5.3.3 Initial Stiffness

The theoretical initial stiffness of Specimen R2 was calculated by the same procedure as used for Specimens O1 and R1. The theoretical stiffness of Specimen R2 was

\[ K_{\text{theoretical}} = 43.8 \text{kN/mm} \]

The theoretical stiffness \( K_{\text{theoretical}} \) is shown in Fig. 5.13. The stiffnesses estimated at 75% of the theoretical ideal storey horizontal load strength of the specimen were 21.1kN/mm for the positive loading cycle and 19.6kN/mm for the negative loading cycle. The average value of the stiffnesses obtained for the positive and negative loading cycle, 20.3kN/mm was 27% larger than that obtained from Specimen R1. However, the measured initial stiffness of Specimen R2 retrofitted by jacketing the columns, beams and joint was far below the theoretical value as observed for Specimen R1. The main reason for this low stiffness is likely to be caused by micro cracking and reduction of the effectiveness of the existing concrete associated with chipping off the cover concrete in the beams of the as-built specimen to place the new transverse reinforcement. Chipping off also had a detrimental effect of the bond condition along the beam bars in the joint. Therefore chipping off the existing concrete should be minimised if required. However, the measured stiffness of Specimen R2 was nine times that of the as-built Specimen O1, indicating a significant increase in initial stiffness.

The yield displacement measured for the test specimen was 10.7mm which could be converted to 0.33% in terms of a storey drift angle. The yield displacement obtained from Specimen R2 approached the limiting value recommended by the current code [SANZ 1992].

5.3.4 Available Displacement Ductility Factor

The available displacement ductility factor \( \mu_a \) of 10 was obtained for Specimen R2. This value was comparable to that calculated for Specimen R1. As mentioned before, the behaviour of the joint did not affect the overall response of Specimen R2 although the widths of the joint diagonal tension cracks were much larger than those observed for Specimen R1.

It can be concluded that even when no new horizontal shear reinforcement were placed in the joint core, the response required of structures with "adequate ductility" in the current code could be achieved by jacketing the poorly detailed as-built beam-column joints in this study.
5.3.5 **Beam Behaviour**

5.3.5.1 **Beam Shear**

The maximum nominal beam shear stress \( \nu_b = V_b / bd \) measured was 0.12\(^2\) \( f_c' \) during beam negative moment and 0.07\(^2\) \( f_c' \) during beam positive moment, respectively, where \( V_b \) is applied beam shear force obtained from the measured end reaction, \( b \) is the beam width, \( d \) is the effective depth of the beam and \( f_c' \) is the weighted average compressive strength of the two concretes of the beam.

5.3.5.2 **Longitudinal Beam Bar Strains**

The strain distributions obtained using the readings of wire strain gauges attached to the new and existing beam longitudinal bars are shown in Figs. 5.15 and 5.16, respectively. The strains of the new bottom beam bars measured at the column face reached the yield strain during the loading to \( \pm 0.5P_l \) as illustrated in Fig. 5.15(b). On the other hand, the strains of the new top beam bars reached the yield strain in the loading to \( \pm 0.75P_l \) (see Fig. 5.15(a)). In the loading to DF of 2, the measured bar strains at the column face increased to larger than 1.3% and yielding of top and bottom beam bars spread over 0.5 times the beam overall depth from column face in the beam. The measured strains at the centre of the column were less than the yield strain until the loading to DF of 2. The strain profiles measured along the new beam bars in Specimen R2 were very similar to those measured in Specimen R1.

As shown in Fig. 5.16, the strains in the existing top and bottom beam bars at the column face reached the yield strain during the loading to \( \pm 0.75P_l \) or DF of 1. In the loading to DF of 2, the maximum tensile strains measured at the column face were about 2.14% for the top beam bars and 2.33% for the bottom beam bars. Up to the loading to DF of 4, the strains at the centre of the joint were smaller than yield strain and no yield penetration into the joint core was measured. As was observed for Specimen R1, tensile strains in the beam bars were measured through the column depth even in the elastic loading cycles. The tensile strains measured in the joint core would cause beam fixed-end rotation, resulting in a more flexible structure. Although the tensile strains in the beam bars measured at the column face subjected to beam flexural compression force were somewhat smaller than those obtained from Specimen R1, the difference was not so significant.

5.3.5.3 **Beam Curvature Ductility Factor**

The curvature ductility factors of the beams were calculated using the same method as used for Specimen R1, described in Section 5.2.5.3. The gauge length for calculating the curvature ductility factor was \( 250 \text{mm} (=0.42h_b) \), where \( h_b \) is beam depth (=600mm). The
Fig. 5.15(a) Strain Profiles of New Top Beam Bar(D12) of Specimen R2
Fig. 5.15(b) Strain Profiles of New Bottom Beam Bar (D12) of Specimen R2
Fig. 5.16(a) Strain Profiles of Existing Top Beam Bar (D24) of Specimen R2
Fig. 5.16(b) Strain Profiles of Existing Bottom Beam Bar(D24) of Specimen R2
curvature ductility factors so obtained were plotted in Fig.5.17. The yield curvatures estimated at ±0.75P, are also shown. Theoretical yield curvatures by conventional section analysis are 0.0033(1/m) during beam positive moment and 0.0037(1/m) during beam negative moment.

In the loading to DF of 2, the curvature ductility factors with beam positive moment were very similar to those with beam negative moment. In the loading to DF of 4, however, the curvature ductility factors with beam positive moment increased rapidly while those with beam negative moment increased gradually. Up to the loading to DF of 8, the maximum curvature ductility factor during beam positive moment was 18 for the west beam and 23 for the east beam. During beam negative moment, the maximum curvature ductility factor was 6 for the east beam and 4 for the west beam. It is evident that the beams retrofitted by jacketing with new reinforced concrete designed to the current code[SANZ 1982(a)] could reach large curvature ductility factors in this study.

5.3.5.4 Equivalent Plastic Hinge Lengths of the Beams

The equivalent plastic hinge length \( L_p \) for the retrofitted beams of Specimen R2 was estimated at the peak of each loading cycle. The procedures for estimating the equivalent plastic hinge length were described in Section 5.2.5.4 in detail. The equivalent plastic hinge lengths for each beam so obtained are shown in Fig.5.18 expressed as a proportion of the beam depth.

As was observed for Specimen R1, the values of the equivalent plastic hinge length exhibited a wide scatter in the loading to DF of 2. In the loading to DF of 4, however, the equivalent plastic hinge length became fairly constant for both beams. Until loading to DF of 8, the range of values of the equivalent plastic hinge lengths were 0.41 to 0.44 times the beam depth during beam positive moment and 0.39 to 0.47 times the beam depth during beam negative moment, respectively. The equivalent plastic hinge length estimated for Specimen R2 was very comparable to that estimated for Specimen R1.

5.3.6 Column Behaviour

5.3.6.1 Column Cracking

During the test, column flexural cracks and bond splitting cracks along the column corner bars were observed. The crack widths of flexural cracks remained very small while the bond splitting cracks extended and connected to the joint diagonal tension cracks.
Fig. 5.17 Curvature Ductility Factors of Beams of Specimen R2
Fig. 5.18 Equivalent Plastic Hinge Length of the Beams of Specimen R2
5.3.6.2 **Longitudinal Column Bar Strains**

The strains in the new longitudinal column bars at the beam face are shown in Fig.5.19. Those strains were measured using the wire strain gauges. Up to the loading to DF of 2, the strains increased gradually and the strains measured at the beam face when subjected to column flexural compression force showed small compressive strain. In the loading to DF of 4, however, in which corner to corner diagonal tension cracks developed in the joint core, those strains shifted to tensile strains. Subsequent loading cycles resulted in a rapid increase in tensile strains. The tensile strains prevailed along the column bars in the joint and the bond forces along the column corner bars to be transmitted into the joint core were significantly reduced. This would cause fixed-end rotation of the columns, resulting in a more flexible structure. As mentioned earlier, bond splitting cracks were observed along the column corner bars at this stage.

When comparing the strain profiles obtained from Specimen R1 (compare Fig.5.8 with Fig.5.19), it is clearly shown that after diagonal tension cracking occur in the joint, the bond forces along the new column corner bars could be hardly generated in the joint without horizontal shear reinforcement.

5.3.7 **Joint Behaviour**

5.3.7.1 **Joint Shear**

The maximum nominal horizontal joint shear stress was $0.27\sqrt{f_{c}^*}$, where $f_{c}^*$ is the weighted average compressive strength of two concretes of the joint core. In the loading to DF of 4, corner to corner diagonal tension cracks developed in the joint core. Those cracks extended and opened as the displacement ductility factor imposed on the test specimen increased.

5.3.7.2 **Strains of Column Intermediate Reinforcement**

The longitudinal column bars of the as-built specimen became the column intermediate reinforcement of the retrofitted specimens R1 and R2. Strains measured along the column intermediate bars in the joint of Specimen R2 are plotted in Fig.5.20.

Up to the loading to DF of 2, the strain profiles of the column intermediate bar indicate the role of flexural reinforcement. In the loading to DF of 4, in which corner to corner diagonal tension cracks were observed in the joint, the strains measured in the joint core increased gradually. In the subsequent loading cycles, rapid increase in tensile strain was observed where joint diagonal tension cracks crossed. On the other hand, the strains
Fig. 5.19(a) Strain Profiles of Column Bars of Specimen R2
Fig. 5.20(a) Intermediate Column Bar Strains of Specimen R2
Fig. 5.20(b) Intermediate Column Bar Strains of Specimen R2
measured at the beam face were fairly constant during the test. The measured strains in the joint core reached the yield strain in the loading to DF of 6 and the maximum strains measured were about 0.6 to 0.8%.

5.3.7.3 Joint Shear Distortion and Expansion

The joint shear distortion and expansion of Specimen R2 are shown in Fig.5.21. Up to the loading to DF of 2, joint shear distortion and expansion increased gradually. In the loading to DF of 4, in which corner to corner diagonal tension cracks developed, those measurements increased rapidly. The rate of increase was rather constant until the end of testing. The maximum joint shear distortion was 0.56%, measured in the second cycle of loading to DF of 8, which was twice that of Specimen R1. On the other hand, a maximum joint expansion of 3.2mm was obtained and this value was approximately three times of that measured for Specimen R1.

The joint shear distortion and expansion obtained from Specimen R2 were much larger than those from Specimen R1 due to the absence of the new joint hoops. However, the joint behaviour did not affect the overall response of this test specimen. This is mainly due to the reduction in the horizontal shear stress in the joint as a result of the enlargement of the column cross section area.

5.3.8 Decomposition of Horizontal Displacement

Fig.5.22 shows the components of the horizontal displacement at the peaks of the selected loading cycles.

The beam displacement due to flexure and fixed-end rotation contributed about 60% to 70% to a storey drift angle during the test. Of this, the beam fixed-end rotation accounted for approximately 35% to 45% while the beam flexure accounted for approximately 25% to 30%. The contribution due to beam fixed-end rotation increased slightly as the test progressed. On the other hand, the contribution due to beam flexure decreased as the test progressed. The trend of the beam contribution to the storey drift angle was very similar to that obtained from Specimen R1.

As shown in Fig.5.22, the fixed-end rotation of the columns contributed about 20% to 40% of a storey drift angle during the test. The contribution of the columns was fairly constant up to the loading to DF of 4. However, the contribution increased gradually in the subsequent loading cycles, as could be expected from the strain profiles of column corner bars shown in Fig.5.19. The contribution of column fixed-end rotation was somewhat larger than that of Specimen R1.
Fig. 5.21 Joint Shear Distortion and Expansion of Specimen R2

Fig. 5.22 Components of Storey Drift Angle of Specimen R2
The contribution due to joint shear distortion increased gradually until the end of the test. The maximum contribution was 16% obtained in the second cycle of loading to DF of 8. Although the absence of the joint hoops caused larger contribution of the joint shear distortion to the storey drift angle when compared with that of Specimen R1, its contribution was not so significant during the test.

5.4 RETROFITTED SPECIMEN R3

5.4.1 The Specimen

Specimen R3 was the retrofitted specimen in which only the columns were jacketed with new reinforced concrete. The compressive cylinder strengths of the existing and new concrete were 43.4MPa and 42.0MPa, respectively, at the time of testing. The ratio of the theoretical ideal flexural strength of the column to that of the beam was 2.4 based on the measured material strengths. When the beam plastic hinge mechanisms developed, the calculated ideal storey horizontal load strength $P_1$ was 139kN.

5.4.2 General Behaviour

The observed cracking after testing and the storey shear force versus horizontal displacement hysteresis curves are shown in Figs.5.23 and 5.24, respectively. The theoretical ideal storey horizontal load strength $P_1$ and the theoretical stiffness $K_{\text{theoretical}}$ are also shown in Fig.5.24. Observed cracking at the peaks of the selected loading cycles are shown in Fig.5.25.

In the loading to $\pm 0.5P_b$, flexural and flexural-shear cracks initiated in the beams. Column flexural cracks were also observed. In the loading to $\pm 0.75P_b$, those cracks extended and the number of the cracks also increased (see Fig.5.25(a)).

In the loading to displacement ductility factor DF of 1, diagonal tension cracks initiated at approximately 45 degree to the beam axis in the west beam (see Fig.5.25(b)). At this stage, the longitudinal beam bars started to yield in tension at the column face.

In the loading to DF of 2, the flexural-shear cracks in the beams extended and tended to open wide. The maximum width of those cracks was measured to be 2.2mm for the beam bottom face and 0.5mm for the beam top face, respectively. In the first negative loading cycle, the diagonal tension cracks in the east beam also developed at approximately 45 degree to the beam axis. In the west beam, the diagonal tension cracks extended. In this case, however, the diagonal tension cracks began to shift direction to become more acute to the
Fig. 5.23  Observed Cracking of Specimen R3 at second cycle of DF=8

Fig. 5.24  Storey Shear Force versus Horizontal Displacement Relationship for Specimen R3
(a) at the peak of 0.75P_i
(R=0.20%)

(b) at the peak of second cycle, DF=1
(R=0.27%)

(c) at the peak of second cycle, DF=2
(R=0.54%)

(d) at the peak of second cycle, DF=4
(R=1.08%)

(e) at the peak of second cycle, DF=6
(R=1.61%)

(f) at the peak of second cycle, DF=8
(R=2.15%)

Fig.5.25 Observed Cracking of the Joint (Specimen R3)
beam axis (see Fig. 5.25(c)). The stirrup placed at 100mm from the column face in the west beam started to yield. Columns and joint showed only minor flexural and shear cracking. The theoretical ideal storey horizontal load strength of the specimen based on the measured material strengths was reached during the first cycle of loading to DF of 2. Only small strength degradation was observed in the second loading cycle (see Fig. 5.24).

In the first cycle of loading to DF of 4, the maximum horizontal load strength of the specimen was reached at a storey drift angle of approximately 1.1% (see Fig. 5.24). The measured maximum horizontal load strength was 144kN for the positive loading cycle and 145kN for the negative loading cycle. The maximum strength was 104% of the theoretical ideal storey horizontal load strength when the plastic hinges were formed in the beams. The flexural-shear cracks extended and opened wide to a maximum crack width of 2.8mm at the beam bottom face and of 1.2mm at the top beam face, respectively. The diagonal tension cracks also extended and opened wide. In the second positive loading cycle, one dominant diagonal tension crack with an inclination of approximately 35 to 45 degree to the beam axis extended toward the flexural compression zone of the beam at the column face in conjunction with the bond splitting cracks along the top beam bar in the west beam. The diagonal tension crack opened wide and resulted in a shear failure during beam negative moment (see Fig. 5.25(d)). The maximum width of the diagonal tension crack was larger than 10mm. Beam shear failure initiated at a storey drift angle of approximately 0.74%. Severe strength degradation during the positive loading cycle was observed in the hysteresis loops shown in Fig. 5.24. In the negative loading cycle, little strength degradation could be found although a diagonal tension crack similar to that observed in the west beam developed. During beam positive moment, the beam shear cracks extended and opened wide with an inclination of larger than 45 degree to the beam axis. Significant tensile strains of larger than 1.5% were measured by all of the wire strain gauges attached to the stirrups.

The east beam also failed in the same manner at a storey drift angle of approximately 0.56% during the first cycle of loading to DF of 6 (see Fig. 5.25(e)). Severe strength and stiffness degradation was obvious during the negative loading cycle as shown in Fig. 5.24. In the subsequent loading cycles, the beam diagonal tension cracks opened more wide to a measured maximum crack width of approximately 25mm. The 6mm diameter stirrups with a spacing of 380mm fractured where crossing the wide diagonal tension cracks. Near the column face, the cover concrete of the beams at the bottom face spalled off significantly (see Figs. 5.25(e) and (f)) and the 135 degree hooks of the stirrups were bent to 90 degrees. Although beam shear cracks at obtuse angle to the beam axis also opened wide with beam positive moment, no indication of shear failure could be found. In the second cycle of loading to DF of 8, only 45% of the measured maximum horizontal load strength developed as shown in Fig. 5.24.
Until the end of the test, only minor flexural cracks could be observed in the columns. For the joint, no critical corner to corner diagonal tension cracks initiated (see Fig. 5.25(f)). Hence it could be concluded that the shape of hysteresis curves was governed by the response of the most damaged elements, namely the beams (see Fig. 5.23).

5.4.3 Initial Stiffness

The theoretical initial stiffness was calculated to be as follows:

$$K_{\text{theoretical}} = 17.3 \text{kN/mm}$$

The procedures for calculating the theoretical initial stiffness were described in Section 3.9. The theoretical stiffness $K_{\text{theoretical}}$ is shown in Fig. 5.24. The stiffness estimated at 75% of the theoretical ideal storey horizontal load strength of the test specimen was 20.0 kN/mm for the positive loading cycle and 12.5 kN/mm for the negative loading cycle. The theoretical stiffness predicted the average value of the measured initial stiffness, 16.2 kN/mm with good accuracy. On the other hand, the initial stiffnesses obtained from Specimens R1 and R2, in which the beams as well as the columns were retrofitted by jacketing, were very low when compared with the theoretical values. It can be concluded that the main reason for the low stiffness of Specimens R1 and R2 is micro cracking and reduction of the effectiveness of the existing concrete due to chipping off the cover concrete of the beams to place the new transverse reinforcement.

The measured yield displacement of the test specimen was 8.58 mm which converted to 0.27% in terms of the storey drift angle. The measured storey drift angle at yielding met the current code requirements [SANZ 1992].

5.4.4 Available Displacement Ductility Factor

An available displacement ductility factor $\mu_a$ of 2.5 was obtained from the measured storey shear force and horizontal displacement hysteresis loops of Specimen R3. The method for calculating the available displacement ductility factor was explained in Section 5.2.4. According to the New Zealand code [SANZ 1992], the test specimen was categorized as "Limited Ductility". The results tested on this specimen which was retrofitted by jacketing the columns alone showed the beam shear failure after the plastic hinges were formed in the beams, resulted in a poor displacement ductility capacity.
5.4.5 Beam Behaviour

5.4.5.1 Longitudinal Beam Bar Strains

The strains along the longitudinal beam bars obtained from the readings of wire strain gauge are shown in Fig.5.26. Fig.5.27 illustrates the beam bar strains measured from the linear potentiometer attached to the steel rods welded to the beam bars. Up to the loading to DF of 1, both figures show the gradual increase in tensile strains along the beam bars and the attainment of the yield strain at the column face. In the loading to DF of 2, the tensile strains measured at the column face increased rapidly. After the second cycle of loading to DF of 4 in which diagonal tension cracks extended and opened wide in the beams, yielding of the beam bars spread in the beams from the column face, as shown in Figs.5.26(b) and 5.27(b).

In the central region of the joint, large tensile strains were not attained until the loading to DF of 6, indicating that the beam bars were well anchored in the joint during the test.

5.4.5.2 Slip of Beam Bars

The slip of the top and bottom beam D24 bars in the joint are shown in Fig.5.28. The clear distance between two ribs of the beam bar used was 11mm. The methods to estimate the bar slip were mentioned in Section 3.7.4.

The bar slip measured at each location increased as the test progressed. The maximum bar slip was only 0.61mm, indicating that no significant slippage initiated until the end of the test. This is because the enlarged column depth of Specimen R3 was great enough to accommodate the development length of the beam bars.

5.4.5.3 Beam Shear Stress and Shear Distortion

In this study, beam shear distortions were estimated using the second set of the linear potentiometers placed diagonally at 50mm away from the column faces as illustrated in Fig.5.29. The linear potentiometers were attached to the steel rods which were welded to top and bottom beam bars.

Fig.5.29 plots the relationship between shear stress level $\nu_b/\sqrt{f_c}$ and shear distortion of the beams so obtained. Also plotted is the shear carried by stirrups using the average strains measured from the wire strain gauges attached on both sides of the stirrup, assuming a 45 degree truss mechanism.
**Top Beam Bar (Positive Loading Direction)**

- $+0.5\pi$
- $+0.75\pi$
- $+DF1(1)$
- $+DF2(1)$
- $+DF4(1)$

**Top Beam Bar (Negative Loading Cycle)**

- $-0.5\pi$
- $-DF1(1)$
- $-DF4(1)$
- $-0.75\pi$
- $-DF2(1)$

Fig. 5.26(a) Strain Profiles of Top Beam Bar Measured by Wire Strain Gauges of Specimen R3
Fig.5.26(b) Strain Profiles of Bottom Beam Bar Measured by Wire Strain Gauges of Specimen R3
Fig. 5.27(a) Strain Profiles of Top Beam Bar Measured by Linear Potentiometers of Specimen R3
Bottom Beam Bar (Positive Loading Cycle)

-0.5Pi
-0.75Pi
+DF1(1)
+DF2(1)
+DF4(1)
+DF6(1)

Strain ($\times 10^{-6}$)

Bottom Beam Bar (Negative Loading Cycle)

-0.5Pi
-0.75Pi
-DF1(1)
-DF2(1)
-DF4(1)
-DF6(1)

Strain ($\times 10^{-6}$)

Fig. 5.27(b) Strain Profiles of Bottom Beam Bar Measured by Linear Potensiometers of Specimen R3
(a) Slip of Top Beam Bar

(b) Slip of Bottom Beam Bar

Fig. 5.28: Measured Slip of Beam Bars of Specimen R3
Fig. 5.29 Relationship between Shear Stress Level and Shear Distortion of Beams of Specimen R3
With beam negative moment, the shear stress at onset of diagonal tension cracks was $0.13\sqrt{f_c}$ for the east beam and $0.12\sqrt{f_c}$ for the west beam. After the development of the diagonal tension cracks in the beam, the beam shear stress increased slightly and the maximum shear stress during beam negative moment was measured to be 1.19MPa($=0.18\sqrt{f_c}$) for the west beam and 1.13MPa($=0.17\sqrt{f_c}$) for the east beam, where $f_c$ is the measured concrete compressive cylinder strength. As shown in Fig.5.29, shear distortion increased rapidly to a value of approximately 0.6 to 0.7% with beam negative moment. At this stage, the stirrups yielded in tension. The shear carried by the stirrups was 0.20MPa which was only 16 to 17% of the maximum shear stress measured in the beams, assuming a 45 deg truss. The aggregate interlock mechanism became ineffective due to the diagonal tension cracks opening wide. Shear carried by dowel action was also reduced due to the bond splitting cracks along the top beam bars. After the breakdown of the aggregate interlock mechanism and dowel force, rapid strength reduction could be observed during the loading to DF of 4 for the west beam and DF of 6 for the east beam, as shown in Fig.5.29. The total shear stress carried approached the shear carried by stirrups during beam negative moment.

With beam positive moment, the maximum shear stress obtained was 0.62MPa($=0.09\sqrt{f_c}$) for the west beam and 0.74MPa($=0.11\sqrt{f_c}$) for the east beam. Until the loading to DF of 6, the shear distortion was less than 0.75% and no strength reduction was observed during beam positive moment as illustrated in Fig.5.29.

### 5.4.5.4 Beam Curvature Ductility Factor

Fig.5.30 shows the curvature ductility factors of the beams obtained at the peaks of the selected loading cycles. The curvatures were obtained from the second set of the linear potentiometers placed at 50mm away from the column faces. The gauge length for calculating the curvatures was 350mm. As discussed in Section 5.2.5.3, the yield curvature $\phi_y$ was calculated using the curvature $\phi_{75}$ measured at $\pm 0.75P_u$ linearly interpolated to $M_i$ where $M_i$ is the ideal flexural strength based on the measured material strengths.

During beam positive moment, the measured curvature ductility factor increased gradually as the imposed displacement ductility factor increased. The maximum curvature ductility factors attained were 14 for the west beam and 17 for the east beam. Despite the poor ductile detailing of the transverse reinforcement in the beam plastic hinge regions, relatively large curvature ductility factors could be attained during beam positive moment.

During beam negative moment, the beam curvature ductility factors increased up to the first cycle of loading to DF of 4 for the west beam and the second cycle of loading to DF of 4 for the east beam. After these loading cycles, the curvature ductility factors decreased as the test progressed. Noting the rapid increase in the shear distortion of the beams observed
Fig. 5.30 Curvature Ductility Factors of Beams of Specimen R3
during beam negative moment in those loading cycles (see Fig. 5.29), it was identified that the shear deformations of the beams became dominant at this stage.

5.4.6 **Column Behaviour**

5.4.6.1 **Column Cracking**

During the test, only minor flexural cracks were observed in the columns (see Fig. 5.25). The widths of those cracks remained very small during the test.

5.4.6.2 **Longitudinal Column Bar Strains**

Fig. 5.31 shows the strains along the corner and intermediate bars in the column measured by wire strain gauges. As the displacement ductility factor imposed on the test specimen increased, the measured strains in the column corner and intermediate bars increased gradually. Up to the loading to DF of 6, the strain distribution of the corner column bar was almost linear and only small tensile strains were measured at the beam face subjected to the flexural compression force of the column. This implies that the large corner bar force was transferred by bond to the joint core concrete without bond deterioration. Although the strain measured at the beam face reached the yield strain in the loading to DF of 4, no significant tensile strains were measured at the other face.

The strain distribution in the intermediate column bars in Fig. 5.31 indicates the role of column flexural resistance. The strains measured at the centre of the joint showed only small tensile strain up to the loading DF of 4 since no corner to corner diagonal tension cracks initiated in the joint core during the test.

It can be concluded that the columns remained essentially elastic during the test.

5.4.7 **Joint Behaviour**

When the beam plastic hinge mechanism was developed, the maximum nominal horizontal shear stress in the enlarged joint was $0.23/f_{c}^{*}$, where $f_{c}^{*}$ is the weighted average concrete compressive strength.

No critical corner to corner diagonal tension cracks were observed during testing (see Fig. 5.25). Although several diagonal tension cracks initiated in the corners of the joint, the widths of those cracks remained very small until the end of the test. The joint behaviour was satisfactory due to the reduction in the horizontal shear stress in the joint core as a result of the enlargement of the joint.
Fig. 5.31 Strain Profiles of Column Bars of Specimen R3
5.4.7.1 Bond Stresses of Beam Bars

Fig.5.32 shows the average bond stress distributions along the beam bars in the joint at the peaks of the selected loading cycles. The bond stresses, assumed to be uniformly distributed over the gauge length of $7.3d_b$, were obtained from the wire strain gauge readings, where $d_b$ is the beam bar diameter.

