$ exceed 0.20$f'_{c}$. It is recommended that $\xi_{u} = 3.5$ and $\xi_{u} = 2$ be used for interior beam-column joints of frames designed for full ductility and for frames designed for limited ductility, respectively.

Eq. 7.81 can be arranged for $V_{\text{tr,h}}$ as

$$V_{\text{tr,h}} \geq \frac{\xi_{u}}{f_{c}} v_{j} V_{\text{jh}} \quad (7.82)$$

Although the terms $V_{\text{jh}}$ and $v_{j}$ are related, Eqs. 7.81 and 7.82 are given in above form to emphasize on the effects of large shear stresses in interior beam-column joints. Therefore the horizontal joint reinforcement required needs to be the largest between that found from Eq. 7.69 and that obtained from Eq. 7.82.
The above formulation differs from the recommended limits given by Cheung et al (1991a) and Paulay and Priestley (1992) who suggested that the joint shear stress should be limited to values smaller than $0.25f'_c$ or 9MPa for joints of one way frames or $0.20f'_c$ or 7MPa for joints of two way frames.

7.3.4.8 **Interior Beam-Column Joints in Two-Way Frames**

So far, the design of interior beam-column joints has been confined to joints of one-way frames. The same procedure currently included in the New Zealand Concrete Design Code [NZS 3101 (1982)] will be adopted in this study. The provisions for two-way joints are based on judgement and of limited experimental evidence. Only two tests have been carried out on two-way interior beam-column joints in New Zealand. In one test Beckingsale et al (1980) applied an axial compressive load of $0.50A_gf'_c$ to the columns. In another test, Cheung et al (1991) tested a two-way subassemblage including the slab. No axial load was applied to the column.

The code provisions recognize the possibility of plastic hinges occurring simultaneously at each face of the joint and that the critical inclined plane of failure is oriented as a linear combination of the planes of failure in each direction. The design is simplified by enabling the joint reinforcement to be found from independent analysis in perpendicular directions where the beneficial effects of the axial compression load in the column in each of the analysis is apportioned in proportion to the joint horizontal shear force in each direction to avoid superposition. Following the same procedure, the term $P_u$ in Eq. 7.67 (c) can be replaced by $C_jP_u$ where

$$C_j = \frac{V_{jb}}{V_{jx} + V_{jz}}$$

(7.83)

in which $V_{sb}$ is the joint horizontal shear force in the direction being considered and $V_{jx}$ and $V_{jz}$ are the horizontal shear forces in each direction.

7.3.4.9 **Design Example of an Interior Beam-Column Joint**

Determine the horizontal and vertical joint reinforcement of a one-way interior beam-column joint of a moment resisting frame designed (a) for full ductility, and, (b) for limited ductility. The following data is given

- $h_u = 650\text{mm}$
- $P_u/A_gf'_c = 0.10$
- $h_c = 600\text{mm}$
- $b_c = 500\text{mm}$
- $f'_c = 54\text{MPa}$
- $f_y = f_{yh} = 430\text{MPa}$
The following forces have been estimated from an initial analysis, where no consideration has been given to the probable overstrength.

\[ T_T = 735 \text{kN} \quad T_s = 305 \text{kN} \quad T_n = 800 \text{kN} \quad V_{col} = 320 \text{kN} \]

a) **Design for Full Ductility**

**Step 1** - Check for diagonal compression failure

\[ \lambda = 1.25 \]

\[ V_{jh} = 1.25 \times (735 + 305 + 800 - 320) = 1900 \text{kN} \quad \text{[Eq. 7.41 (e)]} \]

\[ v_t = 1.900/(600 \times 500) = 6.3 \text{MPa} < 0.14 f'_c < 0.20 f'_c \quad \text{[Eq. 7.43]} \]

\[ \xi_v = 3.5 \]

\[ V_{er,h} \geq 3.5 \times 0.14 \times 1900 = 931 \text{kN} \quad \text{[Eq. 7.69]} \]

**Step 2** - Find maximum diameter of bars allowed to pass through the joint

Top bars:

\[ \kappa = 800/735 = 1.09 \]

\[ \xi_m = 1 + 0.8 \times 1.09 = 1.87 \]

\[ \xi_t = 0.7 \]

\[ \alpha = 0.46 \]

\[ d_s \leq 20 \text{mm} \quad \text{[Eq. 7.63]} \]

Use 4 HD20 + 2 HD16 \((A_s = 1,685 \text{mm}^2)\)

\[ \Sigma d_s = 112 \text{mm} \]

Bottom bars:

\[ \kappa = (735 + 305)/800 = 1.30 \]

\[ \xi_m = 1 + 0.8 \times 1.30 = 2.04 > 2, \text{ use } \xi_m = 2 \]

\[ \xi_t = 1 \]

\[ \alpha = 0.46 \]

\[ d_s \leq 21 \text{mm} \quad \text{[Eq. 7.63]} \]

Use 6 HD20 \((A_s = 1,884 \text{mm}^2)\)

\[ \Sigma d_s = 120 \text{mm} \]

**Step 3** - Find the horizontal transverse reinforcement, \(V_{ur,h}\)

Find bond forces required to be transferred by horizontal reinforcement

\[ \xi_p = (1 - 1.3 \times 0.1)^2 = 0.76 \quad \text{[Eq. 7.67 (c)]} \]

Top bars: \(V_{sh,h}\) the smaller of
\[
\frac{0.7 \times 45}{550} \times 0.76 \times 600 \times 112 = 521 \text{ kN} \quad \text{[Eq. 7.53(a)]}
\]

or \[
1.87 \times \frac{0.76}{1.5} \times 1658 \times 1.25 \times 430 = 844 \text{ kN} \quad \text{[Eq. 7.54]}
\]

\[V_{bh} = 521 \text{ kN}\]

Bottom bars: \(V_{bh}\) the smaller of

\[
\frac{1.0 \times 45}{550} \times 0.76 \times 600 \times 120 = 667 \text{ kN} \quad \text{[Eq. 7.67 (a)]}
\]

or \[
2 \times \frac{0.76}{1.5} \times 1,884 \times 1.25 \times 430 = 1,026 \text{ kN} \quad \text{[Eq. 7.68]}
\]

\[V_{bh} = 667 \text{ kN}\]

Hence \(V_{bh} = 667 \text{ kN}\). \(V_{u,h}\) is the largest of 667 kN and 931 kN. Then, horizontal hoops are required to avoid a diagonal compression failure.

Use 5 - 4HD12 hoops, \((A_s = 2,260 \text{mm}^2)\)

Thus, \(V_{u,h} = 972 \text{ kN}\)

**Step 4** - Find the vertical joint reinforcement, \(V_{tr,v}\)

\[
\xi_v = \left(0.17 + 0.83 \times \frac{667}{972}\right) = 0.74 > 0.58 \quad \text{[Eq. 7.80]}
\]

\[
V_{tr,v} = 1.4 \times 0.74 \times \frac{650}{600} \times 667 = 749 \text{ kN} \quad \text{[Eq. 7.79]}
\]

\[A_s = 749 / 430 \times 1,000 = 1,740 \text{ mm}^2\]

Use at least 4 HD24 bars in the central part of the column.

