TESTS ON STRUCTURAL CONCRETE BEAM-COLUMN JOINTS WITH INTERMEDIATE COLUMN BARS

R. Park* and Yeoh Sik Keong**

SYNOPSIS

Three structural concrete interior beam-column joint units were tested. The beams were prestressed by tendons in the top and the bottom of the section but not at mid-depth. The columns were reinforced using Grade 380 longitudinal bars. Transverse shear reinforcement existed in all members and in the joint core. Static cyclic loading was applied to the units to simulate seismic loading. The presence of intermediate column bars was shown to significantly improve the shear capacity of the joint core, and the need for a relatively small neutral axis depth in the plastic hinge regions of beams for ductile behaviour was emphasized.

INTRODUCTION

Tests conducted in New Zealand on prestressed concrete beam-column joint units subjected to simulated seismic loading have been reported previously[1,2]. The ten beam-column units tested by Park and Thompson[2] contained a range of prestressed and non-prestressed steel, and the horizontal shear reinforcement in the joint cores in the form of column hoops was designed according to the method of Appendix A of ACI 318-77[3]. All ten test units had columns which contained four longitudinal (corner) bars only. The shear reinforcement in the joint cores enabled the beams to reach at least 95% of their theoretical strength in the first loading cycle into the inelastic range, accompanied by yielding of hoops in the joint cores of some test units. Further cycles of loading into the inelastic range caused shear failure to occur in the joint core of some units, particularly those units which did not have a prestressing tendon at mid-depth in the beam. It was evident that horizontal shear reinforcement alone in the joint cores, as permitted by the ACI Code[3], was inadequate to carry the shear forces imposed during intense seismic load reversals. It was considered that one of the main reasons why many of the test units had failed in shear in the joint core was that there were no longitudinal column bars present between the corner bars to assist transferring vertical shear force.

The object of the present investigation was to further investigate the shear strength of beam-column joint cores which do not contain a mid-depth prestressing tendon. Three interior beam-column units were tested, all of identical overall dimensions to those tested previously by Park and Thompson[2]. Intermediate column bars (between the corner bars) were present in all three test units as well as joint core shear reinforcement in the form of horizontal hoops. A further variable examined was the effect of the amount of prestressing steel in the beams on the flexural ductility of the beams. The three test units were numbered Units 11, 12 and 13 to follow in sequence the test units of Park and Thompson[2] which were numbered Units 1 to 10. The results summarized in this paper may be seen reported in more detail elsewhere[4].

* Professor and Head, Department of Civil Engineering, University of Canterbury.

** Former Graduate Student, Department of Civil Engineering, University of Canterbury.

DETAILS OF TEST UNITS

Test Units and Loading

The beam-column joint test unit represented part of a multistorey plane frame as shown in Fig. 1. The unit was loaded using the arrangement illustrated in Fig. 1 in which the ends of the columns are held on a vertical line and thus no account was taken of the PΔ effect which occurs in the columns of real frames. The PΔ effect can be assessed from the test results and included if necessary. The axial column load P applied was 996 kN. This axial load was held constant during the tests and was 0.13 to 0.15 of $f_c'A_g$, where f_c' is the concrete cylinder strength and A_g is the gross area of the column. Vertical load was applied to the end of the beams and resulted in horizontal reactive forces being induced at the ends of the columns. By reversing the directions of the beam end loads the effect of earthquake loading was simulated. The displacements at each beam end were kept equal during the loading cycles. In the first two load cycles the units were not loaded to their flexural strength. The subsequent load cycles in the inelastic range consisted of four loading runs in each direction to a beam end deflection of approximately 100 mm or more. This loading pattern was similar to that used previously[2]. The cyclic loading was applied statically and over a period of several days.

The overall dimensions of the test units are shown in Fig. 2. Fig. 3 shows a test unit from the previous series[2] under load. A similar test arrangement was used for Units 11, 12 and 13.

Steel and Concrete Details

The beam and column cross-sections and

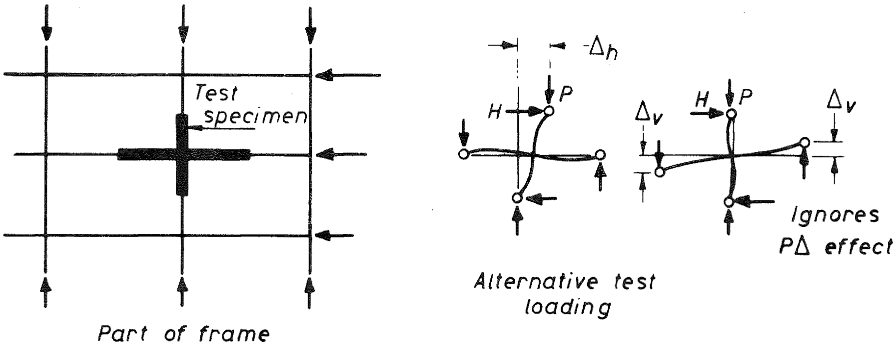
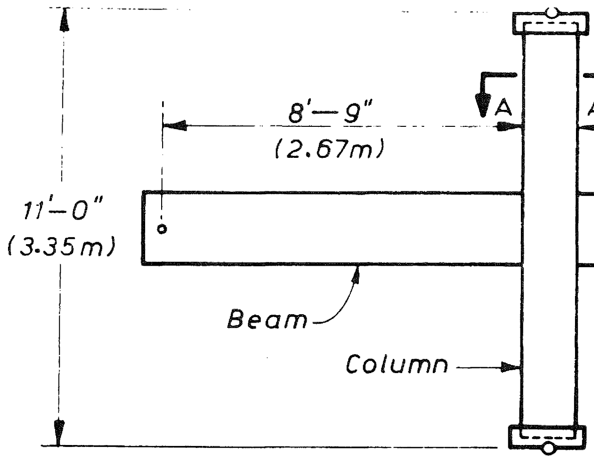


FIG. 1 - BEAM–COLUMN JOINT TEST UNITS AND LOAD APPLICATION



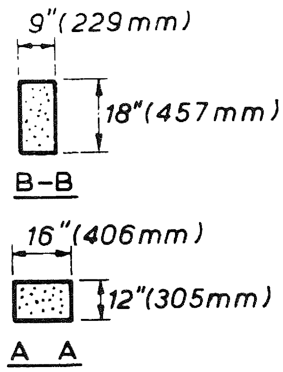


FIG. 2 - DIMENSIONS OF BEAM - COLUMN TEST UNITS

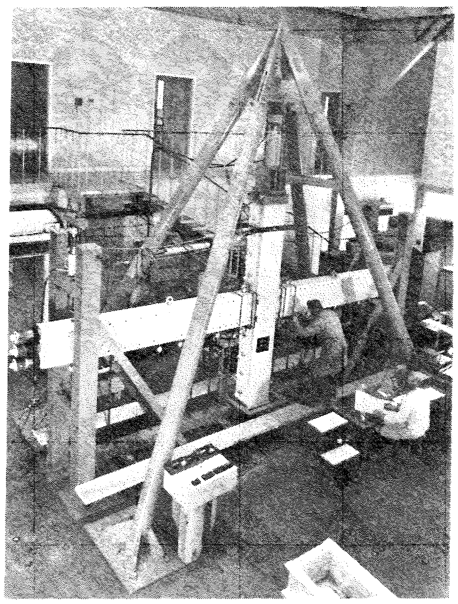


FIG. 3 – A BEAM–COLUMN UNIT UNDER TEST

the steel details are shown in Figs. 4 and 5. A beam section at the theoretical flexural strength is shown in Fig. 6.

Each beam contained two prestressing tendons and four nominal longitudinal non-prestressed bars. The prestressing tendons were post-tensioned to approximately 70% of their tensile strength and grouted. The beams of Units 11 and 12 had approximately the same flexural strength as the beams of the previously tested Units 1 to 10. The beams of Unit 13 had a smaller flexural strength due to the smaller prestressing steel area. At transfer, the uniform compressive stress in the concrete of the beams due to prestress was 9.4, 8.8 and 4.8 MPa for Units 11, 12 and 13, respectively. The stirrup spacing of 89 mm was one-quarter of the effective depth of the beam. The non-prestressed longitudinal steel was from Grade 275 deformed bar and the stirrups from Grade 275 plain bar.