In the elastic loading cycles, the bond stresses were generated mainly over the region where the bars were in tension. Only small bond stresses were developed over the region where the bars were in compression. In the inelastic loading cycles, the bond stresses over the tension zone decreased and the maximum bond stresses were generated toward the centre of the joint. Bond stresses estimated over the region where the bars were in compression were still small. Until the loading to a displacement ductility factor DF of 2, the maximum bond stress was calculated to be $1.3\sqrt{f'_c}$ for bottom beam bar where $f'_c$ is the measured compressive strength of the existing concrete.

It is likely that the bond stress profiles shown in Fig.5.32 represents a good bond condition along the beam bars in the joint since the enlarged column depth was great enough to keep the bond stress of longitudinal beam bars to be an acceptable level.

5.4.8 Decomposition of Horizontal Displacement

The components of the horizontal displacement at the peaks of the selected loading cycles are expressed as a percentage of the storey drift angle in Fig.5.33. The procedures for calculating those components were explained in Section 3.8.

Until the loading to DF of 1, the largest contribution was the beam flexural displacement which contributed 76 to 79% of a storey drift angle. The fixed-end rotation of the columns accounted for about 20% of a storey drift angle. However, this contribution became insignificant as the test progressed since the columns remained essentially elastic during the test.

In the loading to DF of 2, the contribution of the beam shear displacement increased gradually but its magnitude was always less than 5%. In the second cycle of loading to DF of 4, in which the beam shear failure occurred, the beam displacement by shear distortion increased rapidly and contributed 21% of the storey drift angle. A maximum contribution to the storey drift angle of 28% was calculated in the second cycle of loading to DF of 6. On the contrary, the contribution of the beam flexural displacement began to decrease.
Fig. 5.32 Bond Stress Distributions along Beam Bars of Specimen R3
Table 5.1 Test Results

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$f_c$ (MPa)</th>
<th>Maximum Strength $P_{\text{max}}$ (kN)</th>
<th>Initial Stiffness $K_e$ (kN/mm)</th>
<th>Available Displacement Ductility Factor</th>
<th>Joint Cracking Stress $v_{j,cr}$ (MPa)</th>
<th>Maximum Joint Stress $v_{j,\text{max}}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>O1</td>
<td>40.7</td>
<td>89</td>
<td>2.38</td>
<td>NA</td>
<td>0.41$\sqrt{f_c}$</td>
<td>0.61$\sqrt{f_c}$</td>
</tr>
<tr>
<td>R1</td>
<td>50.8*</td>
<td>231</td>
<td>16.7</td>
<td>8</td>
<td>0.27$\sqrt{f_c}$</td>
<td>0.29$\sqrt{f_c}$</td>
</tr>
<tr>
<td>R2</td>
<td>56.0*</td>
<td>223</td>
<td>20.3</td>
<td>10</td>
<td>0.27$\sqrt{f_c}$</td>
<td>0.27$\sqrt{f_c}$</td>
</tr>
<tr>
<td>R3</td>
<td>42.4*</td>
<td>145</td>
<td>16.2</td>
<td>2.5</td>
<td>NA</td>
<td>0.23$\sqrt{f_c}$</td>
</tr>
</tbody>
</table>

$f_c$ : measured compressive cylinder strength of concrete

*: weighted average compressive strength of two concretes of the joint core

$v_{j,cr}$ : nominal horizontal joint shear stress when joint diagonal tension cracks first formed

$v_{j,\text{max}}$ : nominal horizontal joint shear stress when maximum storey horizontal load strength was reached
The maximum contribution of the joint shear displacement to the storey drift angle was only 9% in the loading to DF of 4 and its magnitude decreased as the imposed horizontal displacement increased, indicating that the joint remained in the elastic range.

5.5 EFFECTIVENESS OF THE RETROFIT TECHNIQUE USING CONCRETE JACKETING

Table 5.1 tabulates the test results obtained from Specimens R1, R2 and R3. The results obtained for the as-built Specimen O1 are also shown.

The retrofitted Specimens R1 and R2, in which both the beams and columns were retrofitted by jacketing with new reinforced concrete, had much higher maximum horizontal load strength, stiffness and available displacement ductility capacity than that of the as-built Specimen O1. The maximum nominal horizontal shear stress \( v_{j, \text{max}} \) in the enlarged joint of Specimens R1 and R2 was \( 0.29\sqrt{f_{c}^*} \) and \( 0.27\sqrt{f_{c}^*} \), respectively, where \( f_{c}^* \) is the weighted average compressive strength of the joint core. The joint shear stress was evidently low enough not to result in the joint shear failure, since the joints of Specimens R1 and R2 behaved satisfactory, and almost similarly, with ductile plastic hinge behaviour in the beams in spite of the fact that Specimen R2 had no joint core hoops.

For the retrofitted Specimen R3, which was retrofitted by jacketing the columns alone, the maximum horizontal load strength and stiffness were significantly increased, compared with those of the as-built Specimen O1, as shown in Table 5.1. However, the available displacement ductility factor obtained from Specimen R3 was inferior to those from Specimens R1 and R2 since shear failure of the beams occurred. The maximum nominal horizontal shear stress \( v_{j, \text{max}} \) in the enlarged joint of Specimen R3 was \( 0.23\sqrt{f_{c}^*} \) and the joint behaviour was satisfactory, in spite of the absence of the joint core hoops.

When diagonal tension cracking occurred in the joint core, the nominal horizontal shear stress \( v_{j, \text{cr}} \) in the enlarged joints of Specimen R1 and R2 was \( 0.27\sqrt{f_{c}^*} \). For Specimen R3, corner to corner joint diagonal tension cracks were not observed until the end of the test. The joint shear stress at cracking \( v_{j, \text{cr}} \) of the as-built Specimen O1 was \( 0.41\sqrt{f_{c}} \), which was somewhat larger than those of the retrofitted specimens.

5.6 CONCLUSIONS

Three reinforced concrete beam-interior column joints representing the joint region of a frame constructed in New Zealand before the 1970's were retrofitted by jacketing with new reinforced concrete. One of the as-built interior joint specimen had been tested and damaged before retrofitting. Another as-built specimen, not previously damaged, was retrofitted in
the same manner except that new joint hoops were not placed in the joint core. The other non-damaged as-built specimen was retrofitted by jacketing the columns only.

Based on the test results obtained from the retrofitted Specimens R1, R2 and R3, the following conclusions can be reached:

(1) Results of the simulated seismic load tests showed that the jacketing of columns, beams and joints with new reinforced concrete was a useful technique for enhancing the stiffness, strength and ductility of poorly detailed as-built beam-column joint regions. The tests also showed that the effect of previous damage to the as-built specimen had no significant influence on the overall seismic response of the retrofitted specimen.

(2) It was found that, even when no joint core hoops are present in the existing beam-column joints, no new joint core hoops are required in the added jacket if the existing column is enlarged by jacketing so that the nominal horizontal shear stress in the joint core is reduced to less than 0.3$\sqrt{f_c}$ MPa. This finding is for joints with no axial load on the columns. When axial compressive load is present on columns, a greater horizontal joint shear stress than 0.3$\sqrt{f_c}$ MPa would be tolerable.

(3) The overall response of Specimen R3, which was retrofitted by jacketing the columns alone, was governed by the beam shear failure which occurred after developing the theoretical ideal flexural strength of the beams. A limited ductility response, that is an available displacement ductility factor of 2.5, was attained for this specimen.

(4) Based on this test data, two limiting conditions were identified for the seismic behaviour of the beams with a small amount of shear reinforcement. At a maximum nominal shear stress level of less than 0.11$\sqrt{f_c}$ MPa, the beams did not fail in shear at least up to a curvature ductility factor of 14. However, when the maximum nominal shear stress level approached 0.18$\sqrt{f_c}$ MPa, beam shear failure commenced. At this stage, the hysteresis loops indicated rapid strength degradation, mainly due to the reduced shear carried by the concrete shear resisting mechanism, particularly aggregate interlock.

(5) The measured initial stiffnesses obtained from the retrofitted Specimens R1 and R2 were considerably lower than the theoretical values. One main reason is the damage to the existing concrete associated with surface preparation, especially chipping off the cover concrete of the beam to place the new transverse reinforcement. Therefore it is recommended that chipping off the concrete surrounding the longitudinal reinforcement should be avoided.
CHAPTER 6

EXPERIMENTAL RESULTS OF SPECIMENS O4 AND O5

6.1 INTRODUCTION

Two full-scale beam-interior column joint subassemblages, referred to as Specimens O4 and O5 were constructed and tested under simulated seismic loading. Typical features of the reinforcing details are that the test specimens had no shear reinforcement in the joint core and only a small amount of transverse reinforcement in the beams and columns, deficiencies which are common for in older building frames. Both specimens had the same dimensions and reinforcing details except for the longitudinal beam bar diameter. The beam bar diameter used was 24mm for Specimen O4 and 32mm for Specimen O5, respectively. The main aim of this test was to investigate the effect of the bond condition along the longitudinal beam reinforcement in the joint on the behaviour of the joint without shear reinforcement. This chapter presents the test results for Specimens O4 and O5.

6.2 SPECIMEN O4

6.2.1 Introduction

For Specimen O4, the ratio of the theoretical ideal flexural strength of the column to that of the beam was 1.35 based on the measured material strengths. When the beam plastic hinge mechanism was developed, the ideal storey horizontal load strength $P_1$ was 177kN. The ratio of longitudinal beam bar diameter $d_b$ to column depth $h_c$ was $d_b/h_c=24/600=1/25$. Therefore, the column depth was great enough to accommodate the development length for beam bars according to NZS3101 for ductile frames(SANZ 1982(a)). The concrete of Specimen O4 at the stage of testing had a compressive cylinder strength $f_c$ of 52.9MPa.

6.2.2 General Behaviour

The beams of the test specimen failed in shear during the loading to displacement ductility factor DF of 2 and 4. The test was temporarily terminated during the first negative cycle of loading to DF of 4. The beams were retrofitted to obtain further information about the seismic behaviour of the joint and retested. The retrofit methods used were described in detail in Section 3.3.6.

The observed cracking before retrofitting and the storey shear force versus horizontal displacement relationship are shown in Figs.6.1 and 6.2, respectively. The retrofit and the
Fig. 6.1 Observed Cracking of Specimen O4 at first cycle of DF=+4

Fig. 6.2 Storey Shear Force versus Horizontal Displacement Relationship for Specimen O4
Fig. 6.3 Observed Cracking of Specimen O4 at second cycle of DF = 8

Fig. 6.4 Storey Shear Force versus Horizontal Displacement Relationship for Specimen O4
(a) at the peak of 0.75\pi
(R=0.41\%)

(b) at the peak of second cycle, DF=1
(R=0.55\%)

(c) at the peak of second cycle, DF=2
(R=1.10\%)

(d) at the peak of second cycle, DF=4
(R=2.20\%)

(e) at the peak of second cycle, DF=6
(R=3.30\%)

(f) at the peak of second cycle, DF=8
(R=4.40\%)

Fig.6.5 Observed Cracking of the Joint (Specimen O4)
complete storey shear force versus horizontal displacement response is illustrated in Figs. 6.3 and 6.4, respectively, in which the response before retrofit is expressed by dotted lines and that after retrofit is expressed by solid lines. Observed cracking after testing and that at the peak of the selected loading cycles are shown in Figs 6.3 and 6.5, respectively.

In the loading to ±0.5P, flexural and flexural-shear cracks initiated in the beams and column flexural cracks were also observed. In the loading to ±0.75P, those cracks extended and the number of the cracks also increased. Joint shear cracks initiated at the corners of the joint (see Fig.6.5(a)).

In the loading to displacement ductility factor DF of 1, bond splitting cracks were formed along the column bars in the joint (see Fig.6.5(b)). The diagonal tension cracks initiated near the flexural-shear cracks in the west beam. At this stage, the longitudinal beam bars started to yield in tension at the column face.

In the first positive cycle of loading to DF of 2, one dominant beam diagonal tension crack with an acute angle to the beam axis extended toward the flexural compression zone of the beam at the column face in conjunction with the bond splitting cracks along the top beam bar in the west beam. The diagonal tension crack opened wide and the west beam failed in shear at the storey drift angle of 0.95% (see Figs. 6.2 and 6.5(c)). The maximum width of the beam diagonal tension crack was approximately 5mm. At this stage, the maximum horizontal load strength of the specimen was reached for the positive loading cycle. The measured maximum horizontal load strength was 175kN which was 99% of the ideal storey horizontal load strength when the beam plastic hinge mechanisms were developed. In the first negative cycle of loading to DF of 2, corner to corner diagonal tension cracks initiated at the storey drift angle of 0.53%. The maximum width of the joint diagonal tension crack was 1.8mm. At the peak of this loading cycle, the maximum horizontal load strength of 172kN was reached for the negative loading cycle (see Fig.6.2 or 6.4). In the second cycle of loading to DF of 2, the diagonal tension cracks extended and tended to open wide in the west beam. The splitting cracks along the column bars were also observed in the columns near the beam face and connected to the joint shear cracks (see Fig.6.5(c)).

In the first positive cycle of loading to DF of 4, the beam and joint diagonal tension cracks also extended and opened wide. However, strength degradation was not so significant (see Fig.6.2). During the first negative cycle of loading to DF of 4, the east beam also failed in shear in the same manner at the storey drift angle of approximately 0.94% (see Fig.6.2). At this stage, the test was terminated to retrofit the beams and retested. The horizontal load strength at the peak of the second cycle of loading to DF of 4 after retrofit was 94% of the maximum horizontal load strength obtained before retrofit during positive loading cycle and 99% of that during negative loading cycle, respectively. Joint diagonal tension
cracks opened wide to the crack width of 4mm. Bond splitting cracks along the column bars also extended and opened wide (see Fig.6.5((d)). Although the hysteresis curves were significantly pinched, the strength degradation was not so significant at this loading stage.

In the loading to DF of 6, joint diagonal tension cracks opened wide significantly. Bond splitting cracks along the column bars also extended and opened wide in both the joint and the columns. In the second cycle of loading to DF of 6, severe strength degradation was observed in the hysteresis loops as illustrated in Fig.6.4. The maximum width of the joint shear crack was approximately 6.5mm. At this stage, shear cracks also initiated in the columns (see Fig.6.5(e)). For the beams, concrete crashing in the flexural compression zone was observed. The beam diagonal tension cracks did not extend nor did open wide due to the clamping action of the steel rods placed vertically on the beams.

In the loading to DF of 8, joint diagonal tension cracks opened wide to the crack width of approximately 10mm. Bond splitting cracks along the column bars also extended and opened wide significantly. The joint expansion could be seen by visual observation. In the second cycle of loading to DF of 8, cover concrete along the column bars in the joint spalled off and the joint was severely deteriorated (see Figs.6.3 and 6.5(f)). Column shear cracks also opened wide at this stage. Severe strength degradation and pinching were obvious in the hysteresis loops as shown in Fig.6.4.

After retrofitting, the beam diagonal tension cracks were well confined by the external clamping action of the steel rods attached to the beams, indicating that the aggregate interlock action along the beam diagonal tension cracks were fully mobilized until the end of the test. On the other hand, the joint diagonal tension cracks opened wide in conjunction with the bond splitting cracks along the column bars and the condition of the joint of the test specimen deteriorated. Hence it could be concluded that the shape of hysteresis curves were mainly governed by the response of the most damaged element, the joint (see Fig.6.3).

6.2.3 Initial Stiffness

The theoretical initial stiffness was calculated for Specimen O4 as follows:

\[ K_{\text{theoretical}} = 14.5 \text{kN/mm} \]

The procedures for estimating the theoretical initial stiffness were described in detail in Section 3.9. The theoretical stiffness \( K_{\text{theoretical}} \) is shown in Figs.6.2 and 6.4. The stiffness estimated at 75% of the theoretical ideal storey horizontal load strength of the test specimen was 10.4kN/mm for positive loading cycle and 9.7kN/mm for negative loading
cycle, respectively. The average value of the stiffnesses estimated for the positive and negative loading cycle was 70% of the theoretical stiffness.

The yield displacement of the test specimen estimated at ±0.75P, was 17.6mm. This value could be converted to 0.55% in terms of a storey drift angle. Specimen O4, which has no shear reinforcement in the joint core and good bond condition along the beam bars through the joint was significantly flexible according to the current code [Sanz 1992].

6.2.4 Available Displacement Ductility Factor

The available displacement ductility factor $\mu_a$ of 4.5 was obtained from the measured storey shear force and horizontal displacement relationship of Specimen O4. The method for calculating the available displacement ductility factor was explained in Section 5.2.4. Although severe strength degradation of the test specimen was observed mainly due to the joint shear failure after developing beam plastic hinge mechanisms, a moderate displacement ductility capacity could be obtained for Specimen O4 without joint shear reinforcement.

6.2.5 Beam Behaviour

6.2.5.1 Longitudinal Beam Bar Strains

Strains along the longitudinal beam bars are illustrated in Figs. 6.6 and 6.7. Strain profiles obtained from the wire strain gauge readings are shown in Fig. 6.6 while those measured from the linear potentiometers attached to the steel rods welded to the beam bars are shown in Fig. 6.7. Up to the loading to DF of 1, both figures show the gradual increase in the tensile strains along the beam bars and the yield strain was reached at the column faces in the loading to DF of 1 or 2. In the loading to DF of 2, the tensile strains measured at the column face increased rapidly. For the west beam, the tensile strain measured at column face was not so large in the loading to DF of 2 since the west beam failed in shear during the first cycle of loading to DF of 2.

Typical feature of the strain profile along the beam bars is that tensile strains were measured over the column depth in the joint during the loading to DF of 1. This trend became more apparent in the loading to DF of 2, in which the corner to corner diagonal tension cracking was observed. The tension steel entering the joint found anchorage in the opposite beam at this stage. In the loading to DF of 2 and 4, yield penetration into the joint core was observed as shown in the Figs. 6.6 and 6.7.

It was found from the results tested on Specimen O4 without shear reinforcement that even if the column depth was great enough to accommodate the development length for the
Fig. 6.6(a) Strain Profiles of Top Beam Bar Measured by Wire Strain Gauges of Specimen O4
Bottom Beam Bar (Positive Loading Cycle)

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Bottom Beam Bar (Negative Loading Cycle)

---

Fig. 6.6(b) Strain Profiles of Bottom Beam Bar by Wire Strain Gauges of Specimen O4
Top Beam Bar (Positive Loading Cycle)

Strain (×10^-6)

-0.5πi
-0.75πi
+DF1(1)
+DF2(1)
+DF4(1)

Top Beam Bar (Negative Loading Cycle)

Strain (×10^-6)

-0.5πi
-0.75πi
-DF1(1)
-DF2(1)

Fig. 6.7(a) Strain Profiles of Top Beam Bar Measured by Linear Potentiometers of Specimen O4
Fig. 6.7(b) Strain Profiles of Bottom Beam Bar Measured by Linear Potensiometers of Specimen O4
beam bars required by current design code [SANZ 1982(a)], tensile strains in the beam bars were developed over the column depth at early loading stage, resulting in more flexible structure. The development of the diagonal tension cracks in the joint accelerated this trend.

6.2.5.2 Slip of Beam Bars

As mentioned earlier, the ratio of beam bar diameter $d_b$ of column depth $h_c$ for Specimen O4 was $d_b/h_c = 24/600 = 1/25$, where $h_c$ is the column overall depth and $d_b$ is the longitudinal beam bar diameter. Therefore well anchorage condition for the beam bars could be expected through the joint.

The slip of the top and bottom beam bar in the joint are shown in Fig.6.8. The clear distance between two adjacent ribs of the beam bar was 11mm. The methods to obtain the bar slip were mentioned in detail in Section 3.7.4.

The bar slip measured at each location increased with the test progressed. Until the loading to DF of 6, the maximum bar slip was measured to 2.2mm for top beam bar and 1.9mm for bottom beam bar, respectively. No significant slippage of the beam bars in the joint was measured during the test.

6.2.5.3 Beam Shear Stress and Shear Distortion

Fig.6.9 plots the relationship between shear stress level $v_b/\sqrt{f_c}$ and shear distortion of the beams, where $v_b = V_b/(b_w d)$, $V_b$ is the applied beam shear force, $b_w$ is the beam width and $d$ is the effective depth of beam. Beam shear distortion was calculated using the second set of the linear potentiometers placed diagonally at 50mm away from the column faces as illustrated in Fig.6.9. Also plotted are the shear carried by stirrups using the average strains of the stirrups measured from the wire strain gauges attached on both sides of the stirrup, assuming 45 degree truss mechanism.

For the west beam, the beam diagonal tension crack was developed in the loading to DF of 1. In the loading to DF of 2, beam shear failure initiated at shear stress of $0.16\sqrt{f_c}$ for the loading to negative moment, where $f_c$ is the measured concrete compressive strength. Shear distortion at failure was about 0.25%. Subsequent loading cycles resulted in a rapid increase in shear distortion of up to approximately 1%. The severe strength degradation for the west beam was caused by the diagonal tension cracks opened wide and bond splitting cracks along the top beam bar. At this stage, the stirrup in the west beam located at the closest to the column face yielded in tension and large strain was measured there. For the east beam, the maximum beam shear stress of $0.16\sqrt{f_c}$ was obtained. In the first negative cycle of loading to DF of 4 (shown in Fig.6.9 as DF2(3)), the shear distortion measured in the
Test Sequence

(a) Slip of Top Beam Bar

(b) Slip of Bottom Beam Bar

Fig. 6.8 Measured Slip of Beam Bars of Specimen O4
Fig. 6.9  Relationship between Shear Stress Level and Shear Distortion of Beams of Specimen O4
east beam increased rapidly and the shear stress was reduced to $0.12\sqrt{f_c}$ at shear distortion of 1.61% during negative moment. At this stage, the west beam also failed in shear with positive moment shown in Fig.6.9 and severe strength degradation was measured. The aggregate interlock mechanism became ineffective due to wide open diagonal tension cracks observed in both beams. Shear carried by dowel action was also reduced by the bond splitting cracks along the top beam bar.

6.2.6 Column Behaviour

6.2.6.1 Introduction

In the loading to DF of 1, bond splitting cracks along the column bars initiated in the joint. Those cracks extended and opened wide significantly as the test progressed. Shear cracks were observed in the columns in the loading to DF of 6 and opened wide in the loading to DF of 8.

6.2.6.2 Longitudinal Column Bar Strains

Fig.6.10 shows the strains along the longitudinal column bars in the joint. Those strains were obtained from the wire strain gauges. As the displacement ductility factor imposed on the test specimen increased, the measured strains of the column bars increased gradually. Until the loading to $\pm 0.75P_0$, the strains measured at the beam face showed small compressive strains when subjected to flexural compression force of the column while those showed tensile strains when subjected to flexural tension force of the column. In the loading to $\pm 0.75P_1$ in which joint shear cracks were developed at the corners of the joint, the tensile strain measured at the centre of the joint increased rapidly when compared with the other measurement. In the loading to DF of 1, this trend became more obvious and the tensile strains measured at the centre of the joint were equal to or larger than those measured at beam face when subjected to flexural tension force of the column. This implies that the column bond forces could not be developed in the flexural tension zones of the column in the joint. In the loading to DF of 2 in which a corner to corner diagonal tension crack was observed in the joint, the column bars at the centre of the joint yielded in tension as illustrated in Fig.6.10 (b). Yield strain was reached along the column bar through the joint in the first negative cycle of loading to DF of 2(see Fig.6.10 (a)).

The strain profiles of the column bars in the joint shown in Fig.6.10 indicated that the column bars in the joint core were stressed significantly larger than expected by the column flexural resistance alone, especially after the development of the joint diagonal tension cracks.
Fig. 6.10(a) Strain Profiles of Column Bar of Specimen O4
Fig. 6.10(b) Strain Profiles of Column Bar of Specimen O4
6.2.7 Joint Behaviour

6.2.7.1 Introduction

Joint shear cracks initiated at the corners of the joint in the loading to \( \pm 0.75P_t \). In the loading to DF of 1, bond splitting cracks were formed along the longitudinal column reinforcement in the joint (see Fig.6.5(b)). The maximum nominal horizontal joint shear stress was \( 0.47\sqrt{f'_c} \), where \( f'_c \) is the measured concrete compressive strength. Initial corner to corner diagonal tension crack was observed at storey drift angle of 0.53% in the loading to DF of 2 (see Fig.6.2). The cracking shear stress was found to be \( 0.45\sqrt{f'_c} \). In the second cycle of loading to DF of 6, severe strength degradation was observed mainly due to the joint shear failure.

6.2.7.2 Bond Stresses of the Longitudinal Beam and Column Bars in the Joint

Average bond stresses along the longitudinal beam and column bars in the joint, assuming to be uniformly distributed over the gauge length of 150mm for the beam bars and 250mm for the column bars, were estimated using the wire strain gauge readings. Average bond stresses so obtained for the longitudinal beam and column bars in the joint are plotted in Figs.6.11 and 6.12, respectively. Only the bond stresses at the peak of the positive loading cycle are plotted until the loading to DF of 2 or 4.

In the loading to \( \pm 0.5P_t \), the bond stresses in the beam bars estimated over the region located immediately inside the joint, where the column flexural tension force was imposed, began to decrease before the peak of the loading cycle was reached. Those bond stresses began to decrease at a bond stress of 1.5 to 2.7MPa. As shown in Fig.6.11, only small bond stresses were developed over that region. The maximum bond stress was generated mainly over the region located inside the joint from the beam flexural tension side in the loading to \( \pm 0.5P_t \). After the loading to \( \pm 0.75P_t \), in which joint shear cracks initiated at the corners of the joint, the bond stress profiles along the beam bars changed radically. The bond stresses estimated over the region, where the column flexural compression force was imposed, gradually increased as the test progressed. On the other hand, the bond stresses over the region subjected to the column flexural tension force diminished. As illustrated in Fig.6.11, the bond stresses distributed almost linearly in the inelastic loading cycle. The maximum bond stress over the region subjected to the column flexural compression force was 7.5MPa (\( =1.03\sqrt{f'_c} \)) estimated in the first cycle of loading to DF of 2, where \( f'_c \) is the measured concrete compressive strength. At this stage, however, the bond stress began to decrease over the column depth.
**Bond Stress of Top Beam Bar**

![Graph](image)

**Bond Stress of Bottom Beam Bar**

![Graph](image)

Fig. 6.11 Measured Bond Stresses of Beam Bars of Specimen O4
Fig.6.12 Measured Bond Stresses of Column Bars of Specimen O4
For the bond stress profiles along the longitudinal column bars, similar trends for the beam bars were observed as shown in Fig.6.12. In the loading to ±0.5P₀, the maximum average bond stress was developed over the region where the beam flexural tension force was imposed. After the loading to ±0.75P₀, however, the bond stresses estimated over the region subjected to the beam flexural tension force decreased while the bond stresses estimated over the region subjected to the beam flexural compression force increased rapidly. The maximum bond stress estimated for the column bars was 7.0MPa(=0.96β₀) developed in the first cycle of loading to DF of 1 in which bond splitting cracks initiated (see Fig.6.15(b)). In the loading to DF of 2, the bond stresses began to decrease and significant bond deterioration could be found in the loading of DF of 4 as illustrated in Fig.6.12.