**b) Design for Limited Ductility**

**Step 1** - Check for diagonal compression failure

\[
\lambda_v = 1.15
\]

\[
V_{ph} = 1,748 \text{ kN} \quad \text{[Eq. 7.41 (c)]}
\]

\[
\nu_l = 5.8 \text{MPa} = 0.13f_s' < 0.20f_s' \quad \text{[Eq. 7.43]}
\]

\[
\xi_v = 2
\]
Step 2 - Find the maximum bar diameter allowed to pass through the joint

Top bars:
\[ \xi_m = 1.87 \]
\[ \xi_t = 0.7 \]
\[ \alpha = 0.6 \]
\[ d_b \leq 25\text{mm} \]

Use 4 HD24 \( (A_t = 1,808\text{mm}^2) \)
\[ \Sigma d_b = 96\text{mm} \]

Bottom bars:
\[ \xi_m = 2 \]
\[ \xi_t = 1 \]
\[ \alpha = 0.6 \]
\[ d_b \leq 25\text{mm} \]

Use 4 HD24 \( (A_t = 1,808\text{mm}^2) \)
\[ \Sigma d_b = 96\text{mm} \]

Step 3 - Find the horizontal transverse reinforcement, \( V_{tr,h} \)

\[ \xi_p = 0.76 \] \[ \text{[Eq. 7.67 (c)]} \]

Top Bars: \( V_{sh,t} \) the smaller of
\[ 447\text{kN} \] \[ \text{[Eq. 7.67 (a)]} \]
or \( 906\text{kN} \)
\[ \therefore V_{sh,t} = 447\text{kN} \]

Bottom bars: \( V_{sh,b} \) the smaller of
\[ 534\text{kN} \] \[ \text{[Eq. 7.67 (a)]} \]
or \( 906\text{kN} \)
\[ \therefore V_{sh,b} = 534\text{kN} \]

Hence, \( V_{sh} = 534\text{kN} \)
\( V_{tr,h} \) the largest of 534kN and 489kN.

Use 4 - 4HD10 hoops \( (A_t = 1,256\text{mm}^2) \)
Thus, \( V_{tr,h} = 540\text{kN} \)

Step 4 - Find the vertical joint reinforcement, \( V_{tr,v} \)

\[ \xi_v = 0.99 > 0.58 \] \[ \text{[Eq. 7.65]} \]
Use at least 4 HD24 bars in the central part of the column.

### 7.3.5 Additional Recommendations for the Design of Connections Between Precast Concrete Members Joining at the Beam-Column Joint Region

Stevenson and Beattie (1991) recommended the use of shear keys in the vertical construction joint of beams of System 1 after the performance of a subassemblage of this type showed a pinched hysteretic performance due to sliding of the beams at the column face. However, the test result obtained in Unit 1 indicates that no shear keys are required in the vertical construction joint in the beams at the column faces providing that the beams are properly seated on the concrete cover of the column below. This is because the main crack at the column face will not coincide with the construction joint, making the behaviour similar to that of a monolithic equivalent method of construction.

Based on the above two results it is recommended that the beams possess a series of shear keys along the vertical construction joints unless a minimum seating distance $S_d$ of the beams in the concrete cover of the column below can be guaranteed. A recommended value for $S_d$ is

$$S_d = \frac{V^o}{0.85 f'_c b_w} \geq 30\text{mm} \tag{7.84}$$

where $V^o$ is the beam shear associated with the flexural overstrength at negative moment, $f'_c$ is the concrete cylinder compressive strength of the column and $b_w$ is the width of the precast concrete beam seating in the column.

Concerning the anchorage of the hooked bottom bars protruding from the precast concrete beams, it is recommended that the existing code provisions for exterior beam-column joints be used. For obtaining the $V_{n,b}$ from Eq. 7.67 (a), all the bars in tension and compression being anchored in the joint should be considered in the evaluation of $\Sigma d_i$.

In relation with the precast concrete System 2 (see Section 1.4.1) no additional requirements are necessary. The existing recommendation [Charleson (1991)] of using a grout with a compressive strength of at least 10MPa larger than the concrete compressive strength of the precast member appears reasonable since, in terms of quality assurance, it is very difficult if not impossible to repair the connection between two precast concrete elements joined using a low strength grout.

### 7.4 EFFECTS OF THE BEAM ELONGATION

In a series of tests on spandrel beams of shear walls Paulay (1969) observed that, if the beams are unrestrained, longitudinal elongation at the mid-depth occurs due to the offset of the neutral axis depth. However, no large elongation values were recorded mainly because the rotation in the spandrel beams was kept to low values.
Beekhuis (1971), Binney (1972) and Cheung et al (1991) have reported significant elongations recorded during the testing of full scale beams or beam-column joint subassemblages at the University of Canterbury.

Researchers at the University of Auckland have also reported the beam lengthening during the cyclic reverse loading of various series of tests tested to failure [Fenwick and Fong (1979), Fenwick et al (1981), Megget and Fenwick (1988)]. Fenwick (1990) has classified the beam elongation according to two different sources, namely uni-directional hinging and reversed plastic hinging. In the first case, beam elongation occurs because the negative and positive plastic hinges in a beam are offset which is the case of beams of gravity dominated frames. In the second case both directions of plastic hinges form in the same region.

Fig. 7.25 shows the deformed shape of a component of a reinforced concrete frame that has been monotonically displaced well into the plastic range. In this figure it has been assumed that deformations occur only in the beam plastic hinge regions. Evidently an increase in the beam length will occur due to the offset between the beam mid-depth and the neutral axis depth. By assuming that in beams the neutral axis depth coincides with the centroid of the compression reinforcement, the following expression for the lengthening in the span of the bay of a frame, \( \delta_n \), can be obtained.
where $\theta_i$ is the angle of drift in each storey, $l'_b$ is the distance between the column centrelines and $l_{ph}$ is the distance between the positive and negative plastic hinges in the beam.

The beam lengthening can even be larger than that calculated using Eq. 7.85 when the member is subjected to post-elastic reversal cycles. This is because either the top or the bottom layer of beam reinforcement or both may not fully yield in compression, as it was demonstrated in Section 7.3.2, and hence the cracks in the compression side will not close. Fenwick (1990) has also stated that the wedging action of the aggregate particles that are dislocated between cracks may also contribute to the non-closure of the cracks. This observation was evident in the tests of Units 1, 2 and 3 where the diagonal strut carrying the shear to the column faces was crossed by several cracks that did not close. That is, the diagonal compression force was transferred by contact between the dislocated particles, which led to grinding of the concrete at advance stages during the test.

An upper bound to envelope the beam elongation is to assume that upon reversal the reinforcement that goes into compression remains with the same residual tensile strain. Thus, the maximum expected lengthening of the beam of a frame subjected to large inelastic reversals can be of the order of

$$\delta_{el} = 2\theta_i \frac{l'_b}{l_{ph}} (d - d')$$

Fig. 7.26 illustrates the beam elongations recorded in some different tests in New Zealand. The beam elongation is presented as percentage of the distance between steel centroids $d - d'$. Also shown are the predicted upper and lower bounds found using Eqs. 7.85 and 7.86. The initial recorded beam elongation at low drift levels is always over-predicted by these equations. This is because being still in the elastic range or just beyond it, the deformations causing the interstorey drift are distributed throughout the test subassemblage. However often Eqs. 7.85 and 7.86 bound quite well the beam elongation recorded because, as assumed, the inelastic deformations concentrated in the plastic hinges.

The recorded elongation of the beams shown in Figs. 7.26 (a) and (b) is very close to the upper bound given by Eq. 7.86. In these units the beam bars were anchored in the beam-column joint region so as to avoid a global bar slip due to bond deterioration. Because of the anchorage characteristics, it would be expected that the elongation of the plastic hinges in a beam at an exterior column display a similar trend. Unit 3, tested in this study, had equal amounts of top and bottom beam
(a) Unit 3
(b) Fenwick et al Unit 2B (1981)
(c) Unit 5
(d) Unit 6
Fig. 7.26 - Beam Elongation Recorded in New Zealand Tests.
reinforcement while the beam of Fig. 7.26 (b) had a ratio $\rho' / \rho = 0.8$. It is evident that the elongation is independent of the ratio $\rho' / \rho$.

Fig. 7.26 (c) and (d) depicts the recorded elongation in two interior beam-column joint units. At moderate and large drift levels, the total beam elongation is enveloped by the prediction given by Eqs. 7.85 and 7.86. The elongation is therefore not so large compared with the first two beams due to the effect of global bar slip. For instance Unit 5 showed an early bond failure and its effect is clearly marked in the smaller elongation shown in Fig. 7.26 (d), which is very well defined by the lower bound line.