The columns were not prestressed. The longitudinal steel in the columns was from Grade 380 deformed bar. The columns of Units 11 and 13 had two intermediate bars between the corner bars at mid-depth in the plane of the bending, and the columns of Unit 12 had four intermediate bars between the corner bars in the plane of the bending. The columns of Units 11, 12 and 13 were of approximately the same flexural strength as the columns of the previously tested Units 1 to 10 which only had four (larger) column bars. The columns were stronger than the beams and therefore plastic hinging was not expected to occur in the columns. The column hoops and ties were from Grade 275 plain bar.

The longitudinal steel shown in Figs. 4 and 5 was continued through the joint cores, but not the beam stirrups. For Units 11 and 12 each joint core was reinforced horizontally by eight rectangular hoops from 16 mm diameter bar enclosing all column bars at a mean spacing of 47 mm, and either eight supplementary cross-ties (Unit 11) or eight rectangular hoops (Unit 12) from 10 mm diameter bar passed around the intermediate column bars at the same spacing as the enclosing hoops. For Unit 13 each core was reinforced horizontally by five rectangular hoops from 16 mm diameter bar enclosing all column bars at a mean spacing of 82 mm, and five rectangular hoops from 10 mm diameter bar passed around the four intermediate column bars at the same spacing as the enclosing hoops. The joint core hoops were placed within the outer layers of non-prestressed longitudinal beam steel. All transverse steel was from Grade 275 plain steel bar. The supplementary cross-ties and interior hoops were not considered to make any contribution to the joint core shear capacity since they crossed the section at right angles to the direction of shear force.

The measured concrete strengths at the time of testing the units and the measured steel properties, are shown in Table 1. The concrete had an aggregate : cement : water ratio of 5.4 : 1.0 : 0.45, by weight. Each cylinder strength tabulated was the mean value from eight 150 mm diameter by 300 mm cylinders. The grout had a cement : water: Interplast ratio of 1 : 0.4 : 0.01, by weight, and the compressive strength found from 50 mm diameter cylinders at age 18 days was 37 MPa.

TEST RESULTS FROM THE UNITS

During the test loading of the units the end deflections of the beams were measured and controlled to give the required displacement cycles. Beam rotations in the plastic hinge regions were calculated from readings taken by dial gauges held by steel yokes placed around the beams. The dial gauges recorded the longitudinal deformations of the beams over either 210 mm or 236 mm long gauge lengths adjacent to the columns. Strains were measured on the horizontal shear reinforcement in the direction of applied shear in the joint core using electrical resistance strain gauges. The strain gauges were placed on the hoops so that bending of hoops due to concrete bulging did not alter the strain readings, and hence the measured hoop strains were due to axial tensile force in the steel only.

Figs. 7a, 8a and 9a show the measured vertical deflection at the ends of the beams plotted against the beam moment at the column faces for Units 11, 12 and 13, respectively. Figs. 7b 8b and 9b show the measured average beam curvature over either the 210 mm or 236 mm gauge length of beam adjacent to the column face plotted against beam moment the centre of the gauge length for the three test units. It is evident that in all units the inelastic deformations concentrated in the beam plastic hinge regions, since the shape of the moment-deflection and moment-curvature curves for the plastic hinges for each unit are so similar. In the first four loading runs the units were not loaded to the flexural strength of the beams and the moment-deflection loops show the large elastic recovery characteristic of prestressed concrete. Spalling of cover concrete commenced in the beam plastic hinge regions in loading run 5, which was the first loading run up to the beam flexural strengths. The degradation of strength and stiffness which occurred with subsequent loading runs into the inelastic range was due mainly to further spalling of the concrete cover at large concrete compressive strains, which reduced the section to that within the stirrups. In these loading cycles into the inelastic range the energy dissipation, and the residual deflection after unloading, was significant. The units during testing are shown in Figs. 7d and e, 8d and e, and 9d and e. The cracks have been marked on the units with a felt tipped pen to show their positions more clearly. As the end of the loading cycles was approached, the concrete in the plastic hinge regions in the beams had become extensively damaged and the effective depth and width of the cross-section had become much reduced. This resulted in the applied moment being resisted mainly by the longitudinal prestressing steel in tension and compression.

During testing the diagonal tension cracks in the joint core were observed to be relatively small and were well controlled by the joint core reinforcement. Figs. 7c, 8c and 9c show the strains measured on the joint core hoops during the loading cycles (the loading cycle number is shown beside the strains) and indicate that the maximum