6.2.7.3 Joint Shear Distortion and Expansion

Fig.6.13 shows the joint shear distortion, together with the joint expansion. The methods to obtain the joint shear distortion and joint expansion were defined in Section 3.7.6.

After the loading to ±0.75P₀, the joint shear distortion gradually increased. In the loading to DF of 2 in which a corner to corner diagonal tension crack initiated, the joint shear distortion increased rapidly. At this stage, the joint expansion also began to increase. In the second cycle of loading to DF of 4, the joint shear distortion was measured to be larger than 1%. Joint expansion became notable in this loading cycle as illustrated in Fig.6.13. This could be expected by the relatively large tensile strains along the beam and column bars measured overall depth of the column and beam in the joint. The maximum joint shear distortion of 1.5% was measured in the loading to DF of 6.

It is obvious that the joint shear distortion and expansion increased rapidly after the corner to corner diagonal tension cracks were developed in the joint. The absence of the joint shear reinforcement resulted in significant increase in joint shear distortion and expansion after the joint diagonal tension cracking.

6.2.8 Decomposition of Horizontal Displacement

The components of the horizontal displacement at the peak of the selected loading cycles are expressed as a percentages of the storey drift angle in Fig.6.14. The procedures for calculating those components were explained in Section 3.8.

Until the loading to DF of 1, the main contributions to the horizontal displacement were the beam flexural displacement and the fixed-end rotation of the columns. The beam flexural displacement contributed 32 to 48% of the imposed storey drift angle while the contribution of
Specimen O4

Joint Shear Distortion (%) vs. Test Sequence

- Shear Distortion
- Expansion

Fig. 6.13 Joint Shear Distortion and Expansion of Specimen O4

Specimen O4

Percentage of Storey Drift Angle (%)

Uncounted
Joint
Column Fixed-End Rotation
Beam Shear
Beam Flexure

Fig. 6.14 Components of Storey Drift Angle of Specimen O4
the fixed-end rotation of the columns was 24%. The contribution of the fixed-end rotation of
the columns fairly constant until the loading to DF of 4.

In the loading to DF of 2, in which the west beam failed in shear, the contribution of
the beam shear displacement increased gradually although its magnitude was less than 10%.
In the first cycle of loading to DF of 4, the component by the beam shear displacement
continued to increase up to 12% as could be expected by the shear failure observed in the east
beam. On the contrary, the contribution of the beam flexural displacement became to
decrease. No measurement for the beam flexure and shear, and column fixed-end rotation
were made after the second cycle of loading to DF of 4.

The contribution of the joint shear distortion to the storey drift angle constantly increased
as the imposed horizontal displacement increased as shown in Fig.6.14. Even in the elastic
loading cycle, the joint shear accounted for 8% of the storey drift angle. Rapid increase of
the joint shear contribution could be found in the second cycle of the loading to DF of 4.
Maximum contribution of 37% was obtained in the loading to DF of 6, indicating severe joint
deterioration.

6.3 SPECIMEN O5

6.3.1 Introduction

For Specimen O5, the ratio of the theoretical ideal flexural strength of the column to that
of the beam was 1.52 based on the measured material strengths. When the theoretical
flexural strength of the beam was reached at the column face, the ideal storey horizontal load
strength \( P_t \) was 150kN. The ratio of longitudinal beam bar diameter \( d_b \) to column depth \( h_c \)
was \( d_b/h_c=32/600=1/18.75 \), which did not satisfy the requirements by NZS3101 for ductile
frames[SANZ 1982(a)]. The compressive strength of the concrete cylinder was 32.8MPa at
the time of testing.

6.3.2 General Behaviour

After the first cycle of loading to DF of 4, the beams of the test specimen were
retrofitted using the same method used for Specimen O4 since one dominant diagonal tension
crack extended in the east beam.

The storey shear force versus horizontal displacement relationship is shown in Fig.6.16,
in which the response of Specimen O5 before retrofit is expressed by dotted lines and that after
retrofit is expressed by solid lines. Observed cracking after testing and those at the peak of
the selected loading cycles are shown in Figs 6.15 and 6.17, respectively.
Fig. 6.15 Observed Cracking of Specimen O5
at second cycle of DF=8

Fig. 6.16 Storey Shear Force versus Horizontal Displacement Relationship
for Specimen O5
Fig. 6.17 Observed Cracking of the Joint (Specimen O5)
In the loading to ±0.5P₀, flexural and flexural-shear cracks initiated in the beams. Column flexural cracks were also observed at the beam face. In the subsequent loading cycle to -0.75P₀, a corner to corner diagonal tension crack was formed in the joint (see Fig.6.17(a)). Beam flexural and flexural-shear cracks, and column flexural cracks extended. The number of those cracks also increased.

In the loading to displacement ductility factor DF of 1, bond splitting cracks initiated along the column bars in the joint (see Fig.6.17(b)). In the first positive loading cycle, a joint diagonal tension crack was also observed. At this stage, the longitudinal beam bars started to yield in tension at the column face.

In the loading to DF of 2, a diagonal tension crack with an acute angle to the beam axis initiated in the east beam. Bond splitting cracks along the column bars in the joint became more apparent (see Fig.6.17(c)). Joint diagonal tension cracks opened wide and extended into the columns in the flexural compression zone near the beam face. The maximum width of the joint diagonal tension crack was measured to 1.6mm, which was somewhat smaller than that measured for Specimen O4. Beam flexural cracks at the column face opened wide to the crack width of approximately 4mm and crushing of cover concrete of the beams in the flexural compression region was observed near the column face. In the first cycle of loading to DF of 2, the ideal storey horizontal load strength was reached for both positive and negative cycle (see Fig.6.16). At the peak of this negative loading cycle, the maximum horizontal load strength of 150kN was developed. In the second cycle of loading to DF of 2, the reduction of the horizontal load strength was observed (see Fig.6.16).

In the first positive cycle of loading to DF of 4, the maximum horizontal load strength of 159kN was measured which was 106% of the ideal storey horizontal load strength (see Fig.6.16). In the first cycle of loading to DF of 4, the beam diagonal tension cracks extended in both beams. At this stage, the test was terminated to retrofit the beams and retested. In the subsequent loading cycles, joint diagonal tension cracks extended and opened wide. The maximum width of the joint diagonal tension crack was approximately 4mm which was comparable to the crack width observed for Specimen O4. The bond splitting cracks along the column bars also extended and opened wide. Crushing of cover concrete in the beams at column face became more obvious and flexural cracks at the column face opened wide to the width of approximately 10mm. As shown in Fig.6.16, in the second cycle of loading to DF of 4, severe strength degradation was observed mainly due to the joint distress and the hysteresis curves were significantly pinched.

In the loading to DF of 6, joint diagonal tension cracks opened wide significantly. The maximum width of joint diagonal tension crack was measured to about 8mm for positive
loading cycle and 6mm for negative loading cycle, respectively. Bond splitting cracks along the column bars extended and opened wide significantly both in the joint and in the columns near the beam face. Column shear cracks became more apparent in the second cycle of this loading stage. For the beams, concrete crushing in the flexural compression zone was more significant when compared with that of Specimen O4 and cover concrete started to spall off (see Fig.6.17(e)). As illustrated in Fig.6.16, strength degradation and pinching became more apparent in the hysteresis loops.

In the loading to DF of 8, joint diagonal tension cracks opened wide to approximately 10mm. Bond splitting cracks along the column bars also extended and opened significantly. The columns as well as the joint suffered severe shear cracks (see Fig.6.17(f)). In the second cycle of loading to DF of 8, cover concrete in the joint and the columns started to spall off. The joint and columns were severely deteriorated. Severe strength degradation and pinching were obvious in the hysteresis loops as shown in Fig.6.16.

6.3.3 Initial Stiffness

The theoretical initial stiffness was calculated for Specimen O5 as follows:

\[ K_{\text{theoretical}} = 12.0 \text{kN/mm} \]

The calculated theoretical stiffness \( K_{\text{theoretical}} \) is shown in Fig.6.16. The stiffness estimated at 75% of the theoretical ideal storey horizontal load strength of the test specimen was 6.9 kN/mm for positive loading cycle and 8.3 kN/mm for negative loading cycle, respectively. The average value of the stiffnesses obtained from the positive and negative loading cycle was 64% of the theoretical stiffness. As observed for Specimen O4, the initial stiffness estimated for the test specimen was considerably smaller than the theoretical value.

The yield displacement measured for the test specimen was 19.7mm which could be converted to 0.62% in terms of a storey drift angle. Specimen O5 which has no shear reinforcement in the joint core and bad bond condition along the beam bars through the joint was significantly flexible like Specimen O4 according to the current code requirements [SANZ 1992].

6.3.4 Available Displacement Ductility Factor

The available displacement ductility factor \( \mu_a \) of 2.5 was obtained from the measured storey shear force and horizontal displacement relationship of Specimen O5, indicating a limited ductility response. When compared with the value of 4.5 obtained from Specimen O4, much less displacement ductility factor was observed for Specimen O5.
6.3.5 Beam Behaviour

6.3.5.1 Longitudinal Beam Bar Strains

Longitudinal beam bar strains are illustrated in Figs.6.18 and 6.19. Strain profiles obtained from the wire strain gauge readings are shown in Fig.6.18 while those measured from the linear potentiometers are shown in Fig.6.19. Up to the loading to DF of 1, tensile strains along the beam bars increased gradually and the yield strain was reached at the column face as shown in Fig.6.18. In the loading to DF of 2, the tensile strains measured in the beam plastic hinge regions increased rapidly. As illustrated in Fig.6.19, yield penetration along the beam bars was observed into the joint core in the loading to DF of 4. In the loading to DF of 6, significantly large tensile strains were measured over the column depth.

After the loading to DF of 1, the "compression" reinforcement in the beams at the column face were in tension as shown in Fig.6.18 and the tension steel entering the joint found anchorage in the opposite beam at this stage. This trend was also observed in the measured strain profiles of the longitudinal beam bars of Specimen O4 in which the column depth to beam bar diameter ratio meet the current code requirements[SANZ 1982(a)](see Fig.6.6).

The effect of the column depth to beam bar diameter ratio on the strain profiles of the longitudinal beam bars in the joint could not be found until the loading to DF of 2 in this study.

6.3.5.2 Slip of Beam Bars

The ratio of beam bar diameter to column depth for Specimen O5 was \( d_b/h_c = 32/600 = 1/18.75 \), where \( d_b \) is beam bar diameter and \( h_c \) is column overall depth. Therefore poor anchorage condition for the beam bars in the joint could be expected.

The bar slip of the top and bottom beam bars in the joint of Specimen O5 are shown in Fig.6.20. The clear distance between two adjacent ribs of the beam bar was 18mm.

The bar slip measured at each location increased with the test progressed. In the loading to DF of 4 in which large tensile strains were measured along the beam bars in the joint, the beam bar slip increased rapidly. The measured maximum bar slip until the loading to DF of 6 was 11.7mm for top beam bar and 5.8mm for bottom beam bar, respectively. When compared with the beam bar slip measured for Specimen O4 shown in Fig.6.8, much larger bar slip was observed for Specimen O5 after the loading to DF of 2. The large slip of the beam bars through the joint resulted in severe crushing of concrete in the flexural
Fig. 6.18(a) Strain Profiles of Top Beam Bar Measured by Wire Strain Gauges of Specimen O5
Bottom Beam Bar (Positive Loading Cycle)

Bottom Beam Bar (Negative Loading Cycle)

Fig.6.18(b) Strain Profiles of Bottom Beam Bar by Wire Strain Gauges of Specimen O5
Fig. 6.19(a) Strain Profiles of Top Beam Bar Measured by Linear Potensiometers of Specimen 05
Bottom Beam Bar (Positive Loading Cycle)

Bottom Beam Bar (Negative Loading Cycle)

Fig. 6.19(b) Strain Profiles of Bottom Beam Bar Measured by Linear Potensiometers of Specimen O5
Test Sequence

(a) Slip of Top Beam Bar

Test Sequence

(b) Slip of Bottom Beam Bar

Fig. 6.20 Measured Slip of Beam Bars of Specimen O5
compression zone of the adjacent beams. However, the effect on the overall response of Specimen O5 was not so significant.

6.3.6 Column Behaviour

6.3.6.1 Introduction

In the loading to DF of 1, bond splitting cracks along the column bars initiated in the joint. Those cracks extended and opened wide as the test progressed. Column shear cracks were developed in the column in the loading to DF of 6 and opened wide in the loading to DF of 8. Column shear cracks were more apparent for Specimen O5 when compared with those observed for Specimen O4.

6.3.6.2 Longitudinal Column Bar Strains

Fig.6.21 shows the strains along the longitudinal column bars in the joint. Those strains were obtained from the wire strain gauges. As the displacement ductility factor imposed on the test specimen increased, the measured strains of the column bars increased gradually. In the loading to ±0.5P1, the strain distributions of the column bars in the joint were almost linear and the strains measured at the beam face showed small compressive strains when subjected to flexural compression force of the column. In the loading to ±0.75P1 in which diagonal tension cracks were developed in the joint, tensile strains increased rapidly especially at the centre of the joint. In the loading to DF of 1, this trend became more apparent and the tensile strains measured at the centre of the joint were almost equal to that measured at beam face when subjected to flexural tension force of the column, indicating the reduction of bond stress along the column bar in the flexural tension zones. In the loading to DF of 2, the column bars at the centre of the joint reached the yield strain.

The strain profiles of the column bars in the joint measured for Specimen O5 were very similar to those obtained from Specimen O4(see Fig.6.10). It is likely that after joint diagonal tension cracking the strains of the column bars in the joint core with no shear reinforcement would be larger than expected by the column flexural resistance alone under severe earthquake loading.

6.3.7 Joint Behaviour

6.3.7.1 Introduction

An initial corner to corner diagonal tension crack was observed in the loading to -0.75P1 for Specimen O5(see Fig.6.17(a)). The cracking shear stress was 0.42\sqrt{f'_c}$, where $f'_c$ is the
**Column Bar-CA (Positive Loading Cycle)**

- +0.5Pi
- +0.75Pi
- +DF1(1)
- +DF2(1)
- +DF4(1)

**Column Bar-CA (Negative Loading Cycle)**

- -0.5Pi
- -0.75Pi
- -DF1(1)
- -DF2(1)
- -DF4(1)

---

Fig. 6.21(a) Strain Profiles of Column Bar of Specimen O5
Fig. 6.21(b) Strain Profiles of Column Bar of Specimen O5
measured concrete compressive strength. In the loading to DF of 1, bond splitting cracks initiated along the longitudinal column reinforcement in the joint (see Fig.6.17(b)). The maximum nominal horizontal joint shear stress was calculated to $0.61f'_c$. The maximum joint shear stress was obtained in the loading to DF of 2 after the ideal storey horizontal load strength was reached. As the test progressed, joint diagonal tension cracks extended and opened wide, resulting in the strength reduction of the test specimen.

6.3.7.2 Bond Stresses of the Longitudinal Beam and Column Bars in the Joint

Average bond stresses along the longitudinal beam and column bars in the joint, assuming to be uniformly distributed over the gauge length of 150mm for the beam bars and 250mm for the column bars, were estimated using the wire strain gauge readings. Average bond stresses so obtained for the main beam and column bars in the joint are plotted in Figs.6.22 and 6.23, respectively. Only the bond stresses at the peak of the positive loading cycle are plotted.

In the elastic loading cycle, the maximum bond stresses along the beam bars were generated over the region located secondly or thirdly inside the joint from the beam flexural tension side. Subsequent loading cycles resulted in the decrease in the bond stresses over those regions. As shown in Fig.6.22, until the loading to DF of 2 only small bond stresses were developed over the region located immediately inside the joint, where the column flexural tension force was imposed. On the other hand, the bond stresses estimated over the region, where the column flexural compression force was imposed, gradually increased as the test progressed. As illustrated in Fig.6.22, the bond stresses distributed almost linearly at the peak of the loading of DF of 2 although some irregularities were observed for the bottom beam bar. The maximum bond stress over the region subjected to the column flexural compression force was 6.0MPa (=1.05$f'_c$) estimated in the first cycle of loading to DF of 2, where $f'_c$ is the measured concrete compressive strength.

For the bond stress profiles along the longitudinal column bars in the joint, similar trends for the beam bars were observed as shown in Fig.6.23. In the loading to $\pm0.5P_b$, the maximum average bond stress was developed over the region located immediately inside the joint, where the beam flexural tension force was imposed. After the loading to $\pm0.75P_b$, however, the bond stresses estimated over the region subjected to the beam flexural tension force decreased while the bond stresses estimated over the region subjected to the beam flexural compression force increased rapidly. The maximum bond stress estimated for the column corner bars was 5.0MPa (=0.87$f'_c$) developed in the loading to $\pm0.75P_b$. In the loading to DF of 1, the bond stresses began to decrease and significant bond deterioration could be found in the loading to DF of 4 as illustrated in Fig.6.23.
Bond Stress of Top Beam Bar

![Graph showing bond stress of top beam bar with data points and lines representing different conditions.

D32 Top Bar

D32 Bottom Bar

Bond Stress of Bottom Beam Bar

![Graph showing bond stress of bottom beam bar with data points and lines representing different conditions.

Fig. 6.22 Measured Bond Stresses of Beam Bars of Specimen O5

Key to positive loading direction:

- West
- East
6.3.7.3 Joint Shear Distortion and Expansion

Fig. 6.24 illustrates the joint shear distortion and expansion of Specimen O5.

After the loading to \( \pm 0.75P_0 \), in which corner to corner diagonal tension cracks initiated in the joint, the joint shear distortion gradually increased. In the loading to DF of 2, the joint shear distortion increased rapidly. At this stage, the joint expansion also began to increase. In the first cycle of loading to DF of 4, the joint shear distortion approached to 1%. Although some reduction in shear distortion was observed in the second cycle of loading to DF of 4, joint shear distortion increased up to 1.4% measured in the second cycle of loading to DF of 6. Joint expansion became notable in this loading cycle as illustrated in Fig. 6.24.

When compared with the joint shear distortion and expansion measured for Specimen O4, the shear distortion and expansion measured for Specimen O5 were somewhat smaller than those for Specimen O4 (compare Fig. 6.24 with Fig. 6.13). However the difference was not so significant. The effect of the difference in column depth to beam bar diameter ratio on the joint shear distortion and expansion could not be found in this study.

6.3.8 Decomposition of Horizontal Displacement

The components of the horizontal displacement at the peak of the selected loading cycle are expressed as a percentages of the storey drift angle in Fig. 6.25. No measurements were made for beam flexural and shear displacement after the second cycle of loading to DF of 4.

Until the loading to DF of 4, the main contributions to the storey drift angle were the beam flexural displacement and the fixed-end rotation of the columns. The beam flexure displacement contributed 26 to 47% of the imposed storey drift angle while the contribution of the fixed-end rotation of the columns was 18 to 28%. The fixed-end rotation contribution increased up to 31% in the loading to DF of 6. The contribution of the beam displacement due to shear was less than 2% until the loading to DF of 4.

In the loading to \( \pm 0.75P_0 \), the joint shear distortion accounted for 16% of the storey drift angle. The contribution of the shear distortion of the joint fairly constant until the first cycle of loading to DF of 2. In the second cycle of loading to DF of 2, the component due to joint shear distortion increased rapidly and the maximum value of 31% was obtained. Although subsequent loading cycles resulted in the reduction of the contribution of the joint shear, its contribution was still important. The contribution in the second cycle of loading to DF of 6 was 24% of the storey drift angle.
**Fig.6.24 Joint Shear Distortion and Expansion of Specimen O5**

- **Specimen O5**

**Fig.6.25 Components of Storey Drift Angle of Specimen O5**

- **Specimen O5**
6.4 DISCUSSION OF THE TEST RESULTS

The comparison of the storey shear force and horizontal displacement relationship obtained from Specimen O4 with that from Specimen O5 indicates that the effect of the column depth to beam bar diameter ratio on the seismic behaviour of the joint without shear reinforcement was not significant. The somewhat inferior performance observed for Specimen O5 is mainly due to the lower concrete compressive strength when compared with that for Specimen O4.

The beam and column bar strains observed for Specimens O4 and O5 are again shown in Fig.6.26. As mentioned before, the strain profiles of the beam bars measured in the joint were almost similar despite the fact that the ratio of the column depth to beam bar diameter was different. Some difference may be found in the large tensile strains measured at flexural compression side of the bottom beam bar of Specimen O5. However, the effect of the column depth to beam bar diameter ratio on the beam bar strain profiles in the joint without shear reinforcement could not be found in this study. The strain profiles obtained along the column bars in the joint were also similar as illustrated in Fig.6.26.

The effect of the column depth to beam bar diameter ratio was observed in the loading to DF of 4, in which large beam bar slip initiated through the joint of Specimen O5 with large beam bar diameter. However, the slip of the beam bars through the joint did not affect the overall response of the test specimen significantly.

Based on the limited test data obtained from Specimens O1, O4 and O5, typical features of the strain profiles along the beam and column bars in the joint without shear reinforcement are as follows:

1. Even in the early loading stages, only small bond stresses were generated in the flexural tension side where transverse tension forces are developed, irrespective of the column depth to beam bar diameter ratio. The bond stresses were mainly developed in the flexural compression side where transverse compression forces are present.

2. The tensile strains measured along the longitudinal beam and column bars in the joint core were larger than those predicted by section analysis. This trend became more obvious after joint diagonal tension cracking.

3. In the loading to DF of 1, the "compression" reinforcement in the beam on one side of the column was actually in tension. This observation could also be applicable to the strain profiles of the column bars.
Fig. 6.26(a) Strain Profiles of the Beam and Column Bars in the Joint of Specimen O4
Fig. 6.26(b) Strain Profiles of the Beam and Column Bars in the Joint of Specimen OS
It is believed that in the elastic loading cycles, bond splitting cracks initiated along the beam and column bars in the joint. In the joint without joint hoops and intermediate column bars which could significantly improve bond performance under seismic conditions, those cracks resulted in the decrease of the bond stress over the region subjected to transverse tension force, that is the column or beam flexural tension force, irrespective of the column depth to beam bar diameter ratio. Bond force in terms of the average bond stress in the joint without shear reinforcement could be generated mainly over the region subjected to transverse compression force which could exert clamping action across the bond splitting cracks. For the column corner bars situated outside the beam width, however, the clamping action due to the beam flexural compression force could be hardly mobilized. Therefore, the splitting cracks along the column corner bars in the joint without hoops could be easily developed and extended under the seismic loading, resulting in the reduction of bond resistance along the column corner bars (see Fig. 6.3).

After diagonal tension cracking, the beam and column bars in the joint core were stressed in tension significantly, resulting in the deviation from the strain profiles obtained from section analysis. This mechanism will be discussed in Chapter 8.

For the joint without shear reinforcement, large tensile strains prevail along the beam and column bars in the joint under seismic loading, resulting in the considerable joint expansion. Fixed-end rotation of the members adjacent to the joint also become large, causing flexible structures. The beam and column bar strain profiles in such joints, which represent bond stress distributions, are quite different from those obtained from a well designed joint.

Based on the limited test data, the joint shear stress at diagonal tension cracking was found to be larger than $0.4\sqrt{f_c}$ for the joint without shear reinforcement, where $f_c$ is the measured concrete compressive strength. This value was somewhat larger than the cracking shear stress of $0.3\sqrt{f_c}$ which has been proposed by Priestley and Calvi 1991. When considering the bond stress distributions along the beam and column bars in the joint without shear reinforcement mentioned above, the shear carried by the diagonal compression strut will be increased, reducing the diagonal tension stress introduced into the joint core by bond force. Therefore, the cracking strength of the joint without shear reinforcement will be increased. Further investigation is necessary to obtain the shear strength at cracking for joints without shear reinforcement.

6.5 CONCLUSIONS

From the results tested on Specimens O4 and O5, the following conclusions are reached.
(1) Specimen O4, which had the beam bar diameter of 24mm and the maximum nominal horizontal joint shear stress of 0.47√fc, showed a moderate ductility response during the test. Specimen O5 with the beam bar diameter of 32mm and the maximum nominal joint shear stress of 0.61√fc showed a limited ductility response. For joints without shear reinforcement, the effect of the column depth to beam bar diameter ratio on the seismic behaviour of the joint was not significant.

(2) In the elastic loading cycles, bond stresses along the beam and column bars in the joint without shear reinforcement were reduced in the flexural tension side, irrespective of the ratio of the column depth to beam bar diameter. On the contrary, bond stresses were generated mainly in the flexural compression side as the test progressed.

(3) After joint diagonal tension cracking, large tensile strains prevailed along the beam and column bars in the joint core without shear reinforcement, resulting in large joint expansion.

(4) The initial stiffnesses of the test specimens were found to be very low when compared with the theoretical values. This is mainly because the large tensile strains along the longitudinal beam and column bars prevailed in the joint, causing fixed-end rotation of the members adjacent to the joint.

(5) The nominal horizontal joint shear stresses at diagonal tension cracking for the joint without shear reinforcement were found to be larger than 0.4√fc in this study. This shear stress at cracking was somewhat larger than the value previously proposed for the joints with shear reinforcement, that is 0.3√fc. This is due to the bond stress distribution along the main beam and column bars in the joint when joint reinforcement is not present.
CHAPTER 7

EXPERIMENTAL RESULTS OF SPECIMENS O6 AND O7

7.1 INTRODUCTION

One full-scale beam-exterior column joint subassemblage with beam bar end anchorages typical of the 1950’s reinforced concrete building frames being investigated was constructed. The beam bars were not bent into the joint core and the end extension beyond the bend was four times bar diameter. This specimen was referred to as Specimen O7. In order to investigate the effect of such a configuration of the hooks at the ends of the beam bars, another specimen, referred to as Specimen O6, was also constructed, in which beam bars were bent into the joint core and the extension beyond the bend was twelve times bar diameter. Only one set of 6mm diameter hoops was placed in the joint core of both specimens. Small amounts of transverse reinforcement were provided in the beams and columns. The specimens were tested under simulated seismic loading and their behaviour was compared. This chapter describes the test results for Specimens O6 and O7.

7.2 SPECIMEN O6

7.2.1 Introduction

For Specimen O6 with beam bars bent into the joint core, the ratio of the theoretical ideal flexural strength of the column to that of the beam was 1.97 during beam positive moment and 1.37 during beam negative moment, respectively. When the beam plastic hinge mechanism was developed, the ideal storey horizontal load strength $P_t$ was 43kN during beam positive moment and 62kN during beam negative moment, respectively. The concrete of the specimen at the stage of testing had a compressive cylinder strength $f'_c$ of 34.3MPa.

7.2.2 General Behaviour

The final crack pattern and the storey shear force versus horizontal displacement relationship are shown in Figs.7.1 and 7.2, respectively. Observed cracking at the peak of the selected loading cycles are shown in Fig.7.3.