As Figs. 7.26 (e) and (f) illustrate, the slab does not provide any restraint against the beam elongation. The recorded growth in length along the longitudinal mid-depth axis of the beams is comparatively similar to other tests that did not include a slab.

The only case where Eqs. 7.85 and 7.86 do overpredict the beam elongation is in the recorded growth of Unit 4r. This is because the beam rotations concentrated at the column faces and not at their relocated position as intended.

It is obvious that the effects of beam elongation may be of significant structural and non-structural significance, especially in perimeter frames where the depth of the beams and the number of bays are larger compared with other structural solutions. The elongation of the beams in the upper storeys of a moment resisting is likely to be of the same order of that measured in tests mentioned above because there are no apparent restraints. The magnitude of the elongation is of important concern in the design of the connections of the cladding and glazing systems, as well in proportioning the appropriate seating of the precast concrete floor systems. On the first level of the moment resisting frame or coupled structural walls the effects of beam elongation are more difficult to predict. This is because the columns or the walls may provide some restraining action because of their fixity at ground level. The restraining action will induce an axial compression force in the beams, which will enhance their flexural capacity and reduce the ratio between the beam and column flexural capacities. In the worst case the exterior columns or the interior columns located near the exterior column may eventually form plastic hinges at the beam face as depicted in Fig. 7.27. Yalcin et al (1990) concluded that the restraining action in the test unit caused the redistribution of the shear force in the columns and the decreased of the ratio beam-column capacities.

There are ways to maintain the beam elongation close to the lower bound given by Eq. 7.85. One of them is the use of diagonally reinforced beams. The compression reinforcement will be theoretically subjected to an equal force to the tension reinforcement and therefore the cracks will close or remain rather small. One example is given in Fig. 7.26 (h) of a diagonally reinforced coupling beam tested by Binney (1972). Another alternative is the use of beams with distributed reinforcement. In this type of beam the reinforcement in compression can be easily designed to yield even though if the ratio
of positive to negative beam moments is unequal. So far there has been very limited testing using this alternative.

The current computer software that is commonly in use for structural analysis of frames does not account for this second order effect. It appears to be appropriate to develop some computer programs that address this effect. An initial step could be the development of a program based on an incremental approach up to the formation of the complete collapse mechanism and where the enhancement of strength caused by the induced axial force in the beams and the probable column hinging at the beam face in the first level could be considered. A model to account for the beam elongation to be used in a time-history analysis is currently under development at Auckland University [Lowe and Fenwick (1992)].

7.5 CONCLUSIONS

The conclusions of this chapter concerning the theoretical background for the design of connections between precast concrete members of moment resisting frames designed for earthquake resistance are:
1. Analytical results and experimental evidence indicate that the tension shift of $d$ implicitly adopted in the New Zealand Concrete Design Code for the curtailment of the beams or walls longitudinal reinforcement is too conservative and could be relaxed. A value of $0.65d$ is suggested as a more realistic figure.

2. A series of recommendations for the design of midspan connections of precast concrete beams of perimeter frames has been presented. In the derivation of the equations it was assumed that the point of contraflexure coincides with the midspan of the beam. Truss models in combination with the shear friction concept were extensively used. Truss models may be used to derive equations for connections located in a region in the beam span other than at the point of contraflexure.

3. A different approach to the mechanisms of bond resistance of beam longitudinal bars passing through interior beam-column joints is postulated. The model proposed is based on the assumption that there are two bond mechanisms. Data gathered from the experimental programme as well as from test results presented by other researches in New Zealand was used to calibrate the model. Results obtained using the model indicate that, compared with other proposals, smaller values for the ratio $h_d / d_b$ can be used in the presence of axial load or higher concrete strength. The model has also been calibrated to obtain the maximum beam bar diameter permitted in interior beam-column joints of frames designed for limited ductility. The equations so derived are limited to values of $f'_c \leq 55$ MPa and $P_u \leq 0.5A_f f'_c$.

4. An alternative proposal for the design of interior beam-column joints is presented. A new model is suggested that is a modified version of the conventional concrete strut and parallel truss model. The alternative model recognizes that the angle of the compression field can vary according to the amount of horizontal transverse steel provided in the joint region. The alternative model also recognizes that the amount of horizontal transverse reinforcement in the joint panel may be controlled either by the shear forces from bond in the beam bars or by the need to delay early yielding of the hoops, which may cause a premature diagonal compression failure.

5. It is shown that the cumulative elongation of the beams during loading reversal can be large and that this effect can be of important one to both structural and non-structural elements. The effects of beam elongation are not considered in the current office design analytical tools that model members as one-dimensional elements. The beam elongation can be predicted with reasonable accuracy. Available test results show that the slab is unable to provide any significant restraint against beam elongation.

6. A first step towards the understanding of the effects of beam elongation could be the development or adaptation of a computer program that is able to analyze a frame using an incremental load procedure until a collapse mechanism forms. This analysis should consider
the beam growth and the enhancement of strength caused by the induced axial force in the beams and the probable column hinging at the beam face in the first level. This work could be utilized to establish whether the current recommendations of the capacity design procedure need to be modified to allow for this effect. It is expected that the worst conditions will occur in the first storey of columns of multi-bay perimeter frames.
CHAPTER 8

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

8.1 GENERAL

A study on the seismic behaviour of connections between precast concrete members of moment resisting frames has been conducted in this investigation. The study concentrated on the behaviour of connections at the midspan of beams and at beam-column joints of typical of perimeter frames since data gathered from different consulting engineering firms indicated that this structural solution is widely used in New Zealand in the design of buildings from 9 to 21 storeys high.

The experimental programme comprised the testing of six subassemblages and the repair and retesting of one subassemblage. All units were built with rather smooth construction joints to represent the worst condition found in practice. The dimensions of the test components were almost to full scale. A quasi-static cyclic loading regime with increasing displacements in the inelastic range up to failure was chosen to simulate a severe seismic event.

Further research was undertaken in this study to investigate the cyclic stress-strain behaviour of the two current grades of New Zealand manufactured reinforcing steel.

The main conclusions and recommendations are presented in the remainder of this chapter.

8.2 EXPERIMENTAL EVIDENCE ON THE SEISMIC BEHAVIOUR OF PRECAST CONCRETE MEMBERS OF PERIMETER FRAMES

8.2.1 Tests on Precast Concrete Beams Connected at the Beam Midspan

Four H-shaped precast concrete units were connected in a cast in place joint at the midspan of the beam where during lateral loading the point of contraflexure is expected to occur. They represent the type of connections normally utilized when using Systems 2 or 3 of precast concrete construction in New Zealand (see Section 1.4.1). The beam clear span to overall depth ratio of 3 was deliberately chosen as the smaller value that could normally be used in practice.

Three of the test units, Units 1 to 3, were conventionally reinforced and had different connection details. Another point of interest was the proximity of the connection to the critical region in the beam at the column faces. Unit 1 was connected using 180° hooks with transverse rods in contact with the concave side of the hooks. Unit 2 was spliced using U-shaped "drop in" bars and had transverse rods in contact with the concave side of the hooks. Unit 3 was connected at midspan by non-
contact straight lap splices. The amount of longitudinal reinforcement in the beams was such as to
induce a shear force at the flexural overstrength about equal to the maximum permitted by the New
Zealand Concrete Design Code [NZS 3101 (1982)] of $V^* = 0.3\sqrt{f} bd$ if diagonal reinforcement in the
beams is to be avoided.

The overall behaviour of the conventionally reinforced Units 1 to 3 during quasi-static cyclic
lateral loading was very satisfactory. All test units showed an available displacement ductility factor
of at least 6. The hysteresis loops were severely pinched, as expected due to the influence of the low
aspect ratio and moderate shear, but the flexural strength was maintained to levels of drift beyond 2%. During the load tests the midspan connections displayed an excellent performance, even though the splices were located commencing closer than 2d from the critical region from the faces of the columns. It is recommended that splices can commence at a distance of one effective depth from the critical region.