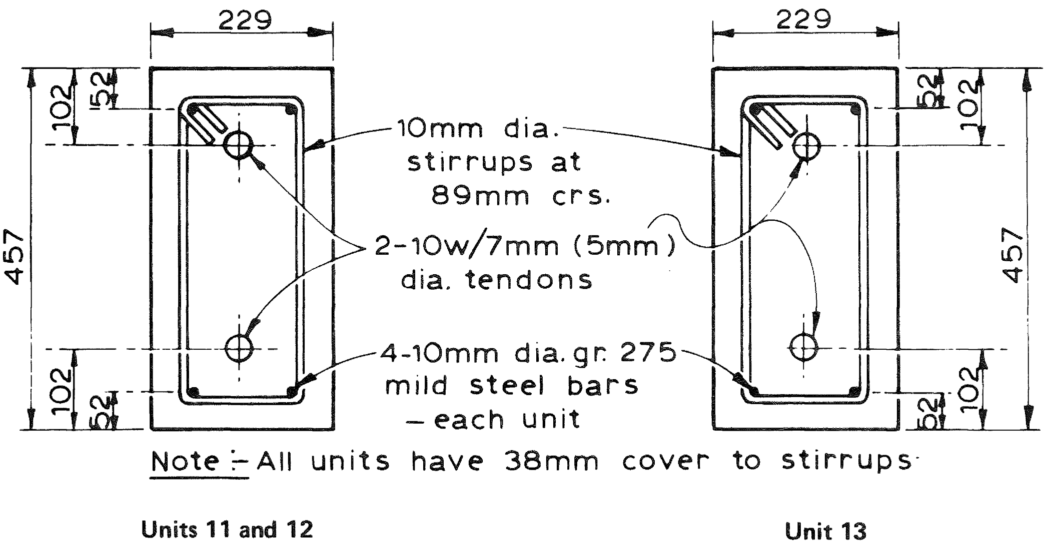


FIG. 4 - BEAM SECTIONS OF TEST UNITS

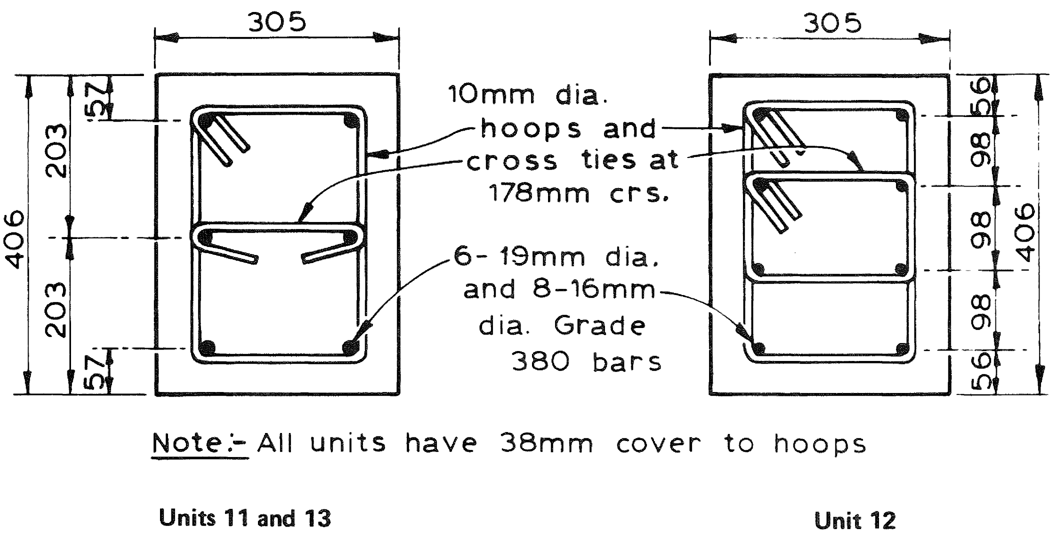


FIG. 5 – COLUMN SECTIONS OF TEST UNITS

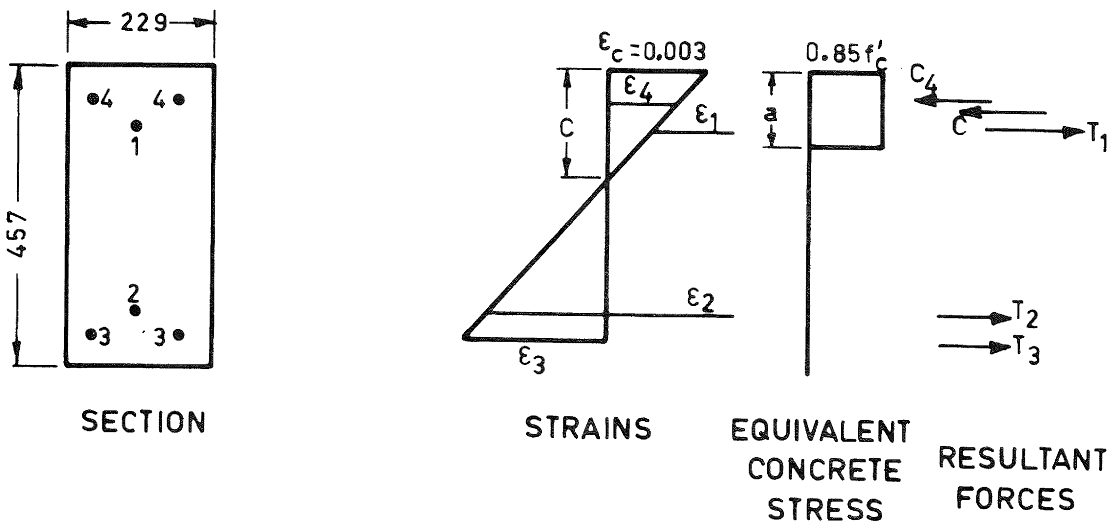
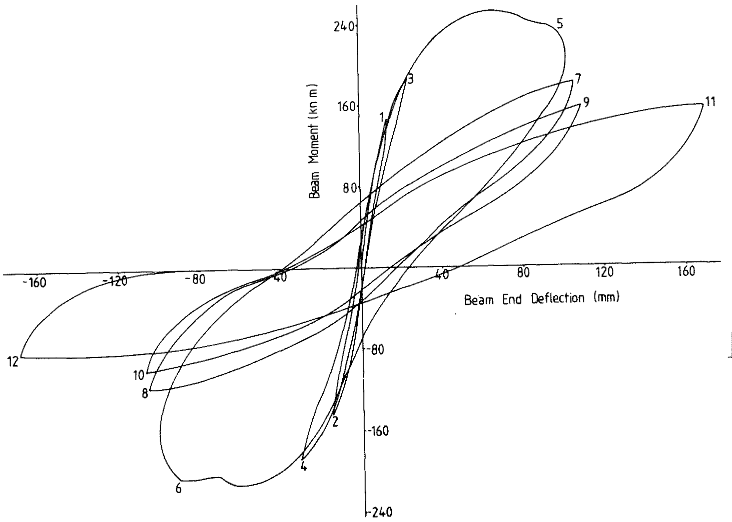
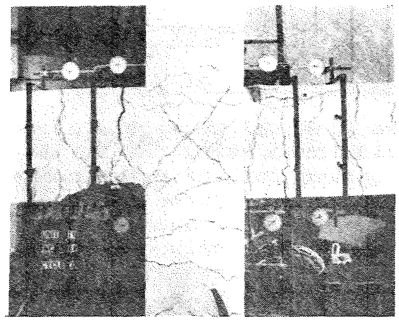


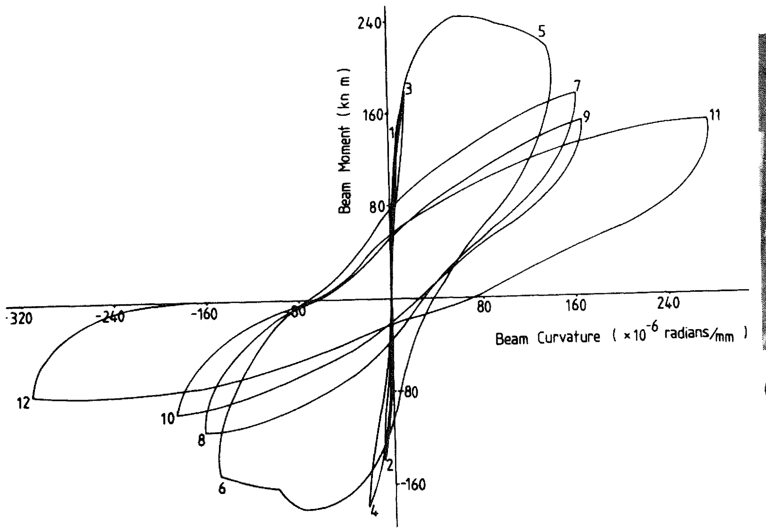
FIG. 6 – BEAM SECTION OF THEORETICAL FLEXURAL STRENGTH



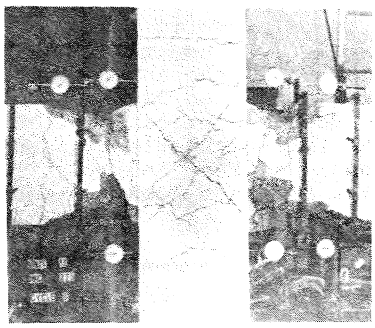
(a) Beam moment at column face versus right beam end deflection



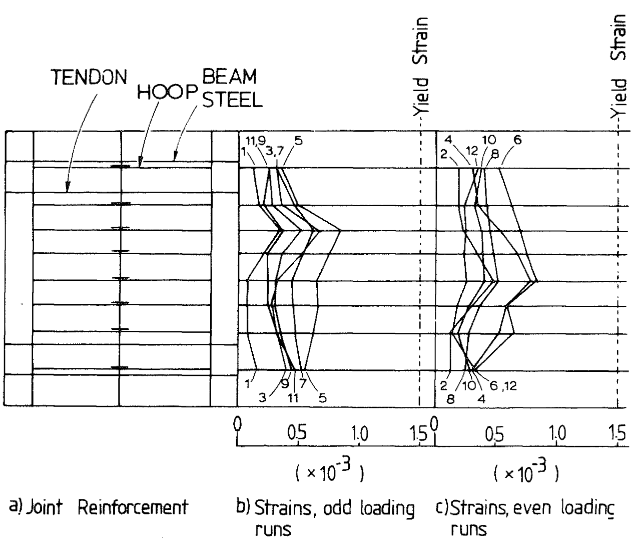
(d) At maximum moment (peak of loading run 5)



(b) Beam moment at 105mm from column face versus average curvature in right beam over 210mm length adjacent to column.



(e) At end of loading cycles (end of loading run 12)



(c) Joint core hoop strains

(f) Buckling of non-prestressed compression steel

FIG. 7 - UNIT 11 TEST RESULTS

hoop stresses reacted were not greater than 60% of the yield strength of the steel.

STRENGTH AND DUCTILITY OF MEMBERS

Flexural Strengths of Members

The flexural strengths of the members were calculated using the actual stress-strain curves for the prestressing steel, the stress-strain curves of the nonprestressed longitudinal steel, and the concrete rectangular compressive stress block of the ACI Code[3]. The measured strengths of the materials for the three test units are shown in Table 1. An extreme fibre concrete compressive strain of 0.003 was assumed[3] and the flexural strength of each unit was calculated by satisfying the requirements of equilibrium of forces on the section and strain compatibility[6]. Perfect bond between steel and concrete was assumed and the strength reduction factor ϕ was taken to be unity. The notation used for the beam sections is shown in Fig. 6; the steel positions 1 and 2 refer to prestressing tendons, and positions 3 and 4 refer to nonprestressed steel. The strain in the tendons due to prestress alone after losses was estimated to be close to 0.005. The iterations to determine the neutral axis depth were continued until the internal forces balanced to within less than 2% of the total tensile force. The theoretical neutral axis depths, internal forces and flexural strengths of the beams, and the theoretical flexural strengths of the columns, are shown in Table 2.