In the loading to ±0.5$P_t$, flexural cracks initiated in the beam and columns. In the subsequent loading cycle to ±0.75$P_t$, those cracks extended and the number of the cracks also increased. Flexural-shear cracks were also observed in the beam (see Fig.7.3(a)).
Fig. 7.1 Observed Cracking of Specimen O6
at first cycle of DF = -12

Fig. 7.2 Storey Shear Force versus Horizontal Displacement Relationship
for Specimen O6
(a) at the peak of 0.75\( \pi \) 
\[ R=0.31\% \]

(b) at the peak of second cycle, \( DF=1 \) 
\[ R=0.41\% \]

(c) at the peak of second cycle, \( DF=2 \) 
\[ R=0.83\% \]

(d) at the peak of second cycle, \( DF=4 \) 
\[ R=1.66\% \]

(e) at the peak of second cycle, \( DF=6 \) 
\[ R=2.49\% \]

(f) at the peak of second cycle, \( DF=8 \) 
\[ R=3.33\% \]

Fig. 7.3  Observed Cracking of the Joint (Specimen O6)
In the loading to displacement ductility factor DF of 1, flexural–shear cracks in the beam became more evident. Joint shear cracks initiated at the two corners adjacent to the beam (see Fig. 7.3(b)). At this stage, bottom beam bars started to yield in tension at the column face.

In the loading to DF of 2, beam flexural and flexural–shear cracks extended and opened wide to a maximum crack width of 3.5mm. Top beam bars also yielded at the column face in this loading cycles. The ideal storey horizontal load strengths of the test specimen was reached during the positive and negative loading cycles (see Fig. 7.2).

In the first cycle of loading to DF of 4, corner to corner diagonal tension cracks formed in the joint core. Also observed were bond splitting cracks along the outer column bars (see Fig. 7.3(d)). Joint diagonal tension cracks extended and opened wide to a maximum crack width of 0.5mm. In the beam, cover concrete of the flexural compression zone in the plastic hinge region started to crush and bond splitting cracks initiated along the main beam bars near the column face. Beam flexural cracks at the column face opened wide to a maximum crack width of approximately 9mm. The maximum horizontal load strength was attained in the first cycle of loading to DF of 4 which was 47.2kN for the positive loading cycle and 63.6kN for the negative loading cycle, respectively. No significant strength degradation and pinching were observed in the hysteresis loops shown in Fig. 7.2.

In the loading to DF of 6, joint diagonal tension cracks extended and connected to the splitting cracks along the column bars. The maximum width of the joint diagonal tension cracks and bond splitting cracks along the outer column bar was 11.4mm and 2.5mm, respectively. In the first negative cycle of loading to DF of 6, one dominant diagonal tension crack initiated in the beam with an angle of approximately 45 degree to the beam axis and opened wide to a maximum crack width of 11mm (see Fig. 7.3(e)). Although some pinching was observed in the hysteresis curves, the reduction of the horizontal load strength was small.

In the loading to DF of 8, the damage of the joint due to diagonal tension cracks and bond splitting cracks along the column bars became more apparent. The maximum crack width was measured to 3mm for the joint diagonal tension cracks. In the beam, the diagonal tension crack during beam negative moment also initiated in this loading cycle (see Fig. 7.3(f)). Bond splitting cracks along the beam bars extended and the cover concrete started to spall off. Hysteresis loops were significantly pinched mainly due to the cracks opened wide in the joint and beam. However, the horizontal load strength of the specimen was observed to be larger than 80% of the measured maximum horizontal load strength (see Fig. 7.2).

In the loading to DF of 10, the beam diagonal tension cracks and splitting cracks along the main beam bars opened wide significantly. Most of the cover concrete in the beam flexural compression zone spalled off in the plastic hinge region. Sliding movement along
the beam diagonal tension cracks were observed up to 18mm in vertical direction during beam positive moment. The maximum width of the beam diagonal tension cracks was 4mm during the positive loading cycle and 15mm during the negative loading cycle, respectively. Joint diagonal tension cracks also opened wide to a maximum crack width of about 4mm. The horizontal load strength of the specimen could still be maintained larger than 80% of the measured maximum horizontal load strength (see Fig.7.2).

In the loading to DF of 12, joint diagonal tension cracks opened wide to a maximum crack width of approximately 10mm. Bond splitting cracks along the outer column bars opened wide significantly. The hysteresis loops were significantly pinched and severe strength degradation was observed in the second cycle of this loading due to beam and joint shear failure (see Figs. 7.1 and 7.2).

It is evident that for Specimen O6 in which beam bars were bent into the joint core, a ductile response could be achieved in spite of the fact that the joint and beam suffered severe diagonal tension cracking after the beam plastic hinge mechanism was developed.

### 7.2.3 Initial Stiffness

The theoretical initial stiffness was calculated for Specimen O6 as follows:

\[ K_{\text{theoretical}} = 9.0 \text{kN/mm} \]

The procedures for calculating the theoretical initial stiffness were described in detail in Section 3.9. The calculated theoretical stiffness \(K_{\text{theoretical}}\) is shown in Fig.7.2. The measured stiffness at 75% of the theoretical ideal storey horizontal load strength of the test specimen was 4.9kN/mm for the positive loading cycle and 3.5kN/mm for the negative loading cycle, respectively. The average value of the stiffnesses obtained for the positive and negative loading cycle was only 47% of the theoretical stiffness.

The yield displacement of the test specimen estimated from the stiffness at 0.75\(P_t\), extrapolated linearly to \(P_1\) was 13.3mm which could be converted to 0.42% in terms of the storey drift angle. The test specimen was rather flexible according to the current code requirements [Sanz 1992].

### 7.2.4 Available Displacement Ductility Factor

The available displacement ductility factor \(\mu_a\) was calculated to be larger than 10 from the measured storey shear force and horizontal displacement relationship of Specimen O6. The method for calculating the available displacement ductility factor was explained in Section
5.2.4. Although the test specimen suffered beam and joint diagonal tension cracking during the test, large ductility capacity was attained for Specimen O6 with beam bars bent into the joint core.

7.2.5 **Beam Behaviour**

7.2.5.1 **Longitudinal Beam Bar Strains**

Strains measured along the longitudinal beam bars are shown in Figs. 7.4 and 7.5. Fig. 7.4 illustrates the strain profiles obtained from the wire strain gauge readings while Fig. 7.5 indicates those measured from the linear potentiometers attached to the steel rods welded to the beam bars.

In the elastic loading cycles, the gradual increase in tensile strains along the beam bars was observed. In the loading to DF of 1, the bottom beam bars reached the yield strain at the column face and subsequent loading cycle resulted in significant large tensile strain of larger than 2.5% (see Fig. 7.4(b)). In the loading to DF of 2, top beam bars also yielded in tension at the column face (see Fig. 7.4(a)). The strains measured in the joint approached the yield strain at this loading stage.

After loading to DF of 4, in which joint diagonal tension cracks initiated, the strains measured in the joint core showed large tensile strains (see Fig. 7.5), indicating yield penetration into the joint core. It could be expected that the bond forces diminished along the straight portion of the beam bars from the inner column face to the hook and the steel tensile forces were resisted around the bend of the hooks.

Yielding of the beam flexural reinforcement were also observed into the beam from the column face in the loading to DF of 4 (see Fig. 7.5) and large tensile strains were measured in the loading to DF of 6, in which diagonal tension cracks extended and opened wide in the beam. For the bottom beam bar, yielding spread over a length larger than 1.5d, where d is the beam effective depth.

7.2.5.2 **Slip of Beam Bars**

The slip of the top and bottom beam bars in the joint are shown in Fig. 7.6. The clear distance between two adjacent ribs of the beam bar was 11mm. The extension of the hook was twelve times bar diameter. The methods to obtain the bar slip were described in Section 3.7.4.
**Top Beam Bar (Positive Loading Cycle)**

- $0.5\pi$ P
- $0.75\pi$ P
- $+Df1(1)$
- $+Df2(1)$

**Top Beam Bar (Negative Loading Cycle)**

- $-0.5\pi$ P
- $-0.75\pi$ P
- $-Df1(1)$
- $-Df2(1)$

Fig. 7.4(a)  Strain Profiles of Top Beam Bar Measured by Wire Strain Gauges of Specimen O6
Fig. 7.4(b) Strain Profiles of Bottom Beam Bar by Wire Strain Gauges of Specimen O6
Fig. 7.5(a) Strain Profiles of Bottom Beam Bar Measured by Linear Potentiometers of Specimen O6
Fig. 7.5(b) Strain Profiles of Top Beam Bar Measured by Linear Potentiometers of Specimen 06
Fig. 7.6 Measured Slip of Beam Bars of Specimen O6
The bar slip measured at each location increased with the test progressed. Until loading to DF of 8, the maximum bar slip was measured to 3.3 mm for top beam bar and 3.7 mm for bottom beam bar, respectively. During the test, no significant slippage was measured for Specimen O6.

7.2.5.3 Beam Shear Stress and Shear Distortion

Fig. 7.7 plots the relationship between shear stress $\sigma_b/\sqrt{f_c}$ and shear distortion of the beam, where $\sigma_b = V_b/(bd)$, $V_b$ is the applied beam shear force, $b$ is the beam width and $d$ is the effective depth of the beam and $f_c$ is the measured concrete compressive strength. Beam shear distortion was obtained from the second set of the linear potentiometers placed diagonally at 50 mm away from the column face as illustrated in Fig. 7.7. Also plotted is the shear carried by stirrups using the average strains of the stirrups measured from the wire strain gauges attached on both sides of the stirrup, assuming 45 degree truss mechanism.

During the test, the maximum nominal shear stress of the beam reached $0.14\sqrt{f_c}$ with beam negative moment and $0.11\sqrt{f_c}$ with beam positive moment.

For the negative loading cycles, which was during beam positive moment, diagonal tension cracks initiated in the loading to DF of 4 and those cracks extended and opened wide in the loading to DF of 6. At this stage, the stirrup in the beam plastic hinge region reached the yield strain and shear distortion increased rapidly up to larger than 2% as shown in Fig. 7.7. Subsequent loading cycles resulted in a large increase in shear distortion. The aggregate interlock mechanism became ineffective due to the one dominant diagonal tension crack which opened wide. However, only a little reduction of the beam shear strength was observed up to the loading to DF of 8.

For the positive loading cycles, which was during beam negative moment, diagonal tension cracks were developed and extended in the loading to DF of 8. The shear distortion during this loading cycle was measured up to 1.4% (see Fig. 7.7). The strength degradation was not so significant as was observed during beam positive moment.

7.2.5.4 Beam Curvature Ductility Factor

Until loading to DF of 8, the measured beam flexural strength during beam positive moment was 11% less than the theoretical ideal flexural strength based on the measured material strengths and was 3% less during beam negative moment. Fig. 7.8 shows the curvature ductility factor of the beam estimated at the peak of the selected loading cycles. The curvature was obtained from the second set of the linear potentiometers placed at 50 mm away from the column face, as shown in Fig. 7.8. The gauge length for estimating the curvature
**Specimen O6 Beam Shear**

![Graph showing Specimen O6 Beam Shear](image)

Fig. 7.7 Relationship between Shear Stress Level and Shear Distortion of Beam of Specimen O6

**Fig. 7.8 Curvature Ductility Factors of Beam of Specimen O6**
was 350mm. The yield curvature $\phi_y$ was obtained by using the curvature at 0.75P, extrapolated linearly P.

Up to the loading to DF of 4, the curvature ductility factor increased gradually as the test progressed. In the second cycle of loading to DF of 4, the measured curvature ductility factor began to decrease during beam positive moment. The reduction in the beam curvature ductility factor became more obvious in the loading to DF of 6. The rapid increase in beam shear distortion in this loading cycle shown in Fig.7.7 showed that shear deformations dominated for the deformation of the beam.

With beam negative moment, the measured curvature ductility factor increased up to the first cycle of loading to DF of 6 as illustrated in Fig.7.8. In the subsequent loading cycles, the curvature ductility factor did not increase. This was consistent to the increase in the shear distortion of the beam shown in Fig.7.7. The maximum curvature ductility factor was calculated to be about 9 for beam negative moment.

7.2.6 Column Behaviour

7.2.6.1 Introduction

In the loading to DF of 4, bond splitting cracks initiated along the column bars in the joint. Those cracks extended and opened wide significantly as the test progressed.

7.2.6.2 Longitudinal Column Bar Strains

Fig.7.9 shows the strains measured along the longitudinal column bars in the joint. Those strains were obtained from the wire strain gauges. As the displacement ductility factor imposed on the test specimen increased, the measured strains of the column bars increased gradually. Up to the loading to DF of 1, the strains measured in the column flexural compression zone were small compression or tension and fairly constant. In the loading to DF of 2, the strains measured throughout the joint began to increase. This trend became more obvious in the loading to DF of 4 in which the diagonal tension cracks formed in the joint. The tensile strains at the centre of the joint was notable. The strains at the centre of the joint and in the column flexural tension zone reached the yield strain in this loading cycle. Generally the inner column bar was more stressed in tension than the outer column bar as shown in Fig.7.9.
Column Bar-CA (Positive Loading Cycle)

Column Bar-CA (Negative Loading Cycle)

Fig. 7.9(a) Strain Profiles of Column Bar of Specimen O6
Fig. 7.9(b) Strain Profiles of Column Bar of Specimen O6
7.2.7 Joint Behaviour

7.2.7.1 Introduction

Joint diagonal tension cracks initiated at the nominal horizontal joint shear stress of $0.31\sqrt{f'_c}$ in the loading to DF of 4, where $f'_c$ is the measured concrete compressive strength. In the subsequent loading cycles, joint diagonal tension cracks extended and opened wide. The joint deteriorated as a result of diagonal tension cracking and bond splitting cracks along the column bars. The maximum nominal horizontal joint shear stress was calculated to be $0.32\sqrt{f'_c}$ for the positive loading cycle which gave the beam negative moment.

7.2.7.2 Joint Hoop Strains

The strains of three hoops, one located at the centre of the joint and the others at the beam face were measured using wire strain gauges. Fig.7.10 illustrates the hoop strains measured at the peak of the selected loading cycles.

After the diagonal tension cracks formed in the joint, the joint hoop strain increased gradually with an increase in the displacement ductility factor imposed on the test specimen. In the loading to DF of 4, the hoop placed at the centre of the joint started to yield and subsequent loading cycles caused a rapid increase in the tensile strain. As shown in Fig.7.10, the strains of the hoops placed at the beam face were far below the yield strain up to the loading to DF of 8, indicating small contribution of those hoops to the joint behaviour of the test specimen.

7.2.7.3 Joint Shear Distortion and Expansion

Fig.7.11 shows the joint shear distortion, together with the joint expansion. The methods to obtain the joint shear distortion and expansion were defined in Section 3.7.6.

After the loading to DF of 2, the joint shear distortion gradually increased. In the loading to DF of 4 in which corner to corner joint diagonal tension cracks initiated, the joint shear distortion increased rapidly. From the first cycle of loading to DF of 4, the joint shear distortion and expansion increased almost linearly up to the loading to DF of 8 for the positive loading cycle, that is during beam negative moment (see Fig.7.11). For the negative loading cycle, which is during beam positive moment, the joint shear distortion increased in the second cycle of loading to DF of 4. As mentioned in Section 7.2.5.4, the decrease in the beam curvature ductility observed in this loading cycle was partially attributed to the increase in the joint shear distortion. The maximum joint shear distortion was measured to be 1.5% during the positive loading cycle and 1.1% during the negative loading cycle. The maximum
Fig. 7.10 Hoop Strain Profiles of Specimen O6
Fig. 7.11 Joint Shear Distortion and Expansion of Specimen O6
joint expansion measured during both the positive and negative loading cycles were almost the same during the test.

7.2.8 Decomposition of Horizontal Displacement

The components of the horizontal displacement at the peak of the selected loading cycles are expressed as a percentage of the storey drift angle in Fig.7.12. The procedures for calculating those components were explained in Section 3.8.

Until loading to DF of 2, the main contribution was the beam flexural displacement. The beam flexural displacement contributed approximately 50 to 60% of the imposed storey drift angle in the loading to DF of 1. In the loading to DF of 2, in which plastic hinge was formed in the beam, the component of the beam flexural displacement increased to 75 to 90%. In the second cycle of loading to DF of 4, the contribution by beam flexure started to decrease as shown in Fig.7.12. Its contribution was only 20 to 30% in the loading to DF of 8. On the other hand, the component of the beam shear displacement began to increase in the loading to DF of 6 for the negative loading cycle, indicating that beam shear behaviour became dominant.

In the loading to DF of 2, the contribution of the joint shear distortion increased gradually, especially for the positive loading cycle, that was during beam negative moment. In the first cycle of loading to DF of 4, the joint shear contributed to the storey drift by 20% in the positive loading cycle. The contribution by joint shear continued to increase and its contribution was about 30% for the positive loading cycle and 20% for the negative loading cycle, respectively.

No measurements were made for the column displacement. Considering that the uncounted components were mainly due to column displacement, however, the contribution from column displacement started to increase in the second cycle of loading to DF of 4, in which the bond splitting cracks along the column bars were observed in the joint.

7.3 SPECIMEN O7

7.3.1 Introduction

Specimen O7 had the beam bars which were not bent into joint core. The ratio of the theoretical ideal flexural strength of the column to that of the beam was 2.00 during beam positive moment and 1.37 for beam negative moment, respectively. When the plastic hinge mechanism was formed in the beam, the ideal storey horizontal load strength $P_i$ was 42kN
Fig. 7.12 Component of Storey Drift Angle of Specimen O6
during beam positive moment and 61kN during beam negative moment. The compressive cylinder strength of the concrete was 31.0MPa at the time testing.

7.3.2 General Behaviour

Observed cracking after testing and the storey shear force versus horizontal displacement relationship are shown in Figs.7.13 and 7.14, respectively. Crack patterns observed at the peak of the selected loading cycles are shown in Fig.7.15.

In the loading to ±0.5P, flexural cracks initiated in the beam. Also observed were column flexural cracks at the column face.

In the loading to ±0.75P, beam and column flexural cracks extended and the number of those cracks also increased. Flexural-shear cracks formed in the beam. In the positive loading cycle, joint diagonal tension cracks initiated (see Fig.7.15(a)). The diagonal tension cracks were less inclined to the column axis when compared with those observed for Specimen O6 (see Fig.7.13(d)). The maximum horizontal load strength of the test specimen for the positive loading cycle was 46kN in this loading cycle. The maximum horizontal load strength was only 75% of the theoretical ideal horizontal load strength of the specimen.

In the loading to displacement ductility factor DF of 1, the joint diagonal tension cracks opened wide and extended to the column flexural compression zone of the top column (see Fig.7.15(b)). The width of the diagonal tension cracks in the joint was measured to be 1.3mm. No new cracks were observed in the beam. As shown in Fig.7.14, the horizontal load strength was not increased in the positive loading cycles greater than DF of 1. Bond splitting cracks also initiated along the outer column bar at this loading stage.

In the loading to DF of 2, joint diagonal tension cracks were developed in the negative loading cycle (see Fig.7.15(c)). Those cracks opened wide and extended to the column flexural compression zone of the bottom column in conjunction with bond splitting cracks along the outer column bars. The maximum width of the joint diagonal tension cracks was 5mm for the positive loading cycle and 1.8mm for the negative loading cycle, respectively. In the negative loading cycles, the maximum horizontal load strength attained was 37kN, which was 88% of the ideal horizontal load strength. In the second loading cycle, some strength degradation was observed in the hysteresis loops (see Fig.7.14).

In the loading to DF of 4, joint diagonal tension cracks extended into the beam and opened wide to a maximum crack width of 11mm for the positive loading cycle and 7mm for the negative loading cycle, respectively. Bond splitting cracks along the column bars connected with the joint diagonal cracks which also extended and opened wide. The diagonal
Fig. 7.13 Observed Cracking of Specimen O7 at first cycle of DF = 8

Fig. 7.14 Storey Shear Force versus Horizontal Displacement Relationship for Specimen O7
Fig. 7.15 Observed Cracking of the Joint (Specimen O7)
tension cracks opened in the horizontal direction rather than the perpendicular to the crack, indicating that the beam was being pulled out from the cracks. In the beam, only the flexural cracks along the column face tended to open. Severe strength degradation was observed for the negative loading cycle as shown in Fig.7.14.

In the loading to DF of 6, the maximum width of joint diagonal tension cracks was measured to be 18mm for the positive loading cycle and 11mm for the negative loading cycle. The joint deteriorated severely due to diagonal tension cracking (see Fig.7.15(e)). The bond splitting cracks along the column bar were also serious. The flexural crack of the top column opened wide in the positive loading cycle while that of the bottom column opened in the negative loading cycle. The damage in the beam was very slight.

In the loading to DF of 8, the maximum width of larger than 25mm was measured for the joint diagonal tension cracks. Concrete started to spall off in the joint. The joint suffered severe diagonal tension cracking. For the negative loading cycle, shear failure of the bottom column initiated in this loading cycle (see Fig.7.15(f)).

It was evident that for Specimen O7 in which the beam bars were not bent into the joint core, the response was governed by the joint shear failure. The test specimen could not reach the ideal horizontal load strength when the beam plastic hinge mechanism was developed. The maximum horizontal load strength of the specimen was determined by the development of the initial diagonal tension cracking in the joint for both loading directions.

7.3.3 Initial Stiffness

The theoretical initial stiffness was calculated for Specimen O7 as follows:

\[ K_{\text{theoretical}} = 8.5 \text{kN/mm} \]

The theoretical stiffness \( K_{\text{theoretical}} \) is shown in Fig.7.14. The measured stiffness at 75% of the theoretical ideal storey horizontal load strength of the test specimen was 3.8kN/mm for the positive loading cycle and 3.0kN/mm for the negative loading cycle. The average value of the stiffnesses estimated for the positive and negative loading cycles was comparable to that estimated for Specimen O6. However, the average value was only 40% of the theoretical stiffness. The effect of the configuration of the hooks at the ends of the beam bars on the initial stiffness of the test specimen was found to be insignificant.

The yield displacement of the test specimen was obtained to be 15.2mm which could be converted to 0.48% in terms of the storey drift angle. As observed for Specimen O6, the test specimen was rather flexible according to the current code requirements [SANZ 1992].
7.3.4 Beam Behaviour

7.3.4.1 Longitudinal Beam Bar Strains

Strains profiles measured along the longitudinal beam bars are shown in Figs.7.16 and 7.17. Fig.7.16 illustrates the strain profiles obtained from the wire strain gauge readings while Fig.7.17 shows those measured from the linear potentiometers attached to the steel rods welded to the beam bars.

In the elastic loading cycles, the gradual increase in tensile strains along the beam bars was observed. In the subsequent loading cycles, however, the strains measured in the beam were fairly constant as shown in Figs.7.16 and 7.17. Yield strain was not reached in the beam plastic hinge region. In the loading to DF of 2, however, the top beam bar in the joint reached the yield strain. In the loading to DF of 6, the bottom beam bar also yielded in tension in the joint core. The tensile strains along the beam bars measured in the joint core increased as the test progressed.

It could be concluded that the beam remained essentially in the elastic range during the test. However, the beam bars in the joint core were subjected to large tensile strains due to the joint diagonal tension cracking.

7.3.4.2 Slip of Beam Bars

The slip of the top and bottom beam bars in the joint are shown in Fig.7.18. The clear distance between two adjacent ribs of the beam bar was 11mm. The extension of the hook beyond the bend was four times bar diameter. The slip was estimated from the concrete at the column centre line as mentioned in Section 3.7.4.

As could be expected, the beam bars tended to be pulled out when subjected to tensile force. When subjected to compressive force, however, the bars did not push in. In the late loading stage, the bars tended to be pulled out even when subjected to compressive force as shown in Fig.7.18. The slip of top beam bar was small and relatively constant during the test. On the other hand, the slip of bottom beam bar increased gradually from the loading to DF of 2. The maximum slip was measured to 4.4mm for bottom beam bar.

In spite of the fact that the extension of the hook was only four times bar diameter, the slip measured from the column centre line was not so significant during the test.
Fig. 7.16(a) Strain Profiles of Top Beam Bar Measured by Wire Strain Gauges of Specimen O7
Bottom Beam Bar (Positive Loading Cycle)

Bottom Beam Bar (Negative Loading Cycle)

Fig. 7.16(b) Strain Profiles of Bottom Beam Bar by Wire Strain Gauges of Specimen O7
Fig. 7.17(a) Strain Profiles of Top Beam Bar Measured by Linear Potentiometers of Specimen O7
Fig. 7.17(b) Strain Profiles of Bottom Beam Bar Measured by Linear Potentiometers of Specimen O7
Fig. 7.18  Measured Slip of Beam Bars of Specimen O7
7.3.4.3 Beam Shear Stress and Curvature

During the test, the maximum nominal shear stress in the beam was obtained to $0.10\sqrt{f_c}$ during beam negative moment and $0.09\sqrt{f_c}$ during beam positive moment, respectively, where $f_c$ is the measured concrete compressive strength. Those shear stresses were low enough not to result in diagonal tension cracks in the beam (see Fig.7.15(f)).

The maximum curvature obtained from the second set of the linear potentiometers from the column face was 0.003304(1/m) during beam positive moment and 0.004885(1/m) during beam negative moment. Those curvatures were far below the yield curvature calculated from the section analysis, which were 0.004925(1/m) during beam positive moment and 0.005768(1/m) during beam negative moment, respectively.

It was again shown that the beam of Specimen 07 was essentially elastic during testing.

7.3.5 Column Behaviour

7.3.5.1 Introduction

In the loading to DF of 1, bond splitting cracks along the outer column bar initiated. Those cracks were connected to the joint diagonal tension cracks and opened wide as the test progressed.

7.3.5.2 Longitudinal Column Bar Strains

Fig.7.19 shows the strains measured along the longitudinal column bars in the joint. Those strains were obtained from the wire strain gauges. For the outer column bar, small compressive or tensile strains were measured in the column flexural compression zone until loading to DF of 1. In the loading to DF of 2, the tensile strains measured at the column flexural compression zone gradually increased. However, the strains measured on the outer column bar were relatively small as shown in Fig.7.19(a).

For the inner column bar, the strain measured at the centre of the joint increased rapidly in the loading to +0.75P, in which diagonal tension cracks initiated. In the subsequent loading cycles, the tensile strains increased consistently along the column bars through the joint as shown in Fig.7.19(b). In the loading to DF of 2, the inner column bar at the beam face reached the yield strain. It could be expected that the bond forces of the inner column bars were reduced over the flexural tension zone.
Figure 7.19(a) Strain Profiles of Column Bar of Specimen O7
**Column Bar-CB (Positive Loading Cycle)**

![Graph showing strain profiles for positive loading cycle]

**Column Bar-CB (Negative Loading Cycle)**

![Graph showing strain profiles for negative loading cycle]

Fig. 7.19(b) Strain Profiles of Column Bar of Specimen O7
As was observed for Specimen O6, the inner column bar was more stressed than the outer column bar. This is mainly due to the effect of the forces applied from the beam. Typical feature of the column bar strain profiles observed for Specimens O6 and O7 with small amount of transverse reinforcement in the joint core was that the strains measured along the column bars in the joint were relatively large after diagonal tension cracking. When compared with the column bar strain profiles in the joint measured for Specimen O6, the column bars at the flexural compression zone were more stressed in tension for Specimen O7 with the beam bars not bent into the joint core.