The beam of the other unit, Unit 4, had beams with strong end regions to relocate the plastic hinges away from the column faces. All inelastic deformations were expected to concentrate at the midspan of the beam where the diagonal reinforcement had been detailed. The connection detail at the beam midspan had the diagonal reinforcing bars mig-welded to mild steel plates, which were interconnected using a sandwich plate and high strength friction grip bolts. Three quarters of the beam diagonal bars were artificially strain aged to check this effect since the bend of the bars was located in the critical region of the beam.

The test of Unit 4 showed only a limited ductile response. This unexpected result was
cased by three-dimensional effects originated by the lay-out of the reinforcement in the strong end
regions. Splitting of the concrete between the diagonal reinforcement and the additional reinforcement
in the critical region at the bend of the diagonal reinforcement as well as the high bearing stresses in
the concrete in contact with the diagonal bars were the main causes for the limited ductile performance.
These effects were not considered in the initial design. The unit when repaired and retested unit showed
that a full ductile behaviour is attainable providing that the three-dimensional effects are considered and
a crushing failure in the concrete in contact with the bend of the diagonal bars is avoided. The
connection detail at the beam midspan showed a satisfactory performance in both tests. The strain
ageing of the diagonal bars did not result in any adverse effects. However, it cannot be concluded that
this phenomena might not be critical since strain age embrittlement is very susceptible to low
temperature values and the temperature during the tests was about 17°C.

8.2.2 Tests on Precast Concrete Subassemblages Connected at the Beam-Column Joint Region

Two cruciform precast concrete subassemblages, Units 5 and 6, connected at the
beam-column joint region were tested as part of the experimental programme.

Unit 5 was typical of a precast concrete method of construction used in New Zealand,
System 1 (see Section 1.4.1), where the lower part of the beams is precast and the bottom longitudinal
bars protrude from the beams and are anchored in the core of the beam-column joint. The main points
of interest in this type of construction were the influence of the cold joints and the effect of anchoring the bottom beam bars in the core of the beam-column joint, a practice which is not currently allowed by the New Zealand Concrete Design Code.

This unit displayed a satisfactory ductile behaviour. At the end of the test on Unit 5 bond failure of the top beam bars had occurred. These bars had commenced to slip through the beam-column joint at an early stage during the test. The bar diameter of these bars complied with the current New Zealand code recommendations and it is believed that the main cause for the slipping was the "top bar" effect along with the poor quality of the fresh concrete cast.

The construction joints did not adversely affect the behaviour of the unit. Sliding along the vertical construction joints was not observed. This was because the beams were seated on the column cover. A recommendation regarding the minimum width of seating has been given in this study, if shear keys are to be avoided in this region.

The anchorage of the hooked bottom bars, which protruded from the beams into the joint core, had no adverse effect on the performance of the beam-column joint. It is recommended that the New Zealand Concrete Design Code should allow this type of detailing. The existing provisions for anchoring beam longitudinal bars in exterior beam-column joints can be applied for anchoring hooked bars in interior beam-column joints.

Unit 6 was a typical sub-component of System 2 (see Section 1.4.1). The precast member included the beam and the beam-column joint. The longitudinal bars protruding from the lower column passed through preformed vertical holes in the precast concrete member. The joint between the lower part of the precast beam element and the column concrete, and the column bars passing through the preformed holes were grouted in one operation. The principal points of interest were the effects of the grout on the bond of the column bars and the effects of the construction joints.

This study also evaluated the properties of different commercial and self-mixed low viscosity grouts. All grouts showed significant segregation when fed by gravity through a hose. It is suggested that several other alternatives for grouting the preformed holes in this system be evaluated.

Unit 6 displayed excellent overall behaviour during testing. The cold joints and anchorage details had no adverse effects on the performance of the subassemblage. The existing recommendation [Charleson (1991)] of using a grout with a compressive strength of at least 10MPa larger than the concrete compressive strength of the precast concrete member being grouted was satisfactory adequate.

8.2.3 Differences between Monolithic and Precast Concrete Subassemblages

The measurements taken in the experimental programme showed that in that neither the construction joints, nor the connecting details used, had an important influence in the elastic or post-elastic response of the test units. Hence, it is recommended that the precast concrete systems tested can be analyzed as conventional monolithic systems.
8.2.4 Evaluation of the Initial Stiffness

In all tests the measured stiffness of the units was substantially smaller than that predicted using a conventional analysis that took into account only half of the gross section properties to allow for cracking and considered only shear and flexural deformations in the beams and the columns. The large differences observed were mainly due to the "fixed-end rotation" in the beams and the columns at the joint faces. The fixed-end rotation is caused by strain penetration of the longitudinal bars anchored in the joint. This source of flexibility accounted for 20% to 35% of the total elastic displacements in the test subassemblages. Another important source of flexibility was caused by the curvature in the members being affected by the tension shift in the reinforcement (curvature shift). Deformations caused by shear in the beam-column joint panel and in the beams and columns were smaller than initially estimated. In Unit 4 the main source of difference in stiffness was the curvature shift in the beam strong ends.

8.2.5 Beam Elongation

The elongation of the beams which occurred during testing concentrated in the plastic hinge regions. Initially and under monotonic loading some lengthening of the beams can be expected, and was predicted, because of the offset of the neutral axis depth and the mid-depth of the beam. During cyclic load conditions the beam elongation tends to be cumulative because, for conventionally reinforced beams, top and/or bottom reinforcement that yields in tension does not yield back in compression to zero strain when the loading is reversed.

The elongations of the beams tested was quite significant. A maximum elongation of 45mm was recorded in the test on Unit 6.

8.2.6 Influence of the Aspect Ratio of the Beams on the Seismic Performance

The deformations in the final stages of all tests, except for the test on Unit 5, were governed by shear displacements in the plastic hinge regions. Shear deformations led to pinching of the hysteresis loops and a reduction in the energy dissipated. From the test results it can be concluded that symmetrically reinforced beams with a clear span/overall smaller than 4.5 display a pinched hysteretic performance at large displacement ductility values even if the nominal shear stresses are as low as 0.2\(\sqrt{f_c}\). The main factor determining this behaviour is the aspect ratio of the member.

8.2.7 The Effects of Short Extension Lengths of the Hooks

An extension of only 8\(d_h\) was provided to the hooks of the main reinforcement in all units instead of an extension of 12\(d_h\) recommended by the Concrete Design Code. No adverse effects due to the contravention of this requirement were observed in the test and consequently it is suggested that the extension need not exceed 8\(d_h\).
8.3 ANALYTICAL CONSIDERATIONS FOR THE DESIGN OF CONNECTIONS BETWEEN PRECAST CONCRETE MEMBERS

An evaluation of the "tension shift" effect shows that the current New Zealand code provisions are unduly conservative and a relaxation for the curtailment rules for the longitudinal reinforcement in walls and beams is possible. For the current design criteria for transverse reinforcement in these elements, it can be demonstrated that a conservative design envelope is still attained when the "tension shift" is reduced from 1.0d to 0.65d.

Truss models and the shear friction concept were effectively used in the development of the design recommendations given in this study.

The recommendations presented for the design of connections at the midspan of beams are only applicable if the connection detail coincides with the point of contraflexure, as it often the case for precast concrete beams of perimeter frames.

The derivation for an alternative approach for the design of interior beam-column joints was also undertaken. This approach is based on a postulate of the bond mechanisms acting on the longitudinal bars passing through the joint region. It was found that the amount of horizontal reinforcement in joints with high shear stresses can be controlled by the need to suppress a premature diagonal compression failure due to excessive yielding of the reinforcement.