The maximum moments reached by the beams during the loading cycles were 23%, 16% and 19% greater than the theoretical flexural strengths for Units 11, 12 and 13, respectively. One reason for this greater measured flexural capacity was that the maximum measured moment was reached at an extreme fibre concrete compressive strain greater than the value of 0.003 assumed in the flexural strength calculations, and hence the actual maximum tensile force in the prestressing tendon in the tension zone would have been greater than calculated. Also, the extra confinement of the compressed beam concrete at the critical section due to the presence of the column and its reinforcement would have enhanced the strength and ductility of the beam concrete adjacent to the column faces.

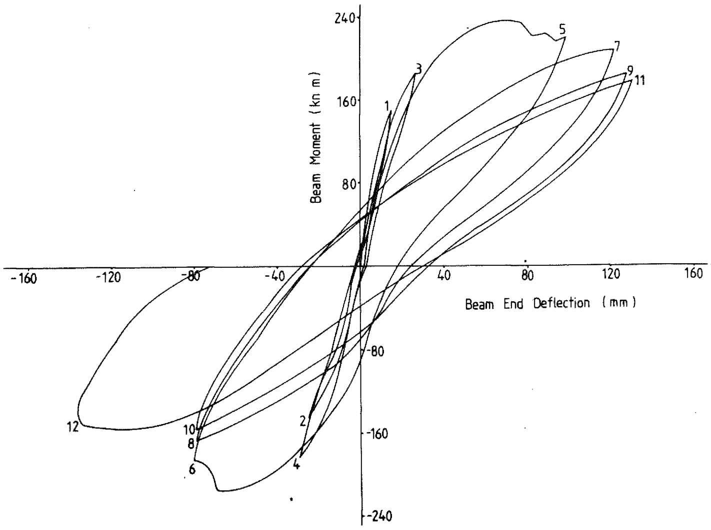
Flexural Ductility of Beams

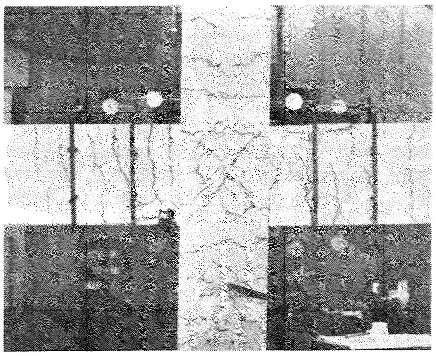
As discussed previously, the degradation of the strength and stiffness of the units once maximum moment had been reached was due mainly to the spalling of the concrete cover in the plastic hinge regions of the beams at large compressive strains, which reduced the effective concrete section to that within the stirrups. The degradation of the strength and stiffness during the loading cycles in the inelastic range was significant, as is shown in Figs. 7a, 8a and 9a. However it needs to be borne in mind that the 38 mm thickness of concrete cover to the stirrups in the beams was a relatively large proportion of the beam section dimensions. That is, when the concrete cover spalls, 33% of the section width and 11% of the section effective depth to the tendon is lost, which has a large effect on the effective moment of inertia of the section and the lever arm. In larger members the loss of cover concrete will not have such an appreciable influence on the concrete section. Loss of cover concrete cannot be prevented by additional confining steel, and hence the cover to stirrups should be made as small as allowed by the other design constraints.

Figs. 7f and 8f show buckling of the 10 mm diameter nonprestressed compression steel in the beam plastic hinge regions. Thus the stirrup spacing of 89 mm, which was 8.9 times the longitudinal bar diameter, was insufficient to prevent this buckling. Cyclic (reversed) loading of steel causes a reduction in the tangent modulus of the steel at low levels of stress, owing to the Bauschinger effect[6], and buckling will occur unless the bars are supported laterally at relatively close centres. The draft SANZ Concrete Design Code[5] recommends a stirrup spacing of not more than 6 longitudinal bar diameters to prevent bar buckling. The findings of these tests gives some further justification for this code recommendation.

There is a noticeable difference in the extent of strength degradation with cyclic loading between Units 11 and 12, and Unit 13, in Figs. 7a, 8a and 9a. The beams of Units 11 and 12 had more prestressing steel than the beams of Unit 13, and therefore the neutral axis depth at the flexural strength of the beams was greater in Units 11 and 12 than in Unit 13. The ratio a/h was 0.23 to 0.24 for the beams of Units 11 and 12 and 0.13 for the beams of Unit 13, where a = depth of rectangular concrete compressive stress block at the flexural strength and h = overall depth of section. The draft SANZ Concrete Design Code[5] recommends that unless special transverse steel, of the quantity provided in potential plastic hinge zones in columns, is provided in the plastic hinge regions of beams the a/h ratio should not exceed 0.2. This draft SANZ recommendation was deliberately not complied with in the beams of Units 11 and 12. The volume of closed stirrups present in the beams was 1.46% of the volume of the concrete core of the beam, whereas the draft SANZ code would require at least almost three times this volume of confining steel in potential plastic hinge regions of columns with the dimensions and steel positions of these beams. It is evident that the loss of strength of Units 11 and 12, as shown in Figs. 7a and 8a, with cyclic loading gives some additional justification for this code requirement of a relatively small a/h ratio when extensive confining steel is not present. The beams of Unit 13 satisfied the code recommendation for the a/h ratio and showed considerably less strength degradation, as is illustrated in Fig. 9a.

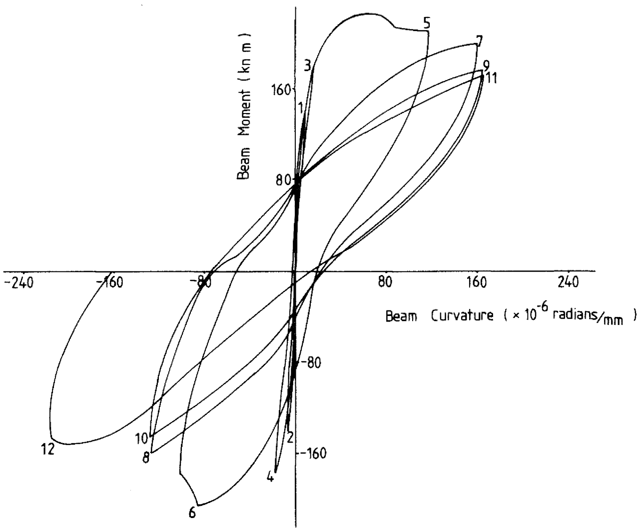
It is apparent that a further improvement in the behaviour of the beams would have resulted from the presence of significant quantities of nonprestressed longitudinal steel, to help maintain the compression capacity of the section when concrete commenced to spall, and to widen out the moment-displacement hysteresis loops, thereby increasing the energy dissipation per cycle. Prestressing tendons present in the compression region of the concrete are able to act as compressio

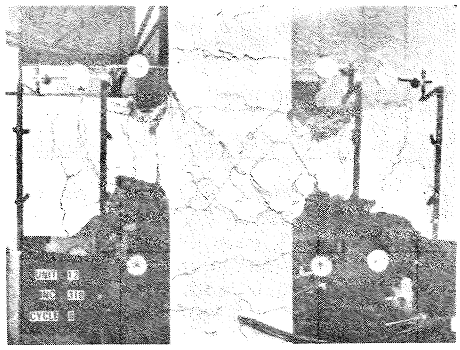




(d) At maximum moment (peak of loading run 5).

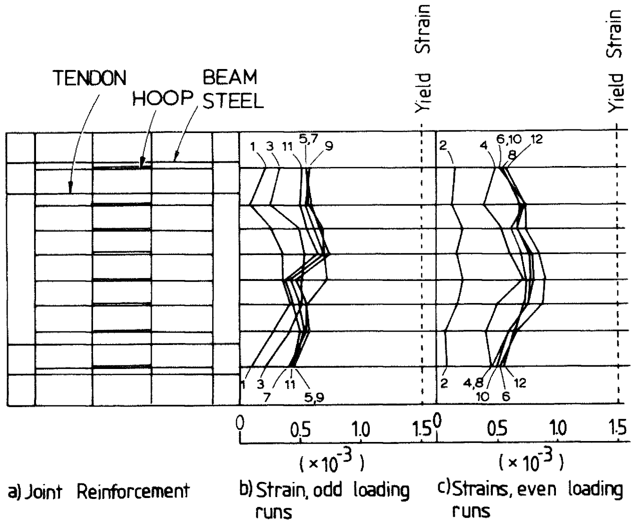
(a) Beam moment at column face versus right beam end deflection



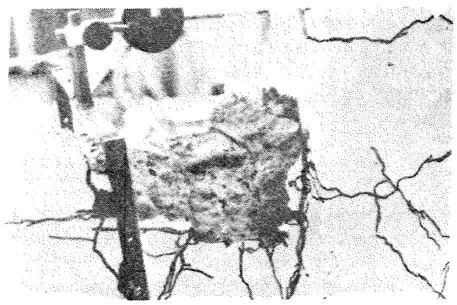


(e) At end of loading cycles (end of loading run 12).