7.3.6 Joint Behaviour

7.3.6.1 Introduction

Joint diagonal tension cracks initiated at the nominal horizontal joint shear stress of $0.25\sqrt{f_c}$ in the loading to $+0.75P_b$, where $f_c$ is the measured concrete compressive strength. In the subsequent loading cycles, joint diagonal tension cracks extended and opened wide. The maximum horizontal load strength of the test specimen was determined by the initial diagonal tension cracking. Maximum nominal horizontal joint shear stress was calculated to $0.25\sqrt{f_c}$ for the positive loading cycle and $0.21\sqrt{f_c}$ for the negative loading cycle, respectively.

7.3.6.2 Joint Hoop Strains

The strains of three hoops located in the joint region are shown in Fig.7.20 at the peak of the selected loading cycles.

After the diagonal tension cracks formed in the joint, the joint hoop strains increased gradually as the test progressed. This trend became more evident in the loading to DF of 1 for the positive loading cycle and DF of 2 for the negative loading cycle. In the loading to DF of 2, the hoops located at the beam face yielded.

When compared with the strain profiles measured for Specimen O6 shown in Fig.7.10, the hoops located at the beam face were more stressed than that at the centre of the joint. When top beam bars were in tension, the strain of the hoop measured near the top beam bars became large. It is likely that the hoops outside of, but close to, the joint core have an important role for the seismic performance of the joint with beam bars not bent into the joint core. This will be discussed later in this chapter.
Fig. 7.20 Hoop Strain Profiles of Specimen O7
7.3.6.3 Joint Shear Distortion and Expansion

Fig. 7.21 shows the joint shear distortion and expansion. The joint shear distortion gradually increased after the loading to +0.75P_1 for the positive loading cycle and in the loading to DF of 2 for the negative loading cycle. In those loading cycles, diagonal tension cracks were observed in the joint. From the loading to DF of 4, joint shear distortion together with joint expansion increased rapidly. The maximum joint shear distortion was measured to approximately 3.5% in the loading to DF of 8. The maximum joint shear distortion and expansion measured for both the positive and negative loading cycle became comparable in the late loading stage. The maximum joint shear distortion and expansion obtained from Specimen O7 were much larger than those for Specimen O6.

7.3.7 Decomposition of Horizontal Displacement

The components of the horizontal displacement at the peak of the selected loading cycles are expressed as a percentage of the storey drift angle in Fig. 7.22. No measurements were made for the column displacement.

In the loading to ±0.75P_1, the beam flexural displacement accounted for 35 to 50% of the storey drift. This value was somewhat smaller than that calculated for Specimen O6. Subsequent loading cycles resulted in the reduction of the contribution of beam flexure. In the loading to DF of 8, its contribution was only 6% for the positive loading cycle and 1% for the negative loading cycle. The contribution of beam shear displacement was negligible during the test.

The contribution of joint shear distortion increased rapidly from the loading to DF of 1 for the positive loading cycles and DF of 2 for the negative loading cycles. A significantly large contribution due to joint shear to the storey drift is clearly shown in Fig. 7.22. The maximum contribution of joint shear distortion was 66% for the positive loading cycle and 68% for the negative loading cycle.

When compared with the test results for Specimen O6, it was evident that the joint shear behaviour governed the overall response of Specimen O7. Hence the beam-exterior column joint with beam bar anchorages typical of older reinforced concrete building frames, in which the beam bars were not bent into the joint core was identified to be possibly the weak link of the frame when subjected to severe seismic loading.
Fig. 7.21 Joint Shear Distortion and Expansion of Specimen O7
Fig. 7.22 Component of Storey Drift Angle of Specimen O7
7.4 DISCUSSION OF THE TEST RESULTS

The test results clearly demonstrated that the anchorage details of the longitudinal beam reinforcement in beam-exterior column joints have a profound effect on their seismic behaviour. This section describes possible stress paths in the exterior joint under seismic loading, depending on the configuration of the beam bar anchorage in the exterior column.

The external actions and corresponding internal forces generated around the exterior joint, in which the beam bars are bent into the joint core are shown in Figs.7.23 and 7.24. At an exterior joint, reliance for beam bar anchorage is placed primarily on a standard hook rather than the straight portion of the beam bars between the inner column face and the hook. This results in a force introduced into the joint core concrete by means of bearing and bond stresses within the bend. The reinforcement detail shown in Fig.7.24 is arranged in such a way that a diagonal compression strut, which is the main shear resisting mechanism in a beam-exterior joint, can develop between the bend of the top beam bars and the lower right-hand corner of the joint, where compression forces in both the horizontal and vertical directions are introduced. The tensile stresses are generated at a right angle to this compression strut which is responsible for the diagonal tension cracking in the joint core (see Fig.7.23).

A lightly reinforced beam-exterior joint with such beam bar anchorage in the exterior column, as is Specimen O6, can have a ductile response under simulated seismic loading when the maximum nominal horizontal joint shear stress was approximately \(0.31\sqrt{f_c}\), where \(f_c\) is the measured concrete compressive strength.

The internal forces in the exterior joint, in which the beam bars are not bent into the joint core is illustrated in Fig.7.25. This anchorage detail of the beam bars in joints commonly used in older building frames is not sufficient to develop the diagonal compression strut mechanism within the joint along a corner to corner diagonal. This is because the diagonal compression strut cannot be locked into the bend of the beam bars in the joint core as shown in Fig.7.24. Instead, the outer column bars are pushed outward. The diagonal tension crack pattern observed for Specimen O7 with beam bars not bent into the joint core was less inclined to the column axis than that for Specimen O6, indicating the difficulty of developing the diagonal compression strut mechanism illustrated in Fig.7.24. In addition, the beam bar anchorage shown in Fig.7.25 does not act to restrain the opening of the joint diagonal tension cracks.

For the configuration of the beam bar anchorage typical of older building frames, an alternative stress path may be possible, in which the angle of the strut, beginning at the lower right-hand corner of the joint, is less inclined as is illustrated in Fig.7.25. This is possible only when adequate column hoops are placed close to the joint core to provide the necessary
Fig. 7.23 Forces Acting on a Beam-Exterior Column Joint

Fig. 7.24 Diagonal Strut Mechanism of the Exterior Joint with Beam Bars Bent into the Joint

Fig. 7.25 An Alternative Shear Mechanism of the Exterior Joint with Beam Bars not Bent into the Joint
horizontal tie forces for the diagonal compression strut. The hoop forces at the inner face of the column will balance another strut which is much more inclined to the column axis from the bend of the top beam bar as shown in Fig. 7.25. The hoop strain profiles observed for Specimen O7 during the test support this alternative stress path. For Specimen O7, however, the amount of column hoops close to the joint core was not large enough to sustain this mechanism since only one set of 6mm diameter hoops could participate in this mechanism, resulting in the joint shear failure.

The beam-exterior column joint with beam reinforcement details shown in Fig. 7.25 failed in shear shortly after the diagonal tension cracking in the joint (Taylor 1974; Nilsson and Losberg 1976 and Scott et al 1994). Therefore, the cracking strength of the joint can be used to assess the shear strength of joints with such reinforcing details.

7.5 CONCLUSIONS

The following conclusions are reached on the basis of the test results from Specimens O6 and O7.

(1) It was identified that the seismic response of beam-exterior column joints was significantly influenced by the reinforcement details of the beam bars in the joint core.

(2) Specimen O7 was a beam-exterior column joint subassembly with the longitudinal reinforcement details commonly used in older reinforced concrete building frames, that is the beam bars were not bent into the joint core. The test conducted on Specimen O7 demonstrated that the seismic performance of the joint would be unsatisfactory in terms of stiffness, strength and ductility capacity of the structure. This is mainly due to the inadequate configuration of the beam bar anchorage and the inadequate amount of shear reinforcement in the joint region.

(3) Specimen O7 failed in shear shortly after the commencement of diagonal tension cracking in the joint. The maximum measured horizontal load strength of the test specimen was only 75 to 88% of the ideal horizontal load strength calculated based on the measured material strengths. The cracking strength can be used to estimate the shear strength of exterior joints with such beam anchorage details.

(4) Specimen O6 was detailed in the same manner as Specimen O7 except that the beam bars were bent into the joint core. The test on Specimen O6 showed a stable and ductile response with plastic hinge forming in the beam. Although only a small amount of shear reinforcement was provided in the joint core, the joint shear stress level of approximately
$0.31\sqrt{f_c}$ was small enough not to result in severe reduction of the horizontal load strength of the specimen under severe seismic loading.

(5) Beam bars not bent into the joint core do not efficiently develop the diagonal compression strut within the joint along a corner to corner diagonal. For an exterior joint with such reinforcing details, however, a diagonal compression strut mechanism may develop with the strut less inclined to the column axis when column hoops are adequately placed close to the joint core. If the column hoops had been adequately placed in the vicinity of the joint core of the test specimen, a better seismic performance might have been obtained. Further research is necessary in this aspect.
CHAPTER 8

ANALYSIS OF THE TEST DATA AND RECOMMENDATIONS FOR SEISMIC ASSESSMENT METHODS

8.1 INTRODUCTION

In this chapter, the seismic behaviour of beam-column joints is discussed with the support of the experimental data obtained from this study as well as from other research. Emphasis is placed on the behaviour of joints without shear reinforcement. Bond mechanisms along the beam and column bars in the joint are first examined and then the shear resisting mechanisms of the joint without shear reinforcement are discussed. Methods to assess the seismic response of such joints are suggested. The seismic behaviour of beams with small quantities of transverse reinforcement is also described in terms of the curvature ductility factor and shear strength.

8.2 SEISMIC BEHAVIOUR OF INTERIOR JOINTS WITHOUT SHEAR REINFORCEMENT BEFORE DIAGONAL TENSION CRACKING

8.2.1 Forces and Crack Pattern

Fig.8.1 shows forces acting on a joint and internal stresses in a joint core induced under seismic action. Crack pattern for a joint before diagonal tension cracking occurs is shown in Fig.8.2(a). Within a joint core, internal diagonal tensile and compressive stresses denoted by $f_t$ and $f_c$ in Fig.8.1 are generated. When column cross sections are very large and/or when beams with very small amounts of flexural reinforcement are used, the diagonal tensile stresses in the beam-column joint core may be small enough not to develop diagonal tension cracks in the joint core. Such joints were demonstrated by the retrofitted Specimen R3 in which the joint shear stress was reduced by enlarging the column cross section (see Fig.8.3).

8.2.2 Bond Behaviour in Joints

Under severe earthquake loading, the bond stresses along the bars passing through the beam-interior column joints of early building frames investigated can be large, because high $d_b/h_c$ ratios are often used for the joints, where $d_b$ is beam bar diameter and $h_c$ is the column depth. Generally these bond stresses considerably exceed the allowable bond stresses associated with the code requirements for development length. In such cases, premature bond deterioration and slip of bars within the joint may initiate under seismic loading. This results in loss of the stiffness of the frame due to the fixed-end rotation of members adjacent to
Fig. 8.1  Forces Acting on a Beam-Interior Column Joint

(a) Before Joint Diagonal Tension Cracking  (b) After Joint Diagonal Tension Cracking

Fig. 8.2  Crack Pattern for a Joint without Shear Reinforcement

Fig. 8.3  Observed Cracking of the Joint without Shear Reinforcement for Specimen R3
the joint[Popov 1984]. The bond response of longitudinal bars, both in beams and columns, plays a very important role in the shear behaviour of a beam-column joint. Therefore, the bond mechanisms along the bars within the beam-interior column joint are first examined. Only the bond of deformed bars is dealt with in this study.

Bond is made up of three components:

1. Chemical adhesion
2. Friction
3. Mechanical interlocking between concrete and steel.

Bond of deformed bars depends primarily on mechanical interlocking. The other two components are of secondary importance[Lutz and Gergely 1967]. Various factors that affect the bond strength, and bond stress and slip relationship for the bars subjected to high-intensity reversed cyclic forces have been identified by several researchers[Eligehausen et al 1983, Ismail and Jirsa 1972(a) and (b)].

Flexural cracks along the beam and column face, which are inevitably formed even during the elastic response of a structure to an earthquake, affect the bond conditions along the bars near the cracks in the joint. Under such condition, some separation of the bar and concrete occurs in the vicinity of the cracks. Internal inclined cracks, which are referred to as "bond cracks", initiate shortly after flexural cracks form due to the tensile stresses in the concrete around the bars caused by the high bearing pressure on the concrete in front of the lugs[Luzs and Gergely 1967]. After the initiation of bond cracks, the bond transfer from steel to the surrounding concrete is mainly achieved by the mechanical interlocking between concrete and steel which induces inclined compressive forces spreading from the deformation lugs into concrete. Circumferential tensile stresses are also generated, which causes splitting cracks[Goto 1971]. The bond stress at a splitting crack in unconfined concrete may be as low as $0.36\sqrt{f_c}$ MPa [Eligehausen et al 1983]. This value reveals that even under elastic loading cycles, bond splitting cracks may be formed in a beam-column joint.

In pullout tests specially designed not to restrain the concrete near the loading end, the bond behaviour in the vicinity of a crack running perpendicular to the bar was investigated by Hayashi et al 1985. The local bond stress and slip relationship obtained from such tests indicates that the maximum bond stress and bond stiffness were significantly lower near the crack than at some distance from the crack. No bond deterioration was observed at the distance of four or more times bar diameters from the crack. It can be expected that even in elastic loading cycles, bond deterioration along the beam and column bars in a beam-column joint initiates in the vicinity of the flexural cracks at the column and beam faces.
Formation of bond splitting cracks can be suppressed if the concrete surrounding the bars is effectively confined. That is, the bond performance can be significantly improved by confinement [Elgehausen et al. 1983]. In a beam-column joint region, such confinement may be achieved by lateral pressure from the compressive stresses in beams and columns adjacent to the joint and also from the transverse reinforcement orthogonally placed in the joint, when available. The compression force transverse to the direction of the embedded bars in the beam-column joint is normally available from flexural compression force induced in the adjacent members during earthquake loading. Joint transverse reinforcement, in the form of intermediate column bars and joint hoops, can prevent a failure along a potential splitting crack. They cannot prevent the initiation of splitting cracks, but they enable bond action to be maintained along the cracks. This results in a more ductile local bond stress versus slip relationship [Elgehausen et al. 1983].

The bond performance of bars passing through beam-column joints is not only influenced by the factors mentioned above. Bar diameter, concrete strength, clear distance between bars, casting direction of concrete and position of bars (top bar effect) also affect the bond behaviour along the bars in the joint.

For longitudinal beam bars passing through a joint, the least bond resistance is found in the region where the bars are in tension in the joint because of the early formation of splitting cracks caused by high circumferential tensile stresses and because of the flexural cracks at the column face. The vertical column bars there in tension also cause adverse effects. The best bond performance is achieved in the region of the longitudinal beam bars near the end of the joint where the bars are in compression. This is because at this end, confinement is provided by the flexural compression force of the columns acting on the joint. The expansion of the compressed bar due to the Poisson effect also causes compressive stresses in the concrete surrounding the bar.

A transition region between the tension and compression regions of the bar exists in a beam-column joint [Elgehausen et al. 1983]. It should be noted that bond performance in the transition region of the joint without vertical and horizontal reinforcement crossing the interior of the joint will be significantly inferior to that of a well designed joint, since no confinement is provided in that region. Therefore, a more severe bond condition can be expected along beam and column bars in a joint without intermediate vertical bars and without horizontal transverse reinforcement.

### 8.2.3 Shear Mechanisms in Joints

As mentioned in Section 8.2.1, diagonal tensile and compressive stresses develop in the joint core concrete of a beam-column joint when subjected to seismic loading. The diagonal
Compressive stresses are introduced into the joint core mainly by beam and column concrete compressive forces due to flexure. On the other hand, bond forces along the main beam and column bars transmitted into the joint core induce the diagonal tensile stresses in the joint core concrete. When bond stresses are high enough to cause significant bond deterioration, the "compression" reinforcement of the beams and columns may actually be in tension at or near the face of the joint. In that case, beam or column concrete compressive forces due to flexure will need to be increased to compensate for the reduction of the compression force in the "compression" reinforcement, resulting in an increase in the diagonal compressive forces in the joint core concrete and a reduction in the diagonal tensile stresses introduced into the joint core by bond forces.

When the nominal horizontal joint shear stress $\tau_{jh}$ is small, diagonal tension cracks may not initiate in the joint core. Then the shear force transmitted into the uncracked joint core will be resisted by means of diagonal tensile and compressive stresses in the core concrete, irrespective of whether joint shear reinforcement is present or not.

The joint shear stress for the case when diagonal tension cracks initiates in the joint core is examined in the following sections, where the joint shear stress at cracking is expressed in terms of the nominal horizontal shear stress of the joint.

8.3 Joint Shear Strength at the Stage of Developing Diagonal Tension Cracking

8.3.1 Introduction

To assess the seismic performance of early building frames studied in this research, it is very important to investigate the shear strengths of the joints without shear reinforcement. As described in Chapter 2, one approach to the assessment of the shear strengths of such joints is to assume that the shear strength is reached at the stage of initial diagonal tension cracking in the joint core. When the concrete jacketing technique is used to retrofit an existing building frame, the placement of new horizontal joint shear reinforcement can be eliminated if the nominal joint shear stress can be reduced to a level at which diagonal tension cracking does not occur. The joint shear stress can be reduced by enlarging the column cross section. According to the test results in this study, the joint failure can be prevented by simply enlarging the joint section, without placing new hoops in the joint core. For example, the retrofitted Specimen R2 without new joint hoops in which the nominal joint shear stress $\tau_{jh}$ was reduced to approximately $0.3\sqrt{f_c}$ demonstrated almost the same behaviour as the retrofitted Specimen R1 with new joint hoops. The stress level $\tau_{jh}=0.3\sqrt{f_c}$ corresponded approximately to the joint shear stress level at which diagonal tension cracks initiated in the joint core.
It is obviously of importance to determine the nominal joint shear stress at cracking for a wide range of variables, so as to assist designers with the assessment of likely joint behaviour. With this in mind the experimental data obtained from the eighty beam-interior column joints without transverse beams tested in Japan [Bessho et al 1986, Ohhtsuki et al 1986, Goto et al 1987, Teraoka et al 1987, Kamimura and Nagatsuka 1988, Bessho et al 1989, Yamauchi et al 1990, Fujii and Morita 1990, Jinno et al 1991, and Kashiwazaki and Noguchi 1991] were used to investigate the nominal joint shear stress at cracking.

8.3.2 The Test Specimens Studied

The conditions of the specimens tested in Japan, from which the data were collected, were as follows:

(1) Normal weight concrete was used for the joint region
(2) Deformed steel bars were used for the longitudinal column and beam reinforcement
(3) Both the column depth \( h_c \) and the beam depth \( h_b \) were larger than 160mm
(4) The joint core was reinforced with either joint hoops and/or column intermediate bars
(5) Longitudinal beam and column bars passed through the joint core without special anchorage details or devices

The dimensions of the selected test specimens were 160×200mm to 500×500mm for the beam cross sections and 220×220mm to 570×570mm for the column cross sections. The aspect ratio of the joint \( h_c/h_b \) was 0.86 to 1.60. Most specimens were designed to develop beam plastic hinging.

The measured compressive cylinder strength \( f_c \) of the concrete ranged from 23.3MPa to 92.6MPa while the yield strength \( f_y \) of the beam bar ranged from 345MPa to 1,069MPa.

8.3.3 Analysis of Test Data

The following factors can be considered to affect the joint shear stress at the stage when the joint diagonal tension cracks form:

(1) Compressive strength of concrete
(2) Axial load acting on the column
(3) Bond condition of the longitudinal beam bars passing through the joint
Horizontal joint shear forces \( V_{jh} \) were obtained from the beam face moments \( M_b \) and column shear forces \( V_c \) acting on the joint as illustrated in Fig.2.24. The nominal horizontal joint shear stress \( \sigma_{j,cr} \) at the onset of diagonal cracking was calculated from the effective joint area shown in Fig.2.25 [SANZ 1982(a)] and is

\[
\sigma_{j,cr} = V_{j,cr} / b_j h_j
\]

where \( V_{j,cr} \) is the horizontal joint shear force at cracking, \( b_j \) is the effective joint width defined as shown in Fig.2.25 and \( h_j \) is the effective joint depth (=\( h_c \)), where \( h_c \) is the overall column depth.

First, the effect of the concrete compressive strength on the joint shear stress at cracking was examined based on the measured concrete cylinder strengths. The results are plotted in Fig.8.4, with data points for the axial load levels \( P_u/(A_g f_c) \) of 0.12 and 0.18, where \( P_u \) is the axial load on column, \( A_g \) is the gross area of column cross section and \( f_c \) is the measured compressive strength of concrete cylinder. From each group of the test data where the applied axial load levels are the same, it can be said that the nominal horizontal joint shear stress at first diagonal cracking increased with an increase in concrete compressive strength approximately linearly.

Also, when the same concrete compressive strength was used, the nominal joint shear stress at cracking increased with an increase in the axial load level. Fig.8.5 compares the nominal horizontal joint shear stresses at cracking for various axial compressive stresses on the column, \( P_u/A_g \). Test units with different concrete compressive strengths are identified by different symbols. Again it is evident that the cracking stress was significantly affected by the level of axial compressive stress on the column. The increase in the cracking stress was approximately linear with an increase in the axial compressive stress and the rate of the increase was not affected by the concrete compressive strength significantly.

The effect of the bond condition along beam bars passing through the joint on the cracking stress was also examined. If the bond strength is assumed to be proportional to the square root of the concrete compressive strength \( f_c \), a bond index BI suggested by Kitayama et al 1987 can be used to gauge the severity of bond condition along beam bars passing through the joint. The beam bar bond index BI is defined as the total bar force to be transferred through the joint divided by the bar surface area and \( \sqrt{f_c} \), and is given by

\[
BI = \frac{2 f_y d_b \pi d_b^2}{4 \pi d_b h_c \sqrt{f_c}}
= \frac{f_y d_b}{2 h_c \sqrt{f_c}}
\]  

(8.2)
Fig. 8.4  Concrete Compressive Cylinder Strength versus Joint Shear Stress at Cracking

Fig. 8.5  Axial Compression Stress on Column versus Joint Shear Stress at Cracking
where \( f_y \): measured yield strength of beam bar, \( d_b \): diameter of beam bar and \( h_o \): column overall depth.

The nominal horizontal joint shear stress at cracking is plotted versus the beam bar bond index BI in Fig.8.6. No relationship can be seen between the cracking stress and the beam bar bond index from this figure. As mentioned in Section 8.2.3, however, good bond condition of beam bars (low BI values) in the joint results in larger diagonal tensile stresses in the joint core concrete when compared with poor bond condition (high BI value). In other words, poor bond condition of beam bars may increase the joint shear stress at cracking due to the reduction of diagonal tensile stress induced by beam bars. This was demonstrated by the test on a specimen conducted by Goto et al 1987. Beam bars in the joint of the specimen was set in vinyl tubes so that no steel bar forces could be transferred to the joint core concrete by means of bond. During the test, no joint diagonal tension cracks could be observed in spite of the maximum joint shear stress of \( 0.6\sqrt{f_c} \) MPa calculated for the specimen. The bond condition of the specimen was extremely bad and is unlikely to occur in real beam-column joints. Therefore it may be concluded from the data plotted in Fig.8.6 that for the beam bar bond index in the investigated range of 1.25 to 2.75, the bond condition of beam bars in the joint core does not affect the joint shear stress at cracking significantly. Further research is required in this aspect.

### 8.3.4 Principal Tensile Stress at Cracking

As mentioned in Chapter 2, one approach to predict the joint shear stress at cracking is to assume that initial joint diagonal tension cracking occurs when the principal tensile stress of the joint core, \( f_{cr} \), indicated by Mohr's circle for stress, reaches the diagonal tensile strength of the concrete, \( f_t \). The principal tensile stress at joint diagonal tension cracking \( f_{cr} \) can be found from the following equation.

\[
f_{cr} = -\frac{P_u}{2A_g} + \sqrt{\left(\frac{P_u}{2A_g}\right)^2 + (v_{j,cr})^2}
\]

where \( P_u \) is axial load on column, \( A_g \) is gross area of column cross section and \( v_{j,cr} \) is the nominal joint shear stress at cracking. Note that \( f_{cr} \) is positive in tension and that \( P_u \) is positive in compression in the above Eq.8.3. There are approximations in Eq.8.3. For example, the concrete compressive stress at the centre of the joint may not be \( P_u/A_g \) since the flow of forces through the joint is more complex. However, the results obtained from the eighty test data are plotted against the concrete compressive strength in Fig.8.7. Assuming that the concrete cracking stress varies in proportion to \( f_c^{2/3} \) rather than \( \sqrt{f_c} \) [Raphael 1984], the mean value of \( f_{cr} \) is 0.17\( f_c^{2/3} \) MPa for the test data. Hence, although the test data scattered widely, at the onset of joint diagonal tension cracking it can be assumed that
Fig. 8.6  Beam Bar Bond Index versus Joint Shear Stress at Cracking

Fig. 8.7  Concrete Compressive Strength versus Principal Tensile Stress at Cracking
\[ f_{cr} = 0.17f_c^{2/3} \] (8.4)

with a coefficient of variation of 23.3%. For design purposes, the following equation with the 95% lower confidence limit can be used for the concrete compressive strength in the range of 20 to 100MPa:

\[ f'_{cr} = 0.51(0.17f_c^{2/3}) \] (8.5)

This expression is shown in Fig.8.7.

8.4 SEISMIC BEHAVIOUR OF INTERIOR JOINTS WITHOUT SHEAR REINFORCEMENT AFTER DIAGONAL TENSION CRACKING

8.4.1 Forces and Crack Pattern

Figs.8.1 and 8.2(b) show forces acting and a crack pattern for a beam-interior column joint without shear reinforcement after diagonal tension cracking, respectively. Under seismic actions, large shear and bond stresses may be introduced into the joints by these forces, irrespective of whether plastic hinges develop at column faces or beam faces. These forces may cause a failure of the joint core due to the breakdown of the diagonal tension or diagonal compression mechanisms. As mentioned in Chapter 2, it was found that the joints of reinforced concrete building frames designed prior to about 1970 may have such deficiencies since in the joint core typically no shear reinforcement is provided and also longitudinal beam bars of large diameter pass through the joint with relatively small depth, causing high bond stresses.

8.4.2 Shear Mechanisms in Joints with Good Bond Condition along the Beam Bars

Fig.8.8 illustrates the actions on a beam-interior column joint subjected to horizontal seismic loading. For the sake of simplicity, it is assumed that the bending moments introduced are the same at all four sides of the joint. The tensile resultant force is denoted by \( T \), and the compressive resultant forces in the concrete and steel are shown by the symbols \( C_c \) and \( C_s \), respectively. Axial load on the column is not considered. The horizontal joint shear force \( V_{jh} \) in the joint core can be expressed from Fig.8.8 as follows:

\[ V_{jh} = T + (C_s + C_c) - V_c \] (8.6)
Fig. 8.8 Forces Acting on a Beam-Interior Column Joint

(a) Top Beam Bar

Steel Stresses

Bond Stresses
to strut

(b) When Bond Stresses along Beam Bars are Low

Steel Stresses

Bond Stresses
to strut
to diagonal tensile stresses

(c) When Bond Stresses along Beam Bars are High

Fig. 8.9 Bond Stress Distributions along the Top Beam Bar in a Joint
Similarly the vertical joint shear force, $V_{jv}$, can also be obtained from the internal column forces, $T_r$, $C_s$ and $C_c$ and the beam shear force, $V_b$. Assuming that the bond stresses along the bars passing through the joint vary linearly, approximate distributions of steel stresses, when the bond stresses are low and high, are shown in Fig.8.9.