8.4 STRESS-STRAIN BEHAVIOUR OF NEW ZEALAND MANUFACTURED REINFORCING STEEL

An experimental programme aimed at studying the influence of different variables on the stress-strain behaviour of New Zealand manufactured reinforcing steel, namely Grade 300 steel and Grade 430 steel, was undertaken in this study. The research was jointly proposed, designed, and carried out by Dr. L. Dodd, a Ph.D graduate, and the author. This chapter is therefore included in Dr. Dodd's Ph.D. thesis (1992).

Approximately 120 test coupons were axially loaded under various conditions. The main variables studied were:

a) The cyclic stress-strain behaviour during various loading histories.

b) The strain rate effects.

c) The strain ageing effects.

d) The effects of bar deformations.
It was found that the stress-strain behaviour of reinforcing steel in the tension and compression regions is identical only if compared using in the true stress-natural strain coordinate system. This is an old finding that has been overlooked in most analytical models.

The shape of the Bauschinger curve is independent of the monotonic stress-strain curve and it is affected by the carbon content of the steel.

The strain rate has a significant effect on the stress-strain behaviour of the steel in the yield plateau region. In the strain hardening region this difference is not so significant. The effect on strain rate is greater for Grade 300 steel than for Grade 430 steel.

The strain ageing effects are not significant for New Zealand manufactured Grade 430 steel because of the presence of vanadium. For Grade 300 steel, the strain ageing effects can be significant in the strain range beyond the yield plateau region where a recovery in stiffness and increase in strength was observed. In the yield plateau region, Grade 300 steel displayed only a recovery in the initial stiffness. It is suggested that the implications of strain ageing should be considered in post-earthquake repair work.

Buckling of longitudinal reinforcing steel can lead to premature fracture due to extremely large local compression strains at the roots of the deformations that cause crack initiation. These cracks may propagate as the bar is straightened when axially tensioned. It is important to consider this phenomenon when using energy or damage models for reinforcing steel.

The cyclic stress-strain behaviour of reinforcing steel of the machined coupons tested under quasi-static conditions was used to calibrate an analytical model based on the recorded data in the true stress-natural strain coordinate system. A computer program, STEEL, was written in Fortran language and is included in Appendix A.

8.5 SUGGESTIONS FOR FUTURE RESEARCH

8.5.1 Tests on Connections between Precast Concrete Beams

It could be valuable to provide information on the seismic behaviour of connections between precast concrete beams joined in a cast in place joint in a region other than at the beam ends and at the point of contraflexure. This information is relevant for the use of precast concrete systems of gravity dominated frames where the connections are required to transfer forces originated by shear and bending.

8.5.2 Beam Elongation

The effects of beam elongation are not considered in the current office design analytical tools that model members as one-dimensional elements. A review of the existing literature has shown that
the phenomena of beam elongation needs to be considered in design. Also, the available results show that the slab is unable to provide any significant restraint against beam elongation.

A first step towards the understanding of the effects of beam elongation could be the development or adaptation of a computer program that is able to analyze a frame using an incremental load procedure until a collapse mechanism forms. This analysis should consider the beam growth and the enhancement of strength caused by the induced axial force in the beams and the probable column hinging at the beam face in the first level. This work could be utilized to establish whether the current recommendations of the capacity design procedure need to be modified to allow for this effect. It is expected that the worst conditions will occur in the first storey of columns of multi-bay perimeter frames.

Experimental work is also required to provide evidence on the possible effects of beam elongation. Tests on multi-bay frames are required to observe the behaviour of the first storey of columns. Besides, more information needs to be given to architects and designers on the effects that the beam elongation might have on the cladding and glazing systems, as well as on the seating and support detailing necessary for precast concrete slab floor systems.

The use of beams with vertically distributed longitudinal reinforcement is a possible solution to reduce the amount of elongation expected to occur in a conventionally reinforced frame. This type of construction offers several other advantages, such as the use of smaller diameter bars of Grade 430 steel passing through the beam-column joint region. It is likely that this system may perform much better in transferring high shear forces in the plastic hinges.

8.5.3 Bond Mechanisms Acting on the Beam Bars on the Beam-Column Joint

Equations which could be used to derive the maximum bar diameter permitted to pass through an interior beam-column joint were calibrated using the existing data. However, there is a lack of information regarding joints subjected to high axial compressive load and joints constructed using high strength concrete.

8.5.4 Evaluation of the Stiffness of Reinforced Concrete Frames

There is extensive literature showing that the current recommendations for calculating the initial "elastic" stiffness of frame components are not sufficiently accurate. The current methods lead to higher design lateral forces because the larger calculated stiffness. Also, the interstorey drifts may actually be much larger than currently estimated.

Analytical work could be conducted to provide simple design recommendations that allows for the effects of fixed-end rotation in beams and columns, for the curvature shift and for the shear deformations in beams, columns and beam-column joints.
8.5.5 Reinforcing Steel

The critical radius of buckling at which the "compression" cracks in deformed bars propagate and cause fracture may be defined in terms of the bar diameter and perhaps the stress-strain history. It is recommended that experimental and analytical work be conducted to endeavour to determine if such a critical buckling radius does exist, and if so, how it can be defined in simple terms.

The existing stress-strain model for reinforcing steel presented in this study can be complemented by adding a subroutine that accounts for the possible buckling in the reinforcing steel under the different boundary conditions provided by the transverse reinforcement in a column.

8.5.6 Experimental Techniques

Alternative experimental techniques to enable the accurate measurement of the inelastic strains in the bars passing through a beam-column joint need to be investigated. These measurements are often hampered by debonding of the strain gauges at moderate levels of strain (1 to 2%) or by local and global bar slipping that causes the break down of the wire connection. One proposed alternative is to split the bar to be instrumented and place spot high elongation strain gages along the diameter of one half bar. The other half bar could have a slot to channel the wires out and could be epoxied to the instrumented half bar.
REFERENCES


ACI Committee 318, (1989), "Building Requirements for Reinforced Concrete (ACI 318-89) and Commentary", American Concrete Institute, Detroit, 353 pp.


Bauschinger, J., (1886), "On the Change of Elastic Limit and Strength of Iron and Steel, by Drawing Out, by Heating and Cooling, and by Repetition of Loading", (Sumarised translation from


Earthquake Engineering Papers of George W. Housner", (1990), Civil Engineering Classics, ASCE, New York, pp. 186-198).


Mahin, S.A. and Bertero, V.V., (1972), "Rate of Loading Effects on Uncracked and Repaired Reinforced Concrete Members", Report No. EERC 72-9, Earthquake Engineering Research Center, University of California, Berkeley, 148 pp.


The Department of Research of the Beijing Institute of Architectural Design, (Undated), "Research on Monolithic-Precast Concrete Frames", Beijing, 16 pp.


Thompson, K.J. and Park, R., (1975), "Ductility of Concrete Frames under Cyclic Loading", Research Report No. 75-14, Department of Civil Engineering, University of Canterbury, Christchurch, 341 pp.


Thürlimann, B., (1979), "Reinforced Concrete Members in Torsion and Shear", International Association for Bridge and Structural Engineering (IABSE) Colloquium, Copenhagen, pp. 119-130.


APPENDIX A

Computer Program: STEELTEST

Code: VAX-11 FORTRAN

Computer: VAX 11/750
This program calls Subroutine STEEL to determine the cyclic stress history of reinforcing steel given the tensile skeleton curve properties and the strain history as described by Chapter 2 of this thesis.