(b) Beam moment at 105mm from column face versus average curvature in right beam over 210mm length adjacent to column.



(c) Joint core hoop strains



(f) Buckling of non-prestressed compression steel

FIG. 8 - UNIT 12 TEST RESULTS

reinforcement at large deformatons, as was evident from these tests and the previous tests[2]. However, nonprestressed steel is capable of acting as compression steel before the compressed concrete spalls and hence will maintain the moment capacity better. Also, because of the lower steel yield strength, the bond stresses required to anchor beam steel passing through columns will be less critical for nonprestressed steel than for prestressing tendons.

Shear Strength Members

Shear in the members was not critical. The nominal shear stress applied to the beams at the theoretical flexural strength was $0.14\sqrt{f'_c}$, $0.15\sqrt{f'_c}$ and $0.09\sqrt{f'_c}$ for the beams of Units 11, 12 and 13, respectively. This beam shear could be carried by the stirrups alone without assistance from the concrete shear resisting mechanisms. The shear applied to the columns during the tests required some assistance from the concrete shear resisting mechanism, since the column hoops alone could not carry the maximum applied shear force. However since no plastic hinges formed in the columns this was acceptable. No significant shear cracking was observed in the beams or the columns during testing.

STRENGTH OF JOINT CORES

Shear Forces to be Carried by Joint Cores

When the flexural strength of the beams is reached, the beam-column joint core is bounded by flexural cracks at the beam and column faces. The internal beam and column forces due to flexure and shear acting on the faces of the joint core when the flexural strength of the beams is reached can be accurately calculated. The horizontal shear force to be carried by the joint core can be obtained from these forces. As shown in Fig. 10 the maximum horizontal shear force occurs in the middle region of the beam depth between the neutral axis positions of the beam sections. Using the notation of Figs. 6 and 10, the maximum horizontal shear force acting above (or below) a horizontal plane in this middle region is

$$V_{jh} = T_2 + T_3 + C + C_4 - T_1 - V' \qquad (1)$$

Similarly the maximum vertical shear force to be carried may be found the same way by considering the vertical column internal forces and the beam shear acting to one side of a vertical plane in the middle of the joint core.

Theoretical Shear Strength of Joint Core

The mechanism of shear transfer across beam-column joint cores has been discussed previously, for example[6,7,2]. Fig. 11a shows a reinforced concrete joint and the forces present when seismic loading acts on the frame. The shear in the joint core is induced by the beam and column forces as discussed above. These forces can result in high diagonal tension stresses in the joint core causing diagonal tension cracks to form. Two mechanisms capable of transmitting shear forces across the joint core are shown in Fig. 11b and c. The first is a diagonal compression strut mechanism which transfers the compressive forces in the concrete, and the shear forces at the faces of the joint core, across the joint core (Fig. 11b). The second is a truss mechanism involving both horizontal and vertical shear reinforcement and a diagonal compression field in the concrete which transfers the shears introduced at the faces of the joint core by the bond stresses from the reinforcing bar forces across the joint core (Fig 11c). The shear transferred across the joint core by these two mechanisms would appear to be additive. The diagonal compression strut mechanism can be likened to "the shear carried by the concrete" and the truss action mechanism can be likened to "the shear carried by the shear reinforcement".

It has also been pointed out previously, for example[6,7,2], that cyclic (reversed) loading reduces the effectiveness of the diagonal compression strut mechanism, because the opening and closing of diagonal tension cracks in alternating directions weakens the diagonal compression strut. Also, yielding of longitudinal beam tension steel at the column faces will mean that when the direction of loading is reversed an open crack will remain in the beam at the column face in the "compression zone". This will result in all the compression force in the beam being transferred to the joint core by the compression steel, until that steel yields and the crack closes enabling some of the beam compressive force to be introduced to the joint core by the concrete. Thus during cyclic loading the shear transferred by the diagonal compression strut mechanism may become insignificant and the whole of the shear may need to be transferred by truss action of shear reinforcement acting with a diagonal compression field in the joint core. This reduction in the shear carried by the concrete during cyclic loading has been observed in many past tests, and occurred particularly in those frames with significant quantities of nonprestressed longitudinal steel[2]. If prestressing steel is present in the beams, and the cracks close on unloading, the reduction in the shear carried by the concrete is not so significant.

Note also that the mechanism of shear transfer by truss action in Fig. 11c requires both vertical and horizontal shear reinforcement. However, the vertical shear reinforcement necessary for truss action can be reduced if a large axial compressive load exists on the column, causing the neutral axis to be deep in the column.

Code Approaches to Joint Shear Design

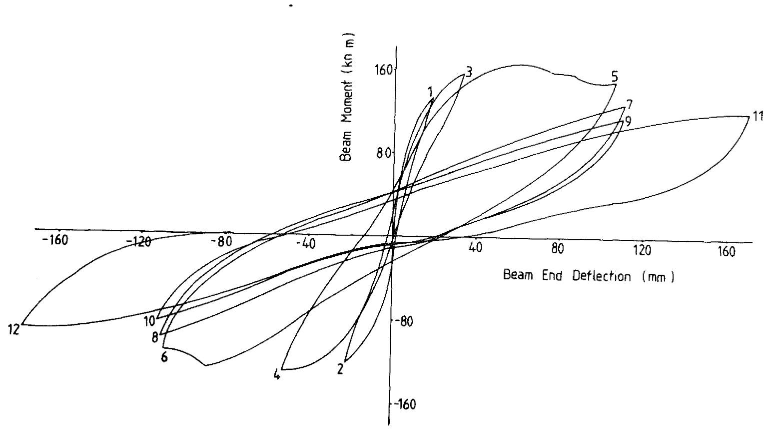
In the following equations the strength reduction factor ϕ has been taken to be unity.

(a) ACI Code[3]

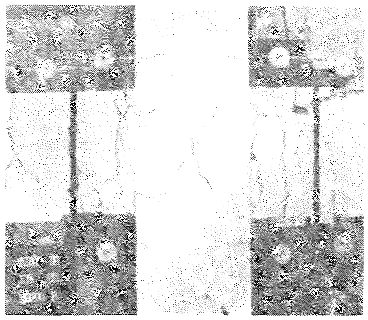
The horizontal shear strength of joint core is given by the sum of the shear carried by the concrete and the horizontal shear reinforcement as:

$$V_{jh} = 0.17(1 + 0.0725\frac{N_u}{A_g})\ \sqrt{f'_c}\ bd + \frac{A_v f_y d}{s} \qquad (2)$$

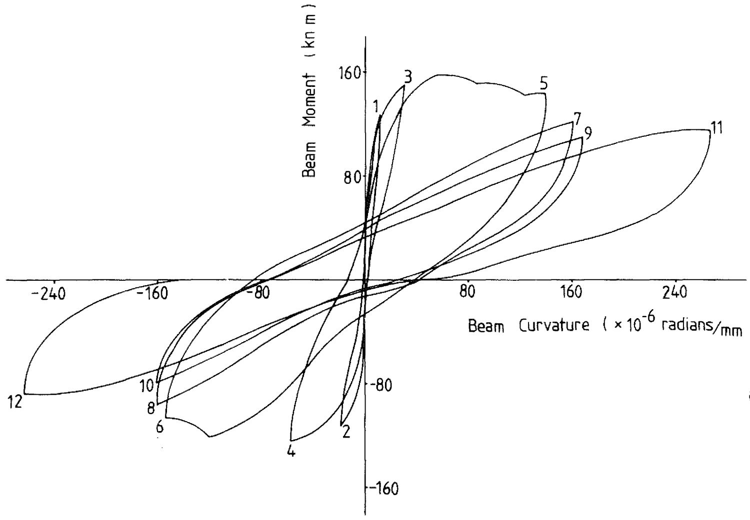
where N_u = factored axial load on column, A_g = gross area of column, f'_c = concrete cylinder strength, b = column width, d = column effective depth, A_v = area of