Two mechanisms of joint core shear resistance which occur after diagonal tension cracking, namely a diagonal compression strut mechanism and a truss mechanism, have been postulated by Park and Paulay (Park and Paulay 1975). The diagonal compression strut mechanism shown in Fig.8.10(a) transfers mainly the forces from the concrete compression zones of the adjacent beams and columns across the joint core. After the development of flexural cracks at the beam or column face, it is appropriate to assume that the shear force in each of the adjoining members is introduced to the joint core mainly through the concrete compression zones in the beams and columns, respectively. Then the internal compression forces $C_c$ and $C_c$ and shearing forces $V_b$ and $V_c$ are transferred to the diagonal concrete strut. Steel forces are transferred by bond predominantly to the joint core concrete that surrounds the bars. The bond force $\Delta T_c$ from the beam bars over the length of the neutral axis depth of the column c is assumed to be transmitted to the concrete strut. A similar force $\Delta T_c$ from the longitudinal column bars is also transferred to the diagonal concrete strut. The concrete compression forces together with shearing forces and bond forces transmitted within the compression zone balance each other by means of the diagonal compression force $D_c$ without the aid of any shear reinforcement in the joint core, apart from the role of confinement for the diagonal concrete strut.

The remaining steel bond forces $\Delta T_s$ and $\Delta T_s$ should be also in equilibrium by means of a diagonal compression field with a capacity of $D_s$, where $\Delta T_s = C_s + T - \Delta T_c$ and $\Delta T_s = C_s + T - \Delta T_s'$. When no bond deteriorations initiate along the beam and column bars in the joint, those bond forces $\Delta T_s$ and $\Delta T_s'$ may be large. The stress distribution of the beam bar in the joint under such condition is shown in Fig.8.9(b), in which uniformly distributed bond stresses are assumed. Prior to diagonal tension cracking of the joint, the joint shear force is transferred through the joint, causing diagonal compressive stress $f_c$ and tensile stresses $f_s$ in the core concrete (see Fig.8.1), as described in Section 8.2.3. After diagonal tension cracks form in the joint, the ability of the concrete to transmit the tensile stresses is severely reduced. Unless appropriate reinforcement is provided, shear failure may occur in the joint. When the reinforcement is present, tension forces are induced in the reinforcement due to the loss of tension capacity of the diagonally cracked concrete in the joint. This may enable the joint to carry the necessary shear force after diagonal tension cracking. Fig.8.10(b) illustrates the upper half of the joint without intermediate column bars. Assuming no tension capacity in the cracked concrete, vertical tension force $\Sigma T_c$ is required in the main column bars to balance the necessary horizontal shear force $\Delta T_s$ and a diagonal compression force $D_s$. From consideration of the equilibrium shown in Fig.8.10(b), the required vertical tension force is
(a) Shear Carried by Diagonal Strut

(b) Shear Carried by Tension Force of Column Bars

(c) Shear Carried by Tension Force of Beam Bars

(d) Compression Stresses at the Boundaries of the Joint Core

Fig. 8.10  Shear Transfer in a Joint without Shear Reinforcement When Bond Stresses along Beam Bars are Low
\[ \Sigma T_c = \Delta T_s \tan \alpha \]  \hspace{1cm} (8.7)

where \( \alpha \) is the inclination of the diagonal compression force with respect to the horizontal centre axis. \( \tan \alpha \) may be approximated by

\[ \tan \alpha = h_b / h_c \]  \hspace{1cm} (8.8)

where \( h_b \) is the overall depth of column and \( h_c \) is the overall depth of beam. Therefore additional tension forces are introduced in the column bars in the joint core.

The left half of the joint without hoops is shown in Fig. 8.10(c). The column bar bond force \( \Delta T_s \) could also resolve itself into a diagonal compression force and the horizontal tension force supplied by main beam bars \( \Sigma T_b \) as shown in this figure. The additional tension force of the beam bars is then

\[ \Sigma T_b = \Delta T_s / \tan \alpha \]  \hspace{1cm} (8.9)

The additional tension forces expressed by Eqs. 8.7 and 8.9 are assumed to be generated in the longitudinal beam and column reinforcement within the joint core only. It is also assumed that these longitudinal reinforcement in tension within the joint core are well anchored in the adjacent beams and columns so that vertical and horizontal compression stresses are developed at the boundaries of the joint core concrete by means of bond stresses for anchorage, as shown in Fig. 8.10(d). These compression stresses enable the core concrete to transfer the necessary shear stresses at the boundaries of the joint core by means of a diagonal compression field after joint diagonal tension cracking.

The additional tension forces mentioned above may result in significantly large tensile stresses in the column and beam bars in the joint core when no bond deterioration initiates along those bars in the joint.

8.4.3 Shear Mechanisms in Joints with Severe Bond Condition along the Beam Bars

In this section, the shear mechanisms in a joint without shear reinforcement is explained for the case when severe bond deterioration occurs along the longitudinal beam and column bars in the joint. When premature bond deterioration occurs along the beam bars in the joint, the compression forces of the beam bars may be completely lost at the column face where beam flexural compression force is applied. In such cases, the neutral axis depth from the compression side of the beam section, \( c \) will become larger. The stress distribution along
the top beam bar, for this case is illustrated in Fig.8.9(c). As shown in this figure, the major part of the horizontal bond forces along the beam bars can be transmitted to the diagonal compression strut in the shaded region. Similarly, most of the vertical forces developed in the column bars $\Delta T_s'$ will be transmitted to the same region of the joint. The concrete compression, shear and bond forces at the lower right-hand corner of the joint will be combined into an equal and opposing diagonal compression force $D_e$ as shown in Fig.8.11(a).

It is evident that bond forces along the beam and column bars can be disposed of more easily within a wider diagonal compression strut. In such case, the remaining steel bond forces $\Delta T_s$ and $\Delta T_s'$ will be small, as shown in Fig.8.11(b). Those bond forces are resisted by means of diagonal tensile stresses in the concrete $f_t$ and additional tension forces in the beam and column bars $T_{b1}$ and $T_{c1}$ at a section between the two adjacent diagonal tension cracks (see Fig.8.11(c)). At a crack, the tensile stress in the concrete goes to zero and additional tension force $T_{c2}$ is required in the main column bars to balance the beam bar bond forces $\Delta T_s$ and a diagonal compression force. Similarly, additional tension force $T_{b2}$ is also required in the main beam bars to balance the column bar bond force $\Delta T_s'$ and a diagonal compression force, as illustrated in Fig.8.11(d). However, the magnitude of the additional tension forces both at a crack and at a section between the cracks will be small. It should be noted that local shear stresses on the crack surface are not shown in Fig.8.11(d).

8.4.4 Shear Mechanisms in the Joints of the Test Specimens

8.4.4.1 Introduction

The shear mechanisms in the joint without shear reinforcement have been described in the previous sections. As mentioned in Section 8.4.2, additional tension forces may be generated in the longitudinal beam and column bars in the joint core. To obtain the additional tensile stresses, it is necessary to determine realistic bond stress distributions of the longitudinal beam and column bars within the joint. As described in Chapters 4 and 6, however, the bond forces in terms of the average bond stresses change along the beam and column bars passing through the joint, depending on the stress conditions of the adjacent members acting on the joint. The estimated bond stress distributions also change as the tensile stresses induced at the tension side of the beam bars increase. In this section, the bond stress distributions along the beam bars in the joint without shear reinforcement are first examined, based on the test observations, and then a bond stress distribution along the beam bars is proposed to obtain the additional tension forces in the beam and column bars in the joint core. Finally, based on the joint shear mechanisms postulated in this study, the stress distributions along the beam and column bars in the joint are estimated for the specimens tested for this study and compared with the test results.
(a) Shear Carried by Diagonal Strut

(b) Cracked Joint Core

(c) Forces Transmitted Between Cracks  (d) Forces Transmitted Across a Crack

Fig. 8.11  Shear Transfer in a Joint without Shear Reinforcement
   When Bond Stresses along Beam Bars are High
8.4.4.2 Bond Stress Distributions in the Joints of the Test Specimens

According to the test observations, the bond stress distributions along the beam bars in the joint without shear reinforcement could be expressed at several stress levels of the beam bar at the tension side, as shown in Fig. 8.12. A notable feature of the bond behaviour in such a joint is that bond deterioration initiates even in the early loading stages.

When beam bar stresses approach about 50% of the yield strength, bond resistance is significantly reduced and no bond stresses can develop over the region of approximately six bar diameter from the column face where beam flexural tension force is applied (see Fig. 8.12(b)). A maximum bond stress of about $0.6\sqrt{f_c}$ MPa was estimated in the transition region at this stage. At bar stress of about 75% of the yield strength, the location of the maximum bond stress moves somewhat towards the compression side of the beam bar in the transition region (see Fig. 8.12(c)). Fig. 8.12(d) illustrates the bond stress distribution when the yield strength is reached in the loading to a displacement ductility factor of 1 or 2. The bond stresses increase almost linearly towards compression side of the beam, in proportion to the flexural compression force of the column acting on the joint. It was observed that the stresses in the beam bars turned to be in tension along the whole length of the joint at this stage. The maximum bond stress was estimated from the measurements to be about $1.0\sqrt{f_c}$ MPa. No measurements were made after the loading to DF of 2. However, it is likely that the bond stress distributions did not basically change further since only the flexural compression forces of the column offer confinement of the concrete surrounding the beam bars in the joint without shear reinforcement. It can be also expected that the bond resistance of the beam bar will be reduced over the region subjected to the column flexural compression force due to the effect of the high-intensity reversed cyclic forces and many diagonal tension cracks formed crossing the beam bars.

Fig. 8.13 illustrates the assumed steel stress distribution of the beam bar in a joint without shear reinforcement. Also shown is the corresponding bond stress distribution which expresses the premature bond deterioration along the beam bars where the bars are in tension. The bond stresses increase linearly over the region subjected to column flexural tension forces and uniformly distributed bond stresses are assumed over the region subjected to column flexural compression forces. The bond force $\Delta T_s$ causing the additional tension forces in the longitudinal column bars in the joint core can be derived using the neutral axis depth of the column as shown in Fig. 8.13.
Steel Stresses

\[ f_s < 0 \quad \text{small compressive stress} \quad \frac{M_c}{T} = 6d_b \]

\[ f_s = 0.5f_y \]

Steel Stresses

\[ f_s = 0 \]

\[ f_s = 0.75f_y \]

Bond Stresses

\[ u'_{\text{omax}} = 0.6\sqrt{f_c} \]

(c) When \( f_s = 0.75f_y \)

Bond Stresses

\[ u'_{\text{omax}} = 0.8\sqrt{f_c} \]

(d) When \( f_s = f_y \)

Fig. 8.12 Bond Stress Distributions along the Beam Bars in the Joint without Shear Reinforcement

Steel Stresses

\[ f_s \]

to tension force of column bars

\[ u_{o,\text{max}} = \frac{2h_c}{(h_c + c)} u_o \]

Bond Stresses

\[ u_o = \frac{(f_s - f_e) \lambda_e}{\pi d_b h_c} \]

Fig. 8.13 Assumed Bond Stress Distribution along the Beam Bars for Test Specimens
8.4.4.3 Stress Distributions along the Beam and Column Bars in the Joints of the Test Specimens

Fig.8.14 shows the method which can be used to calculate the main beam bar steel stresses within the joint, which takes into account the effect of the joint shear force on the beam bar forces in the joint core. The beam bar stresses at the column face, $f_s$ and $f_a$ can be obtained from section analysis. The theoretical stress distribution caused by flexure alone within the joint can be estimated using the assumed bond stress distribution mentioned in the previous section (shown in Fig.8.13 or Fig.8.14(b)) and is illustrated in Fig.8.14(c). The beam bar stresses are distributed linearly over the region subjected to column flexural compression forces and parabolically over the region subjected to column flexural tension forces, as shown in Fig.8.14(c). The additional tension stresses may be induced in the main beam bars in the joint core to balance the column bond force $\Delta T_s$ and a diagonal compression force (see Fig.8.10(c)). The column bar bond force $\Delta T_s$ can be calculated assuming the similar bond stress distribution along the column bars to that along the beam bars as shown in Fig.8.14(b). The additional tension stresses so obtained are shown in Fig.8.14(d), indicating larger tensile stresses along the beam bars in the joint core when compared with those by flexure alone.

The measured strains along the column and beam bars in the joint of the test specimens were converted to bar stresses, namely the measured stresses. The results of the bar stress distributions are shown in Fig.8.15. Only the bar stresses, when the maximum strengths of the test specimens were attained, are plotted. Also plotted are the theoretical stress distributions caused by flexure alone, and those predicted by the method which takes into account the effect of the joint shear force on the column and beam bar forces in the joint core mentioned in Section 8.4.2, referred to as "predicted stresses". For the predicted stresses, the additional tensile stresses due to the bond force $\Delta T_s$ or $\Delta T_a$ were assumed to be the same in all beam or column bars.

As shown in Fig.8.15, the stresses calculated from the readings of the strain gauges were significantly larger than those calculated as due to flexure alone. Even when bond deterioration along the main beam bars is assumed over the region subjected to column flexural tension forces, the large tensile stresses measured in the joint core cannot be explained by flexure alone. On the other hand, the predicted stress profiles, which took into account the additional tensile stresses induced by the joint shear forces $\Delta T_s$ or $\Delta T_a$, approached the measured values. The main difference between the measured and predicted stress profiles can be found at the beam or column faces where the bars are to be in compression due to flexure alone. For Specimen O1, the predicted column bar stresses reached the yield stress in the joint core as well as at the beam face. For the predicted beam bar stresses obtained for Specimens O4 and O5, the yield stress was reached both in the joint core and at the column.
Fig. 8.14 Method to Calculate the Beam Bar Stresses in a Joint
Fig. 8.15(a) Stress Distributions along Beam and Column Bars in the Joint for Specimen O1
Fig. 8.15(b) Stress Distributions along Beam and Column Bars in the Joint for Specimen O4
Fig. 8.15(c) Stress Distributions along Beam and Column Bars in the Joint for Specimen C
face. It should be noted that in order to develop the ideal strengths of the test specimens, the maximum value of the predicted steel stresses in the joint core is approximately 125% of the yield stress although it is shown as 100% of the yield stress in Fig.8.15. This means that the column or beam bars in the joint core yield in tension before developing the column or beam plastic hinges. However, if the steel forces transmitted into the joint core had been assumed to be smaller, the additional tensile stresses induced by the joint shear might have been reduced. As mentioned earlier, the predicted stresses at the beam or column faces were given by section analysis. When assuming the bond force distribution as illustrated in Fig.8.13, large bond forces must be generated in the regions subjected to flexural compression force of the column. For example, the bond stress along the beam bar over that region for Specimen O1 can be estimated to be $2.0\sqrt{f_c}$ MPa, where $f_c$ is the measured compressive cylinder strength of concrete. If such large bond stresses can not be developed, the "compression" reinforcement may be in tension at the column face. This may result in a reduction of the steel forces transmitted into the joint, reducing the additional tensile stresses in the column and beam bars in the joint core.

For the test specimens, the horizontal joint core shear stresses $v_{jh}$ was approximately $0.6\sqrt{f_c}$ MPa when the ideal storey horizontal load strengths were reached. If the joint shear force had been much larger than that developed in the test specimens, the column or beam bars might have yielded in the joint core before developing the ideal flexural strength of the column or beam. This phenomenon was found in the test on the specimen conducted by Blaikie 1988. Blaikie's test results showed that the test specimen without horizontal joint shear reinforcement developed only 70% of the beam flexural strength due to the beam bar yielding not at the column face but in the joint core. The maximum nominal horizontal shear stress in the joint core of the test specimen was $1.0\sqrt{f_c}$ MPa. It may be expected that the effect of the joint shear force on the stresses induced in the steel bars in the joint core was quite large for Blaikie's specimen when compared with the specimens tested for this study. This may result in significantly large tensile stresses in the beam bars in the joint core.

It has been shown from the predicted stress profiles along the longitudinal beam and column bars in the joint that the shear strength of the joint without shear reinforcement may be governed by the column or beam bar yielding in the joint core and that the large bond stresses are developed in the flexural compression zones of the joint after diagonal tension cracking occurs.
8.4.5 Joint Shear Strength

8.4.5.1 Joint Shear Strength When Governed by Diagonal Compression Failure

It is evident that when significant bond deterioration initiates, the joint shear forces are transmitted mainly by means of the diagonal compression strut mechanism. In that case, diagonal compression failure will control the strength of the joint. Also, when bond stresses are low and significant shear is transferred by a diagonal compression field acting with a large quantity of joint shear reinforcement, diagonal compression may control the strength of the joint.

It has been widely recognized that the presence of tensile strains in the horizontal and/or vertical directions in the joint reduce the diagonal compressive strength of the concrete[Stevens et al 1991]. When the joint shear force is large, significant diagonal tension cracking in both direction will occur in the joint core (see Fig. 8.16), particularly when shear reinforcement is not present in the joint core. Under reversed cyclic loading in the inelastic range, as a consequence of earthquake forces, the diagonal tension cracks become large and disintegration of the concrete begins because of the repeated opening and closing of the cracks along which shear sliding movements occur. This is associated with drastic volumetric increase in the joint core concrete unless adequate confinement is provided. This phenomenon is likely to further reduce the diagonal compressive strength of the concrete.

8.4.5.2 Maximum Joint Shear Strength When Governed by Diagonal Compression

A rational approach to predict the shear behaviour of members, using the compression field theory was developed by Collins and Mitchell 1980. This theory has been applied in Canada to the design of beams and columns for shear and torsion[CSA 1984]. With the aim of preventing the crushing of concrete due to diagonal compression before the yield of the transverse and longitudinal bars, some useful design charts were derived from the work of Collins and Mitchell to define the limits of the angle of the diagonal compressive stresses in the concrete \( \alpha \) for a given level of nominal transverse shear stress and for given tensile strains[Collins and Mitchell 1980]. In a somewhat simplified form the derivation gives

\[
10^o + \frac{35 (V_{jh} / f'c)}{0.42 - 50e_i} < (90^o - \alpha) < 80^o - \frac{35 (V_{jh} / f'c)}{0.42 - 65e_i}
\]

where \( e_i \) and \( e_l \) are tensile strains in column transverse and longitudinal directions, \( 90^o - \alpha \) is angle of inclination of the diagonal compressive stresses to the longitudinal axis of the column.
Fig. 8.16 Observed Cracking of the Joint without Shear Reinforcement, Specimen O4

Fig. 8.17 Diagonal Compression Field in a Joint
in degrees, \( \alpha \) is the angle of inclination of potential failure plane in the joint to horizontal (see Fig. 8.17), \( \nu'_{\text{jh}} \) is nominal transverse shear stress and \( f'_c \) is compressive strength of concrete.

The value of \( \varepsilon_t \), \( \varepsilon_l \) and \( \alpha \) need to be estimated in order to determine from Eq. 8.10 the nominal horizontal joint shear stress at the stage of diagonal compression failure of the joint core concrete. When columns are expected to remain in the elastic range, the right hand side of Eq. 8.10 will govern since \( \varepsilon_t > \varepsilon_l \). When the aspect ratio of the joint is close to one, that is, \( h_c = h_b \), and no axial compression load is applied to the column, the value of the angle of \( \alpha \) will be close to 45 degree (see Fig. 8.17), where \( h_c \) is overall depth of column and \( h_b \) is overall depth of beam. When the joint has little or no shear reinforcement, the tensile strain in the transverse direction of the diagonal compression strut at the stage of joint shear failure will be much larger than that for a well-designed joint.

Fig. 8.18 plots the relationship between the maximum nominal horizontal joint shear stress \( \nu'_{\text{jh}} \) and the compressive strength of the concrete cylinder measured for the six beam-interior column specimens without shear reinforcement tested by other researchers [Hanson and Corner 1972, Bessho et al 1986, Blaikie 1988, Pessiki et al 1990 and Kawachi et al 1992]. The nominal horizontal joint shear stress \( \nu'_{\text{jh}} \) was defined as \( \nu'_{\text{jh}} = V_{\text{jh}} / (b_j d_c) \), where \( V_{\text{jh}} \) is the horizontal joint shear force, \( b_j \) is the effective width of the joint and \( d_c \) is the effective depth of the column. The maximum joint shear stresses of all the specimens were reached before the theoretical flexural strengths of the beams were attained, except for the specimen of Hanson and Corner 1972. Although the available test data is limited, the maximum nominal joint shear stress increases almost in proportion to the measured compressive strength of concrete. This indicates that for these specimens the maximum joint shear stress was strongly affected by the diagonal compression failure of the joint core concrete [Stevens et al 1991]. Based on this limited test data, the following equation could be derived to give the lower bound for the test results.

\[
\nu'_{\text{jh}} = 0.19 f'_c
\]  

(8.11)

When the above Eq. 8.11 is used to estimate the nominal joint shear stress at diagonal compression failure, it is found from Eq. 8.10 that the transverse tensile strain \( \varepsilon_t \) in the joint without shear reinforcement is approximately 0.35%.

In Eq. 8.11, the nominal shear stress \( \nu'_{\text{jh}} \) was calculated using the effective depth of the column \( d_c \) in stead of its full depth \( h_c \). Assuming \( d_c = 7/8(0.9 h_c) \) as a typical value for \( d_c \), it is found as shown in Fig. 8.19, that the limiting value for \( \nu_{\text{jh}} \) expressed by \( \nu_{\text{jh}} = V_{\text{jh}} / (b_j h_c) \), where \( V_{\text{jh}} \) is the horizontal joint shear force, \( b_j \) is the effective width of the joint, \( h_c \) is the overall column depth, is
Fig. 8.18 Joint Shear Stress $v_{jh}$ versus Concrete Compressive Strength Relationship

Fig. 8.19 Joint Shear Stress $v_{jh}$ versus Concrete Compressive Strength Relationship
\[ v_{jh} = 0.17 f_c \] (8.12)

When the maximum joint shear stress is traditionally assumed to be in proportion to \( \sqrt{f_c} \), Eq. 8.12 can be replaced by the following more conservative equation for when the concrete compressive strength is greater than 30MPa (see Fig. 8.19).

\[ v_{jh} = 1.0 \sqrt{f_c} \] (8.13)

### 8.4.5.3 Degradation of Joint Shear Strength When Governed by Diagonal Compression

As mentioned before, the upper limit of joint shear strength depends on the diagonal compressive strength of the joint core concrete. Of particular interest is the deterioration of joint shear strength under seismic forces. Diagonal tension cracking of the joint core in alternating directions due to seismic loading will reduce the diagonal compressive strength of the concrete. Therefore joint shear strengths may degrade as the imposed displacement ductility factor of the structure increases.

It has been quantified by Vecchio and Collins 1986 that the reduction of the compressive strength of the concrete in the direction of the principal compressive stress in the concrete is a function not only of the principal compressive strain, but also of the coexisting principal tensile strains, in which the strains are defined in terms of average values over the distances large enough to include several cracks. This reduction will be significant in a joint without shear reinforcement under earthquake loading because reversed cyclic loading will cause the principal tensile strain in the joint core to continue to increase with each cycle. This means that the diagonal compressive strength of the joint core concrete will decrease with each cycle until eventually failure may occur by concrete crushing.

The effect of the principal tensile strain in the joint core concrete on the joint shear strength can be assessed using the test results obtained from the specimens. The principal tensile strains were determined from the joint diagonal deformations measured during the test. For the four specimens, the joint aspect ratio was approximately 1.0 and no axial load existed on the columns, so that the directions of the joint diagonals were almost perpendicular to each other and each direction of the joint diagonals approximately coincided with the critical diagonal tension cracks in the joint. The measurements along the joint diagonals will give the principal tensile strains only when the direction of the critical crack is normal to principal tensile strain direction.

Fig. 8.20 shows the relationship between joint shear stress expressed in terms of \( f_c \) and principal tensile strains obtained from the measurements of the diagonal deformation of the
Fig. 8.20 Joint Shear Stress versus Principal Tensile Strain Relationship

Fig. 8.21 Joint Shear Stress versus Displacement Ductility Factor Relationship
joint. The principal tensile strains continued to increase with an increase in the displacement ductility factor DF, irrespective of the joint shear stresses. A maximum principal tensile strain of up to 0.7% was measured for Specimen R2 with a maximum joint shear stress of 0.05f_c without strength degradation being observed up to a DF of 8. On the other hand, the other specimens with a larger joint shear stress showed degradation of joint shear strength with increasing DF as shown in Fig.8.20. Strength degradation began at a principal tensile strain of about 1%, independent of the joint shear stress. The maximum principal tensile strain measured for Specimen R2 was not large enough to cause strength degradation. It is likely that the degradation of the joint shear strengths is indicated by an increase in the principal tensile strains.

It can be clearly seen in Fig.8.20 that the larger the joint shear stress, the larger the increase in principal tensile strain with constant DF. The values measured in the loading to DF of 2 are shown by the shaded area. The principal tensile strains obtained in that loading stage became significantly larger as the joint shear stress increased. This will result in more rapid strength degradation for the specimens with the larger shear stress induced in the joint core. The critical principal tensile strain of approximately 1% was reached in the loading to DF of 1 for the specimen with the maximum joint shear stress of 0.18f_c and to DF of 4 for the specimens with the joint shear stresses of 0.07f_c to 0.11f_c.

Fig.8.21 plots the relationship between the joint shear stress and the displacement ductility factor DF for the test specimens studied. The seismic behaviour of these specimens without shear reinforcement are classified into the following three categories. When the maximum joint shear stress v_{jh} is less than 0.05f_c, the joint did not fail in shear up to a displacement ductility factor DF of 8, and the joint behaviour did not affect the ductility of the adjacent members in which a flexural plastic hinge was developed. At a joint shear stress v_{jh} of 0.17f_c, joint shear failure initiated at a displacement ductility factor DF of 1, followed by rapid strength degradation as shown in Fig.8.21. When the joint shear stress v_{jh} was 0.07f_c to 0.11f_c, joint shear failure initiated during the loading to DF of 4 or 6 and the joint shear strength degraded moderately.

Based on the test data mentioned above, a model shown in Fig.8.22 is proposed for shear strength degradation of the joints without shear reinforcement. The test data is shown by solid circles and linear interpolation was used between the test data. As mentioned before, the maximum attainable shear stress of the joint without shear reinforcement is estimated to be 0.17f_c in terms of nominal horizontal joint shear stress. The proposed model indicates that the larger the joint shear stress v_{jh}, the more rapid the strength degradation. The available displacement ductility factor is 1 at a joint shear stress v_{jh} of 0.17f_c, while at a joint shear stress v_{jh} of less than 0.05f_c the available displacement ductility factor is at least 8. The proposed model is based on the results from beam-column joint specimens tested without
axial load acting on the columns. The effect of the axial load on the degradation of the joint shear strength needs to be investigated in future research.

The presence of joint hoops will restrict the increase in principal tensile strains in the joint core, resulting in delay in the strength degradation. However, when the quantity of joint hoops is not sufficient for the hoops to remain in the elastic range during seismic loading, the joint shear strength may degrade in the fashion shown in Fig.8.22.