The input and output file names are prompted for and an additional input file "Factor.Dat" is required. All input is in free-format. Factor.Dat requires the following variables:

- **Conv**: Conversion factor for input strain
- **OmegFact**: Factor for "Omega", the Bauschinger curve softness term

**Input File**

**Card 1** - Steel Tensile Skeleton Curve Data (in engineering coordinates)

- **Fy**: Yield Strength
- **Fsu**: Stress at peak load
- **ESH**: Strain at initiation of strain hardening
- **ESU**: Strain at peak load
- **Youngs**: Modulus of elasticity
- **ESHI**: Strain for a point on the strain hardening curve
- **FSHI**: Stress for the point corresponding to ESHI

**Card 2+** - Stress-Strain History (one card per point)

- **Es**: Engineering Strain Value
- **Fsm**: Measured Engineering Stress Value

**Output File**

**Card 1+**

- **Es*100.**: Strain in percent
- **Fsm**: Measured Stress
- **Fs**: Calculated Stress
- **I**: Point Number

Implicit None

Integer I  ! counter
Integer IOS  ! input/output status
Integer LMR  ! Last Major Reversal direction. Value of "s" after reversal
Integer BFlag(2)  ! strain hardening (0) or Bauschinger (1) curve
Real C1 ! Temporary constant
Real Conv ! strain factor (1 for strain 1.0E6 for microstrain)
Real Epa(2) ! Strain at end of linear branch (1=tension, 2=compression)
Real EpaM(2) ! Major reversal Epa
Real Epo(2) ! Maximum "natural" shift (1=compression, 2=tension)
Real EpoMax ! The maximum magnitude of Epo(1) and Epo(2)
Real Eps ! Natural strain
Real Epr(2) ! Reversal strain (1=tension, 2=compression)
Real EprM(2) ! Major reversal strain (1=tension, 2=compression)
Real EpSH ! Natural coordinate strain hardening strain
Real EpSHI ! Intermediate strain hardening curve natural strain
Real EpsLast ! Natural strain at last increment
Real EpsOld ! Natural strain at second to last increment
Real Epsu ! Natural coordinate "ultimate" strain
Real EpsuSh(2) ! Shifted "ultimate" strain value (1=tension, 2=compression)
Real Epy ! The yield strain, Fy/Youngs (a positive value)
Real Es ! Engineering Strain
Real ESH ! Engineering coordinate strain hardening strain
Real ESHI ! Intermediate strain hardening curve engineering strain
Real Esu ! Engineering coordinate "ultimate" strain
Real Fpa(2) ! Stress at end of linear branch (1=tension, 2=compression)
Real FpaM(2) ! Major reversal Fpa
Real Fpr(2) ! Reversal stress (1=tension, 2=compression)
Real FprM(2) ! Major reversal stress (1=tension, 2=compression)
Real Fps ! True coordinates stress
Real FpsH ! True stress at initiation of strain hardening curve
Real FpSHI ! Intermediate strain hardening curve true stress
Real FpsLast ! True stress at last increment
Real FSHI ! Intermediate strain hardening curve engineering stress
Real Fsm ! Engineering coordinate measured stress
Real Fs ! Engineering coordinate "ultimate" stress
Real Fsu ! Engineering Stress
Real FY ! Yield Stress
Real OmegFac ! Multiplication factor for Omega
Real Power(2) ! Exponent in normalised Bauschinger eq. (1=tens., 2=comp.)
Real PowerM(2) ! Major reversal Power
Real SHPower ! Exponent which governs the strain-hardening curve
Real Youngs ! Youngs modulus
Real YTan ! Tangential modulus
Real YpTan ! True coordinates tangential modulus
Real YpTanM(2) ! Tangential modulus at major reversals (1=tens, 2=comp)
Real YpTanLast ! Tangential modulus at last increment
Real YoungsUn ! Unloading modulus
Character*30 InFile,OutFil

C Read Input file for factored data
C
OPEN (UNIT=1,NAME='Factor.dat',STATUS='OLD')
Read (1,*) Conv,OmegFac
Write (6,800) Conv,OmegFac

C Required Input ENGINEERING Stress and Strain values
C
10 Write(6,810)
   Read (5,820) InFile
OPEN (UNIT=1,NAME=InFile,STATUS='OLD',Err=10)
Write(6,830)
Read (5,820) OutFil
OPEN (UNIT=7,NAME=OutFil,STATUS='NEW')
Read (1,*) Fy,Fsu,ESH,ESU,Youngs,ESHI,FSHI

C
C INITIALISE
C
Ep = Fy/Youngs
EpSH = log(1+ESH/Conv)
EpSU = log(1+ESU/Conv)
Fpsu = Fsu*(1+ESU/Conv)
EpSUSh(1) = EpSU
EpSUSh(2) = -EpSU
YoungsUn = Youngs
LMR = 0
BFlag(1) = 0
BFlag(2) = 0
Epa(1) = 0.
Epa(2) = 0.
EpaM(1) = 0.
EpaM(2) = 0.
Epo(1) = 0.
Epo(2) = 0.
EpoMax = 0.
Epr(1) = 0.
Epr(2) = 0.
EprM(1) = 0.
EprM(2) = 0.
I = 0

C
C Calculate the power term for the strain hardening branch
C
EpSHI = log(1+ESHI/Conv)
FpSH = Fy *(1+ESH/Conv)
FpSHI = FSHI*(1+ESHI/Conv)
Cl = FpSH-Fpsu+Fpsu*(EpSU-EpSH)
SHPower = log((FpSHI+Fpsu*(EpSU-EpSHI)-Fpsu)/Cl) +
          / log((EpSU-EpSHI)/(EpSU-EpSH))

C
Read (1,*,IOStat=IOS) Es,fsm

C
Do While (IOS.EQ.0)
I = I+1
Es = Es/Conv
Call Steel (Es,EpsLast,FpsLast,YpTanLast,EpsOld,Fy,Epy,  ! Input
           EpsE,EpSU,FpsSU,Youngs,SHPower,          ! Input
           Epr,Fpr,Epa,Fpa,Epo,EpoMax,EpSUSh,YoungsUn, ! Changeable
           Eps,Fps,Fs,YpTan,YTan,                   ! Output
           OmegFac)                                  ! Temporary Variable
C
Write (7,840)Eps*1.E6,Fsm*(1+Es),Fps,I
Write (7,840)Es*1.E2,Fsm,Fs,I
If (Eps.NE.EpsLast) then
  EpsOld = EpsLast
  EpsLast = Eps
FpsLast = Fps
YpTanLast = YpTan
End If
Read (1,* ,IOStat=IOS) Es, fsm
End Do
C
Stop
C
800 Format ( ' Data read from Input file "Factor.Dat" '/
* ' Strain Conversion Factor: ',El2.4/
* ' Omega Multiplication Factor: ',El2.4/)
810 Format ( ' What is the input file name?')
820 Format (A)
830 Format ( ' What is the output file name?')
840 Format (3E15.6,I6)
C
End
C
Subroutine Steel (Es,EpsLast,FpsLast,YpTanLast,EpsOld,Fy,Epy, ! Input
* EpsH,Epsu,Fpsy,Youngs,SHPower,
* Epr,Fpr,Eps,Fpa,Epo,EpoMax,EpsuSh,YoungsUn,
* Eps,Fps,Fs,YpTan,YTan,
* OmegFac) ! Temporary Variable
C
C==============================================================================
C Written by L.L. Dodd and J. Restrepo 1991
C
C This subroutine determines the cyclic stress history of reinforcing steel
C given the tensile skeleton curve properties and the strain history as
C described by Chapter 2 of this thesis.
C==============================================================================
C
C PASSED VARIABLES
C
Implicit None
Integer LMR ! Last Major Reversal direction. Value of "s" after reversal
Integer BFlag(2) ! Strain hardening (0) or Bauschinger (1) curve
C
Real Epa(2) ! Strain at end of linear branch (1=tension, 2=compression)
Real EpaM(2) ! Major reversal Epa
Real Epo(2) ! Maximum "natural" shift (1=compression, 2=tension)
Real EpoMax ! The maximum magnitude of Epo(1) and Epo(2)
Real Eps ! Natural strain
Real Epr(2) ! Reversal strain (1=tension, 2=compression)
Real EprM(2) ! Major reversal strain (1=tension, 2=compression)
Real EpSH ! Natural coordinate strain hardening strain
Real EpsLast ! Natural strain at last increment
Real EpsOld ! Natural strain at second to last increment
Real Epsu ! Natural coordinate "ultimate" strain
Real EpsuSh(2) ! Shifted "ultimate" strain value (1=tension, 2=compression)
Real Epy ! The yield strain, Fy/Youngs (a positive value)
Real Es ! Engineering Strain
Real Fpa(2) ! Stress at end of linear branch (1=tension, 2=compression)
Real FpaM(2) ! Major reversal Fpa
Real Fpr(2) ! Reversal stress (1=tension, 2=compression)
Real FprM(2) ! Major reversal stress (1=tension, 2=compression)
Real Fps ! True coordinates stress
Real FpsLast ! True stress at last increment
Real Fpsu ! True coordinate "ultimate" stress (slope at ultimate)
Real Fs ! Engineering Stress
Real Fy ! Yield Stress
Real OmegFac ! Multiplication factor for Omega
Real Power(2) ! Exponent in normalised Bauschinger eq. (1=tens., 2=comp.)
Real PowerM(2)! Major reversal Power
Real SHPower ! Exponent which governs the strain-hardening curve
Real Youngs ! Youngs modulus
Real YTan ! Tangential modulus
Real YpTan ! True coordinates tangential modulus
Real YpTanM(2)! Tangential modulus at major reversals (1=tens, 2=comp)
Real YpTanLast! Tangential modulus at last increment
Real YoungsUn ! Unloading modulus