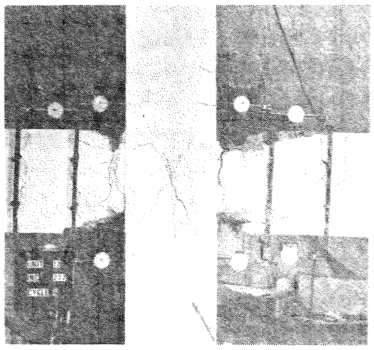
(a) **Beam moment at column face versus right beam end deflection.**



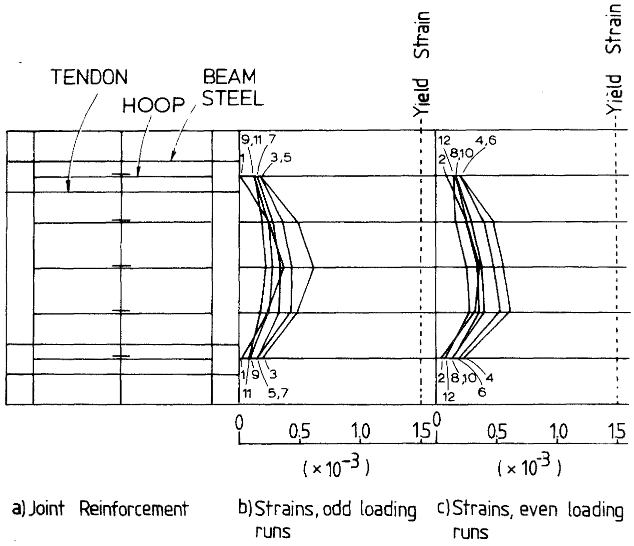
(d) **At maximum moment (peak of loading run 5)**



(b) **Beam moment at 118mm from column face versus average curvature in right beam over 236mm length adjacent to column.**



(e) **At end of loading cycles (end of loading run 12)**



(c) **Joint core hoop strains**

FIG. 9 - UNIT 13 TEST RESULTS

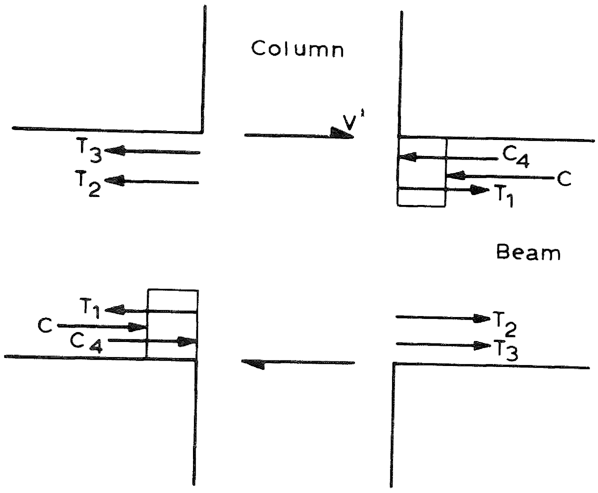
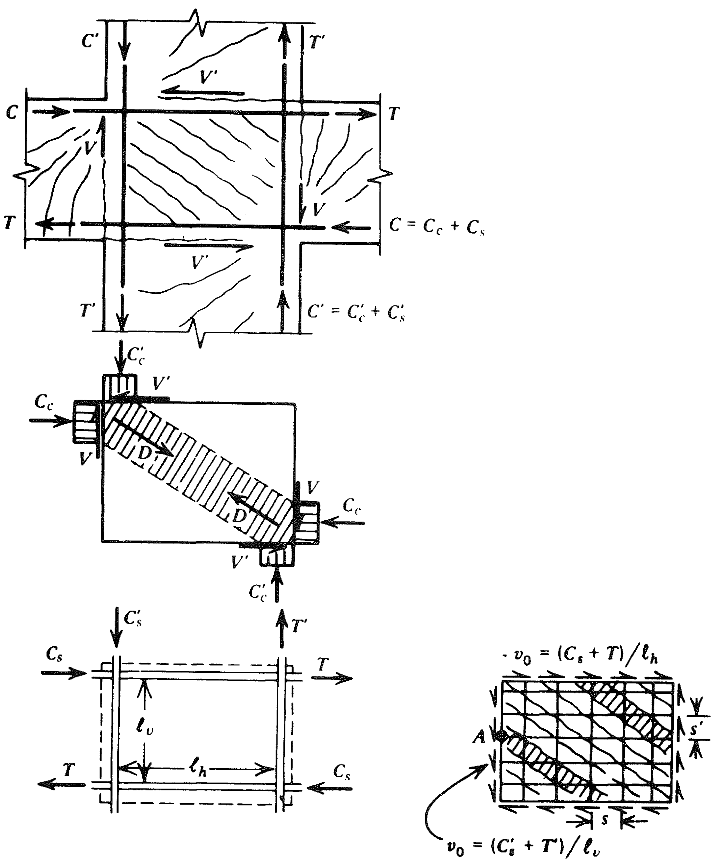


FIG. 10 - FORCES CAUSING HORIZONTAL SHEAR FORCE IN BEAM-COLUMN JOINT CORE



(a) Forces acting on a beam-column joint core.

(b) Shear transfer by diagonal compression strut.

(c) Shear transfer by truss action of shear reinforcement.

FIG. 11 - MECHANISMS OF SHEAR TRANSFER OF REINFORCED CONCRETE BEAM-COLUMN JOINT CORE

horizontal shear reinforcement within distance s, f_y = yield strength of shear reinforcement, and s = spacing of horizontal shear reinforcement.

No recommendation is made with regard to the vertical shear strength of the joint core or vertical shear reinforcement.

(b) Draft SANZ Concrete Design Code(5)

The horizontal shear strength of the joint core of the symmetrically reinforced beam-column units is given by the sum of the shear carried by the concrete and the shear carried by the horizontal shear reinforcement as:

$$V_{jh} = V_{ch} + V_{sh} \tag{3}$$

where

$$V_{ch} = 0.25(1 + \frac{f'_c}{25})\sqrt{\frac{N_u}{A_g} - \frac{f'_c}{10}}\ (bh) \tag{4}$$

and

$$V_{sh} = A_{jh}\ f_y \tag{5}$$

The required vertical shear reinforcement in the joint core, normally consisting of intermediate column bars placed in the plane of bending between the corner bars of the column, is given by

$$A_{sv} = V_{sv}/f_y \tag{6}$$

where

$$V_{sv} = V_{jv} - V_{cv} \tag{7}$$

and

$$V_{cv} = 0.5V_{jv}(1 + \frac{N_u}{0.6f'_cA_g}) \tag{8}$$

where f'_c = concrete cylinder strength, N_u = factored axial load on column, A_g = gross area of column, b = column width, h = column overall depth, A_{jh} = total area of horizontal shear reinforcement between the outer-most layers of top and bottom beam reinforcement, f_y = yield strength of shear reinforcement, and V_{jv} = vertical shear force across joint core, which according to the Commentary(5) may be approximated as

$$V_{uv} = V_{jh}\gamma \tag{9}$$

where γ = ratio of beam overall depth to column overall depth. The intermediate column bars used as vertical shear reinforcement can also serve as part of the reinforcement in the column required for axial load and flexure.

Discussion of Calculated Shear Strengths of Joint Cores of Units 11, 12 and 13

Table 3 shows the theoretical maximum applied horizontal shear forces, calculated from Eq. 1 using the beam internal forces and column shear at the beam ultimate moment given in Table 1, compared with the theoretical horizontal shear strengths calculated using the ACI Code approach (Eq. 2) and the draft SANZ Code approach (Eqs. 3, 4 and 5) computed using the measured material strengths. It is evident that the horizontal shear strengths of the joint cores of Units 11, 12 and 13 were just adequate according to both code approaches.