8.5 SEISMIC BEHAVIOUR OF BEAMS WITH SMALL QUANTITIES OF TRANSVERSE REINFORCEMENT

8.5.1 Introduction

Transverse reinforcement is required in members to provide confinement of compressed concrete, restraint against buckling of longitudinal compression reinforcement and shear resistance. Inadequate quantities and detailing of transverse reinforcement are often found in the members of early reinforced concrete building frames. This may result in a reduction in the flexural ductility and shear failure of the members. This section examines the seismic behaviour of the beams with small quantities of transverse reinforcement in terms of curvature ductility and shear strength.
8.5.2 Curvature Ductility of Beam Sections

8.5.2.1 General

In spite of the poor ductile detailing in the plastic hinge region, the available curvature ductility factors of the beams obtained from conventional section analysis can be relatively large. As mentioned in Chapter 2, a section analysis showed that the curvature ductility factor of larger than 10 can be achieved for the typical beam section of the building frame being currently investigated. In this analysis, the maximum compressive strain of concrete was assumed to be $\varepsilon_{cu}=0.004$. Experimental evidence obtained from Specimens R3 and O6 also demonstrated the large curvature ductility capacities of the beams, provided that beam shear failure could be avoided.

It was found that in early building frames beam bars of large diameter often pass through columns of relatively small depth. Hence the anchorage of the beam bars in the joint cores may be poor. During severe earthquake loading, the plastic hinges in beams normally form near the beam-column joints. In such case, the beam bars may be in tension through the joint and the "compression" reinforcement of the beam on one side of the column may be actually in tension. Hence that steel will not act as "compression" reinforcement. This has been demonstrated by the results obtained from the tests on the beam-interior column joints without shear reinforcement conducted in this study. When the "compression" reinforcement is in tension, the available curvature ductility factor may be reduced. This section examines the effect of the stress conditions of the "compression" reinforcement on the curvature ductility capacity of the beam. The available curvature ductility capacities of the beams, taking into account the possible stress conditions of the "compression" reinforcement, are presented.

8.5.2.2 Calculation of Curvature Ductility Factors

The curvature ductility factor is expressed as $\phi_u/\phi_y$, where $\phi_y$ is the curvature when the tension reinforcement reaches the yield strain $f_y/E_s$, $\phi_u$ is the ultimate curvature when the concrete compressive strain in the extreme fibre reaches a specified limiting value, $f_y$ is the yield strength of steel and $E_s$ is modulus of elasticity of steel. The compressed concrete in the beam was treated as unconfined since typically only a small amount of transverse reinforcement was placed in the plastic hinge regions of the members of old building frames. The value for limiting concrete compressive strain was conservatively assumed to be $\varepsilon_{cu}=0.004$ [Scott et al 1982].

Fig.8.23 shows the strain and stress diagrams of a beam section at stages corresponding to the first yield and ultimate curvatures. It is assumed in Fig.8.23 that plane sections remain plane after bending except that the strain in the "compression" reinforcement is not governed
Fig. 8.23 A beam section with flexure in which both the top and bottom reinforcement are in tension.

by that section behaviour. For a given neutral axis depth $c$, the curvature at first yield $\phi_\gamma$ and the curvature at ultimate $\phi_u$ are calculated by (see Fig. 8.23)

\[
\text{Curvature at first yield: } \phi_\gamma = \frac{f_y}{E_s} \frac{1}{d - c} \quad (8.14)
\]

\[
\text{Curvature at ultimate: } \phi_u = \frac{\varepsilon_{cu}}{c} \quad (8.15)
\]

where $d$ is the depth from extreme compression fibre to the centroid of the tension reinforcement. The neutral axis depths at first yield and at ultimate can be found from analysis by satisfying the conditions of equilibrium for internal forces in the section and the compatibility of strains for a given strain of "compression" reinforcement. The concrete compression force for a given concrete strain in the extreme compression fibre can be obtained from the stress-strain relationship of the concrete. For chosen strains in the top and bottom reinforcement, the steel tensile forces can also be determined from the stress-strain relationship of the reinforcement. The stress-strain curve for unconfined concrete was expressed by the Kent and Park model [Kent and Park 1971], taking into account the nonlinear behaviour of the unconfined compressed concrete before and after yielding of the tension reinforcement. The stress-strain curve for longitudinal reinforcement was expressed by a bi-linear relation and did not take into account the strain hardening.
8.5.2.3 The Effect of the Stress Level of Compression Reinforcement

The effect of the stress level in the "compression" reinforcement on the neutral axis depths, curvatures, moment capacities and curvature ductility capacities was investigated for a typical beam cross section of the building frame investigated.

Fig. 8.24 shows the typical beam cross section. The bottom and top reinforcement ratio $\rho$ and $\rho'$ was 0.67% and 1.34%, respectively. The steel yield strength $f_y$ was assumed to be 300MPa while the concrete compressive strength was assumed to be 30MPa.

The relationship between the neutral axis depth expressed as $c/d$ and level of stress in the "compression" reinforcement $f_1/f_y$ is illustrated in Fig. 8.24, where $c$ is the depth of the neutral axis from the extreme compression fibre, $d$ is the distance from the extreme compression fibre to the centroid of the tension reinforcement, $f_1$ is the stress in the "compression" reinforcement and $f_y$ is the steel yield strength (see Fig. 8.23). The stress in the "compression" reinforcement is positive if in tension and negative if in compression. This means that the stress in the "compression" reinforcement $f_1/f_y$ becomes more tensile as the bond along the main beam bars in the joint deteriorates. It should be noted that for the given beam section the neutral axis depth at ultimate calculated for positive moment is small so that the neutral axis lies above the "compression" reinforcement. Therefore, the "compression" reinforcement was in tension even when the perfect bond was assumed along the beam bars in the joint.

As could be expected, the neutral axis depths at yield and ultimate increased as the tensile stress in the "compression" reinforcement was increased. This trend became more obvious for positive moment in which the amount of the "compression" reinforcement was larger. When the tensile stress in the "compression" reinforcement approached the steel yield strength, that was $f_1/f_y = 1.0$, the neutral axis depth during positive and negative moment became almost the same. When the tensile stress in the "compression" reinforcement reached the yield strength, the neutral axis depth was increased by 30 to 90% at yield and 100 to 170% at ultimate, respectively, when compared with those obtained for the perfect bond condition along the main beam bars.

Fig. 8.25 shows the relationship between yield and ultimate moment and stress level in the "compression" reinforcement. The yield and ultimate moments decreased as the tensile stress in the "compression" reinforcement was increased. However, the effect of the stress level in the "compression" reinforcement on the ultimate moment was not so significant. When the tensile stress level in the "compression" reinforcement $f_1/f_y$ was 1.0, the decrease in the ultimate moment was 10% for positive moment and 5% for negative moment, respectively, when compared with those with the perfect bond condition along the beam bars.
At Yield

Neutral Axis Depth c/d

Stress Level of "Compression" Reinforcement $f_1/f_y$ (tension in positive)

Perfect Bond

At Ultimate

Neutral Axis Depth c/d

Stress Level of "Compression" Reinforcement $f_1/f_y$ (tension in positive)

Beam section

Fig. 8.24 Neutral Axis Depth Related to Stress Level in the "Compression" Reinforcement
Yield Moment

Stress Level of "Compression" Reinforcement $f_l/f_y$ (tension in positive moment)

Ultimate Moment

Stress Level of "Compression" Reinforcement $f_l/f_y$ (tension in positive moment)

Fig. 8.25  Moment Capacities Related to Stress Level in the "Compression" Reinforcement
Fig. 8.26  Curvature Related to Stress Level in the "Compression" Reinforcement
Fig. 8.26 illustrates the effect of the stress level in the "compression" reinforcement on the yield and ultimate curvatures. As the tensile stress level in the "compression" reinforcement was increased, the yield curvature increased. In contrast, the value of the ultimate curvature decreased. This is because of the increase in the neutral axis depth, caused by the tensile stress induced in the "compression" reinforcement. The effect of the stress level in the "compression" reinforcement was more obvious when the "compression" reinforcement ratio was increased, as during positive moment. When the bond along the main beam bars in the joint was completely destroyed, which was the case when the stress level in the "compression" reinforcement was $f_l/f_y=1.0$, the yield curvature increased by 25 to 50% and the ultimate curvature decreased by 40 to 50% for the given beam cross section. When the stress in the "compression" reinforcement approached the yield strength, the yield and ultimate curvatures obtained during positive moment became close to the values obtained during negative moment.

The relationship between the curvature ductility factor and the stress level in the "compression" reinforcement is shown in Fig. 8.27. As shown in this figure, the curvature
ductility factor was significantly reduced when the tensile stress in the "compression" reinforcement was increased, especially during positive moment. This trend is a result of changes in the neutral axis depth at yield and ultimate as described before. An available curvature ductility factor of larger than 10 obtained for the perfect bond condition of the "compression" reinforcement approached approximately 5 for the given beam section as the tensile stress in the "compression" reinforcement was increased. When the tensile stress in the "compression" reinforcement was 40% of the yield strength, the curvature ductility factor was reduced to approximately one half of that for when the perfect bond was assumed for the main beam bars.

8.5.2.4 Curvature Ductility Factors of Beam Sections Taking into Account the Bond Conditions along the Main Beam Bars in the Beam-Interior Column Joints

In order to obtain a realistic estimate of the available curvature ductility capacity of a beam section, it is necessary to estimate the stress level in the "compression" reinforcement of the beam adjacent to the joint. The stress level in the "compression" reinforcement depends on the ratio of the column depth to the beam bar diameter, the stress level in the tension reinforcement and the bond stress conditions along the longitudinal reinforcement in the joint. As mentioned before, the bond stress distribution along the main beam bars in the joint without shear reinforcement was expressed as shown in Fig. 8.28, when the axial load acting on the column was zero, after the tension reinforcement reached the yield strength and diagonal tension cracking was initiated. The bond stress distribution so obtained was based mainly on the experimental results in this study. By using the bond stress distribution along the longitudinal reinforcement in the joint without shear reinforcement illustrated in Fig. 8.28, the tensile stress in the "compression" reinforcement can be calculated for the given strengths of the concrete and longitudinal reinforcement. Fig. 8.28 shows the tensile stress in the "compression" reinforcement plotted against the ratio of the column depth to the beam bar diameter \( \frac{h_c}{d_b} \) when the concrete compressive strength and the steel yield strength were assumed to be 30MPa and 300MPa, respectively, where \( h_c \) is the column overall depth and \( d_b \) is the beam bar diameter. When the ratio of the column depth to beam bar diameter is small, large tensile stress can be expected to be generated in the "compression" reinforcement.

Figs. 8.29(a), (b) and (c) illustrate the curvature ductility factors of beam sections, plotted against the tension reinforcement ratio \( \rho \) for a practical range of ratio of column depth to beam bar diameter \( \frac{h_c}{d_b} \) and ratio of top reinforcement area to bottom reinforcement area \( \rho' / \rho \). The beams have a concrete compressive strength of 30MPa and a steel yield strength of 300MPa. The available curvature ductility factors decreased when the tension reinforcement ratio \( \rho \) was increased. When \( \rho \) was larger than 1.5%, the available curvature ductility factor was reduced to be less than 5 for the range of the ratio \( \frac{h_c}{d_b} \) of 12.5 to 25. When \( \rho \) was
Fig. 8.28 Tensile Stress in the "Compression" Reinforcement Related to $h_c/d_b$

$\frac{f_c}{f_y} = 30 \text{MPa}$

$\rho = \rho_b/(bd)$

$\rho' = \rho_b/(bd)$

Beam section

$\rho'/\rho = 1.0$

Fig. 8.29(a) Variation of Curvature Ductility Factor with $\rho$ ($\rho'/\rho = 1.0$)
$\rho'/\rho=1.5$ (Positive Moment)

Curvature Ductility Factor

$\rho (%)$

$\rho'=A_s'/bd$

$\rho=A_d/bd$

Beam section

$\rho'/\rho=1.5$ (Negative Moment)

Curvature Ductility Factor

$\rho (%)$

Fig. 8.29(b) Variation of Curvature Ductility Factor with $\rho$ ($\rho'/\rho=1.5$)
$\rho'/\rho = 2.0$ (Positive Moment)

$\rho = \frac{A_s}{bd}$

Beam section

$\rho'/\rho = 2.0$ (Negative Moment)

Fig. 8.29(c) Variation of Curvature Ductility Factor with $\rho$ ($\rho'/\rho = 2.0$)
0.5% and \( \rho'/\rho \) was less than 1.5, an available curvature ductility factor larger than 10 was attained, irrespective of the column depth to beam bar diameter ratio. When the ratio of the column depth to beam bar diameter was decreased and \( \rho=0.5\% \), the curvature ductility factor significantly decreased, especially during positive moment, indicating that the larger the "compression" reinforcement ratio, the larger the reduction of the available curvature ductility factors of the beam sections. When \( \rho \) was larger than 1.0\%, the effect of the ratio \( h_c/d_b \) on the curvature ductility factor became insignificant in the range of \( h_c/d_b \) of 12.5 to 25. As Fig.8.29 indicates, the available curvature ductility factors were about 10 for beam sections with \( \rho=0.5\% \) and about 5 for those with \( \rho=1.0\% \). Much smaller available curvature ductility factors were found for beam sections with \( \rho \) larger than 1.0\%.

It was found from the dynamic analysis of the building frames designed in the late 1950's discussed in Chapter 2 that the maximum curvature ductility demand of the beams was about 10 under the severe earthquake motion. The ratio of the column depth to beam bar diameter \( h_c/d_b \) is typically in the range of 12.5 to 19 for the building frames. In such case, only a beam with tension reinforcement ratio \( \rho \) less than 0.5\% can survive the earthquake. The effect of the tensile stress in the "compression" reinforcement due to bond deterioration along the beam bars in the joint on the available curvature ductility factor of the beam section will be critical during positive moment since "compression" reinforcement ratio is larger.

It should be mentioned that the bond stress distribution along the beam bars in the joint shown in Fig.8.28 was obtained from the results of tests on the beam-interior joint specimens without axial load on the columns. Usually the columns in the building frame are subjected to axial compression load. In such case, much better bond condition can be expected for the beam bars through the joint due to the transverse compression force acting on the beam bars in the joint[Taylor and Clarke 1976 and Eliehehausen et al 1983]. Therefore, the "compression" reinforcement may not be stressed in tension as significantly as indicated above, resulting in the available curvature ductility factors of the beam sections being larger than those calculated above. It should also be mentioned that the value assumed for the limiting concrete compressive strain in the extreme fibre at ultimate, \( \varepsilon_{cu}=0.004 \), is a lower bound for the strain at crushing of the unconfined concrete[Scott et al 1982]. If a value higher than 0.004 is used in the ultimate curvature calculation, a greater flexural ductility will be obtained since the ultimate curvature depends very much on the value of the extreme fibre strain. Further research is necessary into these aspects.
8.5.3 Seismic Shear Strength of the Beam

8.5.3.1 Previous Models

The available curvature ductility factors described in Section 8.5.2 are applicable only if buckling of compression reinforcement in the beams and shear failure of the beams, columns and joints can be prevented under seismic loading. Shear strength in plastic hinge regions degrades as the ductility demand increases, due to reduced shear carried by the concrete shear resisting mechanisms. Extensive research has been conducted to establish a shear design procedure which enables the relationships between shear strength and displacement ductility factor to be obtained for columns [Ang et al 1988, Priestley and Calvi 1991, Aschheim and Moehle 1992, Wong et al 1993, and Priestley et al 1993]. Those relationships may be used to evaluate the potential shear failure and the available displacement ductility factor in conjunction with the shear demand of the member. This section briefly reviews the proposed models for shear strength. Predictions of the shear strength from those models are compared with the results of tests on Specimen R3, in which the beams failed in shear during beam negative moment. A modification to the concrete shear resisting mechanisms of an existing model [Priestley and Calvi 1991] is recommended for estimating the beam shear strength.

8.5.3.2 Current Design Code Equations for Shear Strength

Current design codes [ACI 318 1989 and SANZ 1982(a)] assume that all the shear reinforcement across a shear failure plane with an angle of 45 deg to the member axis reaches the yield strength, and the shear carried by the shear reinforcement $V_s$ can be expressed by

$$V_s = \frac{A_v f_{yv} d}{s} \hspace{1cm} (8.16)$$

where $A_v$ is the area of shear reinforcement at spacing $s$, $f_{yv}$ is the yield strength of shear reinforcement and $d$ is the effective depth of the member.

The prediction of the shear resisted by the 45 degree truss model is usually conservative, particularly for beams with a small amount of shear reinforcement. Consequently, it has become accepted design practice to add an empirical correction term to the 45 deg truss equation. The correction term is commonly referred to the "concrete contribution" $V_c$ and is taken as the shear at the commencement of diagonal tension cracking.

For members without axial load, the ACI and the New Zealand Standards Association express the "concrete contribution" as follows:
ACI simplified equation  \[ V_c = 0.17 \sqrt{f_c} bd \text{ (MPa)} \]  (8.17)

NZ equation  \[ V_c = (0.07 + 10 \rho_w) \sqrt{f_c} bd < 0.2 \sqrt{f_c} bd \text{ (MPa)} \]  (8.18)

where \( f_c \) is the concrete compressive strength, \( b \) is the member width, \( d \) is the effective depth of the member and \( \rho_w \) is the longitudinal tension steel ratio.

The nominal shear strength \( V_n \) is then given by an additive equation as follows:

\[ V_n = V_c + V_s \]  (8.19)

It should be noted that Eqs. 8.17 to 8.19 refer to regions of members outside plastic hinge regions. For within plastic hinge regions, both the ACI and NZ codes define reduced values for the concrete components \( V_c \).

Recently several design codes[CSA 1984, CEB-FIP Code 1990, and AIJ 1990] have adopted a more rational approach, using "diagonal compression field theory"[Collins and Mitchell 1980] or "plastic theory"[Nielsen 1984] for predicting the shear strength, allowing a wide range of values for the permissible angle of inclination of the principal compressive stress to the axis of the member. However, it has not yet been considered qualitatively by those design codes, except for the AIJ approach, that shear strength at plastic hinges degrades as the displacement ductility demand increases[AIJ 1990].

8.5.3.3 Proposed Models for Predicting the Seismic Shear Strength

Current design code equations cannot predict the real shear strength of a member since they are intended to provide a conservative estimate of the shear strength for safety. Besides, they do not clearly indicate the influence of the displacement ductility factor on the shear strength of the member. Considerable experimental and analytical research[Ang et al 1988, Priestley and Calvi 1991, Aschheim and Mochle 1992, Wong et al 1993, and Priestley et al 1993] has been carried out to propose more realistic shear strength equations which are related to the displacement ductility factor. Most of the proposed models express the degradation of shear strength as a reduced shear carried by the concrete mechanisms \( V_c \) due to reversal cyclic loading at the plastic hinges.

Model by Ang et al

Ang, Priestley and Paulay 1988 reviewed the existing U.S. and New Zealand design expressions for the shear strength of circular columns and compared them with the results from a comprehensive test programme involving 25 circular columns. It was identified by their work that current U.S. and New Zealand design equations for the concrete contribution are
very conservative. Based on the experimental results, new design equations were suggested for the initial shear strength \( V_i \) as follows:

\[
V_i = V_{ci} + V_{si}
\]

\[
V_{ci} = 0.37 \alpha (1 + \frac{3P}{f_c' A_g}) \sqrt{f'_c A_e} \text{ (MPa)}
\]

where \( \alpha = \frac{2}{(M/VD)>1.0} \)

\[
V_{si} = \frac{\pi}{2} A_v f_{yy} D' / s
\]

where \( V_{ci} \) is the initial shear strength carried by concrete, \( V_{si} \) is the initial shear strength carried by shear reinforcement, \( f_c' \) is the concrete compressive strength, \( M/VD \) is the column aspect ratio, \( D \) is the gross column diameter, \( D' \) is the diameter of confined core, \( A_g \) is the gross cross sectional area, \( A_e \) is the effective shear area=0.8\( A_g \), \( A_v \) is the area of shear reinforcement, \( f_{yy} \) is the yield strength of the shear reinforcement and \( s \) is the spacing of shear reinforcement.

For the final shear strength after degradation, the concrete contribution of the initial shear strength was reduced. For the shear carried by shear reinforcement, a lower bound plastic theory solution was used to estimate the angle of the diagonal compression strut of the analogous truss mechanism while a 45 degree analogous truss mechanism was used for the initial shear strength. The following equations were suggested for the final shear strength \( V_f \):

\[
V_f = V_{cf} + V_{sf}
\]

\[
V_{cf} = 18.5 \rho_s f_c A_e \leq 185 \sqrt{f_c A_e} \text{ (MPa)}
\]

\[
V_{sf} = \frac{\pi}{2} A_v f_{yy} D' \sqrt{\frac{1 - \varphi}{\varphi}} \leq \frac{2.15 \pi}{2} A_v f_{yy} D' / s
\]

where \( V_{cf} \) is the final shear strength carried by concrete, \( V_{sf} \) is the final shear strength carried by shear reinforcement, \( \rho_s \) is the ratio of hoop or spiral reinforcement volumetric to the concrete core volume and \( \varphi \) is the mechanical reinforcement ratio\( (=\rho_s f_{yv}/(v f_c')) \), \( v \) is a factor for the reduced effective compressive concrete strength of the diagonal strut, \( D' \) is the diameter of confined core, \( A_e \) is the effective shear area, \( A_v \) is the area of shear reinforcement, \( f_{yy} \) is the yield strength of shear reinforcement and \( s \) is the spacing of shear reinforcement.

Maximum contributions of the concrete and shear reinforcement to the final shear strength were obtained when \( \rho_s \) is 0.01 and the angle of diagonal struts is 25 deg to the
horizontal axis. A model for shear strength degradation with increasing displacement ductility has been suggested, which was similar to one proposed by the Applied Technology Council for retrofitting highway bridges [ATC 6-2 1983]. The initial shear strength $V_1$ was assumed to apply for displacement ductility factors of up to 2. At higher ductilities, the shear strength degrades until a final value $V_f$ was attained when the flexural ductility capacity was reached. Methods for estimating the flexural ductility capacities are presented elsewhere [Priestley and Park 1987].

Recent work by Wong et al 1993 proposed more general form of Ang et al's equations, including the effect of displacement history. It was shown that the reduction of shear strength with displacement ductility was more severe under biaxial seismic attack.

**Model for Columns by Priestley et al**

On the basis of the work of Ang et al 1988 and Wong et al 1993, the model proposed by Ang et al was modified. The strength enhancement provided by axial compression was separated from the "concrete contribution" of the shear strength and considered to result from arch action [Priestley et al 1993]. The shear strength of a column was considered to consist of three independent components: the concrete component $V_c$, the axial load component $V_p$, and the shear reinforcement component $V_s$. Thus

$$V_n = V_c + V_p + V_s$$

(8.26)

The concrete component was given by

$$V_c = k \sqrt{f_c A_c}$$

(8.27)

where $k$ depends on the imposed displacement ductility factor, varying between 0.29 for the initial shear strength (MPa) and 0.1 for the final shear strength (MPa). The concrete component for the initial shear strength was applied to displacement ductility factor of up to 2 and degraded linearly to a displacement ductility factor of 4 for the final shear strength when the column is expected to be subjected to uniaxial displacement ductility demand.

The axial load component was obtained from

$$V_p = P \tan \theta_{st}$$

$$= \frac{(D - c) \cdot P}{2a}$$

(8.28)
where \( P \) is the axial load on column, \( \theta_{wi} \) is the inclination of the strut by arch action, \( D \) is the overall section depth, \( c \) is the depth of the compression zone and either \( a = L \) for a cantilever column or \( a = L/2 \) for a column in double bending, where \( L \) is the column height. Note that \( V_p \) does not degrade with increasing displacement ductility factor.

The shear reinforcement component was based on a truss mechanism with an angle of 30 deg to the vertical axis rather than 45 deg, based on the visual observation of diagonal tension cracking during the tests. The shear carried by the shear reinforcement \( V_s \) is then given by

\[
\text{For circular column} \quad V_s = \frac{2A_v f_{yy} D'}{s} \cot(30 \text{ deg})
\]

\[
\text{For rectangular column} \quad V_s = \frac{A_v f_{yy} D'}{s} \cot(30 \text{ deg})
\]

where \( A_v \) is the area of shear reinforcement, \( f_{yy} \) is the yield strength of shear reinforcement and \( D' \) is the distance between centres of hoop or spiral.

**Model by Aschheim et al**

Aschheim and Moehle 1992 reviewed the columns damaged in previous Californian earthquakes and laboratory data to determine the coefficient \( k \) in the concrete contribution for shear resistance \( V_c = k f'c A_e \), where \( f'c \) is the concrete compressive cylinder strength and \( A_e \) is the effective cross section area. The estimate of \( k \) obtained was as follows:

\[
\text{For spirally reinforced columns} \quad k = \frac{0.03 p_s f_{yy}}{\mu} \text{ (psi)}
\]

\[
\text{For rectangular reinforced columns} \quad k = \frac{0.06 p_w f_{yy}}{\mu} \text{ (psi)}
\]

where \( p_s \) is the spiral reinforcement ratio, \( p_w \) is the web reinforcement ratio, \( f_{yy} \) is the yield strength of hoop or spiral reinforcement and \( \mu \) is the displacement ductility factor.

It was assumed that the shear carried by shear reinforcement was defined by a 45 deg truss mechanism and did not change with increasing displacement ductility.

**AIJ Approach**

In Japan, a theoretical shear design method has been proposed and adopted in the recommendations of the Architectural Institute of Japan[AIJ 1990]. The method was based on the superposition of the truss and the arch mechanisms, and limiting the diagonal
compressive stress in the concrete resulting from combined truss and the arch action. The angle of the truss mechanism was estimated using a lower bound plastic theory solution with the limited value of 22.5 deg to the axis of the member. For the initial shear strength, the effectiveness factor of diagonally compressed concrete $v_0$ was obtained using the equation by Nielsen 1984 as follows:

$$v_0 = 0.7 - f'_c / 200 \text{ (MPa)}$$

(8.33)

The shear carried by truss mechanism was given by

$$V_s = \frac{A_w f_y v_j}{s} \cot(\theta_i)$$

(8.34)

$$\cot(\theta_i) = \min \left( \frac{\sqrt{(1-\varphi)}}{\varphi} , 2 , \frac{j_i}{h \tan \theta_{st}} \right)$$

(8.35)

where $j_i$ is the distance between the upper and lower stringers and for a beam with multi-layered longitudinal reinforcement it is taken as the distance between the plastic centroids of the tension and compression longitudinal reinforcement, $s$ is the spacing of the shear reinforcement, $\theta_i$ is the angle of the compression strut to member axis in truss mechanism, $\varphi$ is the mechanical reinforcement ratio ($= p_w f_y v/(v_0 f'_c)$), $p_w$ is the shear reinforcement ratio ($= A_w / b_s$), $A_v$ is the area of shear reinforcement, $b$ is the width of the section, $f_y$ is the yield strength of shear reinforcement (when $f_y > 25 f'_c$, $f_y = 25 f'_c$), $h$ is the section depth and $\theta_{st}$ is the angle of the compression strut to the member axis in arch action.