C
C INTERNAL VARIABLES
C

Real a !
Real Cl ! Temporary constant
Real C2 ! Temporary constant
Real Delta ! Strain change from previous increment
Real Epp ! Abs((Epsush(K) - Epa(M))/Epsu)
Real FNorm ! Abs(Fpp/Fpt)
Real Fpp ! Fpsu*(s*l.-EpsuSh(K)+Epa(M)) - Fpa(M)
Real Fpt ! Fpsu*(2-EpsuSh(1)+EpsuSh(2))
Real FpSH ! Strain hardening natural strain (Fy*(1+Epsh))
Real Omega ! Percent area term for Bauschinger curve

Integer MaxFlag ! Flag to tell if reversal point is a new max O-no,1-yes
Integer S ! Straining direction: -1 for compressing, 1 for tensioning
Integer K ! Index value 2 1
Integer M ! Index value 1 2
Integer L ! Index value: K for LMR*s.NE.-1, M otherwise

Eps = log(1+Es)
Delta = Eps - EpsLast
If (Delta.Eq.0) Delta = EpsLast - EpsOld
If (Delta.Gt.0.0) then ! tensioning
    M = 2
    K = 1
    S = 1
Else ! compressing
    M = 1
    K = 2
    S = -1
End If
If (Eps*s.Gt.EpsuSH(K)*s) then
    Fps = 0.
    Fpsu = 0.
    Write (6,800)
End If

RETURN
If (((EpsLast-EpsOld)*Delta).Lt.O.) then ! Reversal

C
C STRAIN REVERSAL
C
C
If ((Epo(1).Eq.O.).And.(Epo(2).Eq.O.).And. (Abs(EpsLast).Lt.Epy)) then ! Elastic
Continue
Else If (((LMR*s.EQ.-1).And.((EpsLast-Epa(K))*s).GE.0.) then ! Linear Range
Continue
Else If (((LMR*s.EQ.1).And.(EpsLast-Epr(M))*s).GE.0.) then ! Linear Range
Continue
Else
MaxFlag = 0
If (s*Epo(K).GT.s*(EpsLast-FpsLastfYoungsUn)) then
! Max abs strain in direction
MaxFlag = 1
Epo(K) = EpsLast-FpsLast/YoungsUn
EpsuSh(K) = Epsu*f+*Epo(K)
If (Abs(Epo(K)).GT.EpoMax) then
EpoMax = Abs(Epo(K)) ! New Max Strain
YoungsUn = Youngs*(0.82 + 1./(5.55+1000.*EpoMax)) ! Unloading Mod.
End If
End If
End If
LMR = s
Epr(M) = EpsLast
Fpr(M) = FpsLast
Epa(M) = EpsLast + s*Fy/YoungsUn
Fpa(M) = FpsLast + s*Fy
If (((BFlag{K}.EQ.O.OR.BFlag(M).EQ.O).AND.MaxFlag.EQ.1).OR. + s*(FprM(K)-Fpr(M)).GT.2*Fy .OR. + ((Epr(M)-EpaM(K))/(EprM(M)-EpaM(K)).GT.1.0 .And. + BFlag(K).EQ.1)) then
C
C MAJOR REVERSAL
C
C Reversal from skeleton curve or more than 2Fy from previous major reversal
C Reverse to a MAJOR BAUSCHINGER CURVE
C
EprM(M) = Epr(M)
FprM(M) = Fpr(M)
EpaM(M) = Epa(M)
FpaM(M) = Fpa(M)
YpTanM(M) = YpTanLast
Epr(K) = Epr(M)
EprM(K) = Epr(M)
If ((Epo(2)-Epo(1)).LT.(EpSh-Epy)) then ! Between Yield Plateaus
Power(K) = 0.35
Else ! Bauschinger Curve
BFlag(K) = 1
Fpt = Fpsu*(2-EpsuSh(l)+EpsuSh(2))
Fpp = Fpsu*(s1-EpsuSh(K)+Epa(M)) - Fpa(M)  
Epp = Abs((EpsuSh(K) - Epa(M))/Epsu)  
FNorm = Abs(Fpp/Fpt)  
Omega = ((0.001+1.08E−3/(1.043−Epp))/0.18*(FNorm−0.69)  
+0.085) * OmegFac  
If (Omega.GT.0.086) Omega = 0.085  
Power(K) = 56.689*(Omega−0.077)**2−4.921*(Omega−0.077)+0.1  
End If  
PowerM(K) = Power(K)  
Else If ((Epr(M)-EprM(M))*s.LT.O) then  
EprM(M) = Epr(M)  
FprM(M) = Fpr(M)  
YpTanM(M) = YpTanLast  
Power(K) = 0.35  
Else  
Power(K) = 0.35  
End If  
End If  
End If  
End If  
Else  
C REVERSAL FROM A MINOR BAUSCHINGER CURVE  
End If  
End If  
End If  
End If  
Else  
Power(K) = 0.35  
End If  
End If  
End If  
Else  
Call Bausch(Eps,EpaM(M),FpaM(M),EpsuSh(K),Fpsu*s,YoungsUn,  
Fpsu,PowerM(K),  
Fps,YpTan)  
Else  
C GOVERNING STRESS-STRAIN CURVES  
End If  
End If  
End If  
End If  
End If
MINOR BAUSCHINGER CURVE moving toward previous minor or major peversal point

Call Bausch (Eps,Epa(M),Fpa(M),EprM(K),FprM(K),YoungsUn, ! Input
 + YpTanM(K),Power(K), ! Input
 + Fps,YpTan) ! Output

End If

Else

Fps = Fpr(L) + (Eps-Epr(L))*YoungsUn
YpTan = YoungsUn
End If

C Elastic Branch

Else If ((Epo(1).eq.0.).And.(Epo(2).eq.0.).And.
* (Abs(Eps).LE.Epy)) then

Fps = Eps*Youngs
YpTan = Youngs

C Skeleton Curve

Else If (s*(Eps-Epo(M)-s*FyjYoungsUn).GE.-1.E-5) Then

If (s*(Eps-Epo(K)-s*Epy).LE.(EpSH-Epy)) then

Fps = Fy*s*(1+Es) ! Yield Plateau
YpTan = Fy

Else

Fps = Fy*(1+Epsh) ! Strain Hardening
Cl = FpSH - Fpsu + Fpsu*(Epsu-Epsh)
C2 = (Epsu-s*(Eps-Epo(K)))/(Epsu-Epsh)
Yps = s*Cl*C2**SHPower - Fpsu*(s*Epsu-(Eps-Epo(K))) + s*Fpsu
YpTan = -SHPower*(Cl/(Epsu-Epsh))*C2**(SHPower-1) + FpSU