Table 4 shows the theoretical maximum applied vertical shear force calculated from Eqs. 1 and 9, and the area of vertical shear reinforcement required by the draft SANZ Code approach (Eqs. 6, 7 and 8) calculated using the measured material strengths, compared with the areas actually existing in Units 11, 12 and 13. It is apparent that the amount of vertical shear reinforcement actually present in the joint cores in the form of intermediate column bars was 56%, 81% and 99% of that required by the draft SANZ Code approach for Units 11, 12 and 13, respectively. Thus only Unit 13 satisfied the draft SANZ Code and according to that code distress could be expected in the joint cores of Units 11 and 12. Note that the ACI Code does not require any consideration of vertical shear forces in joint cores or shear reinforcement to carry them.

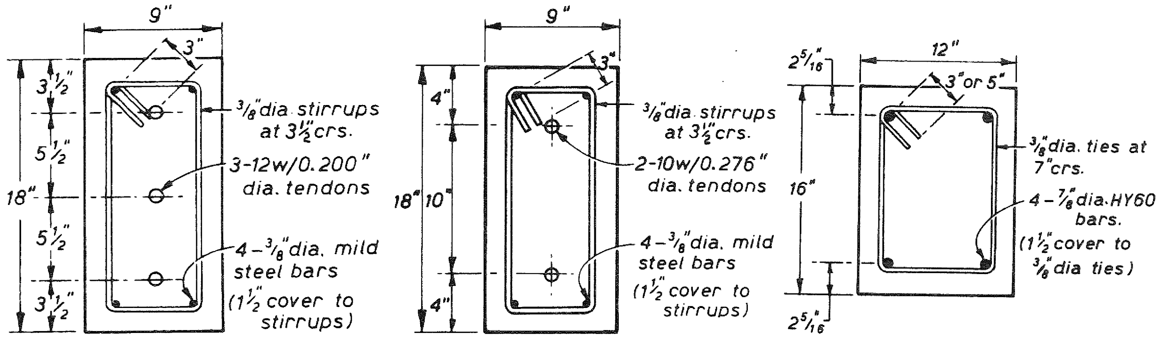
It is apparent from the low measured strains the horizontal joint core hoops of Units 11, 12 and 13, shown in Figs. 7c, 8c and 9c, and the limited diagonal tension cracking observed on those joint cores during the tests, that in fact joint core shear was adequately provided for in those units. Of particular interest is that for Units 11 and 12, in which the areas of intermediate column bars were significantly smaller than required by the draft SANZ Code, the maximum measured joint core hoop stresses were only 0.56 f_y and 0.60 f_y, respectively, where f_y is the yield strength of the hoops. Also, for Unit 13, in which the intermediate column bars satisfied the draft SANZ Code, the maximum measured joint core hoop stress was only 0.41 f_y.

It appears from the test results that an unnecessary degree of conservatism exists in the draft SANZ Code requirements for joint core shear reinforcement for these units.

Discussion of Calculated Shear Strengths of Units 4, 5 and 9

It is of interest to review some of the test results previously obtained(2) in the light of present test results. Fig. 12 shows the beam and column sections of the previously tested Units 4, 5 and 9. The joint cores of Units 4 and 5 had the same quantity of hoop steel as Units 11 and 12, whereas the joint core of Unit 9 had a greater quantity of hoop steel (approximately one-third greater). The columns of Units 4, 5 and 9 had only four (corner) bars whereas Units 11 and 12 had six and eight bars respectively, including some intermediate column bars. The beams of Unit 4 had three prestressing tendons, including one at mid-depth, whereas the beams of Units 5, 9, 11 and 12 had two prestressing tendons. The flexural strengths of the members of Units 4, 5, 9, 11 and 12 were similar.

Fig. 13 shows the joint core hoop strains measured for Units 4 and 5. It is apparent from Fig. 13a that the mid-depth prestressing tendon in the beam of Unit 4 was able to control the diagonal tension cracking of the joint core effectively and



(a) Unit 4 beam section (b) Units 5 and 9 beam sections (c) Units 4, 5 and 9 column sections

FIG. 12 - SECTIONS OF MEMBERS OF UNITS 4,5 AND 9
(1 in = 25.4mm)

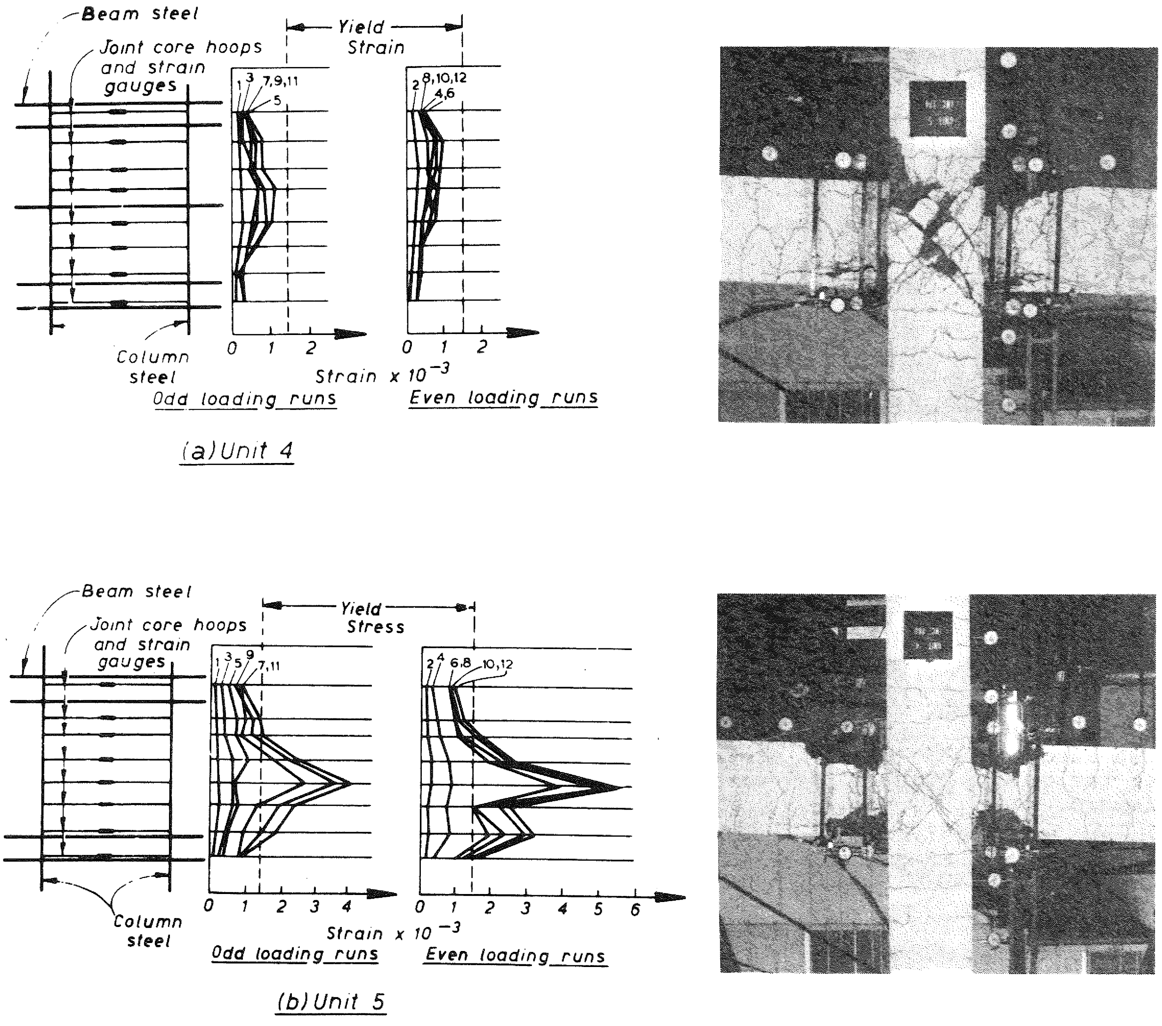


FIG. 13 — JOINT CORE HOOP STRAINS AND DAMAGE AT END OF TESTS OF UNITS 4 AND 5

to prevent joint core shear failure in spite of the lack of intermediate column bars in the column. However in some of the previously tested units (for example Units 2, 6 and 8) a mid-depth prestressing tendon in the beam was not found to prevent joint core shear failure when only four (corner) bars were present in the columns. Therefore it can be concluded that when mid-depth beam tendons are present it is better to use them to replace some of the horizontal hoops in the joint core and to have intermediate column bars present as well.