The shear force carried by the arch mechanism $V_c$ was given by

$$V_c = 0.5 b h (1 - \beta) v_0 f'_c \tan \theta_{st}$$

(8.36)

where $\tan \theta_{st} = \sqrt{(L/h)^2 + 1 - L/h}$, $\beta = [1 + \cot^2 \theta_h] p_w f_y v/(v_0 f'_c)$, $f'_c$ is the concrete compressive cylinder strength, and $L$ is the clear span of a member.

The AIJ Approach assumes that the shear carried by arch action decreases with increase in the amount of shear reinforcement.

In order to allow for the degradation of diagonally compressed concrete due to diagonal cracking in two directions under reversed cyclic loading, the effective compressive strength of concrete in a plastic hinge region was reduced in proportion to the inelastic hinge rotation angle $\theta_p$ as follows:
\[ \begin{align*}
v f_c &= (1.0 - 15R_p)v_0 f_c & \text{for } 0 < R_p < 0.05 \quad (8.37) \\
&= 0.25 v_0 f_c & \text{for } 0.05 < R_p \quad (8.38)
\end{align*} \]

where \( v \) is an effectiveness factor for hinge region and \( R_p \) is an inelastic hinge rotation angle in radians. For practical purposes \( R_p \) can be taken as the total hinge rotation angle or member rotation angle. The maximum allowable value of \( \cot \theta_i \) in truss action was reduced, taking into account the loss of interlocking action along the cracked surfaces in the plastic hinge region. The following equations were derived based on the experimental data:

\[ \begin{align*}
\cot \theta_i &= 2.0 - 50R_p & \text{for } 0 < R_p < 0.02 \quad (8.39) \\
&= 1 & \text{for } 0.02 < R_p \quad (8.40)
\end{align*} \]

**Model for Beams by Priestley**

Priestley and Calvi 1991 proposed a simple model for the concrete contribution of the beam. The concrete contribution for the initial shear strength was calculated using the New Zealand code equations[SANZ 1982(a)]. After a displacement ductility factor of 2 is imposed, the concrete contribution was reduced linearly to a displacement ductility factor of 4 where the concrete contribution was totally ignored. The shear carried by the shear reinforcement was defined using a 45 deg truss mechanism for both the initial and final shear strength calculations.

**8.5.3.4 Nominal Shear Stress in the Beam**

Fig.8.30 shows observed cracking of Specimens R3 and O6, in which the beams suffered severe diagonal tension cracking. The relationships between the nominal shear stress and displacement ductility factor obtained for the beams of Specimens R3 and O6 are illustrated in Fig.8.31.

For Specimen R3, shear failure in the beams commenced with negative moment applied when the maximum nominal shear stress in the beams reached about 0.18\( \sqrt{f_c} \), where \( f_c \) is the measured concrete compressive cylinder strength. At a beam displacement ductility factor of approximately 2, the shear strength degraded rapidly with increasing displacement ductility as shown in Fig.8.31(a). It is clearly shown in Figs.8.30(a) and 8.31(a) that the concrete contribution to the shear resistance of the beam with inadequate quantities of transverse reinforcement, namely aggregate interlock[Fenwick and Paulay 1968], across the flexural compression zone and dowel action, were significantly reduced at a displacement ductility of
(a) Specimen R3

(b) Specimen O6

Fig. 8.30 Observed Cracking at the Second Cycle to DF of -8
Fig. 8.31 Beam Shear Stress versus Displacement Ductility Factor Relationship
about 4. With positive moment applied, the maximum nominal shear stress in the beams was $0.11\sqrt{f_c}$. Shear failure did not occur with positive moment applied and a ductile response was attained for that direction of shear force as shown in Fig.8.31(a).

For Specimen O6, the maximum nominal shear stress in the beam was $0.14\sqrt{f_c}$ with negative moment applied and $0.11\sqrt{f_c}$ with positive moment applied. Only a small reduction in shear strength of the beam was observed for both directions of shear force as illustrated in Fig.8.31(b). Although severe diagonal tension cracking was observed during positive moment applied (see Fig.8.30(b)), the concrete contribution to shear resistance did not degrade significantly. Based on the limited test data obtained in this study, it was found that it was not until the maximum nominal shear stress in the beam reached approximately $0.18\sqrt{f_c}$ that the shear strength was reduced due to the degradation in the concrete shear resisting mechanisms. It should be mentioned that this nominal shear stress of $0.18\sqrt{f_c}$ is almost identical to the diagonal tension cracking stress recommended by ACI 318 1989.

8.5.3.5 Comparison of the Proposed Models with Test Data

The relationship between the shear strength and the displacement ductility factor of the beam of Specimen R3 during negative moment, in which shear failure in the beams commenced, was estimated using the proposed models described in Section 8.5.3.3 and compared with the test results in Fig.8.32. Also shown is the shear force corresponding to the ideal flexural strength $V_{lf}$ of the beam. The predicted shear strengths were calculated using the measured material strengths.

It is shown in Fig.8.32 that the models proposed for columns by Ang et al and Priestley et al overestimate the test results. On the other hand, Ashheim et al's model generally gives a conservative estimate, especially at low displacement ductility levels. The AIJ approach and Priestley et al's model for beams predict the available displacement ductility factor with good accuracy as shown in Fig.8.32, where the available displacement ductility factor is defined as when shear failure commenced. However, the AIJ approach underestimates the effect of the displacement ductility on the shear strength after the beam failed in shear. The model proposed for the beams by Priestley et al gives a good estimate for the influence of the displacement ductility factor on the shear strength.

In summary, a comparison of experimental results in this study with the proposed shear strength models indicated that only the model proposed for beams by Priestley and Calvi 1991 could provide a good estimate for the shear behaviour of the beams of the test specimen, which failed in shear during negative moment.
Specimen R3 (Negative Moment)

Displacement Ductility Factor

Beam Shear Strength (kN)

Ang et al. Model
Priestley et al. Model for Column
AD Approach
Priestley et al. Model for Beam
Ashheim et al. Model

Test Data

Fig. 8.32 Comparison of the Proposed Models with Test Data

Shear Carried by Concrete

Displacement Ductility Factor

$k = (V_{max} - V_s) / (b d f_c)$

Test Data by Iwasaki et al
Recommended Model
Model by Priestley et al. ($\rho_w > 1.3\%$)

Fig. 8.33 Degradation of the Concrete Contribution of Shear Strength
8.5.3.6 Concrete Contribution for Shear Strength of the Beam

As mentioned before, the concrete contribution for the initial shear strength in the model of Priestley and Calvi 1991 was based on the New Zealand Code equations[SANZ 1982(a)]. However, it has been shown that existing design equations are very conservative for the initial shear strength[Ang et al 1988, and Mattock and Wang 1984]. Therefore, a more realistic estimate is necessary for the concrete contribution to the initial shear strength.

The experimental results obtained by Iwasaki et al 1985 were reviewed to assess the shear carried by the concrete $V_c$. Only test data obtained from columns which failed in shear with rectangular cross section and without axial load acting were used to investigate $V_c$ for beams. The dimensions, reinforcement ratio and measured material strengths for the selected test specimens were as follows:

- column aspect ratio $2.0 < M/VD < 6.0$
- shear reinforcement ratio $0.08% < \rho_w < 0.51%$
- tension reinforcement ratio $0.88% < \rho_w < 2.12%$
- measured concrete compressive strength $25.2\text{MPa} < f_c < 33.3\text{MPa}$

The shear carried by concrete $V_c$ may be expressed by

$$V_c = k\sqrt{f_c}bd$$

(8.41)

The maximum shear strength $V_{\text{max}}$ obtained from the selected test specimens, when the axial load is zero, was defined as the sum of the contributions from the concrete $V_c$ and from the shear reinforcement $V_s$. Thus

$$V_{\text{max}} = V_c + V_s$$

(8.42)

The angle of the truss mechanism needs to be estimated to define the term $V_s$. When it is assumed that the angle of the truss mechanism coincides with the angle of diagonal tension cracking, a 45 degree truss mechanism is a good estimate for beams without axial load. Visual observation of diagonal tension cracks supported the assumption of an approximately 45 deg strut angle for beams(see Fig.8.30) and for columns without axial load[Ang et al 1988]. Hence the term $V_s$ was defined as

$$V_s = \frac{A_v f_{vyd}}{s}$$

(8.43)

where $A_v$ is the area of shear reinforcement at spacing $s$, $f_{vy}$ is the yield strength of shear reinforcement, and $d$ is the effective depth of the member.
The coefficient \( k \) in the concrete contribution \( V_c \) is then calculated as

\[
k = \frac{V_{\text{max}} - V_s}{\sqrt{f'_c} \cdot b d}
\]  

(8.44)

Fig.8.33 shows the relationship between the displacement ductility factor and the value \( k \) so calculated. Also shown is the shear carried by the concrete according to the model of Priestley and Calvi 1991.

Two significant trends are apparent in the scattered data of Fig.8.33. The first is that the concrete contribution represented by the value for \( k \) decreases considerably when the displacement ductility factor approaches 4. The second is that the shear carried by the concrete does not go to zero even when the displacement ductility factor is larger than 4.

It was identified in Fig.8.33 that the concrete contribution of the beam for the initial shear strength in the model proposed by Priestley and Calvi 1991 was very conservative. This is because the model uses the code equation which gives a conservative prediction of the initial shear strength. Although the available test data is limited, Fig.8.33 suggests that the concrete term \( V_c \) for the initial shear strength used in Priestley et al’s equation could be replaced by \( 0.3\sqrt{f'_c} \cdot b d \). It was also found that the \( V_c \) for the final shear strength could be larger than \( 0.04\sqrt{f'_c} \cdot b d \), as shown in Fig.8.33.

8.6 CONCLUSIONS

One approach to the assessment of the shear strength of beam-column joints without shear reinforcement is to assume that the shear strength is reached at the stage of initial diagonal tension cracking in the joint core. Based on the test results from eighty beam-interior column joint specimens tested by other researchers, the joint shear strengths at diagonal tension cracking were investigated in terms of the principal tensile stresses in the joint. It was found that Eq.8.5 can be used as one method to assess the shear strength of the joint without shear reinforcement.

The shear mechanisms of beam-column joints without shear reinforcement after diagonal tension cracking were developed and examined using the test results. In the model developed, it was assumed that for a joint without shear reinforcement the longitudinal beam and column bars passing through the joint core act as both flexural reinforcement and joint shear reinforcement. As a consequence, large tensile stresses were developed in these bars in the joint core, which are compared with the test results obtained from this study in Fig.8.15. The results of this phenomenon are:
1. That the horizontal and vertical expansion of the joint core may be significantly large.

2. That as can be expected from the behaviour illustrated in Fig.8.15, the bond condition in the joint core may be quite different from that in a well designed joint. It is likely that only small steel forces in the flexural tension zones can be transmitted to the core concrete by means of bond while the large bond forces must be developed in the flexural compression zones.

3. That the flexural compression reinforcement in the beams and columns adjacent to the joint may be in tension. The loss of compression force in the steel may impair the ductility of the members.

The maximum shear strength of a beam-column joint with no shear reinforcement depends on the available steel bar forces to carry the joint shear forces. The additional forces, which are induced in the longitudinal beam and column bars due to the joint shear mechanisms postulated in this study, could limit the development of the maximum flexural strengths of the members adjacent to the joint. Therefore, when assessing an existing moment resisting frame with no joint shear reinforcement, the effect of the joint shear force on the longitudinal steel bars in the joint core should be investigated.

When diagonal compression failure governs the joint strength, the maximum shear strength of the joint can be estimated using Eq.8.12. The shear strength degradation model shown in Fig.8.22 is proposed for joints without shear reinforcement and can be used to estimate the available displacement ductility factor of an existing structure when joint shear failure occurs.

The seismic behaviour of beams with inadequate quantities of transverse reinforcement was examined in terms of the curvature ductility capacity and the shear strength.

When beam bars of large diameter pass through a column with relatively small depth, as is found in early building frames, the "compression" reinforcement of the beam on one side of the column may actually be in tension. The curvature analysis of the beams, taking into account the effect of the actual stress in the "compression" reinforcement showed that the available curvature ductility factors may be significantly reduced as a result of increasing tensile stress in the "compression" reinforcement. The available curvature ductility factors were calculated to be about 10 for the beam sections with $\rho$ of 0.5% and about 5 for those with $\rho$ of 1.0%. Much smaller curvature ductility factors were available for beam sections with $\rho$ larger than 1.0%.
Those curvature ductility factor values were obtained when the extreme fibre concrete compressive strain of the concrete was taken as \(\varepsilon_{cu}=0.004\) and the bond stress distribution along the beam bars passing through the joint was assumed to be the same as that obtained from the results of tests on the beam-interior column joint specimens without axial load acting on the columns. Those assumptions may result in a conservative estimate of the available curvature ductility factors when columns do carry axial compression load.

A comparison of the experimental results in this study with previously proposed shear strength models indicates that a modification to the concrete shear resisting mechanisms is required for the model to give a realistic prediction of the shear strengths of beams. The experimental results obtained by other researchers were reviewed to assess the shear carried by the concrete \(V_c\) of the beams. It was identified that the concrete contribution of the beam for the initial shear strength in a previously proposed model was very conservative. It is suggested that the concrete term \(V_c\) for the initial shear strength be \(0.3\sqrt{f_{c}b_d}\). The concrete term \(V_c\) decreases as the displacement ductility factor of the beam increases. The shear carried by the concrete mechanisms of the beams did not become zero even when the displacement ductility factors were larger than 4. It was found that \(V_c\) for the final shear strength is larger than \(0.04\sqrt{f_{c}b_d}\).
CHAPTER 9

CONCLUSIONS AND RECOMMENDATIONS

9.1 GENERAL

This study investigated the seismic behaviour of early reinforced concrete building frames constructed prior to 1970 in New Zealand. The emphasis was placed on the behaviour of the beam-column joint regions which are typical of moment resisting perimeter frames of a reinforced concrete building designed and constructed in Christchurch in the late 1950's. It was also attempted to develop concrete jacketing techniques for retrofitting early building frames, including the beam-column joint regions.

9.2 SEISMIC ASSESSMENT OF AN EXISTING REINFORCED CONCRETE BUILDING DESIGNED IN THE LATE 1950'S IN NEW ZEALAND

The seismic performance of a typical reinforced concrete building frame designed and constructed in Christchurch in the late 1950's was assessed.

The results of the seismic assessment indicated that the available lateral load strength of the frame was very close to the design seismic force assuming elastic response obtained from NZS 4203 : 1984 and was larger than that from NZS 4203 : 1992. The inelastic mechanism of the frame was a mixture of flexural and shear failures in the beams and columns. A critical aspect with respect to shear was found in the behaviour of the beam-column joints with little or no shear reinforcement. The relatively large joint shear input during severe earthquakes indicated that the joints of the early concrete frame was likely to be governed by joint shear failure. It was also found that anchorage of the longitudinal beam bars in the joint core of exterior and interior columns of the typical frame would be poor under a severe earthquake.

9.3 EXPERIMENTAL INVESTIGATIONS

9.3.1 The Seismic Behaviour of the Beam-Interior Column Joints without Shear Reinforcement

Three full-scale beam-interior column joints, Specimens O1, O4 and O5, were constructed and tested under simulated seismic loading to investigate the seismic behaviour of joints without shear reinforcement.
Specimen O1 represented a critical joint region of the perimeter frame of the building investigated. The diameter of longitudinal beam bars to column depth ratio was 1/12.5 which did not satisfy the current New Zealand code for ductile frames. The maximum nominal horizontal shear stress in the beam-column joint was $0.61\sqrt{f_c} \text{ MPa}$, where $f_c$ is the measured compressive strength of concrete cylinder. The test on Specimen O1 demonstrated that the performance of the beam-interior column joint region of the frame would be poor in a major earthquake in terms of the stiffness, strength and ductility. This is mainly due to the lack of shear reinforcement and inadequate anchorage of longitudinal beam bars in the joint core.

Specimens O4 and O5 had the same dimensions and reinforcing details except that the beam bar diameter used was 24mm for Specimen O4 and 32mm for Specimen O5, respectively. The main aim of this test was to investigate the effect of the bond condition along the longitudinal beam reinforcement in the joint on the behaviour of the joint without shear reinforcement. The diameter of longitudinal beam bar to column depth ratio was 1/25 for Specimen O4, which is the maximum value allowed by the current New Zealand code and 1/18.75 for Specimen O5, respectively. The maximum nominal horizontal joint shear stress was $0.47\sqrt{f_c} \text{ MPa}$ for Specimen O4 and $0.61\sqrt{f_c} \text{ MPa}$ for Specimen O5. The specimens showed a limited ductile response during the test. The horizontal load strengths of both specimens degraded quickly due to severe joint diagonal tension cracking. The effect of the two different bond conditions along the beam bars in the joint on the seismic behaviour of the joints without shear reinforcement was found not to be significant.

After diagonal tension cracking occurred in the joint core without shear reinforcement, large tensile strains were measured along the beam and column bars in the joint core, indicating significant joint expansion. Bond stresses along the beam and column bars in the joint were mainly generated over the regions where flexural compression forces of the adjacent members acted. The initial stiffnesses of all beam-interior column joint specimens without shear reinforcement were significantly low when compared with the theoretical values, mainly due to the fixed-end rotation of the members adjacent to the joint, in which large tensile strains along the beam and column bars initiated.

9.3.2 The Seismic Behaviour of the Beam-Exterior Column Joints with Beam Bar End Hooks Not Bent into the Joint Core

Two full-scale beam-exterior column joints, Specimens O6 and O7, were constructed, in which only a small amount of shear reinforcement was placed in the beam, the columns and the joint core. The hooks of the longitudinal beam reinforcement of Specimen O7 were not bent into the joint core, which is typical of early building frames, while those of Specimen O6 were bent into the joint core. The test specimens were tested under simulated seismic loading and their behaviour was compared.
The beam-column joint of Specimen O7 failed in shear shortly after diagonal tension cracking in the joint before reaching the ideal horizontal load strength. By comparison, Specimen O6 showed a stable and ductile response with a plastic hinge forming in the beam during the test. It was identified that the seismic performance of the beam-exterior column joint was significantly influenced by the configuration, particularly the direction of the hooks at the ends of the beam bars anchored inside or outside in the joint core. The beam bar hooks when bent into the joint core, were not able to develop the diagonal compression strut mechanism within the joint along a corner to corner diagonal.

It is noted that, in case of Specimen O6, the seismic performance of the joint was satisfactory although only a small amount of shear reinforcement was provided in the joint core. This is because the maximum nominal horizontal joint shear stress of $0.31\sqrt{f_c}$ MPa obtained for Specimen O6 was small enough not to result in the severe reduction of its horizontal load strength and also the configuration of the beam bar hooks, bent into the joint core, was adequate to develop the diagonal compression strut mechanism in the joint under severe seismic loading.

9.3.3 The Seismic Behaviour of the Beams with Small Quantities of Transverse Reinforcement

Based on the results tested on Specimens R3 and O6, two limiting conditions were identified for the seismic behaviour of the beams with small quantities of shear reinforcement. At a maximum nominal beam shear stress of less than $0.14\sqrt{f_c}$ MPa, the beams did not fail in shear. When the maximum beam shear stress approached $0.18\sqrt{f_c}$ MPa, beam shear failure commenced. At this stage, the hysteresis loops showed a rapid strength degradation, mainly due to the reduced shear carried by the concrete shear resisting mechanism, particularly aggregate interlock.

When beam shear failure was avoided, the beam even if the amount of transverse reinforcement is small, showed a ductile response until the end of testing. The curvature ductility factors of at least 14 and 9 were obtained for the beam sections of Specimen R3 and Specimen O6, respectively, during the test.

9.3.4 The Seismic Behaviour of the Beam-Interior Column Joints Retrofitted Using Concrete Jacketing

Three full-scale beam-interior column joint regions with reinforcement details typical of the building frame designed in the late 1950's were retrofitted by jacketing with new reinforced concrete. Specimen R1 was a specimen retrofitted by jacketing both the beams and columns
of the as-built Specimen O1, which was previously damaged under simulated seismic loading. New hoops were also placed in the joint core. Specimen R2 was a specimen retrofitted by jacketing the beams and columns of another as-built specimen, without any previous damage, in the same manner except that new joint hoops were not placed. Specimen R3 was a specimen retrofitted by jacketing the columns of the other non-damaged as-built specimen. New joint hoops were not placed.

Results of the simulated seismic load tests showed that jacketing of columns, beams and joint with new reinforced concrete was a useful technique for enhancing the stiffness, strength and ductility of poorly detailed as-built beam-column joint regions. The tests also showed that the effect of previous damage to the as-built specimen had no significant influence on the overall seismic response of the retrofitted specimen.

It was also found that even when no joint core hoops are present in the existing beam-column joints, no new joint core hoops are required if the existing column is enlarged by jacketing so that the horizontal nominal shear stress in the joint core is reduced to less than $0.3\sqrt{\rho_0}$ MPa. This finding was for joints with no axial load on the columns. When axial compression load is present on columns, a greater horizontal joint shear stress would be tolerable.

The overall response of Specimen R3, which was retrofitted by jacketing the columns alone, was governed by the beam shear failure after developing the theoretical ideal flexural strength of the beam. Limited ductility response, that is available displacement ductility factor of 2.5, was attained for the specimen.

9.4 THEORETICAL CONSIDERATIONS

9.4.1 The Seismic Behaviour of the Beam-Interior Column Joints without Shear Reinforcement

One approach to the assessment of the shear strength of the joint without shear reinforcement is to assume that the shear strength is reached at the stage of initial diagonal tension cracking in the joint core. Based on the results from the eighty beam-interior column joint specimens tested by other researchers, the joint shear strength at diagonal tension cracking was investigated in terms of the principal tensile stresses in the joint. It was found that Eq.8.5 can be used as one method to assess the shear strength of the joint without shear reinforcement.

The shear resisting mechanisms of a joint without shear reinforcement after diagonal tension cracking were developed and examined using the test results. In the model developed
It was assumed that the longitudinal beam and column bars passing through the joint core act as both
structural reinforcement and joint shear reinforcement. As a consequence, large tensile stresses are
developed in these bars in the joint core. These tensile stresses were compared with the test results
obtained from this study in Fig. 8.15.

It was found that the maximum shear strength of a joint with no shear reinforcement depends
on the forces in the longitudinal reinforcement available to carry the joint shear forces. The
additional forces which are induced in these longitudinal bars due to the joint shear mechanisms
studied in this study, could limit the development of the maximum flexural strength of the
members adjacent to the joint. Therefore, when assessing an existing moment resisting frame with
joint shear reinforcement, the effect of the joint shear force on the longitudinal steel bar forces in
the joint core should be investigated.

When diagonal compression failure governs the joint strength, the maximum shear strength of
the joint can be estimated by Eq. 8.12. The shear strength degradation model shown in Fig. 8.22 is
posed for joints without shear reinforcement and can be used to estimate the available
placement ductility factor of an existing structure when joint shear failure occurs.

4.2 The Seismic Behaviour of Beams with Small Quantities of Transverse Reinforcement

The seismic behaviour of the beams with small quantities of transverse reinforcement was
studied in terms of the curvature ductility capacity and shear strength.

When beam bars of large diameter pass through column with relatively small depth, as is often
the case in early building frames, the "compression" reinforcement in the beam on one side of the
column may actually be in tension. This is due to bond slip through the joint and also, if the
section steel area is small, due to the neutral axis depth being small enough for the "compression
steel" to be in tension. The curvature analysis of the beams, taking into account the effect of the
stress conditions of the "compression" reinforcement, showed that the available curvature ductility
capacities were significantly reduced as a result of increasing tensile stress in the "compression"
reinforcement. The available curvature ductility factors were calculated to be about 10 for the beam
sections with \( \rho \) of 0.5% and about 5 for those with \( \rho \) of 1.0%, where \( \rho \) is tension reinforcement
fraction. Much smaller curvature ductility factors were available for beam sections with \( \rho \) larger than
1%.

A comparison of the experimental results in this study with the shear strength models proposed
columns and beams indicates that a modification to the concrete shear resisting mechanisms is
essential for the model to predict realistically the shear strength of the beams.
Experimental results obtained by other researchers were reviewed to assess the shear carried by the concrete $V_c$ of the beam. It was identified that the concrete contribution of the beam for the initial shear strength in the proposed model was very conservative. It was suggested that the concrete term $V_c$ for the initial shear strength be $0.3\sqrt{f_{cu}}bd$. The concrete term $V_c$ decreased as the displacement ductility factor imposed on the beam increased. The shear carried by the concrete mechanisms of the beam was found not to go to zero even when the imposed displacement ductility factor is larger than 4. It was found that $V_c$ for the final shear strength after degradation is larger than $0.04\sqrt{f_{cu}}bd$.

9.5 RECOMMENDATIONS FOR FUTURE RESEARCH

The following research is recommended to obtain a better understanding of the seismic behaviour of older reinforced concrete buildings.

Further experimental research is necessary to investigate the seismic performance of members and subassemblages with dimensions and reinforcing details typical of early reinforced concrete buildings, such as lap splices of beam bars in plastic hinge region.

In particular, the present study involved deformed longitudinal reinforcement. Plain round longitudinal bars were used in New Zealand before the mid-1960's. Tests on beam-column joints involving plain round longitudinal bars are needed.

In this study, the test was carried out on the beam-column joint regions without axial load acting on the columns. Therefore, a next stage of the study should investigate the seismic performance of beam-column joint regions when axial loads are applied to the columns. The following aspects should be examined when axial load is present on the column.

1. Bond performance of the beam bars passing through the joint
2. Maximum joint shear strength
3. Degradation of the joint shear strength

The degradation models for the shear strength of the joints and beams proposed in this study need to be further refined to obtain a more realistic assessment of the behaviour of early building frames under severe earthquakes.

More analytical research is required to examine the overall seismic performance of the retrofitted buildings as well as of the original buildings, providing information regarding effectiveness of the retrofit techniques used. In particular, an analysis of the performance of the
Retrofitted 1950's building studied in this thesis would provide useful comparison of various retrofit methods.

The initial stiffnesses obtained from beam-column joints without shear reinforcement tested in this study were significantly low when compared with theoretical values, mainly due to the large tensile strains along the beam and column bars in the joint core. A method to give a more realistic estimate for the initial stiffness of the frame with dimensions and reinforcing details typical of older building frames is required.

The hooks of longitudinal beam bars entering external columns not bent into the joint core do not efficiently develop the diagonal compression strut within the joint along a corner to corner diagonal. For the exterior joint with this configuration of hooks, however, a diagonal compression strut mechanism may develop at an angle less than 45 degree to the column axis when column hoops are adequately placed close to the joint core. If the column hoops had been adequately placed in the vicinity of the joint core of the test specimen, a better seismic performance might have been obtained. Further research is necessary in this aspect.

Retrofitting of reinforced concrete frames involves alternative procedures of infill walls, steel bracing and jacketing techniques. These techniques were found to be effective and constructible for many existing structures. However, the cost performance of such retrofit techniques is still uncertain since the retrofit procedures often include a complicated construction process. Retrofit methods which make the construction process simpler and more economical are required. These methods may involve the precast elements and new materials.
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