End If

C Reloading to Strain Hardening

Else If (((EprM(K)-Epo(K))*s).GE.Epsh) then

If (((LMR*s.Eq.-1).And.(Eps*s.Gt.Epr(K)*s)).Or.
* (((LMR*s.Eq.1).And.(Eps*s.Gt.Epa(M)*s))) then

Call Bausch (Eps,Epa(M),Fpa(M),EprM(K),FprM(K),YoungsUn, ! Input
 + YpTanM(K),Power(K), ! Input
 + Fps,YpTan) ! Output

Else

Fps = Fpr(L) + (Eps-Epr(L))*YoungsUn
YpTan = YoungsUn
End If

C Between Yield Plateaus

Else

If (((LMR*s.Eq.-1).And.(Eps*s.Gt.Epr(K)*s)).Or.
* (((LMR*s.Eq.1).And.(Eps*s.Gt.Epa(M)*s))) then

If (s*(Eps-Epr(K)).GT.0.) then

C MAJOR BAUSCHINGER CURVE moving toward yield plateau point

Call Bausch (Eps,Epa(M),Fpa(M),Epo(M)+s*Fy/YoungsUn,Fy*s,YoungsUn, ! Input
+  Fy,Power(K),                               ! Input
+   Fps,YpTan)                               ! Output

Else

   MINOR BAUSCHINGER CURVE moving toward previous minor or major reversal point

   Call Bausch (Eps,Epa(M),Fpa(M),EprM(K),FprM(K),YoungsUn,         ! Input
        +  YpTanM(K),Power(K),                                        ! Input
        +  Fps,YpTan)                                                ! Output

   End If

   Else

      Fps = Fpr(L) + (Eps-Epr(L))*YoungsUn
      YpTan = YoungsUn

   End If

   End If

   Fs = Fps/(1+Es)
   YTan = (YpTan-fps)*exp(-2*Eps)

Return

800 Format ('The peak strain has been exceeded, REBAR FRACTURE!'/)

End

C

Subroutine Bausch (Eps,E1,F1,E2,F2,Slopel,Slope2,Power,        ! Input
        +  Fps,YpTan)                                                ! Output

C

This subroutine calculates the stress for a given strain on the Bauschinger
C curve

C

Integer ITest ! Convergence flag
Integer I ! Counter

Real C1 ! (Fpu-Epu*Slopel)/(Fpu-Epu*F2)
Real C2 ! Eps*(Slopel-F2)/(Fpu-F2*Epu)
Real C3 ! 1 - Eppn
Real C4 ! 1 - C3*C3 = 1-(1-Eppn)^2
Real C5 ! C4**Power-C1*Eppn-C2 (function for which Eppn is a root)
Real E1 ! Initial strain on Bauschinger curve
Real Eppn ! Normalised Strain in the Bauschinger Branch
Real Eppn2 ! New estimate of Eppn
Real Eps ! Natural strain
Real E2 ! Final strain on Bauschinger curve
Real Epu ! Strain from linear branch to ultimate (E2 - E1)
Real F1 ! Initial stress on Bauschinger curve
Real Fps ! True coordinates stress
Real Fpu ! Stress from linear branch to ultimate (F2 - F1)
Real F2 ! Final stress on Bauschinger curve
Real Power ! Exponent in the Normalised Bauschinger equation
Real Slope1 ! Initial Slope
Real Slope2 ! Final Slope
Real YpTan ! Tangential modulus

fpu = F2 - F1
Epu = E2 - E1
C1 = (Fpu-Epu*Slopel)/(Fpu-Epu*Slope2)
C2 = (Eps-E1)*(Slopel-Slope2)/(Fpu-Slope2*Epu)
Eppn = (Eps-E1)/(E2-E1)
ITest = 1
I = 0
Do While ((ITest.Eq.1).And.(I.LT.5))
   I = I+1
   C3 = 1-Eppn
   C4 = 1-C3*C3
   C5 = C4**Power-C1*Eppn-C2
   Eppn2 = Eppn - C5/(2*Power*C4**(Power - 1)*C3-C1) ! Newton-Raphson
   If Eppn2.GT.0.02 then
      Eppn = Eppn2
   If (Abs(C5).LE.0.001) ITest = 0
   Else
      Call LinInterp(Eppn,C1,C2,Power)
      ITest = 0
   End If
   End Do
Fps = Eppn*(FPu-Epu*Slopel)+(Eps-El)*Slopel+F1
If (Eppn.LT.0.0001 .OR. (Slopel-Slope2)/slopel.LT.0.01) then
   YpTan = Slopel
Else
   YpTan = (EppnU-C3)*Slopel/(EppnU*C3-Cl)
   YpTan = YpTan*Slopel/(YpTan+Slopel) + Slope2
End If
Return
End

Subroutine LinInterp (Eppn,C1,C2,Power)
Calculate Eppn using an iterative linear interpolation

Integer ITest ! Convergence flag

Real C1 ! (Fpu-Epu*Slopel)/(Fpu-Epu*Slope2)
Real C2 ! Eps*(Slopel-Slope2)/(Fpu-Slope2*Epu)
Real C3 ! 1 - Eppn
Real C4 ! 1 - C3*C3 = 1-(1-Eppn)^2
Real C5 ! C4**Power-C1*Eppn-C2 (function for which Eppn is a root)
Real CSL ! Lower bound of C5
Real CSU ! Upper bound of C5
Real Eppn ! Normalised Strain in the Bauschinger Branch
Real EppnL ! Lower bound of Eppn
Real EppnU ! Upper bound of Eppn
Real Eps ! Natural strain
Real Power ! Exponent in the Normalised Bauschinger equation
EppnU = Eppn
C3 = 1-EppnU
C4 = 1-C3*C3
CSU = C4**Power-C1*EppnU-C2
EppnL = 0
CSL = -C2
ITest = 1
Do While (ITest.Eq.1)
   Do I=1,5
      Eppn = EppnL-CSL*(EppnU-EppnL)/(CSU-CSL)
      C3 = 1-Eppn
      C4 = 1-C3*C3
C5  = C4**Power-C1*Eppn-C2
C
  If (Abs(C5).LE.0.03) then
  ITest = 0
C
  Else
    If (C5.Gt.0) then
      EppnU = Eppn
      C5U  = C5
    Else
      EppnL = Eppn
      C5L  = C5
    End If
  End If
C
End Do
Return
End
APPENDIX B

Construction Details of Loading Frames
Universal Columns

Section A-A

Section B-B

End Brackets

Maximum Service Load: 450 kN
Beam End Supports - LF2

Section C-C

Section D-D

Section E-E

Bottom Bracket
**Column Base Support**

- 520 x 180 x 32 Mild steel plate
- To suit M16 bolts
- (Only one face as in 'Transverse Elevation')

- UC150-23

- 1350 x 600 x 32 Mild steel plate

**Sliding Block**

- Permaglize T20 teflon strip

**Transverse Elevation Column Base Supports**

- 50 x 50 x 10 stiffener
- 180 x 150 x 32 Mild steel plate
- 76 x 76 RHS

- 1350 x 600 x 32 Mild steel plate

**Section FF**
Base Plate - LF2
Top View

Column Base Plate - LF2

Note: Max. service load 450 kN per jack
All M16 bolts are high strength
Maintain dust free sliding mechanism
THE SEISMIC BEHAVIOUR OF CONNECTIONS BETWEEN PRECAST CONCRETE ELEMENTS

José I. RESTREPO-POSADA

ABSTRACT: Six full-scale precast concrete subassemblages connected in a cast in place joint were tested under quasi-static reverse cyclic loading conditions. The test specimens were typical of moment resisting perimeter frames and had connection details commonly used in New Zealand. A research on the stress-strain behaviour of the two grades of New Zealand manufactured reinforcing steel was also conducted.

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