In the case of Unit 5, Fig. 13b shows that the joint core hoops yielded extensively (six of the eight hoops yielded) and diagonal tension cracking became extensive leading to shear failure of the joint core. In Unit 9 which had additional hoops in the joint core but was otherwise identical to Unit 5, shear failure of the joint core still persisted. However in Units 11 and 12 which had the same quantity of joint core hoops as Unit 5, but with intermediate column bars, the joint core remained intact during the tests with only small diagonal tension cracks and small hoop strains. The shear failure of the joint cores of Units 5 and 9 evidently occurred because of the lack of vertical shear reinforcement in those joint cores. It is apparent that merely increasing the amount of horizontal shear reinforcement is no solution to the shear problem in joint cores.

Suggested Modification to Draft SANZ Code Approach for V_{cv}

This study shows clearly that vertical shear reinforcement is necessary in joint cores. However it appears that the amount of vertical shear reinforcement required by the draft SANZ Code, as calculated by Eqs. 6, 7 and 8, is unduly conservative, as indicated by the low measured hoops strains and excellent performance of the joint cores of Units 11, 12 and 13.

However, one reason for the low joint core hoop strains, shown in Figs. 7c, 8c and 9c, would be the degradation of the flexural strength of the beams during the loading cycles in the inelastic range. Note that the joint hoop strains were a maximum in the loading runs when ultimate moment was first reached (around runs 5 and 6) and then in the subsequent loading runs the joint core hoop strains generally decreased. Had the flexural strength of the members been maintained better in these subsequent loading runs, it may be that the diagonal compression strut mechanism would have degraded significantly resulting in a progressive increase in the joint core hoop strains. A reduction in the beam flexural strength due to loss of concrete section caused by spalling can lead to a reduction in the applied horizontal shear force on the joint core. This follows because the beam tensile force $T_2 + T_3$ (see Fig. 10) may become smaller due to the reduced compression, although T_1 reduces and tends to go into compression and compensate for the reduced C. Hence the resulting applied horizontal shear may be smaller than the value before spalling occurred. However, in the previously conducted tests[2] it was found that when the concrete diagonal strut mechanism was adequately maintained by a central prestressing tendon in the beams, and when the flexural strength of the beams showed little degradation during the loading cycles in the inelastic range, the increase in joint core hoop strains was not unreasonable. For example, in the partially prestressed Units 7 and 10 the joint core hoop stress reached approximately $0.78f_y$ and $0.63f_y$, respectively, in the first loading cycle to the ultimate flexural strength, but did not increase to the yield strength in the subsequent loading cycles in the inelastic range even though the beam flexural strength was maintained to greater than 80% of the flexural strength at spalling. Hence it is felt that the performance of the joint cores of Units 11, 12 and 13 would have been satisfactory even if the flexural capacities of the beams had been maintained better during the loading cycles in the inelastic range.

Because of the excellent joint core performance of Units 11, 12 and 13 with a relatively small amount of vertical shear reinforcement, it is felt that the vertical shear carried by the diagonal compression strut, according to the draft SANZ Code[5], as given by Eq. 8, is unduly conservative. The basis of Eq. 8, as explained in the Commentary[5], is that theoretical case studies have shown that approximately 0.5 of the applied vertical joint core shear ($0.5V_{jv}$) can be transferred by the diagonal compression strut when the members are symmetrically reinforced and there is no axial compressive load present ($N_u = 0$). When axial compression is present ($N_u > 0$) the neutral axis of the column is deeper and the column compression can be assumed to provide some vertical restraint to the joint core truss mechanism, thereby reducing the vertical steel requirements. Note that the $0.5V_{jv}$ in Eq. 8 is only an approximation.

On the basis of the results of the current series of tests it would appear to be more reasonable to permit $0.6V_{jv}$ to be carried by the concrete when $N_u = 0$. Eq. 8 would then be modified to become:

$$V_{cv} = 0.6V_{jv}\left(1 + \frac{N_u}{0.6f_c'A_g}\right) \tag{10}$$

The required vertical shear reinforcement given by this modified form of the draft SANZ Code requirement, Eq. 10, calculated using the measured material strengths, is shown in Table 4. The amount of vertical shear reinforcement actually present in the joint cores of Units 11, 12 and 13, in the form of intermediate column bars is 90%, 133% and 154% of that required from Eqs. 6, 7 and 10, respectively. The areas given by the modified equation appear reasonable in view of the low joint core hoop strains measured.

The use of Eq. 10 rather than Eq. 8 has the main advantage of requiring fewer intermediate column bars. This would be a particularly welcome modification for small columns where, for example, one intermediate bar may suffice in each face rather than two. However, the draft SANZ Code requirements that the spacing of vertical shear reinforcement in the plane of bending should not exceed 200 mm, and that there should be at least one intermediate column bar in each side of the column in the plane of bending, should be complied with.

CONCLUSIONS

The need for a relatively small neutral axis depth in the plastic hinge regions of beams for ductile behaviour, when special transverse steel to confine the concrete is not present, was demonstrated by the tests. The desirability of the presence of compression reinforcement with adequate lateral support was also illustrated.

The tests also showed the necessity for vertical shear reinforcement in joint cores to act with the horizontal hoops to form an effective shear resisting mechanism. The vertical shear reinforcement would normally take the form of intermediate column bars placed in the plane of bending between the corner bars. Horizontal shear reinforcement alone in a joint core does not provide an effective truss mechanism for shear resistance. A modified form of the draft SANZ Code equation for vertical shear carried by the concrete is suggested. This modified equation is based on the observed performance of the test units and leads to a smaller amount of vertical shear reinforcement than recommended by the draft SANZ Code.

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TABLE 1

MATERIAL PROPERTIES OF TEST UNITS

Concrete:

Unit	Age at Test, days	Compressive Cylinder Strength, MPa	Modulus of Rupture, MPa
11	75	44.2	8.77
12	107	41.2	7.81
13	53	48.1	9.00

Prestressing Steel:

Diameter mm	Tensile Strength, MPa	Strain at Fracture (51mm gauge length)
5	1,608	0.048
7	1,687	0.046

Nonprestressed Steel:

Diameter* mm	Type	Yield Strength, MPa	Tensile Strength, MPa
19	Deformed	439	732
16	"	431	695
10	"	328	454
16	Plain	303	469
10	"	310	478

* Rounded to nearest mm. Imperial size bars were used.

TABLE 2

THEORETICAL FLEXURAL STRENGTH PROPERTIES OF BEAMS AND COLUMNS

Unit	Beam								Column
	c mm	a mm	T_1 kN	T_2 kN	T_3 kN	C_4 kN	C kN	M_u kNm	M_u * kNm
11	144	105	316	577	47	47	899	207	246
12	149	112	316	577	47	47	895	204	231
13	84.5	59.2	230	300	47	33	553	132	252

* Column M_u calculated for axial load of 996 kN

TABLE 3

HORIZONTAL SHEAR STRENGTH OF JOINT CORES

Unit	Theoretical Maximum Applied Horizontal Shear Force, Eq.1,kN.	Theoretical Horizontal Shear Strength	
		ACI Approach Eq.2, kN.	Draft SANZ Code Approach Eqs.3,4 and 5, kN.
11	1,130	1,120	1,150
12	1,130	1,110	1,150
13	620	730	780
4	980	970	1,300*
5	1,110	980	1,110*
9	1,080	1,280	1,460*

* Not strictly applicable since no vertical shear reinforcement was present.

TABLE 4

VERTICAL SHEAR REINFORCEMENT REQUIRED IN JOINT CORES

Unit	Theoretical Maximum Applied Vertical Shear Force, Eqs. 1 and 9, kN	Area of Vertical Shear Reinforcement		
		Draft SANZ Code Requirement, Eqs.6, 7 and 8. mm^2	Existing in Joint Core mm^2	Modified draft SANZ Code Requirement Eqs.6,7 and 10. mm^2
11	1,270	1,010	570	630
12	1,270	990	800	600
13	700	575	570	370